31

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Dec 2015
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Timber Cassette Floors

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image: Bainsdale Library, Architect: NOWarchitecture
Photographer: NOWarchitecture

Wood Solutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-29-4

Acknowledgments

Authors: Prof Keith Crews, Dr Rijun Shrestha

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: May 2013
Revised: March 2015, April 2016

These materials are published under the brand WoodSolutions by FWPA.
This guide has been reviewed and updated for use in Australia by TDA NSW.

IMPORTANT NOTICE

WoodSolutions Australia is a registered business division of Forest and Wood Products Australia Limited.

Contents

1

Introduction

This Design Guide complements WoodSolutions Design Guide #30: *Timber Concrete Composite Floors.* Timber cassette floor systems are well established in Australia, but are mainly used for residential floor loads comprised of either seasoned sawn timber or engineered wood products (EWP) such as I-joists, in conjunction with sawn and dressed timber, particle board or plywood flooring between 17 and 22 mm in thickness.

This Guide presents a design procedure based on *AS 1720.1: 2010 Timber structures Part 1: Design methods* for composite timber floor structures, manufactured using EWPs such as LVL and glulam and fabricated into 'T or box beam 'cassettes'. The notations throughout this document are based on AS 1720.1.

Timber cassette floors consist of a timber joist (LVL or glulam beam) sandwiched between two timber sheathing layers. The sheathing is rigidly connected to the beams by a combination of adhesives and mechanical fasteners to ensure composite action.

Extensive laboratory testing has been undertaken to validate the design assumptions within this Guide. The results of this testing program have confirmed that, provided the flange to web connections meet the prescriptive requirements contained within this Guide, the floor can be designed using the existing provisions of AS 1720.1 as a fully composite section, with linear elastic behaviour in resisting load actions predicted using AS 1170 Structural Design Action series.

The design of timber concrete composite is covered in WoodSolutions Technical Design Guide *#30: Timber Concrete Composite Floors* for use in commercial and multi-residential timber buildings.

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

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.*

Related publications are listed under References at the end of this Guide.

2

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 standards.

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. This can be replaced with a dynamic analysis if available.

Unless noted otherwise in Figures 2.1 and 2.2, all symbols and letters used in the design procedure conform to those in AS 1720.1.

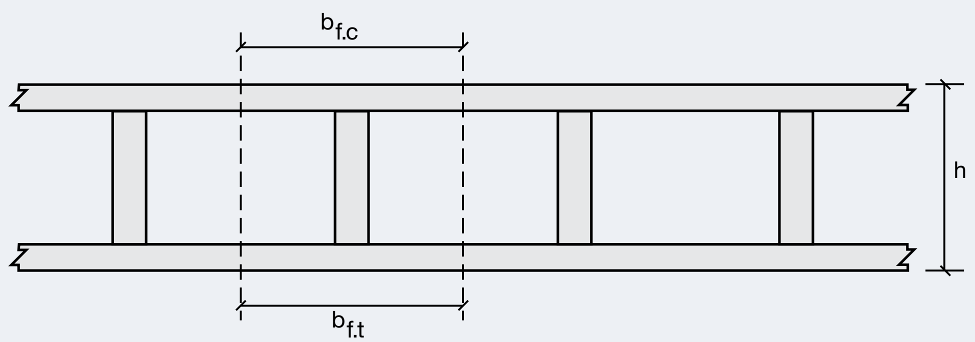


Figure 2.1: Notation for a typical composite timber floor system.

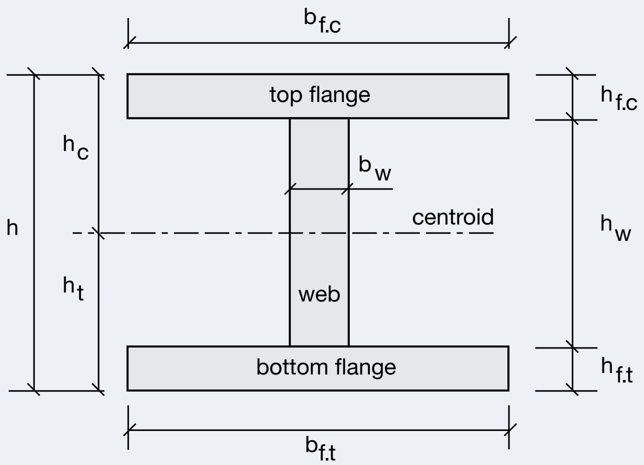


Figure 2.2: Notation for an individual 'cassette' in a typical composite timber floor design procedure.

3 Design Procedure

The design procedure has three fundamental stages:

1. Identifying the cross-sectional characteristics of the cross-section of the composite beam.
2. Evaluation of the strength capacity.
3. Assessment of the serviceability limit states.

3.1 Cross-section Characteristics

In cases where the flanges and webs have differing properties (such as the use of cross-laminated timber), it is necessary to determine the modular ratio and apply this to determine effective widths of members, prior to determination of the section properties.

For irregular sections (e.g. where the top and bottom flanges are different), the location of the centroid must be determined to enable the relevant section properties to be calculated.

It is strongly recommended that the c/c web spacing be such that shear lag effects do not occur in the flanges. This is normally met by satisfying Equations (3-1) to (3-3):

$$b_{f.t} = b_w + 0.1 \times span \qquad \text{for bottom flange} \quad (3\text{-}1)$$

$$b_{f.t} = b_w + 20 \times h_{f.t} \qquad \text{for bottom flange} \quad (3\text{-}2)$$

The minimum value of Equations (3-1) and (3-2) is used for $b_{f,t}$.

$$b_{f.c} \le (b_w + 0.1 \times span) \qquad \text{for top flange} \quad (3\text{-}3)$$

where $b_{f.c}$ and $b_{f.t}$ are width of the top (compressive) and bottom (tension) flanges, respectively, while b_w and $h_{f.t}$ are width (thickness) of the web and height of the bottom (tension) flange, respectively.

3.2 Design for Flexural Effects

The imposed uniformly distributed load (UDL) induces flexure in the webs and a combination of flexural and axial load effects in the flanges. This requires satisfying the requirements of Clause 3.5 of AS 1720.1, for combined bending and axial load effects.

The following equations apply to a simply supported beam and would need to be interpreted correctly for use with continuous beams.

Bending capacity of the section below the centroid is given by:

$$M_d = \phi k_1 k_4 k_6 k_9 k_{12} f'_b Z_{bot} \qquad (3\text{-}4)$$

where f'_b is characteristic bending strength of timber while Z_{bot} and Z_{top} represent section moduli below and above the centroid. Moreover, Φ and ki are capacity and modification factors defined in AS 1720.1 as:

Φ is capacity and modification factors (Section 2, AS 1720.1).

k_1 is modification factor for duration of load (Section 2, AS 1720.1).

k_4 is modification factor for moisture condition (Section 2, AS 1720.1).
In this Guide, seasoned timber is allowed hence, $k_4 = 1$.

k_6 is modification factor for temperature (Section 2, AS 1720.1).
For covered timber structures under ambient conditions $k_6 = 1$.

k_9 is modification factor for strength sharing between parallel members factor
Section 2, AS 1720.1). For glulam and LVL used in parallel systems shall be taken as unity, $k_9 = 1$.

k_{12} is stability factor (Section 3, AS 1720.1).

Axial capacity (compression) of the top flange, N_{d_top} is given by:

$$N_{d-top} = \phi\, k_1 k_4 k_6 k_{12}\, f'_c A_{f.c} \qquad (3\text{-}5)$$

Axial capacity (tension) of the bottom flange, N_{d_bot} is given by:

$$N_{d-bot} = \phi\, k_1 k_4 k_6 k_{11}\, f'_t A_{f.t} \qquad (3\text{-}6)$$

where f'_c and f'_t are characteristic axial strength of timber in compression and tension, respectively, while k_i and Φ are modification and capacity factors as defined in AS 1720.1. $A_{f.c}$ and $A_{f.t}$ represent cross-sectional areas of the top (compressive) and bottom (tension) flanges, respectively.

The axial force induced in each flange as a result of the bending action is calculated using the following equations:

Axial load induced (compression) in the top flange, N^*_c is given by:

$$N^*_c = \frac{M^*\left(h - h_{centroid} - \frac{h_{f.c}}{2}\right)}{I} A_{f.c} \qquad (3\text{-}7)$$

Axial load induced (tension) in the bottom flange, N_t^* is given by:

$$N^*_t = \frac{M^*\left(h_{centroid} - \frac{h_{f.t}}{2}\right)}{I} A_{f.t} \qquad (3\text{-}8)$$

where $h_{f.c}$ and $h_{f.t}$ are heights of the top (compressive) and bottom (tension) flanges, respectively, while $h_{centroid}$ and M^* represent distance from the centroid to the top of the top flange and bending action due to the factored loads specified in AS1170. I is second moment of inertia of the composite section.

Combined bending and compression – top flange:

$$\left(\frac{M^*}{M_d}\right)^2 + \frac{N^*_c}{N_{d\_top}} \le 1.0 \qquad (3\text{-}9)$$

$$\frac{M^*}{M_d} + \frac{N^*_c}{N_{d\_top}} \le 1.0 \qquad (3\text{-}10)$$

Combined bending and tension – bottom flange:

$$k_{12}\frac{M^*}{M_d} + \frac{N^*_t}{N_{d\_bot}} \le 1.0 \qquad (3\text{-}11)$$

$$\frac{M^*}{M_d} - \frac{N^*_t}{N_{d\_bot}}\frac{Z}{A} \le 1.0 \qquad (3\text{-}12)$$

where Z and A are section modulus and section area of the timber module while k_{12} is AS 1720.1 stability factor used in bending strength calculation. M^* and N^* are bending and axial actions due to the factored loads specified in AS 1170.

3.3 Design for Shear Effects

Shear is generally not a limiting state for strength in these types of floor beams.
However, a check of the web for shear is recommended:

$$V_d = \phi\, k_1 k_4 k_6 f'_S A_S \quad (3\text{-}13)$$

where f'_s and A_s are characteristic shear strength timber parallel to the grain and area corresponding to shear (A_s=2/3A) while k_i and Φ are modification and capacity factors as defined in AS 1720.1.

Connection details recommended for achieving fully composite design behaviour are specified in Chapter 4. The first moment of shear area (Q_{top} and Q_{bot}) and hence the shear flow (q_{top} and q_{bot}) at the interface between the web and the flanges can be checked using the following equations:

$$Q_{top} = A_{f.c}\left(h_c - \frac{h_{f.c}}{2}\right) \quad (3\text{-}14)$$

$$Q_{bot} = A_{f.t}\left(h_t - \frac{h_{f.t}}{2}\right) \quad (3\text{-}15)$$

$$q_{top} = \frac{Q_{top} V^*}{I} \quad (3\text{-}16)$$

$$q_{bot} = \frac{Q_{bot} V^*}{I} \quad (3\text{-}17)$$

where h_c and h_t are distances from the centroid to the top of the top flange and the bottom of the bottom flanges, respectively, while (V^*) is the acting shear force at the distance of 1.5d from the supports of the LVL modules.

3.4 Shear Strength at Glue Line

In the case of composite timber section, to ensure there is no failure in the glue line at the interfaces of LVL modules, the shear stress at the interfaces between the flange and the web must be checked according to:

$$\frac{QV^*}{I(2b_w)} \le \text{Min}\ (f'_s),\ (f'_{s,glue}) \quad (3\text{-}18)$$

V^* is the acting shear force at the distance of 1.5d from the supports of the LVL modules and Q is the first moment of area of the LVL cross section at the interface between the flange and the web. d is total depth of the timber cross section. $f'_{s,glue}$ is given by adhesive manufacturers.

3.5 Bearing Strength

The design capacity in bearing must satisfy the condition given in:

$$\phi N_p \ge N_P^* \quad (3\text{-}19)$$

$$\phi N_p = \phi\, k_1 k_4 k_6 k_7 f'_P A_P \quad (3\text{-}20)$$

where, N^*_p is the design load in bearing, f'_p is the bearing strength of the bottom flange, and A_p is the bearing area. k_7 represents modification factor of length and position of bearing as specified in Section 2 AS 1720.1.

3.6 Serviceability – Deflection

Limits on the deflection behaviour need to be determined to suit the functional requirements of the flooring system, in accordance with Guidelines presented in Appendix B of AS 1720.1.

Serviceability of the timber floor is assessed by checking the deflections against the limits defined to suit the functional requirements of the building being designed as:

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(\mathrm{G}+0.7\mathrm{Q})L^4}{384(EI)_{ef}} \le \frac{Span}{300} \quad (3\text{-}21)$$

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(\mathrm{G}+0.4\mathrm{Q})L^4}{384(EI)_{ef}} \le \frac{Span}{400} \quad (3\text{-}22)$$

where G is the self-weight plus the permanent loading and Q is the imposed loading. *L* is the span of the modules and EI_{ef} is the flexural stiffness of the modules. G+0.7Q is the short-term serviceability load combination and G+0.4Q is the long-term serviceability load combination. The minimum required flexural stiffness of the modules can be calculated from:

$$EI_{ef} = \frac{300 \times 5(W^*)L^3}{384} = \frac{300 \times 5(\mathrm{G}+0.7\mathrm{Q})L^3}{384} \quad (3\text{-}23)$$

$$EI_{ef} = \frac{400 \times 5(W^*)L^3}{384} = \frac{400 \times 5(\mathrm{G}+0.4\mathrm{Q})L^3}{384} \quad (3\text{-}24)$$

The serviceability load combinations and deflection limits are presented in Table 3.1.

The minimum required EI_{ef} due to different loading condition shall be less than EI_{ef} of the section.

G	self-weight & permanent loading (permanent)	Span/400
G+Q	imposed loading (instantaneous)	Span/300
G+ 0.7Q	imposed loading (short-term)	Span/300
G+ 0.4Q	imposed loading (long-term)	Span/400
1.0 kN	imposed 'impact' loading (vibration)	1-2 mm

Table 3.1: Load combinations and deflection limit for serviceability limit state design.

Load combination	
1.35G	self-weight & permanent loading (permanent)
1.2G+(1.5ψ)Q,ψ=0.4	imposed loading (long-term)
1.2G+1.5Q	imposed loading (short-term)
1.0 kN	imposed 'impact' loading (vibration)

Table 3.2: Load combinations and deflection limit for ultimate limit state design.

Table 3.2 lists the ultimate load combinations. To consider the long-term deformation of a structure to satisfy a specific serviceability limit state, an appropriate modification factor for creep (see Table 3.3) should be applied to the deformation as specified by AS 1720.1. To consider the creep deformation of the timber modules, the creep factor is applied for the portion of the serviceability load that is permanently applied. Therefore, the minimum required EI_{ef} of timber modules needs to be calculated based on some modification to Equations (3-24) and (3-25) by:

$$EI_{ef} = \frac{300 \times 5(j_2\mathrm{G}+0.7\mathrm{Q})L^3}{384} \quad (3\text{-}25)$$

$$EI_{ef} = \frac{300 \times 5 \times j_2(\mathrm{G}+0.4\mathrm{Q})L^3}{384} \quad (3\text{-}26)$$

where the values of j_2 are presented in Table 3.3.

Loading	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 mid-span deflection under a point load imposed 'impact' loading for vibration is assessed by:

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

where p^* is design action for point load action, for which the value of ψ and $(EI)_{ef}$ are defined to suit the loading condition and duration. The mid-span deflection under a point load imposed 'impact' loading should be less than 2 mm.

3.7 Serviceability – Dynamic Behaviour

In addition to the 1 kN point load vibration check, a more rigorous dynamic assessment can be carried out based on the first fundamental frequency of the timber cassette floor. Note that this formula (above) predicts the behaviour of a simply supported single span beam.

The first natural frequency of the floor systems is generally recommended to be more than 10 Hz, while natural frequencies below 3 Hz and between 5 Hz to 8 Hz should be avoided to prevent walking resonance and human discomfort, respectively. There are several prediction formulas proposed to calculate the first natural frequency of the structure. In this Design Guide, the proposed formula in Eurocode 5, BS EN 2004[1] is used to predict the dynamic behaviour of the timber floor modules:

$$f_1 = \frac{\pi}{2l^2}\left(\frac{EI_{ef}}{m}\right)^{0.5} \quad (3\text{-}28)$$

where f_1 is the fundamental frequency of the floor modules, EI_{ef} is the equivalent bending stiffness of the floor modules in the perpendicular to the beam direction (kNm^2), L is the floor span (m) and m is the mass per unit area (kg/m^2). The mass includes self-weight of the floor and other permanent actions, such as imposed dead load.

4

Manufacturing Provisions

The recommended procedure for connecting flanges to webs is 'gluing and screwing'. The design philosophy is that the glue creates an infinitely stiff bond to resist serviceability load events, while the screws provide a mechanical connection so composite action occurs at the design ultimate load events.

In research by the University of Technology Sydney for the Structural Timber Innovation Company, the glue bond was a PURBOND polyurethane glue, fastened using 14G Type 17 screws (see Figure 4.1) at nominal centres of 400 mm c/c along the entire length of the web. It is recommended this method be a starting point for design. Reference should always be made to timber fabricators for their preferred 'gluing and screwing' method.

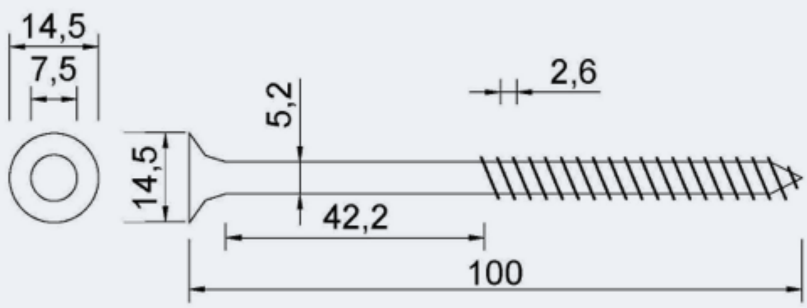


Figure 4.1: Dimensions of the Type 17 screws used for manufacture.

5

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 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 on acoustic performance of timber floors are summarised below:

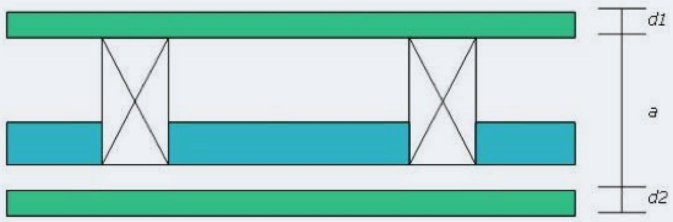


Figure 5.1: Double layer floor with good acoustic properties.

5.1 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 5.1) by providing either 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/m^2. 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 on 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/m^2 and 150 mm thick sound absorbing material with ceiling boards supported on resilient metal channels has 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 = about 2 in Figure 5.1) options, are suitable for satisfactory acoustic properties.[2]

5.2 Guidance on Improving the Impact Sound Insulation

1. Increasing the mass or separating the ceiling from the floor joists can improve impact sound insulation of timber floors.
2. Good impact sound insulation can be achieved for floors constructed with a sub-floor layer (e.g. particleboard, 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 5.2) or in direct connection with walls or columns. Place a resilient under layer between the floor covering and any walls or columns.
6. The addition of transverse stiffeners can improve the high frequency impact insulation of the floor but reducing the joist spacing may not always improve this.

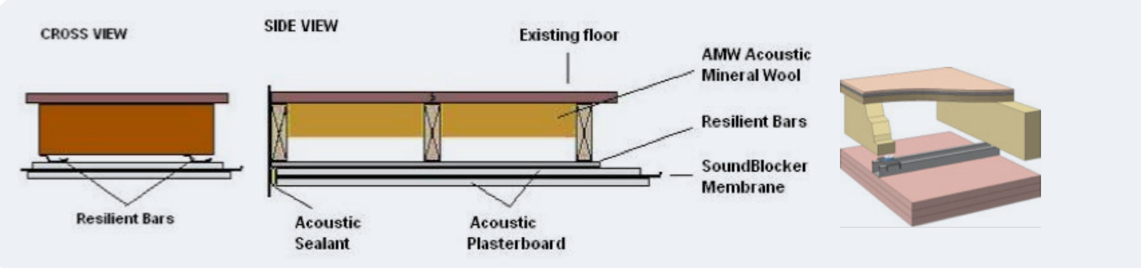


Figure 5.2: Acoustic improvement methods.

A

Appendix A: Worked Example – 8.5 m Timber Floor Span

In this section, the serviceability and ultimate capacities of a timber cassette floor are calculated in accordance with this Guide. As shown in Figure A1.1, LVL 11 was used for the timber cassette floor. The different stages of calculations are summarised in the following sections.

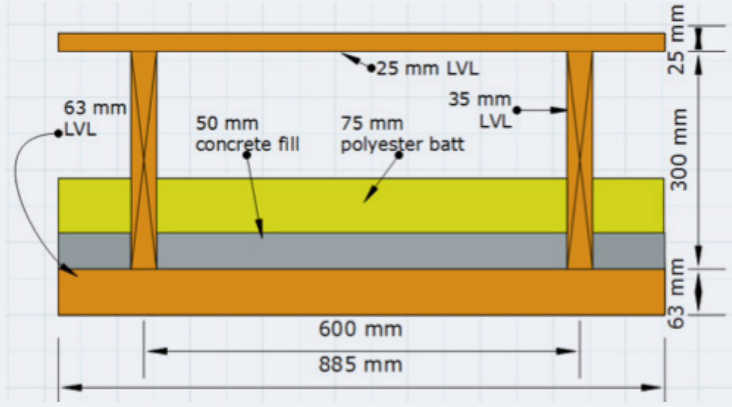


Figure A1.1: Dimensions of 8.5 m timber modules.

A1.1 Material Properties

Material Properties of the timber module based on manufacturer's data are:

• Timber type	LVL 11
• Timber modulus	E_t = 11000 MPa
• Timber density	ρ = 620 kg/m^3
• Timber bending strength	f'_b = 38 MPa
• Timber compression strength	f'_c = 38 MPa
• Timber tensile strength	f'_t = 26 MPa
• Timber shear strength	f'_s = 5.3 MPa

A1.2 Section Properties

$$b_{f.t} \leq \text{Min}\begin{cases} b_w + 0.1 \times span \\ b_w + 20 \times h_{f.t} \end{cases}$$ for bottom flange

$b_{f.t}$ = minimum of (35+0.1 x 8500) and (35+20 x 63) = 885 mm (width of the bottom flange)

b_w = 35 mm (thickness of the web)

$h_{f.t}$ = 63 mm (height of the bottom flange)

$b_{f.c} \leq$ (b_w+0.1 x *span*) for top flange

$b_{f.c}$ = 35+0.1 x 8500 = 885 mm (width of the top flange)

Neutral axis (from bottom to centroid):

y_c = 147.85 mm

A_c = 37175 mm^2

A_t = 61635 mm^2

The calculation of moment of inertia of the timber floor section is given in Table A1.1.
The equivalent section of timber floor is used for the calculation as shown in Figure A1.2.

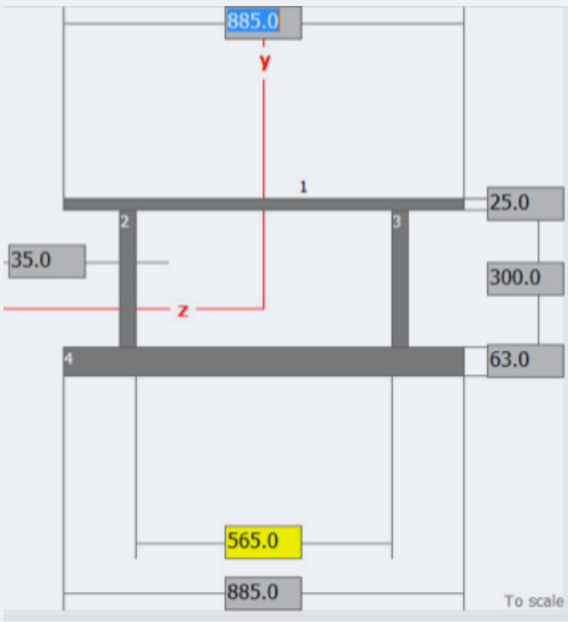


Figure A1.2: Equivalent section of timber floor (mm).

	I =1/12bh^3 (mm^4)	A (mm^2)	d (mm)	I+Ad^2 (mm^4)
Top Flange	1152343.75	22125	228.5	1156348375
Webs	157500000	21000	66	248976000
Bottom Flange	18440966.25	55755	115.5	762226605
I_{tot}				2167550980

Table A1.1: Calculation of second moment of area for the timber floor.

where d is the distance between centroid of each component and the whole section.

$I_{ef} = \Sigma (I+Ad^2) = 21.68 \times 10^8$ mm^4

Section Modulus (top): $Z_{top} = 8.99 \times 10^6$ mm^3

Section Modulus (bottom): $Z_{bot} = 14.74 \times 10^6$ mm^3

$EI_{ef} = 23.84 \times 10^{12}$ N.mm^2

A1.3 Action Loads

The loading input is according to AS 1170 and it includes 2.5 kPa permanent action (DL), 3.0 kPa imposed action (LL) and the self-weight:

Super imposed dead load, SDL = 2.5 kPa

Permanent Load (allows concrete topping, services and partitions): 2.5 (kPa)(b_{tf}) = 2.21 kN/m

Self–Weight:

Timber:

$[(b_{tf})(d_{tf}) + 2(b_w)(d_w) + (b_{bf})(d_{bf})] \times (\rho_{timber})$

= [0.885 x 0.063+0.885 x 0.025 +2 x 0.3 x 0.035] x 620=0.098 x 620x10^{-4}=0.61 kN/m

Spacing (S) = 0.89 m

Self weight of concrete and insulation is included in the SDL.

Self-weight total = 0.61 kN/m

G = (SDL+ Self-weight/S) G = (0.613/0.885+2.5)0.885 = 2.83 kN/m

Live Load (LL) = 3 kPa

Q=LL(btf) = 3x0.885=2.66 kN/m

$M^* = w^*L^2/8$

$V^* = w^*L/2$

Different combinations and the deflection limit for ultimate limit state design are listed in Table A1.2.

A1.4 Serviceability Checks

The load combinations and the deflection criteria for serviceability check are presented in this Guide.

According to AS 1170 and AS 1720.1, Equations (3-22) and (3-23) must be satisfied to check the serviceability performance of timber modules.

Since the moisture of content of timber module (LVL) is less than 15%, the creep factor, j_2 is equal to 1 and 2 for short-term and long-term serviceability check, respectively (Table 0.3).

To consider the creep deformation of the timber modules, the creep factor is applied for the portion of the serviceability load that is permanently applied. Hence, the minimum required short- and long-term EI_{eff} of timber modules need to be calculated based on Equations (3-24) and (3-25), respectively:

$$EI_{ef} = \frac{300 \times 5(j_2 G + 0.7Q)L^3}{384} = 1.13 \times 10^{13}\text{Nmm}^2$$ short-term EI_{ef} (Equation [3-24])

$$EI_{ef} = \frac{300 \times 5 \times j_2(G + 0.4Q)L^3}{384} = 1.87 \times 10^{13}\text{Nmm}^2$$ short-term EI_{ef} (Equation [3-25])

Comparing minimum required EI_{ef} (obtained from Equations (3-24) and (3-25) and EI_{ef} of the section ($23.8\text{x}10^{-13}$ Nmm^2), the specified section satisfied the minimum EI_{ef} requirement.

Considering the effect of creep factor, the short- and long-term deflections of the 8.5 m LVL modules (Equations [3-24] and [3-25]) are equal to 13.4 mm and 11.1 mm, respectively, which are less than Span/300 (or 28.3 mm) and Span/400 (or 21.3 mm), respectively.

$$\Delta = \frac{5(W^*)L^4}{384(EI_{ef})} = \frac{5(G + 0.7Q)L^4}{384(EI_{ef})} = \frac{5(2.83 + 0.7 \times 2.66)8500^4}{384(23.8 \times 10^{12})} \le \frac{Span}{300}$$ 13.4 < 28.3 mm OK

$$\Delta = \frac{5(W^*)L^4}{384(EI)_{ef}} = \frac{5(G + 0.4Q)L^4}{384(EI)_{ef}} = \frac{5(2.83 + 0.4 \times 2.66)8500^4}{384(23.8 \times 10^{12})} \le \frac{Span}{400}$$ 11.1 < 21.3 mm OK

Now consider the short-term deflection check for 1 kN point load – Equation (3-28).

$$\Delta = \frac{PL^3}{48(EI)_{ef}} = \frac{1 \times 8500^3}{48 \times 1.06 \times 10^{13}} \le 2 \text{ mm}$$ 0.54 < 2 mm OK

Short-term deflection check for 1 kN point load) is also equal to 0.54 mm which is smaller than the deflection limit (2 mm).

A1.5 Strength Checks

Characteristic values for structural LVL shall be obtained from the manufacturer. Characteristic values for LVL shall include consideration of the section sizes to which they are intended to apply. Unless otherwise specified by the manufacturer, the characteristic values for LVL for bending and tension shall be modified using Equations specified in Section 8.3 AS 1720.1.

Table A1.2 shows the load combination and the maximum bending and shear force (refer Table 0.2 in body of text). The required modification factors are presented in Table A1.3 and Table A1.4 which are based on AS 1720.1.

Combination	*w** (kNm)	*M** (kNm)	*V** (kN)	N^*_c (kN)	N^*_t (kN)
1.35G	3.8	34.5	16.2	80.5	185.9
1.2G + (1.5ψ)Q, ψ=0.4	5.0	45.1	21.1	105.1	242
1.2G + 1.5Q	7.4	66.7	31.4	155.5	359.4

Table A1.2: Load combinations and deflection limit for ultimate limit state design.

Force	*Φ*	k_4	k_6	k_9	k_{12}
Tension	0.9	1	1	-	-
Compression	0.9	1	1	-	1
Shear	0.9	1	1	-	-
Bending	0.9	1	1	1	1

Table A1.3: AS 1720.1 modification factors.

Load-Combination	k_1	*Comment*
Permanent	0.57	50+ years
Long-term	0.8	5 months
Short-term	0.97	5 hours
Bending	0.9	1

Table A1.4: k_1 modification factors.

Combination		M_{dtop} (kNm)	M_{dbot} (kNm)	N_{dtop} (kN)	N_{dbot} (kN)
1.35G	1-self-weight & permanent loading (permanent)	175.3	287.3	431.3	1086.9
1.2G + (1.5ψ)Q, $\psi = 0.4$	2-imposed loading (long-term)	246	403.3	605.3	1525.5
1.2G + 1.5Q	3-imposed loading (short-term)	298.2	489	734	1849.6

Table A1 5: Axial and flexural strength values.

All the results of the bending and axial ratios for top and bottom flanges are summarised in Table A1.6 for 8.5 m timber modules.

By substituting the values of material properties and the dimensions of the system into Equations (3-4) to (3-7) and considering the values of Tables A1.2 to A1.4, the bending strength ratio for top and bottom flanges and for permanent, long-term and short-term load combinations are calculated as given in Table A1.6.

Load-Comb.	*Bending: Top Flange*	*Bending: Bottom Flanges*	*Axial: Top Flange*	*Axial: Bottom Flanges*
1-Permanent	0.20 ≤ 1 OK	0.12 ≤ 1 OK	0.19≤ 1 OK	0.17 ≤ 1 OK
2-Long-term	0.18 ≤ 1 OK	0.11 ≤ 1 OK	0.17 ≤ 1 OK	0.16 ≤ 1 OK
3-Short-term	0.22 ≤ 1 OK	0.14 ≤ 1 OK	0.21 ≤ 1 OK	0.19 ≤ 1 OK

Table A1.6: Bending and axial load ratio checks.

By substituting the values of material properties and the dimensions of the system into Equations (3-10) to (3-13) – (the components of Equations (3-10) to (3-13) are precisely described in AS 1720.1) – and considering the values of Tables A1-2 to A1-4, the combined bending and axial strength ratio checks for top and bottom flanges and for permanent, long-term and short-term loading are calculated, as given in Table A1-7.

Load Comb	*Bending and Compression (Eq. 3-10)*	*Bending and Compression (Eq. 3-11)*	*Bending and Tension (Eq. 3-12)*	*Bending and Tension (Eq. 3-13)*
1	0.22 OK	0.38 OK	0.29 OK	0.10 OK
2	0.21 OK	0.36 OK	0.27 OK	0.10 OK
3	0.26 OK	0.43 OK	0.33 OK	0.12 OK

Table A1.7: Combined Bending and axial ratio checks.

The results of the shear stress ratio for top and bottom flanges interfaces are reported in Table A1.5. The Equation (3-14) and Tables A1.2 to A1.4 are used for the shear stress ratio checks. The values of shear stress ratio for permanent, long-term and short-term load combination are less than one.

Load-Combination	V_d (kN)	*Max Flexural Shear*
Permanent	179.1	0.09 ≤1 OK
Long-term	251.3	0.08 ≤1 OK
Short-term	304.7	0.10 ≤1 OK

Table A1.8: Shear stress ratio check.

A1.6 Dynamic Performance Check

For the dynamic performance of the LVL modules, the first natural frequency of the system is calculated according to Equation (3-29):

$$f_1 = \frac{\pi}{2l^2}\left(\frac{EI_{ef}}{m}\right)^{0.5} = \frac{\pi}{2(8.5)^2}\left(\frac{10603.25}{0.267}\right) = 4.33 \text{ Hz}$$

The first natural frequency of the system is about 4.33 Hz which is in the safe frequency zone.

B

Appendix B: Notation

Symbols and letters used in the Guide are listed below:

A	cross-sectional area of the entire section
$A_{f.c}$	cross-sectional area of the top (compressive) flange
$A_{f.t}$	cross-sectional area of the bottom (tension) flange
A_s	shear area of the web = 2/3 $b_w h_w$
A_w	cross-sectional area of the web
A_p	bearing area
$b_{f.t}$	width of the bottom (tension) flange
b_w	width (thickness) of the web
d	total depth of the timber cross section
h	overall depth of floor
h_c	distance from the centroid to the top of the top flange
h_t	distance from the centroid to the bottom of the bottom flange
h_w	depth of the web
$h_{f.t}$	height of the bottom (tension) flange
$h_{centroid}$	distance from the bottom of the bottom flange to the centroid = $h_{f.t}$
I	second moment of inertia of the composite section
Z_{top}	section modulus above the centroid (top flange)
Z_{bot}	section modulus below the centroid (bottom flange)
E	value of the modulus of elasticity of the timber members
f_1	fundamental frequency of the floor modules
f'_b	characteristic strength in bending
f'_c	characteristic strength in compression
f'_s	characteristic strength in shear
$f'_{s,glue}$	characteristic shear strength of the adhesive
f'_t	characteristic strength in tension
j_2	stiffness modification factor – load duration specified in AS 1720.1
k_1	duration of load (timber) specified in AS 1720.1
k_4	moisture condition (timber) specified in AS 1720.1
k_6	temperature (timber) specified in AS 1720.1
k_7	length and position of bearing (timber) specified in AS 1720.1
k_9	strength sharing between parallel members specified in AS 1720.1

k_{11} size factor (timber) – this is normally applied to the characteristic strength property by the manufacturer

k_{12} stability factor (timber) specified in AS 1720.1

m mass per unit area

M^* moment action resulting from applied loads

M_{d_top} design moment capacity – top flange

M_{d_bot} design moment capacity – bottom flange

N^*_c axial force (compression) induced in top flange from bending

N^*_t axial force (tension) induced in bottom flange from bending

N_{d_top} design axial capacity (compression) – top flange

N_{d_bot} design axial capacity (tension) – bottom flange

Q_{top} first moment of shear area for top flange

Q_{bot} first moment of shear area for bottom flange

q_{top} shear flow at interface between web and top flange

q_{bot} shear flow at interface between web and bottom flange

V^* maximum shear effect

V_d design shear capacity of the web

Φ capacity factor

References

1. BS EN 2004, *Eurocode 5-Design of Timber Structures-Part 1-1 in General rules and rules for buildings.* 2004, European Committee for Standardization: Brussels, Belgium.

2. Kolb, J., *Systems in Timber Engineering: Loadbearing Structures and Component Layers*. 2008: Springer.

Australian Standards

AS 1170.1, *Structural design actions, in Part 1: Permanent, imposed and other actions*. Standards Australia, 2002.

AS 1720.1, *Timber structures, in Part 1: Design methods.* Standards Australia, 2010.

WoodSolutions Technical Design Guides

WoodSolutions Technical Design Guide #15: *Fire Design*, Forest and Wood Products Australia, Melbourne, Australia, 2016.

WoodSolutions Technical Design Guide #30*: Timber Concrete Composite Floors*. Forest and Wood Products Australia, Melbourne, Australia, 2016.

WoodSolutions Technical Design Guide #35*: Floor Diaphragm*. Forest and Wood Products Australia, Melbourne, Australia, 2016.

Discover more ways to build your knowledge of wood

If you need technical information or inspiration on designing and building with wood, you'll find WoodSolutions has the answers. From technical design and engineering advice to inspiring projects and CPD linked activities, WoodSolutions has a wide range of resources and professional seminars.

www.woodsolutions.com.au

Your central resource for news about all WoodSolutions activities and access to more than three thousand pages of online information and downloadable publications.

Technical Publications

A suite of informative, technical and training guides and handbooks that support the use of wood in residential and commercial buildings.

WoodSolutions Tutorials

A range of practical and inspirational topics to educate and inform design and construction professionals. These free, CPD related, presentations can be delivered at your workplace at a time that suits you.

Seminars and Events

From one day seminars featuring presentations from leading international and Australian speakers to international tours of landmark wood projects, WoodSolutions offer a range of professional development activities.

What is WoodSolutions?

Developed by the Australian forest and wood products industry for design and building professionals, WoodSolutions is a non-proprietary source of information from industry bodies, manufacturers and suppliers.

Wood Solutions™
design and build
woodsolutions.com.au

32

Wood Solutions™
design and build
woodsolutions.com.au

This guide was revised in April 2016
Please check
woodsolutions.com.au/publications
to ensure you have the latest edition

Long Span Roofs - LVL Portal Frames and Trusses

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Downloading of these Technical Design Guides is restricted to an Australian market only. Material is only for use within this market. Documents obtained must not be circulated outside of Australia. The Structural Timber Innovation Company, its shareholders or Forest Wood Products Australia, will not be responsible or liable for any use of this information outside of Australia.

Cover image: Netball Central, Sydney Olympic Park, Architect: Scott Carver
Photographer: Geoff Ambler

WoodSolutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-30-0

Acknowledgments

Authors: Pierre Quenneville, Felix Scheibmair

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: 2013
Revised: April 2016

These materials are published under the brand WoodSolutions by FWPA. This guide has been reviewed and updated for use in Australia by TDA NSW.

WoodSolutions Australia is a registered business division of Forest and Wood Products Australia Limited.

Contents

Contents

1

Introduction

The aim of this Long Span Roof Design Guide is to provide optimised design solutions for a range of long span Laminated Veneer Lumber (LVL) portal frames and trusses under different loadings. These optimised design solutions respect the required structural capacities and deflection performances under the Australian Standards while using the minimum volume of LVL.

Engineers may use the solutions from this Guide as a starting point to their own design and adjust the design factors accordingly for specific design requirements. Engineers and future owners of the proposing properties may also use the solutions from this Guide to estimate the costs and allow comparisons with other design options.

Three types of efficient timber roof configurations have been included in this Guide. They are mono-pitch portal frames, double-pitch portal frames and Pratt trusses. Each of these three roof configurations have been designed over a range of large clear spans (10 metres to 70 metres) and at different clear heights.

Each portal frame and truss design has been done using high-strength engineered timber product – Laminated Veneer Lumber (LVL). All portal frames solutions are designed using a box section of LVL. All truss solutions are designed using solid sections of LVL.

Extensive finite element analysis has been carried out for each roof configuration to ensure an optimised solution has been reached. Result tables and configuration drawings of the final optimised solutions for portal frames and trusses are provided. The Guide also includes supporting materials for helping in design, as given in Section 8. Design examples are provided in the last sections.

2 Scope

2.1 Summary of Design Parameters

This section contains a summary of the design parameters and assumptions for each type of long span roof design. The detailed explanations are provided in later sections.

2.1.1 Portal Frames

Two common types of portal frames are included in design: a portal frame with an internal prop and a gable portal frame.

Portal frame with internal prop

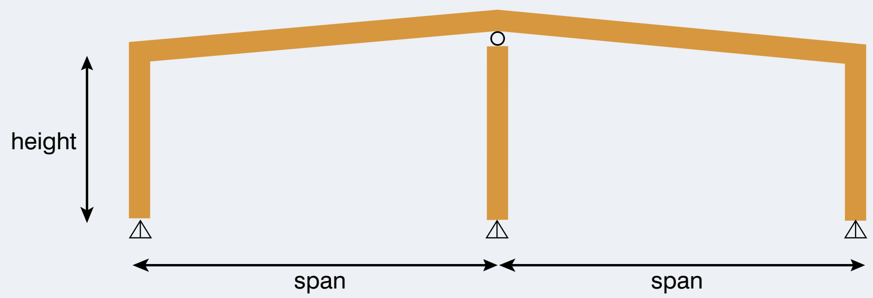


Figure 2.1: Portal frame with internal prop

- Design spans of 20 m, 25 m, 30 m, 35 m, 40 m, 45 m & 50 m
- Side column heights of 5 m, 6 m & 7 m
- Pinned column bases

Gable portal frame

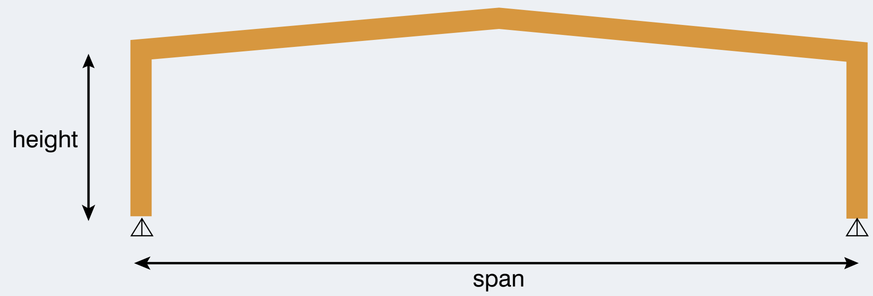


Figure 2.2: Gable portal frame

- Spans of 15 m, 20 m, 25 m, 30 m, 35 m, 40 m & 45 m
- Column heights of 5 m, 6 m & 7 m
- Pinned column bases

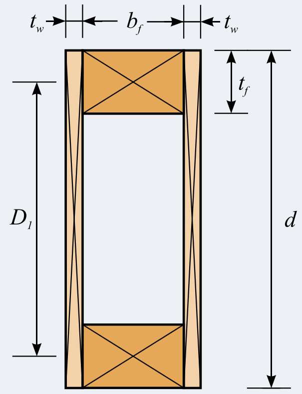


Figure 2.3: Typical box beam.

Cross section and material

All portal frame solutions are designed as box sections of Grade 11 LVL.

- The depth (*d*) of the box sections ranges from 400 mm to 1200 mm.
- The width (b_f) is selected to maximise material utilisation from a 1200 mm-wide LVL billet. Refer to Section 8.3.
- Two optimised solutions are provided for every portal frame, one in 45 mm web thicknesses (t_w), the other in 63 mm web thicknesses (t_w).

All webs of the box sections are with two cross-bands.

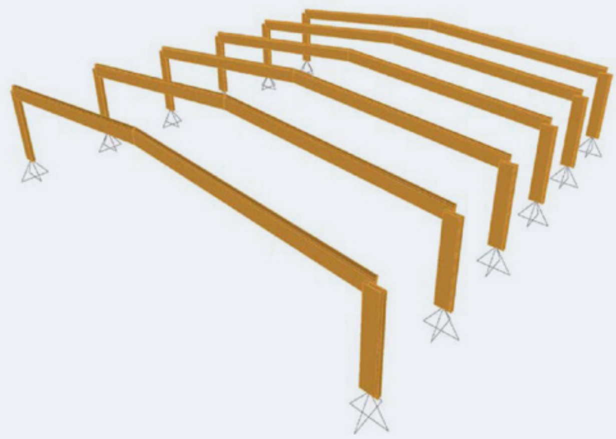


Figure 2.4: Gable portal frame.

Other assumptions

- 8 m frame spacing
- 5° degrees roof pitch
- Importance Level 2
- Design location: Auckland
- Design life: 50 years
- Pre-cambering equal to self-weight deflection
- Fixed rafter to column and apex connections
- Same cross-section throughout the whole portal frame

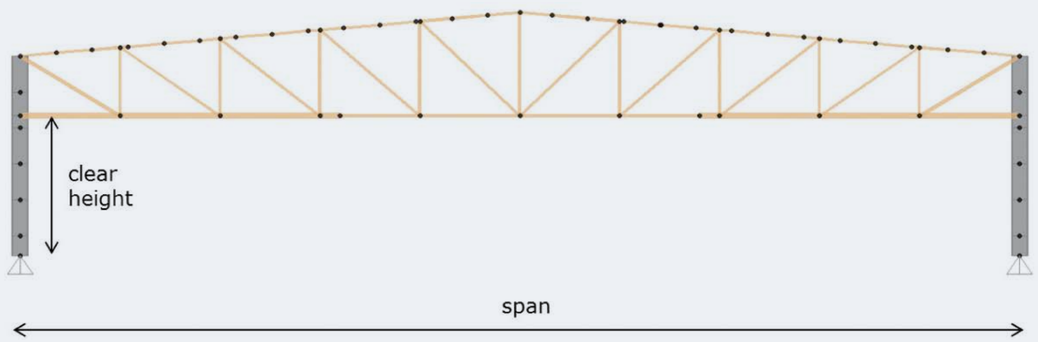


Figure 2.5: Trussed roof.

2.1.2 Truss Design Parameters

- Design spans of 50 m, 60 m, 70 m
- Column clear heights of 7 m
- Pinned column bases

Cross section and material

All truss members are designed as solid sections of Grade 11 LVL.

- The depth (*d*) of the truss members ranges from 150 mm to 400 mm.
- The width (*b*) of the truss members are either 126 mm or 180 mm.

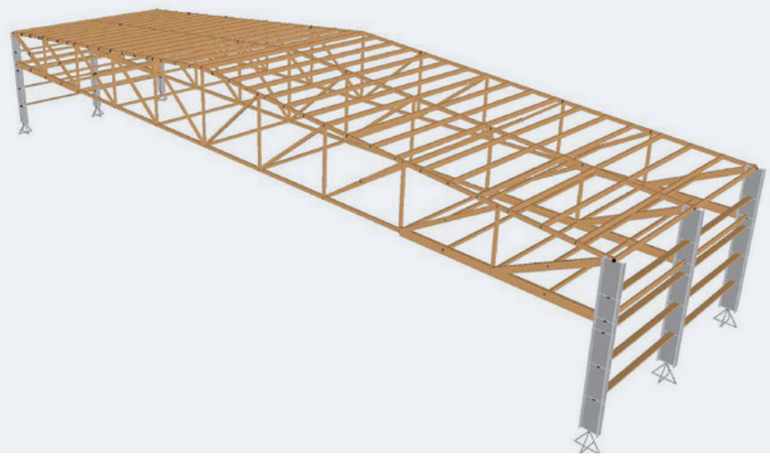


Figure 2.6: Trussed roof.

Other assumptions

- Pratt truss configuration
- 8 m truss spacing
- 5° degrees roof pitch
- Importance Level 2
- Design location: Auckland
- Design life: 50 years
- Pre-cambering equal to self-weight deflection
- Simple pinned connections throughout the truss
- All connection requirements to be met using timber rivets

2.2 Design Solution Criteria

The following sections are provided to clarify the design criteria, design conditions, loadings and assumptions made in each design.

All design solutions must satisfy two important criteria:

1. Optimal solution: All final solutions must achieve the required structural capacities and deflection performances under the Australian Standards while using the minimum volume of timber (see Section 8.4).
2. Minimised wastage: In order to minimise the wastage of LVL within each portal frame and truss project, the flange width of a box section will always be a width that maximises material utilisation from a 1200 mm-wide LVL billet (see Sections 8.3 and 8.4).

Both criteria have a common objective of saving costs. By adopting optimised design solutions that use the minimum volume of timber and have minimised wastage in mind, the material cost for a project will be reduced to a minimum and the LVL supplied for the project will be efficiently used.

2.3 Portal Frame Arrangements

2.3.1 Portal Frame with Internal Prop

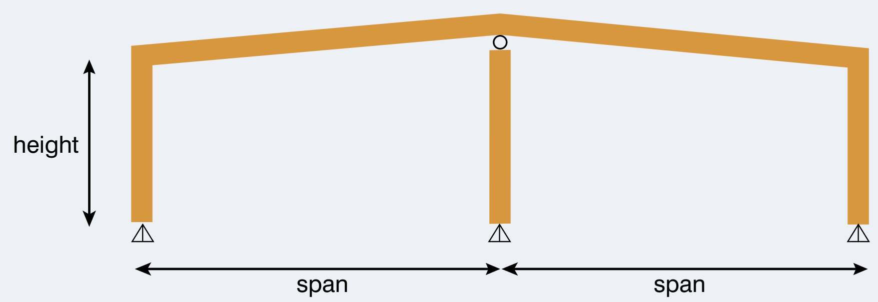


Figure 2.7: Portal frame with internal prop.

This arrangement is constructed by a double-pitch frame with an inner prop column that is pinned to the apex of the double-pitch frame.

The end columns to rafters (knee joints) and the apex joint between the rafters are fixed joints. The inner prop column carries axial loads only, and does not contribute to moment resisting.

2.3.2 Double-Pitch (Gable) Portal Frame

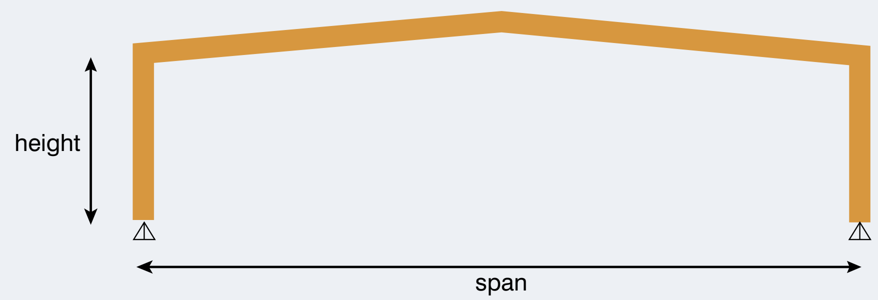


Figure 2.8: Double-pitch (gable) portal frame.

The double-pitch portal frame is the most standard configuration of a portal frame structure. The double-pitch portal frames are assumed to be a ‘two-pin’ configuration. The column to rafter joints (knee joints) and the apex joint between the rafters are fixed joints. The bases of the columns are pinned. The same cross-section is used to construct the entire portal frame.

2.4 Truss Arrangement

The Pratt truss configuration is adopted for all timber trusses. The truss members are assumed to be joined using simple connections throughout.

Steel columns are used to support the trusses. LVL columns are typically not stiff enough and the required cross-section makes them uneconomical.

The bases of the columns are pinned. In all the trusses, bottom chords are braced at every node. The top chords are assumed laterally supported by the roof purlins (spaced at 1.8 metres). For every truss solution, a constant member width has been maintained throughout to allow ease of detailing the connections.

2.5 Frame/Truss Spacing

All portal frames and trusses are designed at 8 metres spacing. Longer spacing is possible using either LVL roof purlins or I-joists. However, a longer spacing will increase frame/truss loads. For optimum use of roof purlin span and spacing, please refer to the WoodSolutions Technical Report*: Timber Portal Frames.*

2.6 Roof Inclination

All portal frames and trusses are designed to have a 5° degree roof pitch.

2.7 Importance Level

All structures are designed in Importance Level 2 under Table B1.2a of the BCA or Table F1 of AS 1170.0 *Structural design actions, General principles.*

2.8 Design Location

All portal frames and trusses designed in this Guide are assumed to be located in Auckland, New Zealand. However, the solutions are applicable to all other areas located within the same AS 1170.2 *Structural Design Actions Part 2: Wind Actions* region A6. None of the designs have been made considering seismic effects. However, if a lightweight steel cladding option is used for wall and roof, the sections provided should be valid for most locations affected by seismic activity.

2.9 Design Life

The design life for most buildings in Australia is 50 years. As a result, the design life of the portal frames and trusses in this Guide is also assumed to be 50 years.

2.10 Pre-Cambering

Pre-cambering is making a beam or rafter slightly curved/bowed upwards, so that under loads, usually gravity loads (also known as self-weight), the beam or rafter would settle into a position. In this way, the deflection of the member is eliminated or partially eliminated. For long span roofs and beams, it is common to pre-camber out the gravity deflection. In this Guide, all portal frames and trusses are assumed to have a pre-camber that is equal to the gravity deflection.

2.11 Connections

No attempt to provide working solutions for any of the connections for the portal frames or trusses has been made in this Guide. However, the magnitude of the forces and moments has been considered in order to ensure that usual portal frame moment connections (using the quick-connect moment connections or nailed gusset moment connections) are feasible or that timber rivet solutions could be used for the truss member connections. For portal frame moment connection solutions, please refer to the WoodSolutions Technical report: *Timber Portal Frames* and WoodSolutions Design Guide #33*: Quick-Connect Moment Connection.*

2.12 Loadings

Permanent, imposed and wind loads were considered in the design of portal frames and trusses. Auckland is a low earthquake activity city, and the designed structures are large span low-rise portal frames and trusses, which means the earthquake action (E) is small compared to the other lateral design action, wind (W). Therefore, earthquake action (E) is not the critical horizontal design action. Also, it is not likely to snow in Auckland. Other than permanent and live loads, wind loads are likely to be dominant in the design actions. Therefore, the permanent, imposed and wind loads are the three main design actions considered in this Guide, the earthquake and snow loads are less significant and it is safe to neglect them in the structural designs of this Guide. For detail considerations of loadings, refer to *AS 1170.0 Structural design actions, General principles.*

2.12.1 Permanent Loads (G)

The Permanent loads (G) are taken as 10 kg/m3 plus the self-weight of the frame or truss.

2.12.2 Imposed Loads (Q)

The imposed loads (Q) are taken as 0.25 kPa for all designs according to Table 3.2 in *AS 1170.1 Structural design actions, Permanent, imposed and other actions.*

2.12.3 Wind Loads (W)

For single-storey large span roof structures, wind actions are one of the dominant types of loads and are most likely to govern the design. Therefore, precise wind actions experienced by the structures in every direction must be carefully modelled and analysed. Wind loads are generally categorized in two directions, Wind Across the building and Wind Along the building.

For each of the Wind Across and Wind Along, direction of wind may result in upward lifting or downward pressing of the roof. As a result, there are an overall of four critical cases of wind actions to be designed for. They are:

- Wind Across – maximum uplift
- Wind Across – minimum uplift
- Wind Along – maximum uplift
- Wind Along – minimum uplift

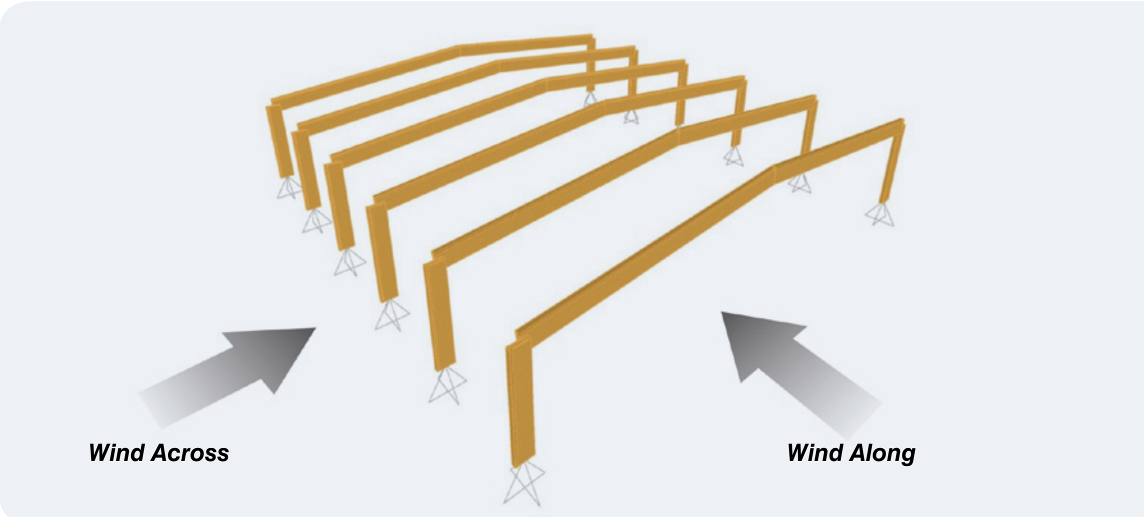


Figure 2.9: Portal frame under two wind directions.

Wind loads generally vary across and along the building, depending on the terrain category, size of the building, wind zone and some other factors.

This Guide provides solutions under Terrain Category 2 and Terrain Category 3.

Supporting materials are provided to help with determining wind loads (refer to Section 8). For a comprehensive wind action calculation, refer to Section W5 of *AS 1170.2 Structural Design Actions Part 2: Wind Actions*. Examples for calculating the wind loads are also provided in Section 7.

2.12.4 Load Combinations

The load combinations for ultimate limit state strength capacity included in the design are:

- [1.35G]
- [1.2G, 1.5Q]
- [0.9G, $W_{across, maximum uplift - ULS}$]
- [0.9G, $W_{along, maximum uplift - ULS}$]
- [1.2G, $W_{across, minimum uplift - ULS}$]
- [1.2G, $W_{along, minimum uplift - ULS}$]

The load combinations for serviceability limit states deflection checks included in the design are:

- [2G]
- [0.7Q]
- [$W_{across, maximum uplift - SLS}$]
- [$W_{along, maximum uplift - SLS}$]
- [$W_{across, minimum uplift - SLS}$]
- [$W_{along, minimum uplift - SLS}$]

It is noted to take away the pre-cambering effect when conducting serviceability deflection checks.

3

Portal Frame with Inner Prop Solutions

The following tables are optimised solutions for portal frames with inner prop under different loadings.

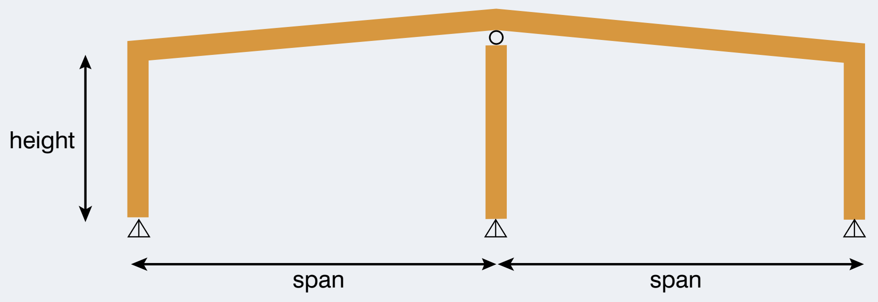


Figure 3.1: Portal frame with inner prop.

3.1 Gravity and Wind Terrain – Category 2

Column Height (m)	Box Beam	Portal Frame Span (m)							
		20		25		30		35	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w)	600 x 45	200 x 45	800 x 45	240 x 45	900 x 45	300 x 45	1000 x 45	300 x 45
	flange size (b_f x t_f)	150 x 90	150 x 63	200 x 90	200 x 90	240 x 45	240 x 63	300 x 90	300 x 90
	Volume of LVL (m^3)	4.31		6.91		9.16		12.21	
6	web size (d x t_w)	600 x 45	200 x 45	800 x 45	240 x 45	1000 x 45	300 x 45	1200 x 45	400 x 45
	flange size (b_f x t_f)	200 x 90	200 x 63	200 x 90	200 x 90	240 x 90	240 x 90	240 x 90	200 x 45
	Volume of LVL (m^3)	5.03		7.19		10.23		12.96	
7	web size (d x t_w)	600 x 45	200 x 45	800 x 45	240 x 45	1000 x 45	300 x 45	1000 x 45	400 x 45
	flange size (b_f x t_f)	200 x 90	200 x 63	240 x 90	240 x 90	240 x 90	240 x 90	400 x 90	400 x 45
	Volume of LVL (m^3)	5.25		7.99		10.44		14.38	

Column Height (m)	Box Beam	Portal Frame Span (m)					
		40		45		50	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w)	1200 x 45	400 x 45	1200 x 45	400 x 45	1200 x 45	400 x 45
	flange size (b_f x t_f)	400 x 90	400 x 63	600 x 90	600 x 63	600 x 126	600 x 63
	Volume of LVL (m^3)	16.99		22.67		29.66	
6	web size (d x t_w)	1200 x 45	400 x 45	1200 x 45	400 x 45	1200 x 45	400 x 45
	flange size (b_f x t_f)	400 x 90	400 x 63	600 x 90	600 x 63	600 x 126	600 x 90
	Volume of LVL (m^3)	17.44		23.22		30.62	
7	web size (d x t_w)	1200 x 45	400 x 45	1200 x 45	400 x 45	1200 x 45	400 x 45
	flange size (b_f x t_f)	400 x 90	400 x 63	600 x 90	600 x 63	600 x 126	600 x 90
	Volume of LVL (m^3)	17.88		23.76		31.29	

Table 3.1: 45 mm web thicknesses.

Column Height (m)	Box Beam	Portal Frame Span (m)							
		20		25		30		35	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	200 x 63 150 x 63	600 x 63 240 x 90	200 x 63 240 x 90	800 x 63 240 x 90	300 x 63 240 x 45	900 x 63 300 x 90	300 x 63 300 x 90
	Volume of LVL (m^3)	5.44		7.64		10.57		14.18	
6	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	200 x 63 150 x 63	800 x 45 150 x 90	240 x 63 150 x 90	900 x 63 200 x 90	300 x 63 200 x 63	1000 x 63 300 x 90	400 x 63 300 x 45
	Volume of LVL (m^3)	5.69		8.42		11.33		15.51	
7	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	200 x 63 150 x 63	800 x 63 200 x 90	240 x 63 200 x 90	800 x 63 300 x 90	300 x 45 300 x 90	900 x 63 400 x 90	300 x 63 400 x 90
	Volume of LVL (m^3)	5.94		9.39		12.11		16.73	

Column Height (m)	Box Beam	Portal Frame Span (m)					
		40		45		50	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	1000 x 63 400 x 90	400 x 63 400 x 45	1000 x 63 600 x 90	400 x 63 600 x 45	1200 x 63 600 x 90	400 x 63 600 x 63
	Volume of LVL (m^3)	18.61		24.41		29.79	
6	web size (d x t_w) flange size (b_f x t_f)	1200 x 63 300 x 90	400 x 63 300 x 63	1000 x 63 600 x 90	400 x 63 600 x 63	1200 x 63 600 x 90	400 x 63 600 x 63
	Volume of LVL (m^3)	19.78		25.20		30.44	
7	web size (d x t_w) flange size (b_f x t_f)	900 x 63 600 x 90	400 x 63 600 x 45	1200 x 63 600 x 90	400 x 63 600 x 63	1200 x 63 600 x 126	400 x 63 600 x 90
	Volume of LVL (m^3)	21.98		28.42		36.39	

Table 3.2: 63 mm web thicknesses.

3.2 Gravity and Wind Terrain – Category 3

Column Height (m)	Box Beam	Portal Frame Span (m)							
		20		25		30		35	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	600 x 45 150 x 90	200 x 45 150 x 45	600 x 45 240 x 90	200 x 45 240 x 90	800 x 45 200 x 90	300 x 45 200 x 90	1000 x 45 240 x 90	300 x 45 240 x 90
	Volume of LVL (m^3)	4.27		6.29		8.07		11.26	
6	web size (d x t_w) flange size (b_f x t_f)	600 x 45 150 x 90	200 x 45 150 x 63	600 x 45 240 x 90	200 x 45 240 x 90	800 x 45 240 x 90	300 x 45 240 x 90	1000 x 45 240 x 90	300 x 45 240 x 90
	Volume of LVL (m^3)	4.51		6.55		8.93		11.59	
7	web size (d x t_w) flange size (b_f x t_f)	600 x 45 150 x 90	200 x 45 150 x 63	600 x 45 240 x 90	200 x 45 240 x 90	800 x 45 240 x 90	300 x 45 240 x 90	1000 x 45 240 x 90	400 x 45 240 x 45
	Volume of LVL (m^3)	4.71		6.80		9.23		11.80	

Column Height (m)	Box Beam	Portal Frame Span (m)					
		40		45		50	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	1200 x 45 240 x 90	400 x 45 240 x 63	1200 x 45 400 x 90	400 x 45 400 x 63	1200 x 45 600 x 90	400 x 45 600 x 63
	Volume of LVL (m^3)	14.22		18.83		24.89	
6	web size (d x t_w) flange size (b_f x t_f)	1200 x 45 240 x 90	400 x 45 240 x 63	1200 x 45 400 x 90	400 x 45 400 x 63	1200 x 45 600 x 90	400 x 45 600 x 63
	Volume of LVL (m^3)	14.59		19.28		25.43	
7	web size (d x t_w) flange size (b_f x t_f)	1200 x 45 240 x 90	400 x 45 240 x 63	1200 x 45 400 x 90	400 x 45 400 x 63	1200 x 45 600 x 90	400 x 45 600 x 63
	Volume of LVL (m^3)	14.95		19.73		25.98	

Table 3.3: 45 mm web thicknesses.

Column Height (m)	Box Beam	Portal Frame Span (m)							
		20		25		30		35	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	400 x 63 240 x 90	200 x 63 240 x 45	600 x 63 200 x 90	200 x 63 200 x 90	800 x 63 200 x 90	300 x 63 200 x 45	800 x 63 300 x 90	300 x 63 300 x 63
	Volume of LVL (m^3)	5.01		7.16		10.03		13.03	
6	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	200 x 63 150 x 45	600 x 63 200 x 90	200 x 63 200 x 90	800 x 63 200 x 90	300 x 63 200 x 45	800 x 63 300 x 90	300 x 63 300 x 63
	Volume of LVL (m^3)	5.65		7.44		10.36		13.42	
7	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	200 x 63 150 x 45	600 x 63 200 x 90	200 x 63 200 x 90	800 x 63 200 x 90	300 x 63 200 x 45	800 x 63 300 x 90	300 x 63 300 x 90
	Volume of LVL (m^3)	5.89		7.73		10.69		13.97	

Column Height (m)	Box Beam	Portal Frame Span (m)					
		40		45		50	
		Frame	Mid-Column	Frame	Mid-Column	Frame	Mid-Column
5	web size (d x t_w) flange size (b_f x t_f)	1000 x 63 300 x 90	400 x 63 300 x 45	1200 x 63 300 x 90	400 x 63 300 x 63	1000 x 63 600 x 90	400 x 63 600 x 45
	Volume of LVL (m^3)	16.91		21.38		26.81	
6	web size (d x t_w) flange size (b_f x t_f)	1000 x 63 300 x 90	400 x 63 300 x 45	1200 x 63 300 x 90	400 x 63 300 x 90	1000 x 63 600 x 90	400 x 63 600 x 45
	Volume of LVL (m^3)	17.35		22.04		27.38	
7	web size (d x t_w) flange size (b_f x t_f)	1000 x 63 300 x 90	400 x 63 300 x 45	1200 x 63 300 x 90	400 x 63 300 x 90	1000 x 63 600 x 90	400 x 63 600 x 45
	Volume of LVL (m^3)	17.79		22.55		27.95	

Table 3.4: 63 mm web thicknesses.

4

Double-Pitch Portal Frame Solutions

The following tables are optimised solutions for double-pitch portal frames under different loadings.

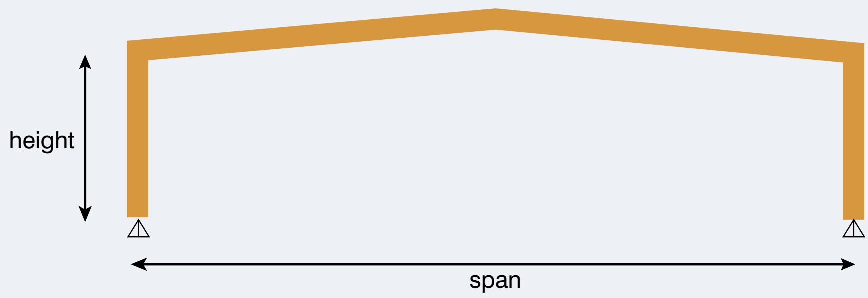


Figure 4.1: Double-pitch portal frame.

4.1 Gravity and Wind Terrain – Category 2

Column Height (m)	Box Beam	Portal Frame Span (m)						
		15	20	25	30	35	40	45
5	web size (d x t_w) flange size (b_f x t_f)	600 x 45 150 x 90	600 x 45 150 x 90	600 x 45 200 x 90	800 x 45 200 x 90	900 x 45 240 x 90	1000 x 45 300 x 90	1200 x 45 300 x 90
	Volume of LVL (m^3)	2.03	2.44	3.16	4.33	5.61	7.22	8.94
6	web size (d x t_w) flange size (b_f x t_f)	600 x 45 150 x 90	600 x 45 200 x 90	600 x 45 240 x 90	800 x 45 200 x 90	900 x 45 240 x 90	1000 x 45 300 x 90	1200 x 45 300 x 90
	Volume of LVL (m^3)	2.19	2.89	3.61	4.55	5.85	7.51	9.26
7	web size (d x t_w) flange size (b_f x t_f)	800 x 45 150 x 90	800 x 45 150 x 90	800 x 45 200 x 90	800 x 45 240 x 90	1000 x 45 240 x 90	1200 x 45 240 x 90	1200 x 45 400 x 90
	Volume of LVL (m^3)	2.88	3.37	4.22	5.08	6.54	8.19	10.65

Table 4.1: 45 mm web thicknesses.

Column Height (m)	Box Beam	Portal Frame Span (m)						
		15	20	25	30	35	40	45
5	web size (d x t_w) flange size (b_f x t_f)	400 x 63 240 x 90	600 x 63 150 x 90	600 x 63 150 x 90	800 x 63 150 x 90	800 x 63 240 x 90	900 x 63 300 x 90	1200 x 63 240 x 90
	Volume of LVL (m^3)	2.35	3.09	3.60	5.13	6.50	8.40	10.73
6	web size (d x t_w) flange size (b_f x t_f)	600 x 63 150 x 90	600 x 63 150 x 90	600 x 63 200 x 90	800 x 63 150 x 90	900 x 63 200 x 90	900 x 63 300 x 90	1200 x 63 240 x 90
	Volume of LVL (m^3)	2.78	3.29	4.14	5.38	7.05	8.73	11.11
7	web size (d x t_w) flange size (b_f x t_f)	800 x 63 150 x 90	800 x 63 150 x 90	800 x 63 150 x 90	800 x 63 200 x 90	800 x 63 300 x 90	1000 x 63 300 x 90	1000 x 63 400 x 90
	Volume of LVL (m^3)	3.71	4.35	5.00	6.03	7.61	9.75	11.72

Table 4.2: 63 mm web thicknesses.

4.2 Gravity and Wind Terrain – Category 3

Column Height (m)	Box Beam	Portal Frame Span (m)						
		15	20	25	30	35	40	45
5	web size ($d \times t_w$) flange size ($b_f \times t_f$)	400 x 45 200 x 90	600 x 45 150 x 90	600 x 45 150 x 90	800 x 45 150 x 90	800 x 45 240 x 90	1000 x 45 240 x 90	1200 x 45 240 x 90
	Volume of LVL (m^3)	1.80	2.44	2.84	3.97	5.20	6.68	8.34
6	web size ($d \times t_w$) flange size ($b_f \times t_f$)	600 x 45 150 x 90	600 x 45 150 x 90	600 x 45 150 x 90	800 x 45 150 x 90	800 x 45 240 x 90	1000 x 45 240 x 90	1200 x 45 240 x 90
	Volume of LVL (m^3)	2.19	2.60	3.00	4.17	5.43	6.95	8.64
7	web size ($d \times t_w$) flange size ($b_f \times t_f$)	600 x 45 240 x 90	800 x 45 150 x 90	800 x 45 150 x 90	800 x 45 150 x 90	800 x 45 240 x 90	1000 x 45 240 x 90	1200 x 45 240 x 90
	Volume of LVL (m^3)	2.82	3.37	3.87	4.37	5.66	7.21	8.95

Table 4.3: 45 mm web thicknesses.

Column Height (m)	Box Beam	Portal Frame Span (m)						
		15	20	25	30	35	40	45
5	web size ($d \times t_w$) flange size ($b_f \times t_f$)	400 x 63 150 x 90	400 x 63 240 x 90	600 x 63 150 x 90	800 x 63 240 x 90	800 x 63 200 x 90	800 x 63 300 x 90	1000 x 63 300 x 90
	Volume of LVL (m^3)	1.94	2.82	3.60	4.77	6.17	7.76	9.93
6	web size ($d \times t_w$) flange size ($b_f \times t_f$)	600 x 63 150 x 90	600 x 63 150 x 90	600 x 63 150 x 90	800 x 63 150 x 90	800 x 63 200 x 90	800 x 63 300 x 90	1000 x 63 300 x 90
	Volume of LVL (m^3)	2.78	3.29	3.81	5.38	6.45	8.07	10.29
7	web size ($d \times t_w$) flange size ($b_f \times t_f$)	600 x 63 200 x 90	600 x 63 240 x 90	800 x 63 150 x 90	800 x 63 150 x 90	800 x 63 200 x 90	900 x 63 240 x 90	1000 x 63 300 x 90
	Volume of LVL (m^3)	3.24	4.05	5.00	5.64	6.72	8.48	10.65

Table 4.4: 63 mm web thicknesses.

Note:

1. The volume of LVL shown is the volume required for constructing one whole double-pitch portal frame, which includes the rafters and two columns, see diagram on the upper corner for clarification.

2. The unit for web size and flange size are millimetres (mm).

5

Truss Solutions

The optimised solutions for each truss span under different loadings are presented in this Section.

5.1 Gravity and Wind Terrain – Category 2

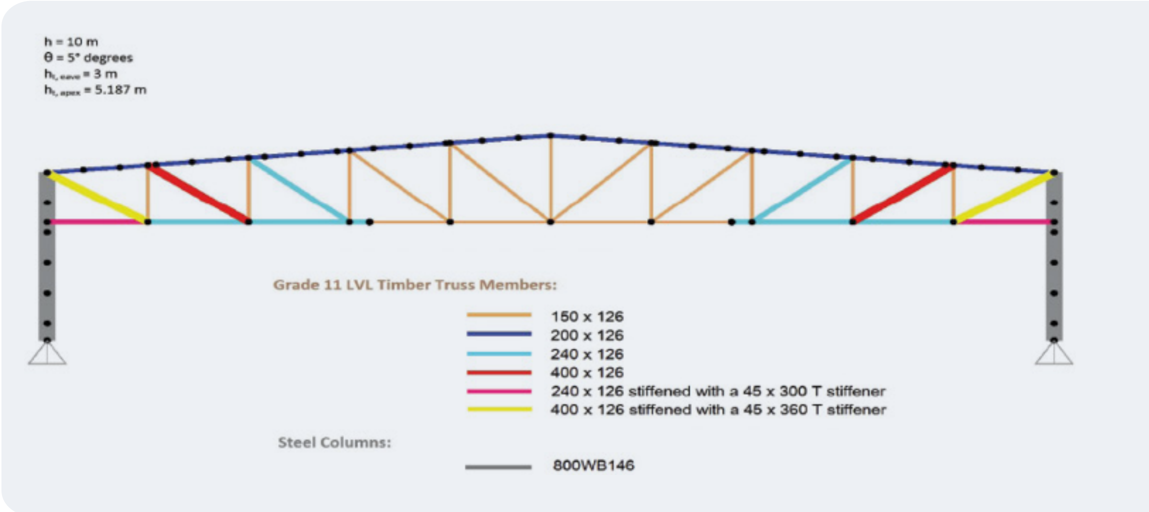


Figure 5.1: 50 m span.

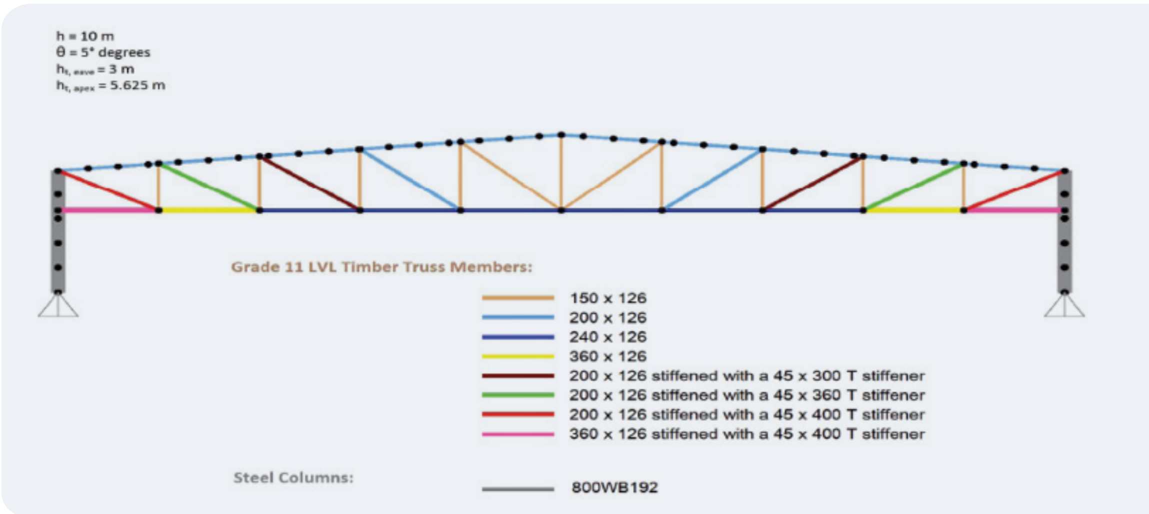


Figure 5.2: 60 m span.

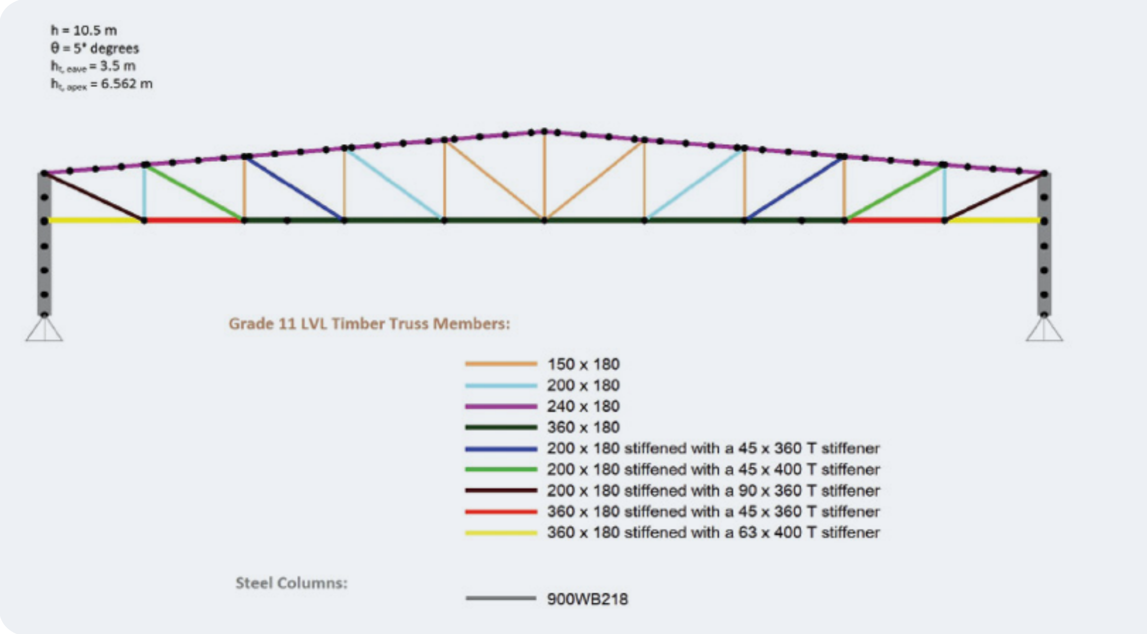


Figure 5.3: 70 m span

5.2 Gravity and Wind Terrain – Category 3

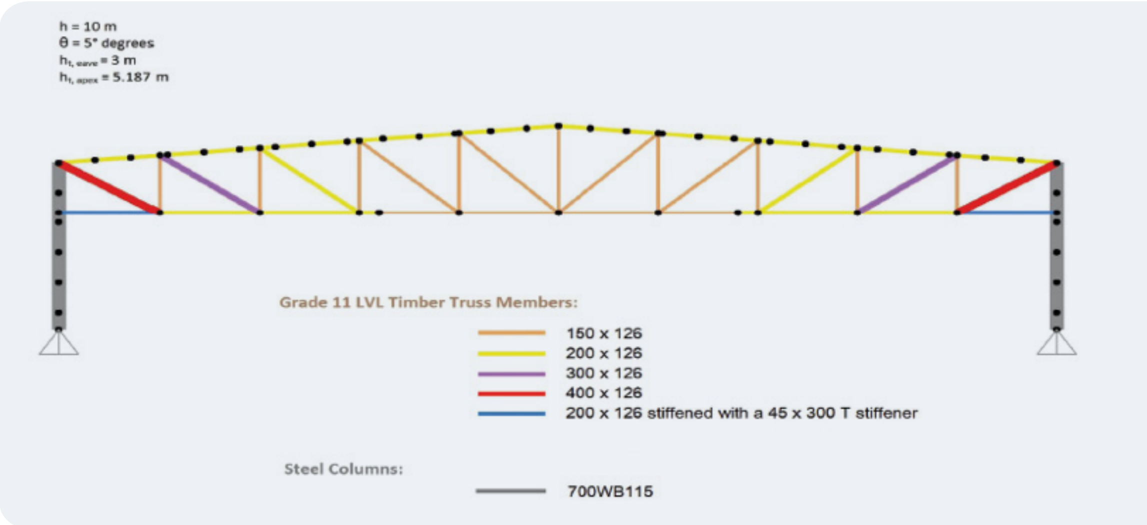


Figure 5.4: 50 m span.

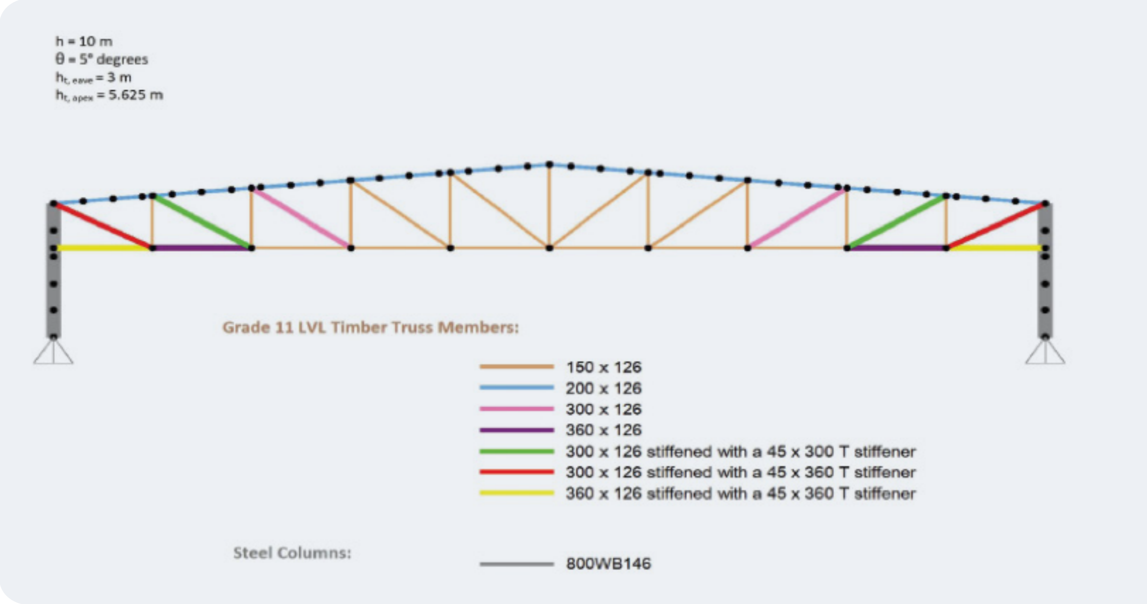


Figure 5.5: 60 m span.

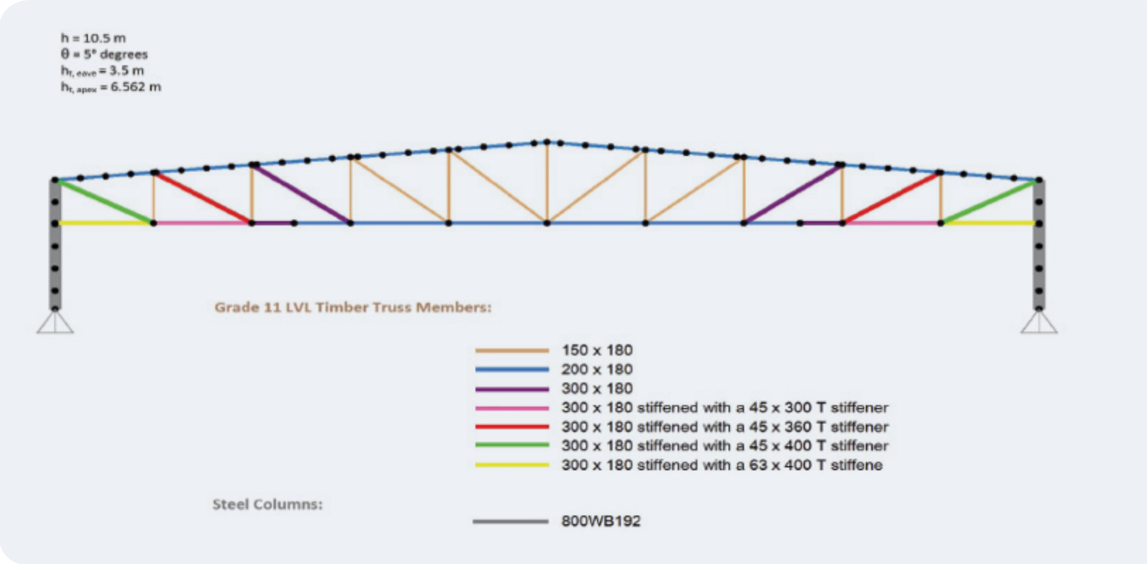


Figure 5.6: 70 m span.

6

Summary of Trends and Findings

6.1 Portal Frames

6.1.1 Double-Pitch Portal Frame

Governing factor

For small spans (15 m, 20 m) of portal frame design, the lateral deflection at the column top governs the design. Final solutions provided for the portal frames have a column top deflection at around 38 mm to 40 mm, where the lateral deflection limit is 40 mm. Cross sections that satisfy the deflection limits exceed the bending and axial load capacity requirements by a large margin, especially for portal frames with the greater clear heights.

For medium spans (25 m, 30 m, and 35 m), the gravity plus wind minimum uplift loading condition is most critical and governs the design, closely followed by the gravity plus live loading condition. Lateral deflection at the top of the column is no longer critical as larger sections are now being used (depth greater than 900 mm), and the box section that is able to resist the required loading is always sufficiently by stiff to resist the lateral deflection.

When the spans are increased to 40 metres and 45 metres, the rafters are now very long and become quite heavy. The mid-span gravity deflection becomes the most critical and governs the design.

Slenderness and load carrying capacity

In 15 and 20 m spans, where the loads are relatively small and the lateral deflection from wind is critical, very slender box sections such as 600 x 63 by 20 x 90, 800 x 63, 20 x 90, 900 x 45 and 20 x 90 are able to provide sufficient strength capacities. But when the spans get larger (30 m, 35 m, 40 m and 45 m), very slender sections must be avoided, since the loads are now getting quite large and very slender sections are not able to provide sufficient load carrying capacities. It is not unusual that some less slender box sections, even with a smaller total cross sectional area, would have a larger load carrying capacity than a cross section that has a larger cross sectional area but more slender.

Effectiveness of increasing dimensions

The following observations can be made. One can conclude that the greater the depth of the box section, the smaller the deflections. Another conclusion possible is that the wider the box section, the greater the load carrying capacity. However, increasing the width of a box section helps relatively little in reducing the deflection. Increasing the depth of a box section also helps to increase the load carrying capacity, but increasing the width is more effective at increasing load carrying capacity as the lateral stability governs the moment resistance in most cases.

Compression in rafters and columns

One can note that for smaller spans (15 m to 35 m), the compression force in the columns is greater than in the rafters. But in larger spans (40 m and 45 m), the compression force in the rafter is greater than in the columns. This is because in smaller spans, the compression force due to the gravity downward/supporting effect in the columns is greater than the mid span pressing effect in the rafter minus the relieving effect caused by the upward pointing 5° degrees roof pitch. But in larger spans, due to the huge mid span downward forces in the rafter, the pressing effect minus the relieving effect caused by the upward pointing 5° degrees roof pitch in the rafter is now greater than the gravity downward/supporting effect in the columns. Therefore, the compression force in the rafter is now greater than the compression force in the columns. Suggestions to reduce the deflection and compression force in the rafter would be to use a steeper roof pitch or increase the pre-camber in the rafter.

Critical loads

Comparing the critical loads that govern the designs between the different Wind Terrain Categories: in Terrain Category 2, wind load governs the design for spans from 15 metres to 25 metres. In Terrain Category 3, wind load governs the design for spans from 15 metres to 20 metres and also in the 7 metres clear height designs.

In all others spans, gravity load governs the design. This indicates that gravity load governs most designs in long span timber portal frame design, especially for spans greater than 20 metres. When designing long span LVL portal frames the gravity loads should be calculated precisely and never be underestimated.

Comparison between 45 mm and 63 mm web thicknesses

Results from the solution tables show that for the same span and height, box sections with 45 mm web thicknesses use less volume of LVL than the 63 mm web thicknesses box sections. This indicates that the 45 mm web thickness box sections are more efficient than the 63 millimetres web thicknesses box sections.

Cross sectional area

This Guide uses web thicknesses of 45 mm or 63 mm and flange thicknesses of 90 mm as the starting thicknesses in design.

The flange thickness is increased if needed. Since the web thicknesses are always thinner than the flange thicknesses, increasing the depth of the box section will result in a smaller increase in cross sectional area than increasing the width of the box section. Therefore, when it is necessary to increase the box section size, try increasing the depth first, before increasing the width. Some 1200 mm depth sections have a smaller total area than a 1000 mm depth section. One should not be surprised when seeing a solution of a larger span or greater height that has a smaller depth. Use the cross sectional area supporting table to compare and use the smallest section that can provide the required ultimate and serviceability capacity.

6.1.2 Internal Propped Portal Frame and Comparison

Mid-span deflection

Since there is an inner column at the centre of the roof, the deflection at the apex joint does not occur. For internal propped portal frames, the vertical deflections are measured at mid span of each of the rafters.

Compression force in the columns

It is found that the compression force in the inner column is greater than the side columns. This is because the inner column supports a greater contributory area than the side columns.

Trends

The trends found from the internal propped portal frame results are similar to the double-pitch portal frame results, except for the following differences.

Comparison of individual spans

Comparing internal propped and double-pitch solutions for the same clear span, internal propped solutions require larger size of sections than double-pitch solutions. This is because the rafter of the internal propped portal frames are straight within each span, and do not have the upward pointing that double-pitch portal frames have at mid span to reduce the force and the deflection.

Comparison of total structure spans

Comparing results of two times the internal propped spans with the double-pitch span, the internal propped solutions use smaller cross sections and smaller volumes of LVL. This indicates that for the same total span, internal propped portal frame is more efficient than double-pitch portal frame. If architecturally allowed, internal propped portal frames are a more efficient option to be used.

6.2 Trusses

6.2.1 Thicknesses

For the 50 metre and 60 metre span trusses, 135 mm thick solid sections of LVL are sufficient to provide the required strengths and deformation performances. However, the 135 mm thick solid sections are not able to provide sufficient strength for the 70 m trusses, even with the use of T stiffeners. As a result, all 70 metres trusses use 180 mm thick solid sections. It is assumed that the LVL solid sections are fabricated from layers of 45 mm thick LVL sections. However, in the design of T stiffeners both 45 mm and 63 mm thicknesses are used.

6.2.2 Effect of T Stiffener on Compression and Tension Capacities

T stiffeners are useful for increasing the compression capacity of the truss members only and do not contribute to resisting tension forces. When both compression and tension forces have exceeded the capacity of a member, the compression force may be overcome by the addition of a T stiffener, but the tension force is still exceeding the capacity. This can only be resolved by increasing the size of the section.

6.2.3 Top Chord

The top chords are governed by tension forces caused by the upward wind forces. It is assumed that the top chords are laterally restrained by the purlins. As a result, the compression capacity will always be sufficient to take the compression loads. But the tension capacity is independent to the lateral restraints. Therefore, the top chords are governed by tension.

6.2.4 Bottom Chord

The outer sections of the bottom chords carry the largest compression forces, especially when the wind is blowing downwards. This is because they transfer all the forces from the truss to the columns. For the same reason, the outer sections of the bottom chord also carry the largest tension forces when the wind is blowing upwards. Since all truss members can have different depths but they must have the same thicknesses for efficiency of load transfer at the connections, the outer sections of the bottom chord are always the critical member for deciding the thickness of the whole truss.

7

Section 7
Example A: Wind Load Calculation

This is a spreadsheet example for Wind Load Calculations. The *n* values are provided in Section 8 - Supporting Materials for Portal Frames.

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure $= n \times C_{fig}$ (kPa)

For this double-pitch portal frame example, the *n* value is taken from Section 8.2. The *n* values provided in this Guide are for wind loads of Terrain Category 1. In order to convert the wind loads to Terrain Category 2 or Terrain Category 3, one will need to multiply the pressures by the ratio table provided in Section 8.

Building Element	Distance from windward edge (m)			C_{pe}	K_a $K_{c,e}$ K_l K_p	$C_{fig,e}$	C_{pi}	$K_{c,i}$	$C_{fig,i}$	$0.5\,\rho_{air}V_{des,\theta}^2C_{dyn} \times 10^{-3}$	P (kPa)	UDL at 8 m spacings
Windward wall	-	to	-	0.7	0.8	0.56	0.7	0.8	0.56	1.449	0.00	0.00
Leeward wall	-	to	-	-0.5	0.8	-0.4	0.7	0.8	0.56	1.449	-1.39	-11.13
Sidewalls	0.000	to	7.984	-0.65	0.8	-0.52	0.7	0.8	0.56	1.449	-1.56	-12.52
Sidewalls	7.984	to	15.968	-0.5	0.8	-0.4	0.7	0.8	0.56	1.449	-1.39	-11.13
Sidewalls	15.968	to	23.952	-0.3	0.8	-0.24	0.7	0.8	0.56	1.449	-1.16	-9.27
Sidewalls	23.952	to	onwards ~	-0.2	0.8	-0.16	0.7	0.8	0.56	1.449	-1.04	-8.35
Roof	0.000	to	3.992	-0.9	0.8	-0.72	0.7	0.8	0.56	1.449	-1.85	-14.84
Roof	3.992	to	7.984	-0.9	0.8	-0.72	0.7	0.8	0.56	1.449	-1.85	-14.84
Roof	7.984	to	15.968	-0.5	0.8	-0.4	0.7	0.8	0.56	1.449	-1.39	-11.13
Roof	15.968	to	23.952	-0.3	0.8	-0.24	0.7	0.8	0.56	1.449	-1.16	-9.27
Roof	23.952	to	onwards ~	-0.2	0.8	-0.16	0.7	0.8	0.56	1.449	-1.04	-8.35

Table 7.1: Wind across (maximum uplift). ***roof angle = 5° h = 7.984 m L = 45 m***

Building Element	Distance from windward edge (m)			C_{pe}	K_a $K_{c,e}$ K_l K_p	$C_{fig,e}$	C_{pi}	$K_{c,i}$	$C_{fig,i}$	$0.5\,\rho_{air}V_{des,\theta}^2C_{dyn} \times 10^{-3}$	P (kPa)	UDL at 8 m spacings
Windward wall	-	to	-	0.7	0.8	0.56	-0.65	0.8	-0.52	1.449	1.56	12.52
Leeward wall	-	to	-	-0.2	0.8	-0.16	-0.65	0.8	-0.52	1.449	0.52	4.17
Sidewalls	0.000	to	7.984	-0.65	0.8	-0.52	-0.65	0.8	-0.52	1.449	0.00	0.00
Sidewalls	7.984	to	15.968	-0.5	0.8	-0.4	-0.65	0.8	-0.52	1.449	0.17	1.39
Sidewalls	15.968	to	23.952	-0.3	0.8	-0.24	-0.65	0.8	-0.52	1.449	0.41	3.25
Sidewalls	23.952	to	onwards ~	-0.2	0.8	-0.16	-0.65	0.8	-0.52	1.449	0.52	4.17
Roof	0.000	to	3.992	-0.4	0.8	-0.32	-0.65	0.8	-0.52	1.449	0.29	2.32
Roof	3.992	to	7.984	-0.4	0.8	-0.32	-0.65	0.8	-0.52	1.449	0.29	2.32
Roof	7.984	to	15.968	0	0.8	0	-0.65	0.8	-0.52	1.449	0.75	6.03
Roof	15.968	to	23.952	0.1	0.8	0.08	-0.65	0.8	-0.52	1.449	0.87	6.96
Roof	23.952	to	onwards ~	0.2	0.8	0.16	-0.65	0.8	-0.52	1.449	0.99	7.88

Table 7.2: Wind across (minimum uplift). ***roof angle = 5° h = 7.984 m L = 45 m***

8 Supporting Materials – Portal Frames

Please see Example A in Section 7 for how to use the n values provided in this section.

8.1 Internal Propped Portal Frame Wind Pressures and Ratios

TC1		10 m			15 m			20 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	5.437	1.056	1.416	5.656	1.059	1.424	5.875	1.062	1.432
	6	6.437	1.070	1.454	6.656	1.073	1.462	6.875	1.076	1.47
	7	7.437	1.084	1.492	7.656	1.087	1.5	7.875	1.090	1.508

TC1		25 m			30 m			35 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	6.094	1.065	1.44	6.312	1.068	1.448	6.531	1.071	1.456
	6	7.094	1.079	1.478	7.312	1.082	1.486	7.531	1.085	1.495
	7	8.094	1.093	1.517	8.312	1.096	1.525	8.531	1.099	1.533

TC1		40 m			45 m			50 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	6.750	1.074	1.464	6.968	1.078	1.475	7.187	1.081	1.484
	6	7.750	1.088	1.503	7.968	1.092	1.514	8.187	1.095	1.522
	7	8.750	1.102	1.542	8.968	1.106	1.553	9.187	1.109	1.561

Table 8.1: Internal propped portal frame – Wind Terrain Category 1 pressures.

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure = $n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC2		10 m			15 m			20 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	5.437	0.918	1.07	5.656	0.922	1.079	5.875	0.926	1.089
	6	6.437	0.936	1.112	6.656	0.940	1.122	6.875	0.944	1.131
	7	7.437	0.954	1.155	7.656	0.958	1.165	7.875	0.962	1.175

TC2		25 m			30 m			35 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	6.094	0.930	1.098	6.312	0.934	1.108	6.531	0.938	1.117
	6	7.094	0.948	1.141	7.312	0.952	1.151	7.531	0.956	1.16
	7	8.094	0.966	1.185	8.312	0.970	1.195	8.531	0.974	1.204

TC2		40 m			45 m			50 m		
		z	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*	*z*	$M_{z,cat}$	*n*
Height (m)	5	6.750	0.941	1.124	6.968	0.945	1.134	7.187	0.949	1.143
	6	7.750	0.959	1.168	7.968	0.963	1.177	8.187	0.967	1.187
	7	8.750	0.977	1.212	8.968	0.981	1.222	9.187	0.985	1.232

Table 8.2: Internal propped portal frame – Wind Terrain Category 2 pressures.

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure = $n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC3		10 m			15 m			20 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.437	0.830	0.875	5.656	0.830	0.875	5.875	0.830	0.875
	6	6.437	0.830	0.875	6.656	0.830	0.875	6.875	0.830	0.875
	7	7.437	0.830	0.875	7.656	0.830	0.875	7.875	0.830	0.875

TC3		25 m			30 m			35 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	6.094	0.830	0.875	6.312	0.830	0.875	6.531	0.830	0.875
	6	7.094	0.830	0.875	7.312	0.830	0.875	7.531	0.830	0.875
	7	8.094	0.830	0.875	8.312	0.830	0.875	8.531	0.830	0.875

TC3		40 m			45 m			50 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	6.750	0.830	0.875	6.968	0.830	0.875	7.187	0.830	0.875
	6	7.750	0.830	0.875	7.968	0.830	0.875	8.187	0.830	0.875
	7	8.750	0.830	0.875	8.968	0.830	0.875	9.187	0.830	0.875

Table 8.3: Internal propped portal frame – Wind Terrain Category 3 pressures

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure $= n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC2/TC1 ratio		10 m		15 m		20 m		25 m		30 m	
		z	ratio	z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	5.437	0.756	5.656	0.758	5.875	0.760	6.094	0.763	6.312	0.765
	6	6.437	0.765	6.656	0.767	6.875	0.769	7.094	0.772	7.312	0.775
	7	7.437	0.774	7.656	0.777	7.875	0.779	8.094	0.781	8.312	0.784

TC2/TC1 ratio		35 m		40 m		45 m		50 m	
		z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	6.531	0.767	6.750	0.768	6.968	0.769	7.187	0.770
	6	7.531	0.776	7.750	0.777	7.968	0.777	8.187	0.780
	7	8.531	0.785	8.750	0.786	8.968	0.787	9.187	0.789

Table 8.4: Internal propped portal frame wind pressures – ratio between Terrain Category 2 & Terrain Category 1.

TC3/TC1 ratio		10 m		15 m		20 m		25 m		30 m	
		z	ratio	z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	5.437	0.618	5.656	0.614	5.875	0.611	6.094	0.608	6.312	0.604
	6	6.437	0.602	6.656	0.598	6.875	0.595	7.094	0.592	7.312	0.589
	7	7.437	0.586	7.656	0.583	7.875	0.580	8.094	0.577	8.312	0.574

TC3/TC1 ratio		35 m		40 m		45 m		50 m	
		z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	6.531	0.601	6.750	0.598	6.968	0.593	7.187	0.590
	6	7.531	0.585	7.750	0.582	7.968	0.578	8.187	0.575
	7	8.531	0.571	8.750	0.567	8.968	0.563	9.187	0.561

Table 8.5: Internal propped portal frame wind pressures – ratio between Terrain Category 3 & Terrain Category 1.

TC1		15 m			20 m			25 m			30 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.328	1.055	1.413	5.437	1.056	1.416	5.547	1.058	1.421	5.656	1.059	1.424
	6	6.328	1.069	1.451	6.437	1.070	1.454	6.547	1.072	1.459	6.656	1.073	1.462
	7	7.328	1.083	1.489	7.437	1.084	1.492	7.547	1.086	1.497	7.656	1.087	1.5

TC1		35 m			40 m			45 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.766	1.061	1.429	5.875	1.062	1.432	5.984	1.064	1.437
	6	6.766	1.075	1.467	6.875	1.076	1.47	6.984	1.078	1.475
	7	7.766	1.089	1.506	7.875	1.090	1.508	7.984	1.092	1.514

Table 8.6: Double-pitch portal frame – Wind Terrain Category 1 pressures.

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure $= n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC2		15 m			20 m			25 m			30 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.328	0.916	1.065	5.437	0.918	1.07	5.547	0.920	1.075	5.656	0.922	1.079
	6	6.328	0.934	1.108	6.437	0.936	1.112	6.547	0.938	1.117	6.656	0.940	1.122
	7	7.328	0.952	1.151	7.437	0.954	1.155	7.547	0.956	1.16	7.656	0.958	1.165

TC2		35 m			40 m			45 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.766	0.924	1.084	5.875	0.926	1.089	5.984	0.928	1.093
	6	6.766	0.942	1.127	6.875	0.944	1.131	6.984	0.946	1.136
	7	7.766	0.960	1.17	7.875	0.962	1.175	7.984	0.964	1.180

Table 8.7: Double-pitch portal frame – Wind Terrain Category 2 pressures.

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure $= n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC3		15 m			20 m			25 m			30 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.328	0.830	0.875	5.437	0.830	0.875	5.547	0.830	0.875	5.656	0.830	0.875
	6	6.328	0.830	0.875	6.437	0.830	0.875	6.547	0.830	0.875	6.656	0.830	0.875
	7	7.328	0.830	0.875	7.437	0.830	0.875	7.547	0.830	0.875	7.656	0.830	0.875

TC3		35 m			40 m			45 m		
		z	$M_{z,cat}$	n	z	$M_{z,cat}$	n	z	$M_{z,cat}$	n
Height (m)	5	5.766	0.830	0.875	5.875	0.830	0.875	5.984	0.830	0.875
	6	6.766	0.830	0.875	6.875	0.830	0.875	6.984	0.830	0.875
	7	7.766	0.830	0.875	7.875	0.830	0.875	7.984	0.830	0.875

Table 8.8: Double-pitch portal frame – Wind Terrain Category 3 pressures.

Note:

$n = 0.5 \times \rho_{air} \times V_{des,\theta}^2 \times C_{dyn} \times 10^3$

Pressure $= n \times C_{fig}$ (kPa)

For Importance Level 3 structures in Region A6 only.

TC2/TC1 ratio		15 m		20 m		25 m		30 m		35 m		40 m		45 m	
		z	ratio	z	ratio	z	ratio	z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	5.328	0.754	5.437	0.756	5.547	0.757	5.656	0.758	5.766	0.759	5.875	0.760	5.984	0.761
	6	6.328	0.764	6.437	0.765	6.547	0.766	6.656	0.767	6.766	0.768	6.875	0.769	6.984	0.770
	7	7.328	0.773	7.437	0.774	7.547	0.775	7.656	0.777	7.766	0.777	7.875	0.779	7.984	0.779

Table 8.9: Double-pitch portal frame wind pressures – ratio between Terrain Category 2 & Terrain Category 1.

TC3/TC1 ratio		15 m		20 m		25 m		30 m		35 m		40 m		45 m	
		z	ratio	z	ratio	z	ratio	z	ratio	z	ratio	z	ratio	z	ratio
Height (m)	5	5.328	0.619	5.437	0.618	5.547	0.616	5.656	0.614	5.766	0.612	5.875	0.611	5.984	0.609
	6	6.328	0.603	6.437	0.602	6.547	0.600	6.656	0.598	6.766	0.596	6.875	0.595	6.984	0.593
	7	7.328	0.588	7.437	0.586	7.547	0.585	7.656	0.583	7.766	0.581	7.875	0.580	7.984	0.578

Table 8.10: Double-pitch portal frame wind pressures – ratio between Terrain Category 3 & Terrain Category 1.

8.3 Flange Width Selection to Minimise Wastage

In order to minimise wastage of LVL in a project, the design flange width (b_f) of the box sections considered in this Guide are ones that can be equally cut from a 1200 mm LVL billet. Therefore, the flange width (b_f) of a box section can be selected from the following two tables:

Starting size (mm)	Cut into No. of pieces	Flange width, b_f (mm)
1200	1	**1200**
1200	2	**600**
1200	3	**400**
1200	4	**300**
1200	5	**240**
1200	6	**200**
1200	8	**150**

Table 8.11: Optimised flange widths.

b_f (mm)	Total width = b_f + 2 x web thicknesses t_w (mm)	
	t_w = 45 mm	t_w = 63 mm
150	240	276
200	290	326
240	330	366
300	390	426
400	490	526
600	690	726

Table 8.12: Total widths.

8.4 Box Section

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
400	45	150	90	0.0630	0.0011	600	45	150	90	0.810	0.00339	800	45	150	90	0.0990	0.0073
400	45	200	90	0.0720	0.0014	600	45	200	90	0.0900	0.00399	800	45	200	90	0.1080	0.0084
400	45	240	90	0.0792	0.0015	600	45	240	90	0.0972	0.00446	800	45	240	90	0.1152	0.0093
400	45	300	90	0.0900	0.0018	600	45	300	90	0.1080	0.00517	800	45	300	90	0.1260	0.0107
400	45	400	90	0.1080	0.0023	600	45	400	90	0.1260	0.00635	800	45	400	90	0.1440	0.0130
400	45	600	90	0.1440	0.0031	600	45	600	90	0.1620	0.00872	800	45	600	90	0.1800	0.0175

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
900	45	150	90	0.1080	0.00991	1000	45	150	90	0.1170	0.0131	1200	45	150	90	0.1350	0.02129
900	45	200	90	0.1170	0.01140	1000	45	200	90	0.1260	0.0150	1200	45	200	90	0.1440	0.02407
900	45	240	90	0.1242	0.01258	1000	45	240	90	0.1332	0.0165	1200	45	240	90	0.1512	0.02630
900	45	300	90	0.1350	0.01436	1000	45	300	90	0.1440	0.0187	1200	45	300	90	0.1620	0.02963
900	45	400	90	0.1530	0.01733	1000	45	400	90	0.1620	0.0225	1200	45	400	90	0.1800	0.03519
900	45	600	90	0.1890	0.02326	1000	45	600	90	0.1980	0.0299	1200	45	600	90	0.2160	0.04630

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
1200	45	150	126	0.1458	0.02391	1200	45	150	180	0.1620	0.02715	1200	45	150	270	0.1890	0.03097
1200	45	200	126	0.1584	0.02756	1200	45	200	180	0.1800	0.03188	1200	45	200	270	0.2160	0.03697
1200	45	240	126	0.1685	0.03048	1200	45	240	180	0.1944	0.03567	1200	45	240	270	0.2376	0.04177
1200	45	300	126	0.1836	0.03486	1200	45	300	180	0.2160	0.04134	1200	45	300	270	0.2700	0.04897
1200	45	400	126	0.2088	0.04216	1200	45	400	180	0.2520	0.05080	1200	45	400	270	0.3240	0.06098
1200	45	600	126	0.2592	0.05676	1200	45	600	180	0.3240	0.06972	1200	45	600	270	0.4320	0.08499

Table 8.13 Cross-Sectional Area and Moment of Inertia- t_w = 45 mm

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
400	63	150	90	0.0774	0.0013	600	63	150	90	0.1620	0.02715	800	63	150	90	0.1890	0.03097
400	63	200	90	0.0864	0.0016	600	63	200	90	0.1800	0.03188	800	63	200	90	0.2160	0.03697
400	63	240	90	0.0936	0.0017	600	63	240	90	0.1944	0.03567	800	63	240	90	0.2376	0.04177
400	63	300	90	0.1044	0.0020	600	63	300	90	0.2160	0.04134	800	63	300	90	0.2700	0.04897
400	63	400	90	0.1224	0.0025	600	63	400	90	0.2520	0.05080	800	63	400	90	0.3240	0.06098
400	63	600	90	0.1584	0.0033	600	63	600	90	0.3240	0.06972	800	63	600	90	0.4320	0.08499

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
900	63	150	90	0.1404	0.01210	1000	63	150	90	0.1530	0.0161	1200	63	150	90	0.1782	0.02648
900	63	200	90	0.1494	0.01358	1000	63	200	90	0.1620	0.0180	1200	63	200	90	0.1872	0.02926
900	63	240	90	0.1566	0.01477	1000	63	240	90	0.1692	0.0195	1200	63	240	90	0.1944	0.03148
900	63	300	90	0.1674	0.01655	1000	63	300	90	0.1800	0.0217	1200	63	300	90	0.2052	0.03481
900	63	400	90	0.1854	0.01951	1000	63	400	90	0.1980	0.0255	1200	63	400	90	0.2232	0.04037
900	63	600	90	0.2214	0.02544	1000	63	600	90	0.2340	0.0329	1200	63	600	90	0.2592	0.05148

d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)	d (mm)	t_w (mm)	b_f (mm)	t_f (mm)	A (m^2)	I (m^4)
1200	63	150	126	0.1890	0.02909	1200	63	150	180	0.2052	0.03234	1200	63	150	270	0.2322	0.03615
1200	63	200	126	0.2016	0.03274	1200	63	200	180	0.2232	0.03707	1200	63	200	270	0.2592	0.04215
1200	63	240	126	0.2117	0.03566	1200	63	240	180	0.2376	0.04085	1200	63	240	270	0.2808	0.04695
1200	63	300	126	0.2268	0.04004	1200	63	300	180	0.2592	0.04653	1200	63	300	270	0.3132	0.05416
1200	63	400	126	0.2520	0.04734	1200	63	400	180	0.2952	0.05599	1200	63	400	270	0.3672	0.06616
1200	63	600	126	0.3024	0.06195	1200	63	600	180	0.3672	0.07491	1200	63	600	270	0.4752	0.09017

Table 8.14: Cross-Sectional Area and Moment of Inertia- t_w = 63 mm.

9

Example B: Truss Design Example

This is an example of the final solution of a 60 metre span truss under Gravity and Wind Terrain Category 1 loadings.

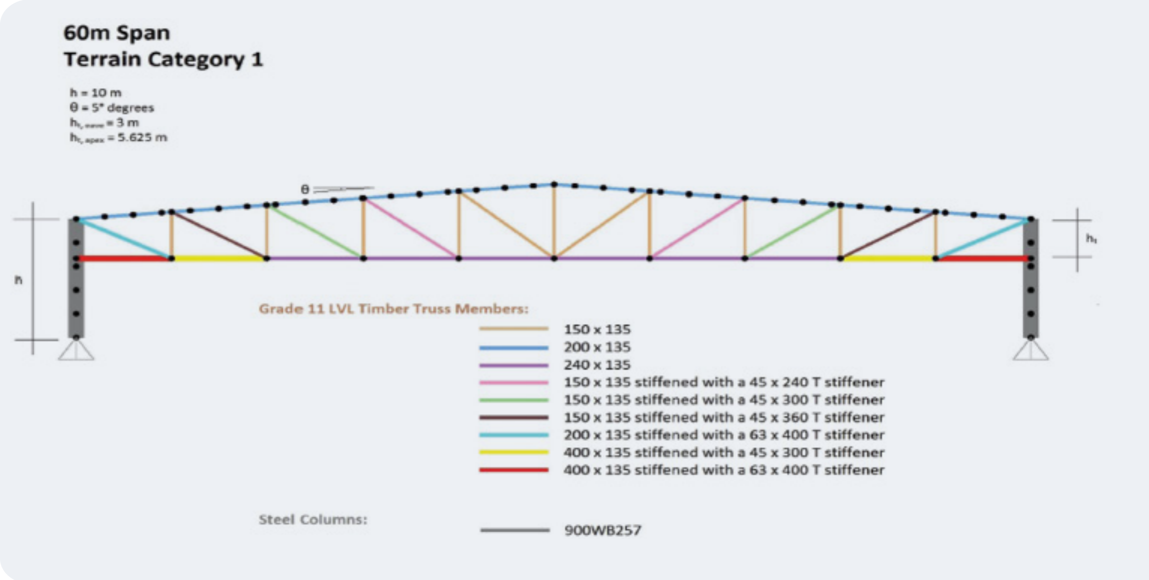


Figure 9.1: 60 m span – Terrain Category 1.

9.1 Deflection Check

Vertical deflections at mid-span									
ΔG	=	98.43	mm	<	$L/360$	=	166.67	mm	OK
ΔQ	=	31.26	mm	<	$L/240$	=	250.00	mm	OK
$\Delta W_{across\ max.}$	=	165.91	mm	<	$L/150$	=	400.00	mm	OK
$\Delta W_{along\ max.}$	=	246.27	mm	<	$L/150$	=	400.00	mm	OK
Horizontal deflection at column tip									
$\Delta W_{along\ max\cdot}$	=	17.14	mm	<	spacing/200	=	40.00	mm	OK

Table 9.1: Deflection check.

9.2 Member Design

9.2.2 Column

Try 900 WB 257

$M^*_{1.2G+Walong\ min}$	=	2091.17	kNm	<	ΦM_{sx}	=	3074.40 kNm	OK
$N^*_{1.2G+Walong\ min}$	=	370.85	kN	<	ΦN_{ncy}	=	1765.85 kN	OK
$(M^*/\Phi M_{nx})^2+(N_c^*/\Phi N_{ncy})$	=	0.67		<	1			OK
$M^*_{0.9G+Walong\ max}$	=	2281.91	kNm	<	ΦM_{sx}	=	3074.40 kNm	OK
$N^*_{t\ 0.9G+Walong\ max}$	=	382.70	kN	<	ΦN_{nt}	=	8240.40 kN	OK
$N^*_{t\ 0.9G+Walong\ max}$	=	382.70	kN	<	0.05 x ΦN_{nt}	=	412.02 kN	No Combined action check required

Table 9.2: Column.

9.2.2 Top Chord

Try 200 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	295.14	kN	<	ΦN_{ncy}	=	612.14	kN	OK
$N^*_{1.2G+1.5Q}$	=	411.48	kN	<	ΦN_{ncy}	=	816.19	kN	OK
$N^*_{1.2G+W\ across\ min.}$	=	520.43	kN	<	ΦN_{ncy}	=	1020.24	kN	OK
$N^*_{1.2G+W\ along\ min.}$	=	567.47	kN	<	ΦN_{ncy}	=	1020.24	kN	OK
Tension									
$N^*_{0.9G+W\ across\ max.}$	=	400.24	kN	<	ΦN_t	=	729.00	kN	OK
$N^*_{0.9G+W\ along\ max.}$	=	682.61	kN	<	ΦN_t	=	729.00	kN	OK

Table 9.3: Top chord.

9.2.3 Bottom Chord

Try 400 x 135 Grade 11 LVL (with T stiffener 63 x 400)

Compression									
$N^*_{1.35G}$	=	558.19	kN	<	ΦN_{ncy}	=	771.28	kN	OK
$N^*_{1.2G+1.5Q}$	=	775.13	kN	<	ΦN_{ncy}	=	1028.37	kN	OK
$N^*_{1.2G+W\ across\ min.}$	=	997.65	kN	<	ΦN_{ncy}	=	1285.46	kN	OK
$N^*_{1.2G+W\ along\ min.}$	=	1053.76	kN	<	ΦN_{ncy}	=	1285.46	kN	OK
Tension									
$N^*_{0.9G+W\ across\ max.}$	=	776.03	kN	<	ΦN_t	=	1239.30	kN	OK
$N^*_{0.9G+W\ along\ max.}$	=	1205.71	kN	<	ΦN_t	=	1239.30	kN	OK

Table 9.4: Outer sections 1 of bottom chord.

Try 400 x 135 Grade 11 LVL (with T stiffener 45 x 300)

Compression									
$N^*_{1.35G}$	=	216.18	kN	<	ΦN_{ncy}	=	248.80	kN	OK
$N^*_{1.2G+1.5Q}$	=	301.48	kN	<	ΦN_{ncy}	=	331.73	kN	OK
$N^*_{1.2G+W\text{ across min.}}$	=	382.63	kN	<	ΦN_{ncy}	=	414.66	kN	OK
$N^*_{1.2G+W\text{ along min.}}$	=	414.65	kN	<	ΦN_{ncy}	=	414.66	kN	OK
Tension									
$N^*_{0.9G+W\text{ across max.}}$	=	283.72	kN	<	ΦN_t	=	1239.30	kN	OK
$N^*_{0.9G+W\text{ along max.}}$	=	497.35	kN	<	ΦN_t	=	1239.30	kN	OK

Table 9.5: Outer sections 2 of bottom chord.

Try 400 x 135 Grade 11 LVL (with T stiffener 45 x 300)

Compression									
$N^*_{1.35G}$	=	28.11	kN	<	ΦN_{ncy}	=	101.33	kN	OK
$N^*_{1.2G+1.5Q}$	=	39.64	kN	<	ΦN_{ncy}	=	135.11	kN	OK
$N^*_{1.2G+W\text{ across min.}}$	=	77.95	kN	<	ΦN_{ncy}	=	168.88	kN	OK
$N^*_{1.2G+W\text{ along min.}}$	=	61.06	kN	<	ΦN_{ncy}	=	168.88	kN	OK
Tension									
$N^*_{0.9G+W\text{ across max.}}$	=	78.74	kN	<	ΦN_t	=	729.00	kN	OK
$N^*_{0.9G+W\text{ along max.}}$	=	102.75	kN	<	ΦN_t	=	729.00	kN	OK

Table 9.6: Outer sections 3 of bottom chord.

Try 240 x 135 Grade 11 LVL

Compression									
$N^*_{0.9G+W\text{ across max.}}$	=	116.02	kN	<	ΦN_{ncy}	=	202.66	kN	OK
$N^*_{0.9G+W\text{ along max.}}$	=	181.84	kN	<	ΦN_{ncy}	=	202.66	kN	OK
Tension									
$N^*_{1.35G}$	=	106.70	kN	<	ΦN_t	=	482.89	kN	OK
$N^*_{1.2G+1.5Q}$	=	149.67	kN	<	ΦN_t	=	643.85	kN	OK
$N^*_{1.2G+W\text{ across min.}}$	=	183.21	kN	<	ΦN_t	=	804.82	kN	OK
$N^*_{1.2G+W\text{ along min.}}$	=	193.90	kN	<	ΦN_t	=	804.82	kN	OK

Table 9.7: Inner sections of bottom chord.

9.2.4 Diagonal Lacings

Try 200 x 135 Grade 11 LVL (with T stiffener 63 x 400)

Compression								
$N^*_{0.9G+W \text{ across max.}}$	=	572.64	kN	<	ΦN_{ncy}	=	865.48	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	792.32	kN	<	ΦN_{ncy}	=	865.48	kN	OK
Tension								
$N^*_{1.35G}$	=	382.90	kN	<	ΦN_t	=	437.40	kN	OK
$N^*_{1.2G+1.5Q}$	=	530.01	kN	<	ΦN_t	=	583.20	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	688.08	kN	<	ΦN_t	=	729.00	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	715.01	kN	<	ΦN_t	=	729.00	kN	OK

Table 9.8: First diagonal lacings.

Try 150 x 135 Grade 11 LVL (with T stiffener 45 x 360)

Compression									
$N^*_{0.9G+W \text{ across max.}}$	=	284.26	kN	<	ΦN_{ncy}	=	488.78	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	458.03	kN	<	ΦN_{ncy}	=	488.78	kN	OK
Tension									
$N^*_{1.35G}$	=	218.56	kN	<	ΦN_t	=	328.05	kN	OK
$N^*_{1.2G+1.5Q}$	=	304.08	kN	<	ΦN_t	=	437.40	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	389.90	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	410.48	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.9: Second diagonal lacings.

Try 150 x 135 Grade 11 LVL (with T stiffener 45 x 300)

Compression									
$N^*_{0.9G+W \text{ across max.}}$	=	131.51	kN	<	ΦN_{ncy}	=	279.39	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	251.69	kN	<	ΦN_{ncy}	=	279.39	kN	OK
Tension									
$N^*_{1.35G}$	=	118.74	kN	<	ΦN_t	=	328.05	kN	OK
$N^*_{1.2G+1.5Q}$	=	166.26	kN	<	ΦN_t	=	437.40	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	207.99	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	224.34	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.10: Third diagonal lacings.

Try 150 x 135 Grade 11 LVL (with T stiffener 45 x 240)

Compression								
$N^*_{0.9G+W \text{ across max.}}$	=	63.30	kN	<	ΦN_{ncy}	=	147.28	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	96.18	kN	<	ΦN_{ncy}	=	147.28	kN	OK
Tension									
$N^*_{1.35G}$	=	46.71	kN	<	ΦN_t	=	328.05	kN	OK
$N^*_{1.2G+1.5Q}$	=	65.62	kN	<	ΦN_t	=	437.40	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	90.67	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	87.64	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.11: Fourth diagonal lacings.

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	15.91	kN	<	ΦN_{ncy}	=	42.65	kN	OK
$N^*_{1.2G+1.5Q}$	=	22.19	kN	<	ΦN_{ncy}	=	56.87	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	40.20	kN	<	ΦN_{ncy}	=	71.09	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	31.88	kN	<	ΦN_{ncy}	=	71.09	kN	OK
Tension									
$N^*_{0.9G+W \text{ across max.}}$	=	45.36	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	42.11	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.12: Fifth diagonal lacings.

9.2.5 Vertical Lacings

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	162.71	kN	<	ΦN_{ncy}	=	212.14	kN	OK
$N^*_{1.2G+1.5Q}$	=	229.03	kN	<	ΦN_{ncy}	=	282.85	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	299.42	kN	<	ΦN_{ncy}	=	353.57	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	311.37	kN	<	ΦN_{ncy}	=	353.57	kN	OK
Tension									
$N^*_{0.9G+W \text{ across max.}}$	=	260.58	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	357.83	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.13: First vertical lacings.

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	105.58	kN	<	ΦN_{ncy}	=	162.38	kN	OK
$N^*_{1.2G+1.5Q}$	=	149.77	kN	<	ΦN_{ncy}	=	216.51	kN	OK
$N^*_{1.2G+W\ across\ min.}$	=	193.77	kN	<	ΦN_{ncy}	=	270.64	kN	OK
$N^*_{1.2G+W\ along\ min.}$	=	203.96	kN	<	ΦN_{ncy}	=	270.64	kN	OK
Tension									
$N^*_{0.9G+W\ across\ max.}$	=	148.53	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{0.9G+W\ along\ max.}$	=	237.15	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.14: Second vertical lacings.

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	62.23	kN	<	ΦN_{ncy}	=	131.77	kN	OK
$N^*_{1.2G+1.5Q}$	=	89.22	kN	<	ΦN_{ncy}	=	175.69	kN	OK
$N^*_{1.2G+W\ across\ min.}$	=	112.22	kN	<	ΦN_{ncy}	=	219.61	kN	OK
$N^*_{1.2G+W\ along\ min.}$	=	121.64	kN	<	ΦN_{ncy}	=	219.61	kN	OK
Tension									
$N^*_{0.9G+W\ across\ max.}$	=	80.13	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{0.9G+W\ along\ max.}$	=	143.33	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.15: Third vertical lacings.

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{1.35G}$	=	24.32	kN	<	ΦN_{ncy}	=	106.62	kN	OK
$N^*_{1.2G+1.5Q}$	=	36.26	kN	<	ΦN_{ncy}	=	142.16	kN	OK
$N^*_{1.2G+W\ across\ min.}$	=	51.46	kN	<	ΦN_{ncy}	=	177.69	kN	OK
$N^*_{1.2G+W\ along\ min.}$	=	49.64	kN	<	ΦN_{ncy}	=	177.69	kN	OK
Tension									
$N^*_{0.9G+W\ across\ max.}$	=	40.23	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{0.9G+W\ along\ max.}$	=	61.27	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.16: Fourth vertical lacings.

Try 150 x 135 Grade 11 LVL

Compression									
$N^*_{0.9G+W \text{ across max.}}$	=	30.20	kN	<	ΦN_{ncy}	=	144.89	kN	OK
$N^*_{0.9G+W \text{ along max.}}$	=	51.69	kN	<	ΦN_{ncy}	=	144.89	kN	OK
Tension									
$N^*_{1.35G}$	=	25.04	kN	<	ΦN_t	=	328.05	kN	OK
$N^*_{1.2G+1.5Q}$	=	32.70	kN	<	ΦN_t	=	437.40	kN	OK
$N^*_{1.2G+W \text{ across min.}}$	=	40.80	kN	<	ΦN_t	=	546.75	kN	OK
$N^*_{1.2G+W \text{ along min.}}$	=	45.28	kN	<	ΦN_t	=	546.75	kN	OK

Table 9.17: Fifth vertical lacings.

References

Australian Standards

AS 1170.0 Structural design actions, General principles. 2002, Standards Australia, Australia

AS 1170.1 Structural design actions, Permanent, imposed and other actions. 2002, Standards Australia, Australia

AS 1170.2 Structural Design Actions, Part 2: Wind Actions. 2002, Standards Australia, Australia

AS 1720.1 Timber structures, Part 1: Design methods. 2010, Standards Australia, Australia

WoodSolutions Technical Design Guides

WoodSolutions Technical Design Guide *#33: Quick-Connect Moment Connection*, Forest and Wood Products Australia, 2016, Melbourne, Australia.

WoodSolutions Technical Report

WoodSolutions Technical Report: *Timber Portal Frames*, Forest and Wood Products Australia, 2016, Melbourne, Australia.

A

Appendix A – Notation

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

A	cross-sectional area
b_f	width of box-beam
C_{dyn}	dynamic response factor, as given in Section 6 of AS 1170.2
C_{fig}	aerodynamic shape factor, as given in Section 5 of AS 1170.2
d	depth of member
E	modulus of elasticity of member
h	column height
$h_{t,eave}$	eave height
$h_{t,apex}$	apex height
I	cross-sectional area
L	span of the structure
M^*	design action moment
M_n	design moment capacity
N^*	design compression force
n	n value used in the calculation of wind pressure
P	design wind pressure in Pascals
t_f	flange thickness
t_w	web thickness
V	nominal shear strength
$V_{des,\theta}$	building orthogonal design wind speeds given in Clause 2.3 AS 1170.2
θ	roof pitch of the building
Φ	capacity factor
ρ_{air}	density of air, which shall be taken as 1.2 kg/m^3

Discover more ways to build your knowledge of wood

If you need technical information or inspiration on designing and building with wood, you'll find WoodSolutions has the answers. From technical design and engineering advice to inspiring projects and CPD linked activities, WoodSolutions has a wide range of resources and professional seminars.

www.woodsolutions.com.au

Your central resource for news about all WoodSolutions activities and access to more than three thousand pages of online information and downloadable publications.

Technical Publications

A suite of informative, technical and training guides and handbooks that support the use of wood in residential and commercial buildings.

WoodSolutions Tutorials

A range of practical and inspirational topics to educate and inform design and construction professionals. These free, CPD related, presentations can be delivered at your workplace at a time that suits you.

Seminars and Events

From one day seminars featuring presentations from leading international and Australian speakers to international tours of landmark wood projects, WoodSolutions offer a range of professional development activities.

What is WoodSolutions?

Developed by the Australian forest and wood products industry for design and building professionals, WoodSolutions is a non-proprietary source of information from industry bodies, manufacturers and suppliers.

Wood
Solutions™
design and build
woodsolutions.com.au

33

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in April 2016

Please check woodsolutions.com.au/publications to ensure you have the latest edition

Quick-Connect Moment Connection

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image: Netball Central, Sydney Olympic Park, Architect: Scott Carver
Photographer: Geoff Ambler

WoodSolutions design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-31-7

Acknowledgments

Authors: Pierre Quenneville, Felix Scheibmair

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: 2011
Revised: April 2016

These materials are published under the brand WoodSolutions by FWPA. This guide has been reviewed and updated for use in Australia by TDA NSW.

IMPORTANT NOTICE

While all care has been taken to ensure the accuracy of the information contained in this publication, Forest and Wood Products Australia Limited (FWPA) and WoodSolutions Australia and all persons associated with them as well as any other contributors make no representations or give any warranty regarding the use, suitability, validity, accuracy, completeness, currency or reliability of the information, including any opinion or advice, contained in this publication. To the maximum extent permitted by law, FWPA disclaims all warranties of any kind, whether express or implied, including but not limited to any warranty that the information is up-to-date, complete, true, legally compliant, accurate, non-misleading or suitable.

To the maximum extent permitted by law, FWPA excludes all liability in contract, tort (including negligence), or otherwise for any injury, loss or damage whatsoever (whether direct, indirect, special or consequential) arising out of or in connection with use or reliance on this publication (and any information, opinions or advice therein) and whether caused by any errors, defects, omissions or misrepresentations in this publication. Individual requirements may vary from those discussed in this publication and you are advised to check with State authorities to ensure building compliance as well as make your own professional assessment of the relevant applicable laws and Standards.

WoodSolutions Australia is a registered business division of Forest and Wood Products Australia Limited.

Contents

1 Introduction

As with most engineering structures, joint design has a major impact on the economy of the building[1]. It can account for between 5 and 50% of the cost of the un-jointed timber members and consume up to 70% of design effort[2].

Portal frame structures are detailed from a combination of hinged and moment-resisting joints. Almost all portal frames require a moment-resisting connection at their knee joint and, depending on the portal frame arrangement, may also include rigid joints at their apex and base connections. The majority of timber portal frame buildings are constructed using a two-pin arrangement, featuring moment-resisting connections at their knee and apex joints. The moment connections are designed to resist axial and shear forces in combination with the moment applied, while the hinged connections are designed to resist axial and shear forces only.

Until recently, the timber industry did not have easy-to-specify connection designs. Previously, the designer was forced to use timber connections such as the nailed gusset connection, which must be specifically designed for each building project undertaken. These existing connections are not only hard to design but are also hard to construct, with large connections requiring multiple nail rings, which may result in more than 1,000 nails per connection side – adding to the time of the erection of the frame on-site and, consequently, costs.

The development of a rod-based connection, introduced in this Guide, overcomes many of the issues experienced with traditional connections. By moving away from the traditional timber connection to a connection in which most of the work is done off-site, the building can be erected much more quickly.

The selection of the type of moment-resisting joint to be used in the building should be made early in the design process. There are a number of methods available for forming moment-resisting joints between portal members, each presenting different benefits and being ideally suited for different types of buildings.

The quick-connect connection must be capable of transferring bending moments, shear forces and axial forces between the portal frame members. The design checks which must be conducted vary, depending on the type of joint. The following sections provide a detailed design approach for a variety of connection types.

1.1 Quick-Connect Moment Connection

The quick-connect joint is a semi-rigid moment-resisting connection that has been developed as an alternative to current nailed solutions for timber portal frame buildings[3]. The quick-connect moment connection has the specific objective of minimising of on-site work required to produce the joint. The joint consists of a rod-based system that can be used as a moment-resisting connection in different connections of a portal frame, such as column to foundation, knee, rafter splice and apex.

The connection bears some conceptual similarity to the partially restrained bolted connections often used in steel construction. It is based on a system of pre-tensioned rods that are placed at the upper and lower extremities of the portal frame members.

When the structure is loaded, a tensile force is applied to one set of rods while the other set remains idle. The compressive force in the connection is transferred in elastic parallel to grain bearing at the sleeve interface. This allows a moment couple to be developed, which facilitates the transfer of load across the joint. The rods are housed in U-shaped timber members, hereafter referred to as timber sleeves, as shown in Figure 1.1. Placing the rods on the extremity of the portal members allows for the full bending moment capacity of the members to be developed at the joint. The timber sleeves are fixed to the portal frame by means of fully threaded continuously threaded timber screws inserted at 60° to the load. The orientation of the screws at this angle results in both reduced demand on the screws and a stiffer connection overall[4].

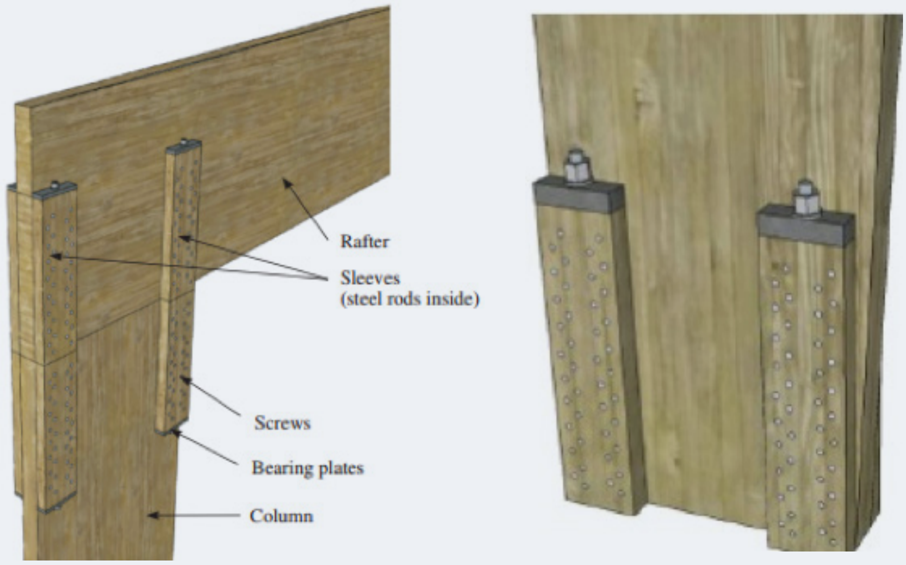


Figure 1.1: Quick-connect moment connection as knee (on right) and column to foundation joints (on left).

The availability of these long, high-strength, fully threaded screws that have been designed specifically for high-load applications in timber allows for the creation of efficient connections between the timber sleeves and the beam or column members.

Unlike traditional timber screws, these screws are hardened after the thread has been rolled. The hardening process increases the screws' bending and torsional capacities. Additionally, a self-drilling tip is cut or moulded into each screw, which allows for application of the screws without predrilling. These attributes make self-drilling timber screws ideal fasteners for high-load timber-to-timber connections.

The use of traditional fasteners such as nails and bolts would result in reduced strength and increased slip at the timber sleeve to beam or column interface, so reducing the efficiency of the connection. The force transfer system for a timber beam to timber column knee joint is shown in Figure 1.2.

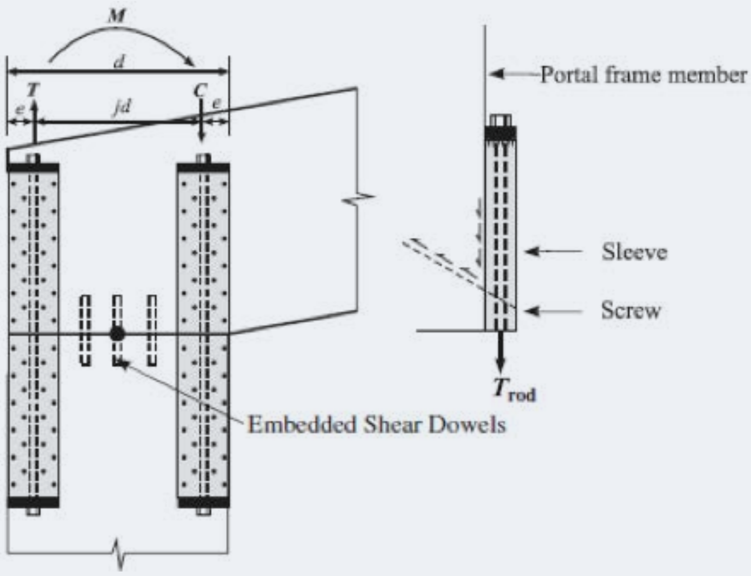


Figure 1.2: Force transfer in the quick-connect connection.

Figure 1.2 shows a portal knee where the rafter is placed atop of the column, as is common in portal frame structures. The rods then extend vertically on either side of the joint and are contained within the timber sleeves. The sleeves are fixed to the portal beam and column with self-drilling timber screws before the connection is taken to site, leaving only the rod to insert and tighten in situ.

Because of the limited components that are required to form the connection, it is easily adapted for use in all parts of the structure. The basic connection configuration always remains the same.

Forces are transferred between the steel rods and the timber sleeves through a steel plate that bears laterally onto the end grain of the timber sleeve. It is also possible to transfer shear forces across the joint in this way; however, for ease of construction, the simplest method for transferring shear is to include dowels between the underside of the portal rafter and the top of the column.

The tolerances for the shear connection must be tighter than those for the long tension rods to ensure that the tension rods are not subjected to shear loading. In practical terms, the connection can be designed and manufactured without special training.

The timber sleeves and bearing block can all be pre-fastened to the portal frame member in a factory environment away from the construction site. This leaves only the rods and steel end bearing plates to be added on site and fastened with nuts. The onsite time required to produce the connection is minimal, saving labour and cranage time during erection and therefore construction costs.

1.2 Application

The quick-connect moment connection can be used for moment-resisting knee, apex and splice joints as well as column-to-foundation connections, offering solutions for a complete portal frame, as shown in Figure 1.3. Also, the quick-connect moment connection can be easily adapted to fix timber members to steel or concrete.

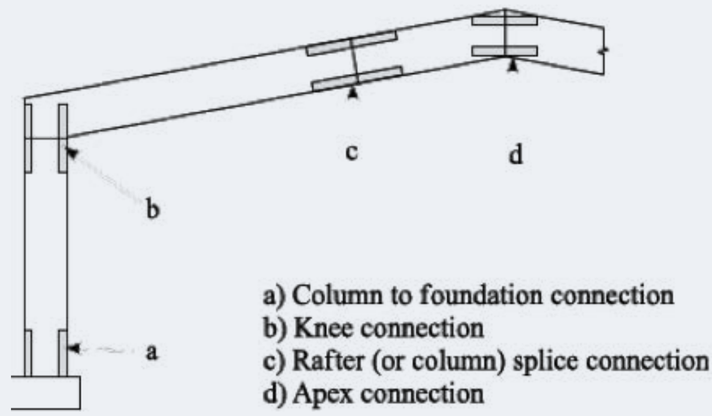


Figure 1.3: Applications of the quick-connect connection in portal frames.

The quick-connect moment connection can be applied to box beam portal frames (see Figure 1.4), with the added advantage that the connecting elements are located within the inside of the box beam, providing a hidden connection and therefore improved aesthetic appeal. The design is exactly the same as with solid sections; the only difference being the location of the steel rods.

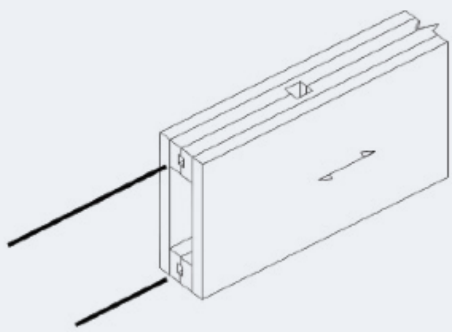


Figure 1.4: Quick-connect moment connection used in a box beam.

With the rods located internally between the two webs of the box beam, access portals must be provided for the rods to be correctly installed. Holes are left in the top and bottom flanges to provide access points so that the bearing plates and nuts can be fitted to the end of the rods. Extra reinforcement may be required for the beam at the locations where the access portals pass through the flange. Such reinforcement is most simply provided by the addition of an extra solid timber member between the webs of the box beam.

2 Design of Quick-Connect Connection

The quick-connect design procedure consists of an iterative process that is used to determine the required characteristics for the sleeves, main tension rods, screws and bearing plates. The connection is designed to resist a given moment; after the components have been specified, joint rotation can be determined.

The design method only relates to the timber-to-timber portal knee connection. The process can easily be adapted for other connection areas such as the column base, beam splice and apex connection.

As moment is transferred, the members and connection components show limited rotation. This rotation must be quantified in the portal frame analysis as it directly affects the bending moments in the structural members and thus the member sizes[5]. Furthermore, joint rotation must be strictly controlled, as small deflections at the joint lead to much larger mid-span deflections. The procedure assumes that a portal frame analysis has been performed in which the stiffness of the connections (k_{rot}), as shown in Figure 2.1, was assumed to be infinity.

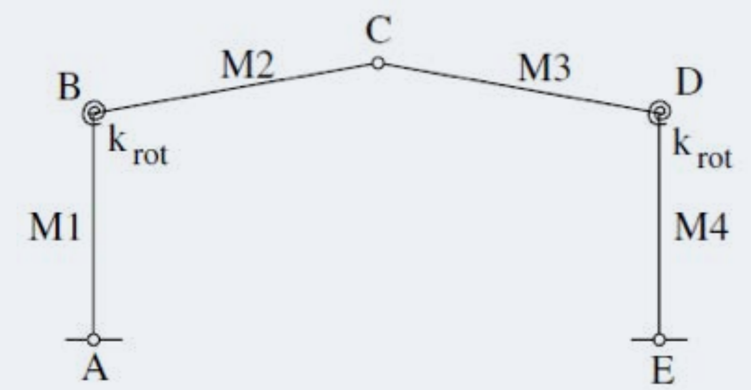


Figure 2.1: Simple structural model showing k_{rot} for a three-pin portal frame.

The designer will have calculated the moment, shear, and axial forces acting on the connection, and will have used these to perform a preliminary member design. Some of the equations used in the design procedure are well accepted design approximations rather than true representations of force and stress distributions.

2.1 Design Portal Frame Members Assuming the Joint is Rigid

For design portal frame members, assuming the joint is rigid, k_{net} is infinite. It is assumed in this case that the designer has already performed a preliminary member design and has established approximate member sizes.

2.2 Design of Connection Detail

2.2.1 Tension and Compression Forces in Joint

The sleeves are sized to allow for correct screw geometry in the connection (i.e. screw spacing and end distances).

Initially, the designer assumes a sleeve size. It is best to assume a larger sleeve size that can be reduced later if the choice is too conservative. For example, two recommended sleeve sizes for a large member with a depth of about 1,000 mm are:

For LVL: 200 mm deep by 75 mm thick (assuming a compressive strength of 45 MPa)

For Glulam: 225 mm deep by 90 mm thick (assuming a compressive strength of 24 MPa)

Design the sleeves to resist the compression force applied to their end grain. To determine the applied compression force, the moment arm, *jd* of the rods needs to be calculated as follows:

$$jd = d - 2e \qquad (2\text{-}1)$$

where

jd moment arm between the steel rods, as shown in Figure 1.2

d depth of member

e distance from the extreme fibre to the centre of the rod, as shown in Figure 1.2.

Equation (2-2) allows preliminary tension and compression forces to be calculated for each set of rods. One set of the rods will act in tension, whilst the other set will not have any load applied. This loading is reversed when the applied force direction is reversed. The sleeves are sized for axial compression loads resulting from the applied moment as well as the tension force in the member. The force per rod is then calculated from:

$$N^*_{e\ sleeve} = N^*_t = \frac{M^*_x}{2\,jd} + \frac{N^*_{t,column}}{4} \qquad (2\text{-}2)$$

2.2.2 Design of Sleeves

The timber sleeve must be capable of resisting the maximum applied compression force (equal to the tension force in the rod). The compression capacity is checked in accordance with AS 1720.1 Timber structures in Part 1: Design methods, taking into account all applicable modification factors such as load duration, service conditions and so on. The sleeve is considered to be fully laterally restrained, therefore AS 1720.1 k_{12} may be ignored.

Figure 2.2: Net area of timber sleeve acting in compression and bearing.

$$N^*_{d,l\ sleeve} \le N^*_{d,l\ sleeve} \qquad (2\text{-}3)$$

$$N^*_{d,l\ sleeve} = \phi\, k_1 k_4 k_6 f'_l A_l \qquad (2\text{-}4)$$

$$V_d = \phi\, k_1 k_4 k_6 f'_S A_S \qquad (2\text{-}5)$$

where

Φ strength reduction factor for sleeve
0.9 for LVL
0.85 for sawn timber and glulam

k_1 duration of load factor for joints for various load combinations
0.57 for [1.35G]
0.77 for [1.2G, 1.5Q]
1.14 for [1.2G, W_u, ψ_cQ]
1.14 for [0.9G, W_u]

f_l characteristic compression strength of timber in sleeve

A_l net cross sectional area of sleeve acting in compression (see Figure 2.2)

$N_{d,\ l\ sleeve}$ nominal compression strength of sleeve.

2.2.3 Design of Screws

Various screw capacities P_{screw} are given in Table 2.1 for LVL and Table 2.2 for GL8. These screw capacities are only valid for those screw types which are listed. Other screw types require verification by testing.

Screw	Article Number	Screw Ø (mm)	Screw Length (mm)	Sleeve thickness (mm)	P_{screw} 5th percent parallel (kN)	P_{screw} 5th percent perp. (kN)	K_{screw} average parallel (N/mm)	K_{screw} average perp. (N/mm)
Würth Assy plus chse hd scr Aw30	00165 36 200	6	200	45	8.5	8.2	8200	7350
				63	9.8	10.1	8340	7200
	00165 58 140	8	140	45	9.7	11.4	11480	7640
				63	13.1	10.9	9700	6930
Würth Assy plus vmp hd scr Aw40	00165 58 200	8	200	45	10.6	11.6	9400	8860
				63	14.7	13.6	9650	7550
	00165 510 200	10	200	45	13.8	13.7	11380	7800
				63	18.2	19.8	12940	12180
Würth AMO III type 2 Aw 30	0234 830 212	7.5	212	45	6.7	6.9	6300	5570
				63	9.2	9.7	6880	6450
	1201020802005	8	200	45	9.5	11.1	8720	9110
				63	14.4	13.8	10750	8060
Spax timber screw	1201021002005	10	200	45	12.9	12.4	12800	9830
				63	16.5	17.1	16490	10180

Table 2.1: Screw properties for quick-connect moment connections in Grade 11 LVL.

Screw	Article Number	Screw Ø (mm)	Screw Length (mm)	Sleeve thickness (mm)	P_{screw} 5th percent parallel (kN)	P_{screw} 5th percent perp. (kN)	K_{screw} average parallel (N/mm)	K_{screw} average perp. (N/mm)
Würth Assy plus chse hd scr Aw30	00165 36 200	6	200	45	4.6	5.1	6680	6320
				90	8.1	9.2	6470	7690
	00165 58 140	8	140	45	7.0	8.7	7320	6300
				90	4.0	4.4	9000	6430
Würth Assy plus vmp hd scr Aw40	00165 58 200	8	200	45	4.1	8.1	5290	7460
				90	11.6	12.2	9420	11680
	00165 510 200	10	200	45	8.6	12.2	8800	7470
				90	13.7	13.6	12430	12520
Würth AMO III type 2 Aw 30	0234 830 212	7.5	212	45	4.9	3.6	5960	5870
				90	8.2	7.2	11390	8500
	1201020802005	8	200	45	9.4	6.9	7830	5880
				90	11.8	11.4	7910	6800
Spax timber screw	1201021002005	10	200	45	12.5	9.5	10470	9730
				90	1	12.1	9530	9830

Table 2.2: Screw properties for quick-connect moment connections in GL8.

To obtain the number of screws required to resist the tension and compression forces at each timber sleeve, the following equation is used:

$$n_s = \frac{N^*_{t,rod}}{\phi k_1 P_{screw}} \qquad (2\text{-}6)$$

where

Φ	strength reduction factor for sleeve 0.9 for LVL 0.85 for sawn timber and glulam
k_1	duration of load factor for joints for various load combinations 0.57 for [1.35G] 0.77 for [1.2G, 1.5Q] 1.14 for [1.2G, W_u, ψ_cQ] 1.14 for [0.9G, W_u]
n_s	number of screws per sleeve required to resist tension forces in the connection
$N^*_{t,rod}$	tension force carried by each steel rod
P_{screw}	5th percentile strength of a single screw as given in the Table 2.1 for LVL and Table 2.2 for GL8.

Screws are arranged in rows along the length of the sleeve.

Screw spacing requirements are shown in Figure 2.3 where:

n_s	number of screws per sleeve
a_4	edge distance
d_a	screw's shank diameter
d_s	depth of sleeve
b	breadth of member
a_3	end distance
D	depth of member (in direction of flexural loading)
t_{sleeve}	thickness of sleeve
t_{LVL}	thickness of LVL.

A minimum spacing of $10d_a$ between screws in a row is recommended. Rows of screws should be staggered with a minimum spacing of $4d_a$ between rows. To ensure screws do not interfere with those on the opposite side of the connection, spacing between rows should be offset on alternate sides of the joint.

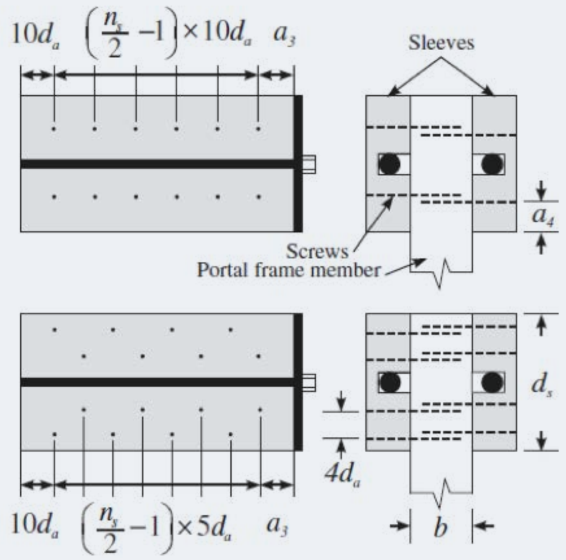


Figure 2.3: Screw spacing requirements for quick-connect connection side view.

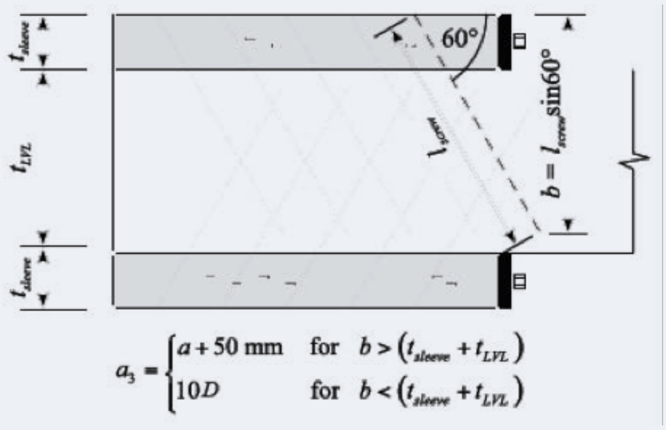


Figure 2.4: End distance.

The characteristic load capacity of the screws varies, depending on whether they are installed parallel or perpendicular to the grain. This is particularly important in knee joints, as different numbers of screws may be required in the portal rafter and column members.

Possible knee joint configurations are shown in Figure 2.5. The extent of the difference in parallel- and perpendicular-to-grain screw strengths also depends on the type of screws used.

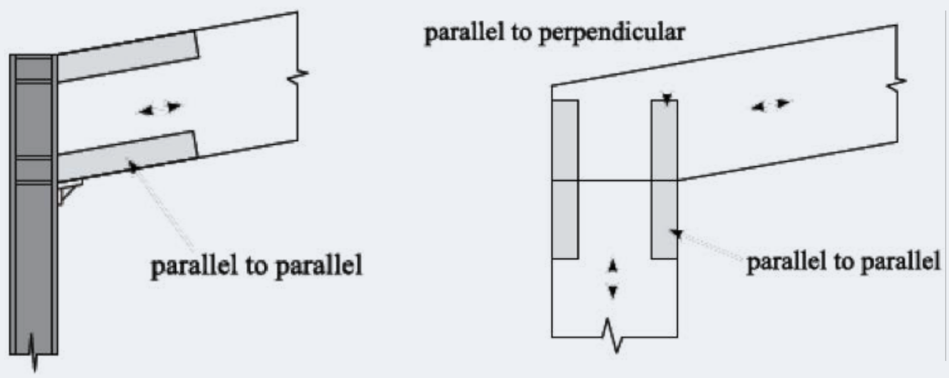


Figure 2.5: Quick-connect moment connection knee joint configurations.

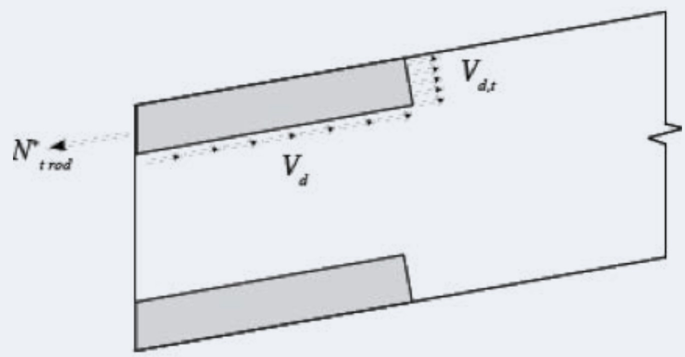


Figure 2.6: Combined shear and tearing in portal frame rafter.

2.2.4 Check Block Tear-Out Resistance in Main Portal Member

The sleeves must be large enough to prevent block tear-out type failure of the main member (timber beam or column). It is assumed the two properties are dependent, as tearing will only occur if shear has occurred, and vice versa. This design check is required only in cases in which the timber sleeves are fixed to the portal member loaded parallel to the grain.

First calculate the shear resistance as:

$$V_d = \Phi k_1 k_4 k_6 f'_s A_s \qquad (2\text{-}7)$$

where

k_1	duration of load factor for joints for various load combinations
	0.57 for [1.35G]
	0.77 for [1.2G, 1.5Q]
	1.14 for [1.2G, W_u, ψQ]
	1.14for [0.9G, W_u]
k_4	Moisture condition only seasoned timber is to be used, therefore 1.0
k_6	Temperature factor modification factor for timber in tropical zones, therefore 1.0
f'_s	shear resistance of portal frame members
A_s	shear area.

The shear area A_s is:

$$A_s = t \times (l_s - a_3) \qquad (2\text{-}8)$$

where

t	thickness of the main member being considered
l_s	length of the timber sleeve attached to portal member loaded parallel to grain
a_3	end distance (see Figure 2.4).

The sleeve must be long enough to accommodate the required number of screws for the connection, adopting the recommended spacings shown in Figure 2.3.

The tearing resistance of the main member is given by:

$$N_{d,t} = \Phi k_1 k_4 k_6 f'_t A_t \qquad (2\text{-}9)$$

where

f'_t	tensile strength of main portal frame member
A_t	tensile area and

$$A_t = t\,(w\text{- edge distance}) \qquad (2\text{-}10)$$

where

w	depth of timber sleeve
t	thickness of main member.

The resistances obtained from Equations (2-6) and (2-8) are then added to give the final tearing and shear resistance of the main member where the timber sleeves are attached:

$$2\,N^*_{t\,rod} \leq N_{d,t} + V_d \qquad (2\text{-}11)$$

2.2.5 Bearing Plate Design

The bearing plates are designed as if each timber sleeve consists of two individual pieces. It may be seen in Figure 2.2 that only the two shaded areas are considered for bearing; the unshaded (white) area is neglected. This approach is conservative.

To specify a bearing plate length, first calculate the required bearing area for ultimate limit state design criteria using:

$$A_{pl} \geq \Phi \frac{N^{*}_{t\,rod}}{k_1\, k_4\, k_6\, f'_l} \qquad (2\text{-}12)$$

where

$N^{*}_{t,\,rod}$	tension force carried by each steel rod
k_1	duration of load factor for joints for various load combinations 0.57 for [1.35G] 0.77 for [1.2G, 1.5Q] 1.14 for [1.2G, W_u, ψQ] 1.14 for [0.9G, W_u]
k_4	Moisture condition only seasoned timber is to be used, therefore 1.0
k_6	Temperature factor modification factor for timber in tropical zones, therefore 1.0
f'_l	characteristic compression stress timber of sleeves
A_{pl}	bearing plate area.

The dimensions of the plate are limited by the thickness and depth of the timber sleeve. To ease calculations, the bearing plate is specified with the same width as the timber sleeve. The total bearing area required can then be divided by this width to obtain the depth of the bearing plate.

Next, the required thickness of the bearing plates must be calculated, considering the serviceability and ultimate limit state load cases. It is likely that the serviceability limit state will govern the design of the plate. It is desirable to limit the deflection of the bearing plate itself to 0.1 mm, as deformations in the knee connections lead to large displacement in the structure as a whole. This limiting value is at the discretion of the designer and experience should determine an appropriate value for the design. To calculate the deflection of the plate, consider it as a beam with two cantilevers mirrored at the rod axis, the deflection equation is therefore:

$$\Delta = \frac{wl^4}{8EI} \qquad (2\text{-}13)$$

where

w	uniformly distributed load acting on bearing plate length of bearing plate either side of the rod (see Figure 2.7)
E	modulus of elasticity of steel bearing plate and I is equal to:

$$I = \frac{bt^3}{12} \qquad (2\text{-}14)$$

where

b	height of the bearing plate (mm), see Figure 2.8
t	thickness of bearing plate (mm), see Figure 2.8.

Check plate satisfies strength requirements using:

$$t_u \geq \sqrt{\frac{3lN^{*}_{t\,rod}}{2\Phi f_y b}} \qquad (2\text{-}15)$$

where

f_y	yield strength of steel
t_u	thickness of bearing plate
b	breadth of bearing plate
$N_{t,rod}$	design tensile force carried by each steel rod.

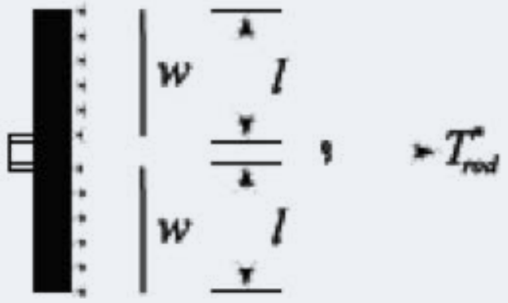


Figure 2.7: Bearing plate double cantilever action on bending.

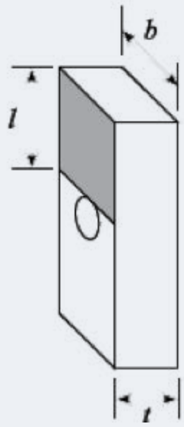


Figure 2.8: Bearing plate dimensions.

2.2.6 Rod Design

The rods in the connection must be checked primarily for tension strength (elongation of the rod is included in later checks). To evaluate the tension strength of each rod it is assumed that the rods do not carry any shear force. This can be ensured by using a shear transfer system in the connection such as embedded steel dowels or a corbel[6].

Determine whether a high or low strength steel rod is applicable to the design.

The tension resistance of the rod must be larger than or equal to the applied tension force:

$$N^*_{t\,rod} \leq N_{tf} \qquad (2\text{-}16)$$

where N_{tf} is the tension resistance of the rod given by:

$$N_{tf} = A_{rod}\, f_{uf} \qquad (2\text{-}17)$$

where f_{uf} is specified minimum tensile strength of steel rod and A_{rod} is the required tensile area of the rod. This can then be used to obtain the diameter of rod required. For adequate performance of the connection, nuts should be tightened during installation to finger tight plus a quarter turn. Further tightening will result in a stiffer connection, but excessive tensioning of the rod will reduce its residual load capacity.

2.3 Evaluation of Rotational Stiffness (k_{rot})

The rotational stiffness (k_{rot}) is the stiffness of the moment connection. It is assumed that deformations remain in the elastic range. The moment is calculated as part of the preliminary design for the portal frame structure; from this, the tension and compression forces in the respective rod groups may be obtained. The rotational stiffness of the joint is:

$$k_{rot} = \frac{M^*}{\theta} \qquad (2\text{-}18)$$

M* is the design bending moment in the joint. The rotation θ is given by the following equation:

$$\theta = \frac{\Delta_{rod,\,ass}}{j\,d} \qquad (2\text{-}19)$$

where the deflection of the rod assembly $\Delta_{rod,ass}$ may be calculated by:

$$\Delta_{rod,ass} = \Delta_{rod} + \Delta_{sleeve} + \Delta_{screws} + \Delta_{plate} \qquad (2\text{-}20)$$

where

$$\Delta_{rod} = \frac{N^*_{t\,rod} L_{rod}}{E_{rod} A_{rod}} \qquad (2\text{-}21)$$

$$\Delta_{sleeve} = \frac{j_{14} N^*_{t\,rod} L_{s,total}}{2E_{sleeve} A_{pl}} \qquad (2\text{-}22)$$

$$\Delta_{screws} = \frac{j_{14}\, N^*_{t\,rod}}{k_{s,perp}\, n_{s,perp} + k_{s,para}\, n_{s,para}} \qquad (2\text{-}23)$$

$$\Delta_{plate} = \text{set bearing plate deformation} \qquad (2\text{-}24)$$

where

$N^*_{t\,rod}$	design tensile force carried by each steel rod
$k_{s,\,para}$	screw connection stiffness factor parallel to the grain
$k_{s,\,perp}$	screw connection stiffness factor perpendicular to the grain
$L_{s,\,total}$	total sleeve length. For a timber to timber connection, add the sleeve length on both sides of the connection.
L_{rod}	length of the rod
$n_{s,\,para}$	number of screws per sleeve parallel to the grain
$N_{,\,perp}$	number of screws per sleeve perpendicular to the grain
A_{pl}	bearing plate area
E_{rod}	modulus of elasticity of rod
E_{sleeve}	modulus of elasticity of timber sleeve
j_{14}	factor for duration of load on shear connections.

Equations (2-20) and (2-21) are both based on the elastic deformation of a member under axial load. This is derived from the standard deflection formula:

$$\Delta = \frac{PL}{EA} \qquad (2\text{-}25)$$

For the deformation of a single rod or timber sleeve, the coefficient P may be substituted for the tension force calculated using Equation (2-2):

$$P = N^*_{t\,rod} \qquad (2\text{-}26)$$

The deflection due to crushing of the timber sleeve is a function of the total sleeve length over both the rafter and the column as given by Equation (2-21). The compression load acting in the timber sleeve decreases from a maximum value (taken as $N^*_{t\,rod}$) at the first screw nearest to the steel bearing block to zero at the interface between the rafter and column.

The screw connection movement is calculated using the k_{screw} coefficient. This coefficient has been determined from connection tests performed on the screw types listed in Table 2.1 for LVL and Table 2.2 for GL8.

2.4 Return to Step 1

Returning to Step 1, it needs to be determined if the bending moment obtained using the given level of rotation is different to the moment obtained from the original assumption of k_{rot} equal to infinite.

Having completed the initial estimates above, determine if there is a difference between the moment calculated, assuming that k_{rot} is equal to infinite, and the moment obtained using the k value using Equation (2-18); done with the help of a structural analysis program. If there is a difference between those moment values, the designer should use an iterative approach to determine a connection size that gives comparative moment values.

2.5 Adjust Member Sizes and Connection Design Accordingly

If an abnormality has been found in the calculation for Step 4, then the designer should use an iterative approach to determine a connection size (Sections 3 and 4).

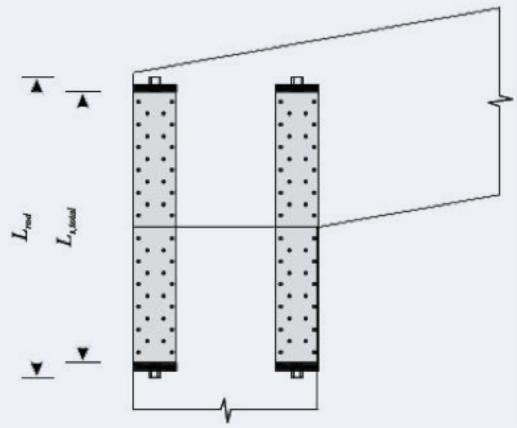


Figure 2.9: Dimensions used in determining the deflection of the rod assembly.

3

Quick-Connect Moment Connect Flow Chart

1. Design portal frame members assuming the joint is rigid, $k_{rot} = \infty$.

2. Design of connection detail:

2.1 Determine compression force in sleeve

$$N^*_{c\ sleeve} = N^*_{t\ rod} = \frac{M^*_x}{2jd} + \frac{N^*_{t\ column}}{4} \quad (2\text{-}2)$$

2.2 Check compression capacity of sleeves

$$N^*_{l\ sleeve} \leq N_{d,l\ sleeve} \quad (2\text{-}3)$$

$$N_{d,\ l\ sleeve} = \Phi k_1 k_4 k_6 f'_l A_l \quad (2\text{-}4)$$

2.3 Design of screws

$$n_s = \frac{N^*_{t\ rod}}{\Phi k_1 P_{screw}} \quad (2\text{-}6)$$

The above calculation should be carried our separately for screws orientated parallel and perpendicular to the grain, due to the different P_{screw} values in each direction.

2.4 Check block tear-out failure in portal member parallel to the sleeves

$$V_d = \Phi k_1 k_4 k_6 f'_s A_s \quad (2\text{-}7)$$

$$N_{d,t} = \Phi k_1 k_4 k_6 f'_t A_t \quad (2\text{-}9)$$

$$2N^*_{t\ rod} \leq N_{d,t} + V_d \quad (2\text{-}11)$$

2.5 Bearing plate design

Limit deflection to 0.1 mm, where the deflection of the bearing plate is given by:

$$\Delta = \frac{wl^4}{8EI} \quad (2\text{-}13)$$

The minimum thickness of the bearing plate to resist bending moments under ultimate limit state loads is given by:

$$t_u \geq \sqrt{\frac{3lN^*_{t\ rod}}{2\Phi f_y b}} \quad (2\text{-}15)$$

2.6 Rod design

$$N^*_{t\ rod} \leq N_{tf} \quad (2\text{-}16)$$

$$N^*_{tf} \leq \Phi A_s f_{uf} \quad (2\text{-}17)$$

2.7 Joint rotation

The rotational stiffness of the joint should be calculated using Equations (2-18) to (2-22). The deflections and internal forces and moments should then be re-evaluated for the new rotational stiffness.

Repeat Steps 1 to 3, adjusting member and connection dimensions sizes as required.

4 Design Example

The following is a design example for a quick-connect moment connection knee joint design.

4.1 Design Actions

For a knee joint, the quick-connect moment connection is arranged such that the steel rods and timber sleeves run perpendicular to the rafter, as shown in Figure 4.1. The design actions are taken at the interface between the rafter and the column. The design actions therefore are taken from the top of the column (see Figure 4.2).

In order to assess which set of design actions are critical, the effect of the load duration factor (k_1) must be considered. To make comparisons between the load combinations, the design actions are each divided by the k_1, corresponding to their respective combination. Design actions adjusted for k_1 are provided in Table 4.2.

4.2 Sleeve Design

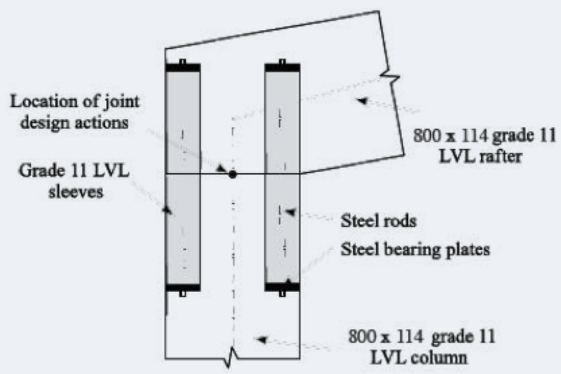
Try 200 x 75 Grade 11 LVL sleeves continuing 30 mm diameter grade 8.8 steel rods (see Figure 4.3).

$j_d = d\text{-}2e = 800 - 2 \times 100 = 600$ mm

Load combination [0.9G, W_u]:

$$N^*_{c\ sleeve} = N^*_{t\ rod} = \frac{M^*_x}{2jd} + \frac{N^*_{t\ column}}{4} \quad (2\text{-}2)$$

$$N^*_{c\ sleeve} = \frac{246.9}{2 \times 600 \times 10^{-3}} + \frac{47.5}{4\cos 10} = 217.8 \text{ kN}$$



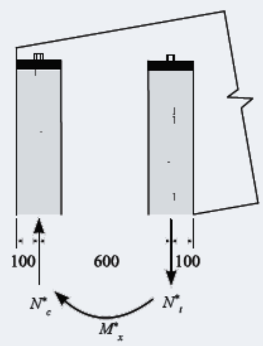


Figure 4.1: Quick-connect knee moment connection.

Figure 4.2: Quick-connect knee connection mechanics.

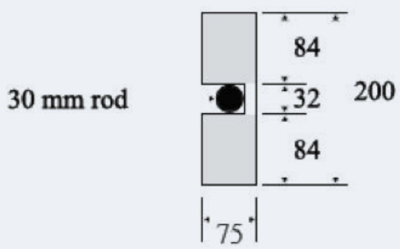


Figure 4.3: Sleeve dimensions.

Note that for this configuration, the tension in the rod is resulting from shear in the rafter. The sleeves are sized for axial compression loads when the rods are in tension. Negative axial forces N^*_{column} are not considered as they do not affect $N^*_{t,rod}$.

The sleeve compression capacity is:

$$N_{d,l\,sleeve} = \phi\, k_1\, k_4\, k_6\, f'_l A_l \qquad (2\text{-}4)$$

The rods are located centrally within the sleeves. Assume for 30 mm rods. A 32 mm hole is cut in the sleeve (see Figure 4.1).

A_l = (200 – 32) x 75 = 12600 mm^2

ϕ = 0.9

k_1 = 1.14 and k_4, k_6 = 1.0

f_c = 45 MPa

$N_{d,l\,sleeve}$ = 0.9 x 1.14 x 1.0 x 1.0 x 45 x 12,600 x 10^{-3}

$N_{d,l\,sleeve}$ = 581.7 kN > $N^*_{c\,sleeve}$ = 217.8 kN, *OK*

Load combination	k_1	***M**** **(kNm)**	***N**** **(kN)**	***V**** **(kN)**
[1.35G]	0.57	-71.1	-24.8	-13.7
[1.2G, 1.5Q]	0.77	-174.5	-60.4	-33.6
[1.2G, W_u]	1.14	-216.1	-77.7	-47.0
[0.9G, W_u]	1.14	246.9	84.8	47.5

Table 4.1: Quick-connect knee joint design actions (point C).

Load combination	k_1	M^*/k_1 **(kNm)**	N^*/k_1 **(kN)**	V^*/k_1 **(kN)**
[1.35G]	0.57	-124.7	-43.5	-24.0
[1.2G, 1.5Q]	0.77	-226.6	-78.4	-43.6
[1.2G, W_u]	1.14	-189.6	-68.2	-41.2
[0.9G, W_u]	1.14	216.6	74.4	41.7

Table 4.2: Quick-connect knee joint design actions adjusted for the load duration factor k_1.

Once the worst case has been determined, all calculations should use the actual load given in Table 4.2, rather than that adjusted for the k_1 factor.

4.3 Screw Design

Calculate the number of screws required:

$$n_s = \frac{N^*_{t\,rod}}{\phi k_1 P_{screw}} \qquad (2\text{-}6)$$

Load combination [0.9G, W_u] gives the greatest tension force in the screws:

$N^*_{t\,rod}$ = 217.8 kN

Try Wuerth 8 mm diameter, 200 mm long screw

The screws act parallel to grain in the column, the number of screws required in the column is then:

$$n_s = \frac{217.8}{0.8 \times 1.14 \times 14.7} = 16.2$$

Adapt 18 Wuerth 8 mm/200 mm long screws to connect each sleeve with portal rafter (9 screws above and below the rod in each sleeve)

In the rafter the screws act perpendicular to the grain:

$$n_s = \frac{217.8}{0.8 \times 1.14 \times 13.6} = 17.6$$

Adapt 18 Wuerth 8 mm/200 mm long screws to connect each sleeve with portal rafter (9 screws above and below the rod in each sleeve)

4.4 Check Block Tear-Out Resistance in Portal Frame Members

The worst tension force occurs under load combination [0.9G, W_u] as a combination of the tension force in the moment couple within the joint, plus the axial tension in the portal column. This block failure cannot occur in the member perpendicular to the sleeves. The force to be resisted is equal to the rod tension force multiplied by the number of rods.

$$N^*_{t\ rod} = 2N^*_{t\ rod} = \text{t x } 217.8 = 435.6 \text{ kN}$$

The shear resistance is:

$$V_d = \phi k_1 k_4 k_6 f'_s A_s$$

$$f'_s = 6 \text{ MPa} \qquad \text{(2-7)}$$

$$A_s = t \times (l_s - a_3)$$

Assume that screws are installed in double rows:

$$b = l_{screw} \sin 60^\circ = 200 \sin 60^\circ = 173.2 \text{ mm}$$

thus: $b = t_{sleeve} + t_{LVL} = 75 + 114 = 189$ mm

and: $a_3 = a + 50$

where: $a = l_{screw} \cos 60^\circ = 200 \cos 60^\circ = 100$ mm

and: $a_3 = 100 + 50 = 150$ mm

therefore, the length of the sleeve in the portal member parallel to the sleeve, $l_{sleeve,\ para*}$

$$l_{sleeve,para} = 10D + \left(\frac{n_s}{2} - 1\right)\frac{10D}{2} + a_3$$

where

D	depth of member
n_s	number of screws per sleeve
a_3	end distance

$$I_{sleeve,para} = 10 \times 8 + (18/2 - 1)\frac{10 \times 8}{2} + 150 = 550 \text{ mm}$$

and the length of the sleeve in the portal member perpendicular to the sleeve, $l_{sleeve,\ para*}$

$$l_{sleeve} = 10D + \left(\frac{n_s}{2} - 1\right)\frac{10D}{2} + a_3$$

where

D	depth of member
n	number of screws per sleeve
a_3	end distance

$$I_{sleeve,prep} = 10 \times 8 + \left(\frac{18}{2} - 1\right)\frac{8 \times 10}{2} + 150$$

$$A_s = 114(550 - 150) = 45600 \text{ mm}^2$$

$$V_d = 0.9 \times 1.14 \times 6 \times 45600 \times 10^{-3} = 280.7 \text{ kN}$$

Tension resistance:

$$N_{d,j} = \phi k_1 k_4 k_6 f'_t A_t \qquad \text{(2-9)}$$

$$f_t = 30 \text{ MPa}$$

$$A_t = t_{LVL}(w_{sleeve} - \text{ edge distance})$$

$$A_t = 114(200 - 22) = 20292 \text{ mm}^2$$

$$N_{d,j} = 0.9 \times 1.14 \times 30 \times 20292 \times 10^{-3} = 624.6 \text{ kN}$$

The combined shear and tearing resistance is then:

$$V_e + N_{d,j} = 280.7 + 624.6 = 905.3 \text{ kN}$$

$$V_e + N_{d,j} = 905.3 > N_t^* = 435.6 \text{ kN}, Ok$$

4.5 Bearing Plate Design

Check bearing area required:

$$A_{pl} \geq \phi \frac{N^*_{t\,rod}}{k_1 k_4 k_6 f_c} = 0.9 \frac{217.8 \times 10^3}{1.14 \times 1 \times 1 \times 45} = 3821 \text{ mm}^2 \qquad \text{(2-12)}$$

Try 150 x 50 steel plates

The plate area is then the same as the sleeve compression area calculated above.

$$A_{pl} = 7500 \text{ mm}^2 > A_{req}, Ok$$

Calculate required bearing plate thickness:

Check deflection:

$$\Delta = \frac{wl^4}{8EI}$$

$$l = \frac{150 - 32}{2} = 59 \text{ mm} \qquad \text{(2-13)}$$

$$w = \frac{N^*_{t\,rod}}{2l} = \frac{217.8}{2 \times 59} = 1.85 \text{ kN/mm}$$

$$E = 200 \times 10^3 \text{ MPa}$$

The required bearing plate thickness is then:

$$I = \frac{ft^3}{12}$$

f=50 mm

Assuming a deflection limit of 0.1 mm, the required second moment of inertia is calculated and then used to determine the required thickness of the bearing plate.

$$0.1 = \frac{1.85 \times 10^3 \times 59^4}{8 \times 200 \times 10^3 \times I}$$

$$I_{req'd} = \frac{1.85 \times 10^3}{8 \times 200 \times 10^3 \times 0.1} = 140107 \text{ mm}^4$$

The required bearing plate thickness is then:

$$t_{req'd} = \sqrt[3]{\frac{12 \times 140107}{50}}$$

$$t_{req'd} = 32.3 \text{ mm}$$

Try 40 mm thick bearing plates

$$I = \frac{50 \times 40^3}{12} = 266667 \text{ mm}^4$$

$$\Delta = \frac{1.85 \times 10^3 \times 59^4}{8 \times 200 \times 10^3 \times 266667} = 0.05 \text{ mm}, Ok$$

Check 40 mm bearing plates in bending:

Try 300 MPa bearing plates

$$t_a \geq \sqrt{\frac{3l \times N^*_{t\,rod}}{2\phi f_y b}} = \sqrt{\frac{3 \times 59 \times 217.8 \times 10^3}{2 \times 0.9 \times 300 \times 50}} = 37.8 \text{ mm}, \ OK \qquad \text{(2-15)}$$

Adopt 150 x 50 x 40 mm steel bearing plates of minimum 300 MPa yield stress

4.6 Rod Design

Try 30 mm diameter, grade 8.8 steel rods

Tensile failure of the rod would occur at the threaded ends; therefore, use the tensile stress area for strength calculations.

$f_{uf} = 830$ MPa

$A_S = 561 \text{ mm}^2$

$$N^*_{t\,rod} \leq N_{tf} \tag{2-16}$$

$$N_{tf} = \phi f_{uf} A_s = 0.8 \times 830 \times 561 \times 10^{-3}$$

$$N_{tf} = 372.5 \text{ kN} > N^*_{t\,rod} = 217.8 \text{ kN}, \ OK \tag{2-17}$$

Adapt M30 grade 8.8 steel rods

4.7 Joint Rotation

Calculate the deformation in the rod assembly:

$$\Delta_{rod\,ass} = \Delta_{rod} + \Delta_{sleeve} + \Delta_{srews} + \Delta_{plate}$$

$$\Delta_{plate} = set\ bearing\ plate\ deformation$$

Elongation of the steel rods:

$$\Delta_{rod} = \frac{N^*_{t\,rod} L_{rod}}{E_{rod} A_{rod}} \tag{2-21}$$

The length of the rod is taken as the length of both sleeves plus the thickness of the bearing plates at each end:

$L_{rod} = 550 + 550 + 2 \times 50 = 1200$ mm

To calculate elongation, use the cross-sectional area of the rod in the non-threaded section:

$$A_{rod} = A_s = 561 \text{ mm}^2$$
$$E_{rod} = 200000 \text{ MPa}$$

$$\Delta_{rod} = \frac{217.8 \times 10^3 \times 1200}{200 \times 10^3 \times 561} = 2.33 \text{ mm}$$

Crushing of the LVL sleeves under bearing plates:

$$\Delta_{sleeve} = \frac{j_{14} N^*_{t\,rod} L_{s,total}}{2 E_{sleeve} A_{pl}} \tag{2-22}$$

$L_{s,total}$ = 550 + 550 = 1100 mm

E_{sleeve} = 11000 MPa for Grade 11 LVL

A_{pl} = 7500 mm^2 (calculated above)

j_{14} = 2.0 (wind loading)

$$\Delta_{sleeve} = \frac{2 \times 217.8 \times 10^3 \times 1100}{2 \times 11000 \times 7500} = 2.9 \text{ mm}$$

Screw slip:

$$\Delta_{screws} = \frac{j_{14} N^*_{trod}}{k_{s,perp} n_{s,perp} + k_{s,para} n_{s,para}} \qquad (2\text{-}23)$$

$$k_{s,perp} = 7550\,\mathrm{N/mm}$$

$$n_{s,perp} = 18$$

$$k_{s,para} = 9650\,\mathrm{N/mm}$$

$$n_{s,para} = 18$$

$$k_{37} = 2(Wind\ Loading)$$

$$\Delta_{screws} = \frac{2 \times 217.8 \times 10^3}{18 \times 7550 + 18 \times 9650} = 1.41\,\mathrm{mm}$$

Bearing plate deflection:

$$\Delta_{plate} = 0.05$$

Total deflection of rod assembly:

$$\Delta_{rod\ ass} = 2.33 + 2.9 + 1.41 + 0.05 = 6.69\,\mathrm{mm}$$

Calculate joint rotation:

$$\theta = \frac{\Delta_{rod\ ass}}{jd}$$

$$\theta = \frac{6.69}{600} = 11.2 \times 10^{-3}\,\mathrm{radians} \qquad (2\text{-}19)$$

Calculate rotational stiffness:

$$k_{rot} = \frac{M^*}{\theta}$$

$$k_{rot} = \frac{246.9 \times 10^3}{11.2 \times 10^{-3}} = 22 \times 10^6\,\mathrm{Nm/radians} \qquad (2\text{-}18)$$

4.8 Apply Rotational Stiffness and Re-Check Member Sizes and Connection Strength

Applying the k_{rot} value calculated above to the initial structural model, the design bending moment at the knee joint was increased by 3%. The axial tension force in the column was increased by 1%. Strength calculations of the portal frame example calculation found the rafter had 12% excess capacity in combined bending and tension, and 12% excess in combined bending and compression. The increase in bending moment due to the rotation of the joint therefore does not require increased member sizes.

Re-check connection with the increased bending moment: load combination [0.9G, W_u]

$$M^*_s = 254.3\,\mathrm{kNm}$$

$$N^*_t = 48\,\mathrm{kN}$$

$$N^*_{c\ sleeve} = N^*_{t\ rod} = \frac{254.3}{2 \times 600 \times 10^{-3}} + \frac{48}{4 \cos 10^0} = 224.1\,\mathrm{kN}$$

Check compression in sleeve (200 x 75 Grade 11 LVL):

$$N^*_{c\ sleeve} = 224.1\,\mathrm{kN}$$

$$N^*_{d,\ t\ sleeve} = 510.5\,\mathrm{kN} > 224.1\,\mathrm{kN},\ OK$$

Adapt 200 x 75 Grade 11 LVL sleeves.

Check tension in screws:

$N^*_{c\ sleeve} = 224.1\ \text{kN}$

$$n_{s\ ,\ para} = \frac{224.1}{0.8 \times 1.14 \times 14.7} = 16.7$$

Adapt 18/ 8 mm Wuerth screws between sleeve and column (9 above and below the rod in each sleeve).

$$n_{s\ ,\ perp} = \frac{224.1}{0.8 \times 1.14 \times 13.6} = 18.1$$

Adapt 20/ 8 mm Wuerth screws between sleeve and column (10 above and below the rod in each sleeve)

Check block tear out resistance in portal frame members:

$N^*_t = 2N^*_{t\ rod} = 2 \times 224.1 = 448.2\ \text{kN}$

As the length of the sleeves in the column remains the same:

$V_d + N_{d,t} = 280.7 + 624.6 = 905.3\ \text{kN}$

$V_d + N_{d,t} = 905.3 > N^*_t = 448.2\ \text{kN},\ OK$

Adopt 590 mm sleeve length for rafter and 550 mm sleeve length for column

Check bearing plates in deflection:

$w = 224.1/(2 \times 59) = 1.90\ \text{kN / mm}$

$$l_{req'd} = \frac{1.9 \times 10^3 \times 59^4}{8 \times 200 \times 10^3 \times 0.1} = 143894\ \text{mm}^4$$

$l_{req'd} < 266666\ \text{mm}^4,\ OK$

Check bearing plates in bending (40 mm, 300 MPa plates):

$$t_u \geq \sqrt{\frac{3 \times 59 \times 224.1 \times 10^3}{2 \times 0.9 \times 300 \times 50}} = 38.3\ \text{mm},\ OK$$

Adopt 40 mm thick steel bearing plates, minimum yield stress = 300 MPa

Check rods in tension:

$T^*_{rod} = 224.1\ \text{kN}$

$N_{tf} = 372.5\ \text{kN} > 224.1\ \text{kN},\ OK$

Adopt 30 mm diameter grade 8.8 steel rods, minimum yield stress = 830 MPa.

References

1. Milner, H.R. and D. Crozier, *Structural Design of Timber Portal Frame Buildings.* 2000: Engineers Australia.

2. Batchelar, M. and K. Mcintosh. *Structural joints in glulam.* in *Proceedings of the 5th World conference in Timber Engineering, Montreux*. 1998.

3. Scheibmair, F. and P. Quenneville, *Moment Connection for Quick Assembly of Timber Portal Frame Buildings: Theory and Validation.* Journal of Structural Engineering, 2014. 140(1): p. 04013022.

4. Blass, H.J. and I. Bejtka, *Screws with continuous threads in timber connections*. Joints in Timber Structures: Stuttgart, Germany, 12-14 September 2001, 2001: p. 193.

5. Dhillon, B. and J. O'Malley *Interactive Design of Semirigid Steel Frames*. Journal of Structural Engineering, 1999. 125(5): p. 556-564.

Australian Standards

AS 1720.1*, Timber structures*, in Part 1: *Design methods*. 2010, Standards Australia: Australia.

Appendix A - Notation

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

A_l	net cross-sectional area of sleeve acting in compression
A_{pl}	bearing plate area
A_{rod}	area of steel rod
A_s	shear plane area of fracture plane in timber member
A_t	tensile plane area of fracture plane in timber member
a_3	end distance, as shown in Figure 2 3
a_4	edge distance, as shown in Figure 2 3
B	breadth of member as shown in Figure 2 3
b_p	breadth of bearing plate as shown in Figure 2 8
D	depth of member (in direction of flexural loading) as shown in Figure 1 2
d_a	diameter of screw
d_p	depth of bearing plate as shown in Figure 2 8
d_s	depth of sleeve as shown in Figure 2 3
E_{sleeve}	modulus of elasticity of timber sleeve
E_{timber}	modulus of elasticity of timber member
E_{rod}	modulus of elasticity of rod
E	modulus of elasticity of member
E	distance from the extreme fibre to the centre of the rod as shown in Figure 1 2
f'_c	characteristic strength in compression parallel to the grain of timber
f'_l	characteristic compression stress timber of sleeves
f'_s	characteristic shear strength of timber
f'_t	characteristic strength in tension parallel to the grain of timber
f_{uf}	specified minimum tensile strength of steel rod
f_y	yield strength of steel
I	moment of inertia
j_d	moment arm between the steel rods, as shown in Figure 1 2
k_{rot}	rotational stiffness of the joint as shown in Figure 2 1
k_s	screw connection stiffness factor
k_1	duration of load (timber)
k_4	moisture condition (timber)
k_6	temperature (timber)
k_{12}	stability factor (timber)
$k_{s,\ perp}$	screw connection stiffness factor perpendicular to the grain
$k_{s,\ para}$	screw connection stiffness factor parallel to the grain
$L_{s,\ total}$	total sleeve length. For a timber to timber connection, add the sleeve length on both sides of the connection

$l_{sleeve, para}$	length of the sleeve in the portal member parallel to the sleeve
$l_{sleeve, perp}$	length of the sleeve in the portal member perpendicular to the sleeve
L	outstanding length of bearing plate either side of the rod as shown in Figure 2.8
L_{rod}	length of the rod
l_s	length of the sleeve attached to portal member loaded parallel to grain
j_{14}	factor for duration of load on shear connections in compliance with AS 1720.1 [1]
M^*_x	design moment
$N^*_{t\ column}$	design tensile force at the members interface
$N_{d,\ l\ sleeve}$	nominal compression strength of sleeve
$N_{L\ sleeve}$	compressive force in the connection as shown in Figure 1 2
$N^*_{c\ sleeve}$	design compression force in one sleeve
$N^*_{t\ ,rod}$	tension force carried by each steel rod"
$N_{t\ rod}$	design tensile force carried by each steel rod
$N_{d,t}$	tearing resistance of the main member
$n_{tension\ rods}$	number of tension rods in connection
N_{tf}	nominal tensile strength of the steel rod
n_s	number of screws per sleeve
$n_{s,\ para}$	number of screws per sleeve parallel to the grain
$n_{s,\ perp}$	number of screws per sleeve perpendicular to the grain
P_{screw}	characteristic resistance of a single screw in single shear
t_{sleeve}	thickness of sleeve
t_{LVL}	thickness of LVL
t_u	thickness of bearing plate
T	nominal tensile strength of portal frame member at fracture plane
T^*	tension force in the connection as shown in Figure 1 2
T	thickness of the main member being considered
t_u	thickness of the main member being considered
V_d	nominal shear strength of portal frame member at fracture plane
w	uniformly distributed load acting on bearing plate = Trod=2l
w	depth of timber sleeve
Δ_{plate}	elastic deflection of plate
Δ	elastic deflection of member
Δ_{rod}	elastic elongation of rod
Δ_{screws}	elastic deflection of screws
Δ_{sleeve}	elastic deformation of sleeve
$\Delta_{rod,ass}$	the deflection of the rod assembly
θ	rotation of the joint
ϕ	capacity factor

34

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Oct 2016
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Timber Rivet Connection

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image cortesy of Specialized Timber Fasteners Ltd, Canada

WoodSolutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-32-4

Acknowledgments

Authors: Pouyan Zarnani, Pierre Quenneville

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: 2011
Revised: October 2016

These materials are published under the brand WoodSolutions by FWPA.
This guide has been reviewed and updated for use in Australia by TDA NSW.

IMPORTANT NOTICE

WoodSolutions Australia is a registered business division of Forest and Wood Products Australia Limited.

Contents

1 Introduction

1.1 Summary

The aim of this Design Guide is to provide an aid for engineers for designing timber rivet connections in structural seasoned wood products including seasoned sawn timber, glulam and laminated veneer lumber (LVL). This Guide includes the design checks for both wood and fastener load-carrying capacities under different loading directions; parallel and perpendicular to grain. By using the Design Guide, an efficient connection design can be made by decreasing the difference between the capacity of the wood and the rivets. It also allows the practitioners to predict the potential brittle (wood block tear-out/splitting), ductile (rivet yielding) and mixed failure modes of the connection.

Section 1 provides an introduction to timber rivets, along with the advantages and applications of timber rivet connections. It also includes characteristic stress tables for LVL, glulam and sawn timber. Flowcharts illustrating the key design steps in the design process are provided in Section 2. Section 3 covers the basic details for timber rivet connections, while Section 4 provides the design rules for rivet connections including rivet capacity, wood block tear-out and splitting capacity. To illustrate the design process, a series of examples are presented in Section 5. Finally, within the Appendices, reference capacity design tables and cost-efficiency analysis for rivets are provided to speed up the computation process.

1.2 Timber Rivets

Connections such as hinged and moment resisting connections are often the most critical parts of any type of structure. Evaluation of timber buildings damaged after extreme wind and earthquake events have shown that weak connections are one of the major causes of problem[1]. As demonstrated over the decades, small-diameter fasteners have shown a significant advantage over large-diameter fasteners such as bolts, which cause large localized stresses and force brittle ruptures in timber. Of this family of fasteners, the timber rivet is a well- established example in timber connection technology[2] . The development of a wood connection providing satisfactory load transfer, joint stiffness, easy manufacture, and good appearance led to the invention of the timber rivets[3].

Timber rivets are hardened steel nails with a rectangular cross-section used in making connections with high load transfer capacity and high stiffness, as shown in Figure 1.1. Rivets are available in three standard lengths; 40, 65, and 90 mm (Figure 1.2). They are always used with a steel plate and the load transfer depends as much on the steel plate capacity as on the rivet connection capacity.

Figure 1.1: Timber rivet connections. *Source: Buchanan, A., Timber design guide New Zealand Timber Industry Federation Inc. Wellington, New Zealand, 2007.*

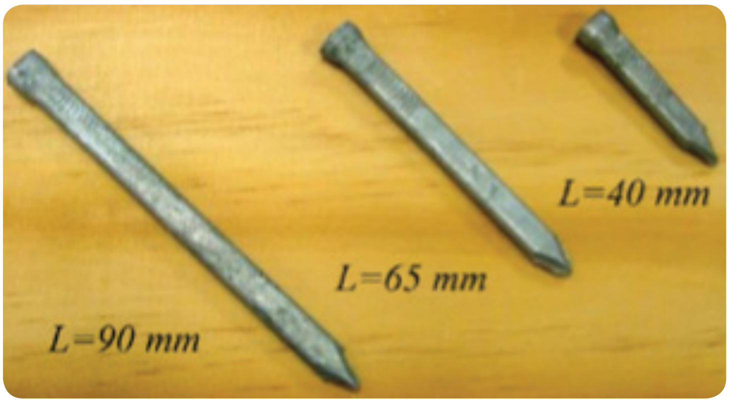


Figure 1.2: Timber rivets.

The rivets are driven through holes in the side plates by the use of either a standard hammer or a palm pneumatic hammer, until the conical heads are firmly seated with a maximum projection of 3.2 mm (half of the tip taper of 6.4 mm). When seated in this manner, the rivet head slightly deforms the steel side plate and wedges in place, creating a fixity that restricts the rivet head from rotating under load. This contributes to the overall stiffness of the connection.

Rivets should not be driven flush, as this will result in losing head fixity. Timber rivets are always driven with the major axis (long side of rectangular nail) parallel to the grain of the timber. They are installed in a spiral pattern from the outside of the group in towards the centre. This way, the pre-stressed fibres will minimise splitting from occurring[4]. Please refer to Section 3 for detailing of the rivets and rivet plates.

Timber Rivets are part of the Canadian CSA-O86[5] and American NDS[6] Wood Standards. However, these Standards have no closed form solution for the strength prediction of this type of connection under wood failure mechanisms. Also, these Standards restrict the use of rivets to specific configurations and for glulam and sawn timber of some limited species[7].

1.2.1 Advantages for Designers

Timber rivet connections offer a number of benefits to designers:

- High load transfer capacity (see Table 1.1)
- Tight-fit dowel action providing stiff connections
- High ductility and the ability to dissipate dynamic loads if detailed to fail in a ductile fashion
- Low variability in strength and deflection properties
- No requirement for pre-drilling and no reduction of timber cross-section
- Ease of installation and inspection in the field
- A cost-effective alternative to other dowel-type fasteners.

Table 1.1: Rivet ultimate design capacity under ductile failure in a double-sided joint with an array of 8*8 i.e. 6 rows of nails with 8 nails in each row, in Radiata Pine LVL.

Rivet Length	40 mm	65 mm	90 mm
Parallel-to-grain loading	350 kN	495 kN	525 kN
Perpendicular-to-grain loading	345 kN	415 kN	440 kN

1.2.2 Applications

Timber rivet connections have been used successfully in Canada and the US for the past four decades in different types of structures (see Figure 1.3) including:

- Long span truss connections (see design example in Section 5.1)
- Long span beam splices
- Beam-to-column and column-to-foundation connections (see design example in Section 5.2)
- Couple moment connections (see design example in Section 5.3)
- Energy dissipating connections
- Shear wall hold-down anchorages (see design example in Section 5.5).

In large structures, where the ductile behaviour and energy dissipation of the connections can be desirable due to the applied wind and seismic loads, the use of rivet connections would assure more structural safety under these dynamic loads[8]. Their high load-transfer and high stiffness characteristics, ease of installation and flexibility in the field make them a highly effective and reliable connector choice.

Figure 1.3: Sample applications of timber rivet connections.
(Image credit: Specialized Timber Fasteners Ltd, Canada)

Figure 1.4: Riveted connections on the Carterton Event Centre, New Zealand.
(Image credit: McIntosh Timber Laminates Ltd)

Figure 1.5: Riveted connections of the Trimble building in Christchurch, New Zealand.
(Image credit: McIntosh Timber Laminates Ltd)

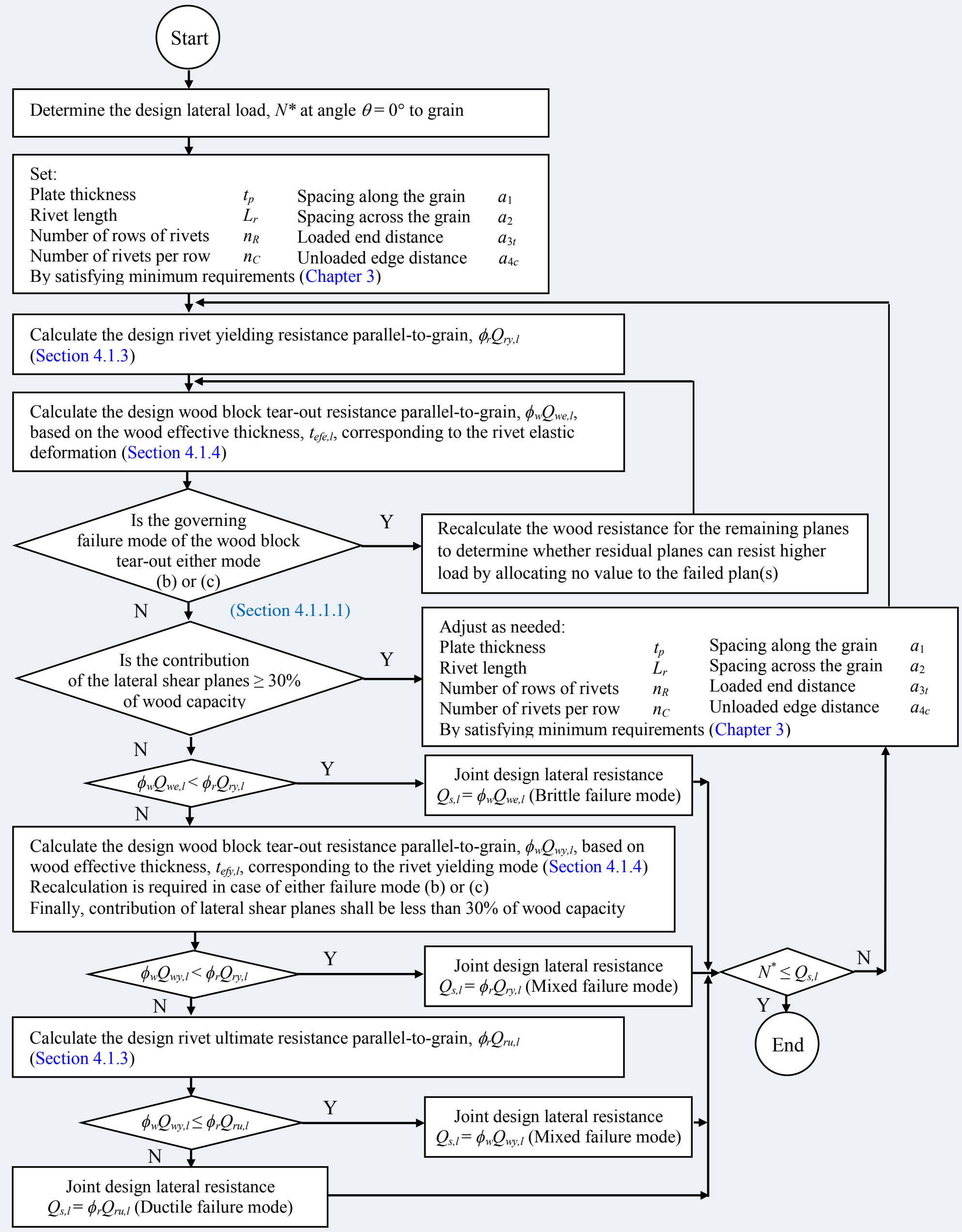
2

Timber Rivet Connection Design Process Flowcharts

The design process for timber rivet connections under different loading directions is illustrated through the following flowcharts. Mainly, two strength limit states are of interest: rivet resistance and wood resistance. Steel plate capacity is not covered in these flowcharts and should be checked using the appropriate steel design code.

2.1 Design Process Flowchart for Load Applied Parallel to Grain

The following flowchart describes the processes required to design a rivet connection.



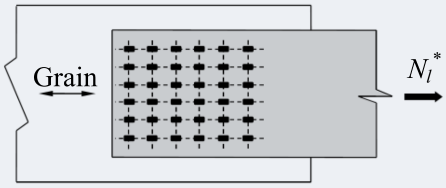
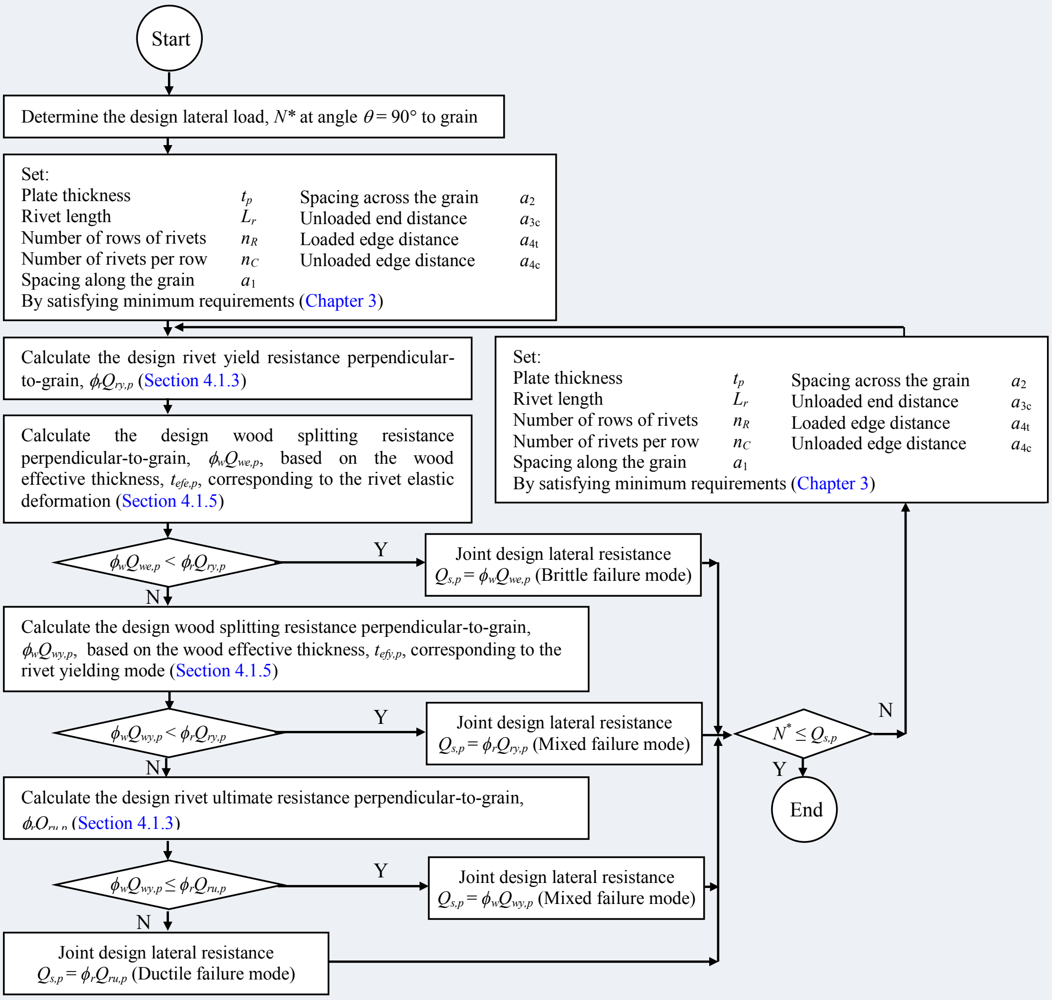


Figure 2.1: Longitudinal loading.

2.2 Design Process Flowchart for Load Applied Perpendicular to Grain



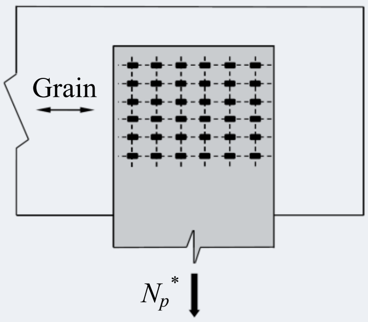
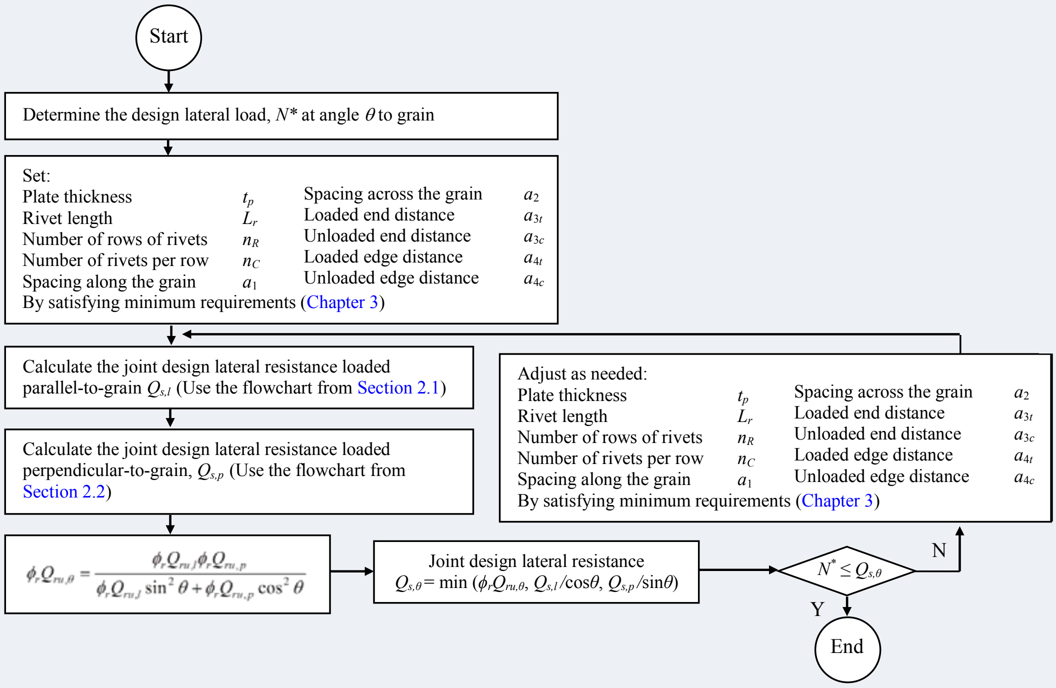


Figure 2.2: Transverse loading.



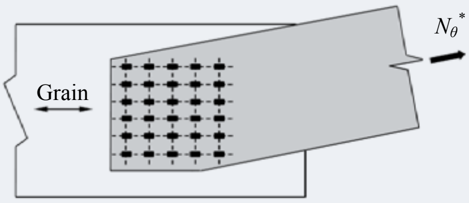


Figure 2.3: Loading at an angle.

3

Detailing of Rivets and Rivet Plates (General Notes and Requirements)

3.1 General Specifications

The design methods specified in Section 4 are for timber rivets that meet the following criteria:

1. Ultimate tensile strength: 1000 MPa, minimum
2. Hardness: Rockwell (C32-39)
3. Finish: hot-dip galvanized
4. Have dimensions as shown in Figure 3.1.

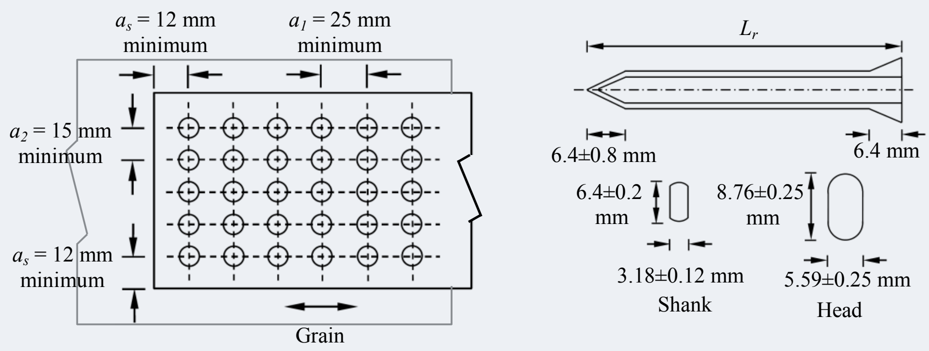


Figure 3.1: Timber rivet joint: rivet details (left); and typical steel plate configuration (right).

Notes:

1. *Hole diameter: 6.9 ± 0.1 mm*
2. *Tolerance in location of holes: 3 mm maximum in any direction.*
3. *Wide face of rivets parallel to grain, regardless of plate orientation.*
4. *All dimensions are prior to galvanizing.*

3.2 Cross-Sectional Requirements

Side plates should have a cross-section adequate for resisting tension, compression and shear forces, but should be not less than 3.2 mm thick. (Minimum thickness is metrication of original research work.) For wet service conditions, side plates should be hot-dip galvanized. Particular attention should be paid to the possible interaction between any timber treatment chemical and the side plate material and protection. Refer to the WoodSolutions Design Guide #5: *Timber service life design* for further information on durability and fastener corrosion protection.

3.3 Placement Requirements

Each rivet should be placed with its major cross-sectional dimension aligned parallel to the grain for the timber element they penetrate. For connectors in trusses, this will generally mean alignment to the element in the joint. The design criteria in Section 4 are based on rivets driven through holes in the side plates until the conical heads are firmly seated with maximum projection of 3.2 mm. Rivets should not be driven flush.

To reduce timber splitting, it is important that timber rivets at the perimeter of the group should be driven first. Successive timber rivets should be driven in a spiral pattern from the outside to the centre of the group.

3.4 Spacing Requirements

The minimum rivet spacing should be:

- perpendicular to grain (a_2) = 15 mm
- parallel to grain (a_1) = 25 mm.

These minimum dimension become the starting of the layout. For C, the maximum penetration of the rivet should be 70% of the thickness of the wood member, as shown in Figure 3.2. For joints with rivets driven from opposite faces of a wood member, the rivet length should be such that the points do not overlap (Figure 3.3).

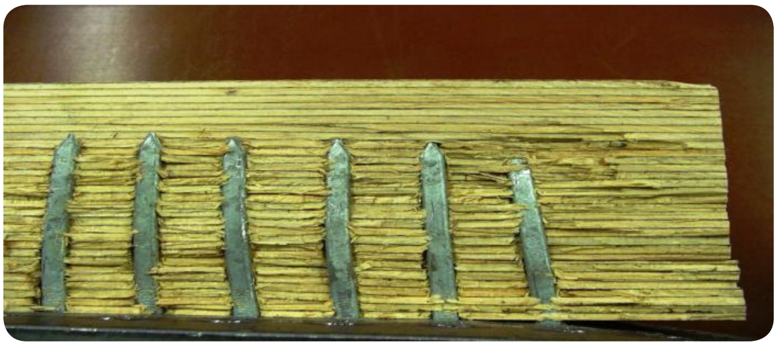


Figure 3.2: One-sided timber rivet connection.

3.5 End and Edge Distances Requirements

Based on research, the minimum end and edge distances are given in Tables 3.1 and 3.2

where

n_R = number of rows of rivets parallel to load

n_C = number of rivets per row

3.5.1 Parallel-to-Grain Loading

For parallel-to-grain loading:

1. Unloaded edge distance, a_{4c}: 25 mm
2. Loaded end distances, a_{3t}, for parallel-to-grain loading are tabulated in Table 3.1.

Table 3.1: Loaded end distance, a_{3t}, for parallel-to-grain loading.

Number of rivets per row n_C	Loaded end distance a_{3t} [mm]
1–6	75
7–10	100
11–12	125
13–14	150
15–16	175
≥17	200

3.5.2 Perpendicular-to-Grain Loading

For parallel-to-grain loading:

1. Unloaded edge distance, a_{4c}: 25 mm
2. Loaded end distances, a_{3t}, for parallel-to-grain loading are tabulated in Table 3.1.

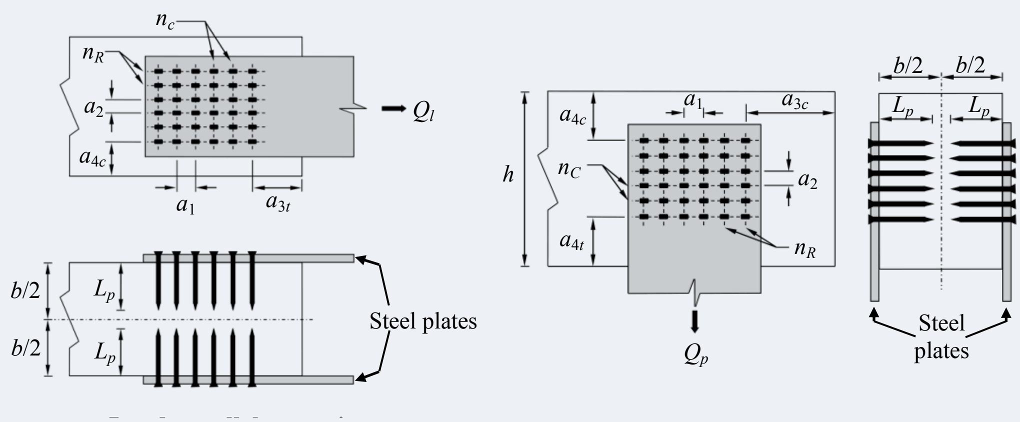
Table 3.2: Unloaded end distance a_{3c}, for perpendicular-to-grain loading.

Number of rows n_R	Unloaded end distance a_{3t} [mm]
1–6	75
7–10	100
11–12	125
13–14	150
15–16	175
≥17	200

3.5.3 Steel Side Plates

In the steel side plates (Figure 3.3):

1. Edge distance to rivet centres, both ways, as: 12 mm
2. Spacing along grain, a_1 : 25 mm
3. Spacing across grain, a_2 : 15 mm



(a) Load parallel to grain (b) Load perpendicular to grain

Figure 3.3: Geometry variables for timber rivet joints.

4 Structural Design Approach

4.1 Lateral Resistance

4.1.1 Failure Mechanism

There are three mechanisms of failure for riveted connections: the brittle tear-out of a plug of wood defined by the rivet's perimeter; the ductile yielding of rivets with localised wood crushing; and the mixed failure mode, which is a brittle failure of the wood with some deflection of the rivets before the rivets reach complete yielding.

The occurrence zone of these potential failure modes is illustrated on a typical load–deflection curve of a timber rivet joint (Figure 4.1). The failures can either be ductile (Figure 4.5); brittle (Figure 4.7a) or mixed (Figure 4.7b). The block tear-out in parallel-to-grain loading (Figure 4.6) and splitting in perpendicular-to-grain loading (Figure 4.10) are the possible wood failures.

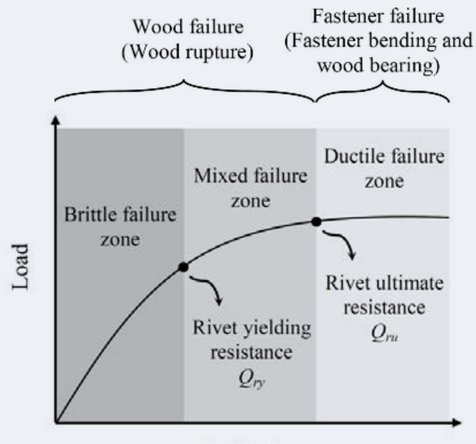


Figure 4.1: Occurrence zone of potential failure modes of timber rivet joints.

4.1.1.1 Brittle failure of wood

In the brittle zone (Figure 4.1), the fasteners deflection is in the elastic range, therefore, the effective wood thickness for the joint corresponds to the elastic deformation of the fasteners, $t_{ef,e}$, as shown in Figure 4.7a. In this failure zone, the wood capacity of the connection, $P_{w,tefe}$, is less than the fastener yielding resistance, $P_{r,yld}$. It should be noted that the $P_{r,yld}$ is not an ultimate failure but constitutes a boundary.

For parallel-to-grain loading, a brittle failure mode occurs when a block of wood bounded by the rivet cluster perimeter is pulled away from the member. As shown in Figure 4.2, the applied load transfers from the wood member to the resisting planes and involves both the tensile and shear capacities of the wood[10,11,12].

Due to the fact that rivets are small in diameter and installed in small spacing, they do not exhibit row shear or splitting failure modes under parallel-to-grain loading which can occur for larger dowel-type fasteners such as bolts.

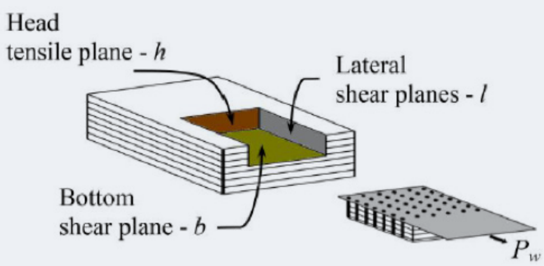


Figure 4.2: Brittle wood block tear-out failure – parallel to grain. *Source: Zarnani, P., Load-carrying capacity and failure mode analysis of timber rivet connections. 2013, ResearchSpace@ Auckland.*

As shown in Figure 4.3, when the joint is subjected to transverse loading, a brittle failure mode can happen where the wood splits along the row of rivets next to the unloaded edge and the crack propagates towards the timber member ends till reaching the unstable zone[12,13].

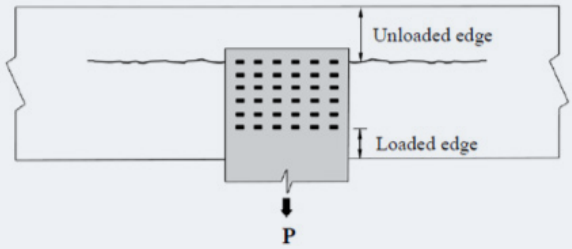


Figure 4.3: Crack growth on either side of the joint along the row of rivets next to the unloaded edge. *Source: Zarnani, P., Load-carrying capacity and failure mode analysis of timber rivet connections. 2013, ResearchSpace@ Auckland.*

4.1.1.2 Mixed Failure

A mixed failure mode (a mixture of brittle and ductile behaviour) is also possible. In mixed failure mode, the wood fails following some deflection of the rivets but before they reach complete yielding. In this failure mode, the thickness of the failed block is significantly smaller than the one associated with the brittle failure mode. As shown in Figure 4 4, the wood effective thickness, tef, depends on the governing yielding mode of the fastener.

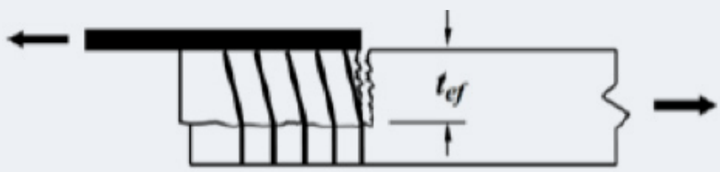


Figure 4.4: Effective wood thickness corresponding to rivet governing yielding mode – mixed failure mode.

As the yield point is reached, the effective wood thickness reduces if the yield mode is not Mode I. This reduction in effective wood thickness, $t_{ef,y}$, leads to the generation of a new connection failure mode (Figure 4.7b). If the wood capacity of the new connection, Pw,tefy, cannot resist the fastener yielding load ($P_{w,tefy} < P_{r,yld}$), a sudden wood failure with slight deflection on the fasteners which is called mixed failure mode occurs. Even if $P_{w,tefy} > P_{r,yld}$, the mixed failure mode can happen as the deflection of the connection progresses if $P_{w,tefy}$ is lower than the connection ultimate ductile strength, $P_{r,ult}$.

4.1.1.3 Ductile failure of rivet

If the wood strength based on effective wood thickness, $t_{ef,y}$ is greater than $P_{r,ult}$, the ductile failure governs and there is no wood rupture (Figure 4.1).

In ductile failure mode, either rivets are loaded longitudinally or transversely, and the rivet compresses the wood up to yielding which results in localised wood crushing. Since rivets are always used in single shear and the rivet head can be considered to be rotationally fixed as it is wedged into the steel plate's hole[15,16], only two yield modes can be possible (Figure 4.5)[17].

The strength of the rivet under different ductile failure modes depends on the embedment resistance of the wood and the bending resistance of the rivet.

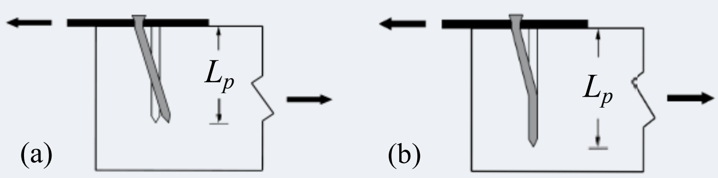


Figure 4.5: Different possible ductile failure modes of rivets: (a) one plastic hinge formation; (b) two plastic hinges

4.1.2 Design Requirement

Using the Load and Resistance Factor Design (LRFD) philosophy, a designer can evaluate the reliability of a structure with regard to its ultimate behaviour under extreme loads (e.g. wind) with significant displacements, where knowledge about the connection capacity beyond the yielding load is critical. In recognition of this fact, the design procedure presented enables a designer to determine the wood and fastener capacities in different possible connection failure modes under ultimate design.

If a designer wants to rely only on the yield limit state as the connection maximum capacity, therefore, the following design procedure can become briefer using:

$$Q_s = \min(\phi_w Q_{we}, \phi_r Q_{ry})$$

where

Q_s = joint design lateral resistance, kN, which should be derived as following the method given in this section.

Joints should be designed in accordance with the following requirement:

$$N^* \leq Q_s \qquad (4.1)$$

where

N^* = design lateral load effects on joint due to factored load, kN

4.1.2.1 For joints loaded parallel to grain

By following the described mechanism for the potential failure modes, the joint ultimate capacity parallel to grain, Pc,ult, can be predicted as follows:

$$Q_s = Q_{s,l}$$

where

$$\begin{aligned} &\phi_w Q_{we,l} \text{ if } \phi_w Q_{we,l} < \phi_r Q_{ry,l} \quad \text{(Brittle mode)} \\ &\phi_r Q_{ry,l} \text{ if } \phi_w Q_{wy,l} < \phi_r Q_{ry,l} \leq \phi_w Q_{we,l} \quad \text{(Mixed mode, small slip)} \\ &\phi_w Q_{wy,l} \text{ if } \phi_r Q_{ry,l} \leq \phi_w Q_{wy,l} < \phi_r Q_{ru,l} \quad \text{(Mixed mode, large slip)} \\ &\phi_r Q_{ru,l} \text{ if } \phi_r Q_{ru,l} < \phi_w Q_{wy,l} \quad \text{(Ductile mode)} \end{aligned} \qquad (4.2)$$

$\phi_w Q_{we,l}$: Design wood block tear-out resistance, parallel to grain, corresponding to rivet elastic deformation, kN (Section 4.1.4)

$\phi_r Q_{ry,l}$: Design rivet yielding resistance, parallel to grain, kN (Section 4.1.3-i)

$\phi_w Q_{wy,l}$: Design wood block tear-out resistance, parallel to grain, corresponding to rivet yielding mode, kN (Section 4.1.4)

$\phi_w Q_{ru,l}$: design rivet ultimate resistance, parallel to grain, kN (Section 4.1.3-i)

The flow charts of the design for rivets loaded parallel, perpendicular and at angle to the grain are given in Section 2.

4.1.2.2 For joints loaded perpendicular to grain

By following the described mechanism for the potential failure modes, the joint ultimate capacity perpendicular to grain, Pc,ult, can be predicted as follows:

$$Q_s = Q_{s,p}$$

where

$$Q_{s,p} = \begin{cases} \phi_w Q_{we,p} \text{ if } \phi_w Q_{we,p} < \phi_r Q_{ry,p} \quad \text{(Brittle mode)} \\ \phi_r Q_{ry,p} \text{ if } \phi_w Q_{wy,p} < \phi_r Q_{ry,p} \leq \phi_w Q_{we,p} \quad \text{(Mixed mode)} \\ \phi_w Q_{wy,p} \text{ if } \phi_r Q_{ry,p} \leq \phi_w Q_{wy,p} < \phi_r Q_{ru,p} \quad \text{(Mixed mode)} \\ \phi_r Q_{ru,p} \text{ if } \phi_r Q_{ru,p} < \phi_w Q_{wy,p} \quad \text{(Ductile mode)} \end{cases} \tag{4.3}$$

4.1.2.3 For joints loaded at an angle, *θ*, to the grain

$$Q_s = Q_{s,\xi}$$

where

$$Q_{s,\xi} = \min(\phi_r Q_{ru,\theta},\ Q_{s,l} / \cos\theta, Q_{s,p} / \sin\theta) \tag{4.4}$$

where

$$\phi_r Q_{ru,\theta} = \frac{\phi_r Q_{ru,l} \phi_r Q_{ru,p}}{\phi_r Q_{ru,l} \sin^2\theta + \phi_r Q_{ru,p} \cos^2\theta} \tag{4.5}$$

4.1.3 Rivet Resistance under Ductile Failure

The analysis and design formulae for the rivet resistance in the following sections are based on two possible ductile failure modes (Figure 4.5).

The design resistance of rivets should be calculated as follows:

4.1.3.1 For parallel-to-grain loading

(i) Rivet yielding resistance:

$\Phi_r Q_{ry,l} = \Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

where

$f_{h,0}$ = embedment strength for rivet bearing, parallel to grain, MPa (Section 4.1.3.3-i)

$f_{hy,0}$ = yielding embedment strength for rivet bearing, parallel to grain, MPa

$M_{r,l}$ = parallel-to-grain moment capacity of the rivet (Section 4.1.3.4-i)

$M_{r,yl}$ = parallel-to-grain yielding moment capacity of the rivet

(ii) Rivet ultimate resistance:

$\Phi_r Q_{ru,l} = \Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

where

$f_{hu,0}$ = ultimate embedment strength for rivet bearing, parallel to grain, MPa

$M_{r,ul}$ = parallel-to-grain ultimate moment capacity of the rivet

$$\phi_r Q_{r,l} = \phi_r k_1 k_f n_p n_R n_C \min(P_{rl,a}, P_{rl,b}) \tag{4.6}$$

where

$P_{rp,a}$ = characteristic strength, perpendicular to grain, for rivet failure mode (a), kN (Section 4.1.3.2-i)

$P_{rp,b}$ = characteristic strength, perpendicular to grain, for rivet failure mode (b), kN (Section 4.1.3.2.ii)

4.1.3.3 Parallel-to-grain rivet characteristic strength

The parallel-to-grain rivet characteristic strength can be calculated as:

(i) For rivet ductile failure mode (a):

$$P_{rl,a} = X_r \left[J_p f_{h,0} L_p d_l \left(\left(\sqrt{2 + \frac{4M_{r,l}}{f_{h,0} d_l L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3} \qquad (4.8)$$

where

X_r = adjustment factor for characteristic resistance (see Appendix A3 for details)
= 0.93 for LVL
= 0.87 for glulam
= 0.84 for sawn timber

J_p = side plate factor
= 1.0 for a side plate thickness, 6.3 mm ≤ t_p
= 0.9 for a side plate thickness, 4.7 ≤ t_p < 6.3 mm
= 0.8 for a side plate thickness, 3.2 ≤ t_p < 4.7 mm

$f_{h,0}$ = embedment strength for rivet bearing parallel to grain, MPa (Section 4.1.3.3-i)

L_p = rivet penetration length, mm
= L_r - t_p - 3.2

where

L_r = rivet length, mm

t_p = side plate thickness, mm

d_l = rivet cross-section dimension bearing on the wood, parallel to grain,
= 3.2 mm

$M_{r,l}$ = parallel-to-grain moment capacity of the rivet (Section 4.1.3.4-i)

f_{ax} = withdrawal resistance per millimetre of penetration, N/mm (Section 4.2)

(ii) For rivet ductile failure mode (b):

$$P_{rl,b} = X_r \left[2J_p \sqrt{M_{r,l} f_{h,0} d_l} + \frac{L_p f_{ax}}{5.33} \right] 10^{-3} \qquad (4.9)$$

The variables are as specified above.

4.1.3.4 Perpendicular-to-grain rivet characteristic strength

The perpendicular-to-grain rivet characteristic strength can be calculated as:

(i) For rivet ductile failure mode (a):

$$P_{rp,a} = X_r \left[J_p f_{h,90} L_p d_p \left(\left(\sqrt{2 + \frac{4M_{r,p}}{f_{h,90} d_p L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3} \qquad (4.10)$$

where

$f_{h,90}$ = embedment strength for rivet bearing, perpendicular to grain, MPa (Section 4.1.3.3-i)

d_p = rivet cross-section dimension bearing on the wood, perpendicular to grain,
= 3.2 mm

$M_{r,p}$ = perpendicular-to-grain moment capacity of the rivet (Section 4.1.3.4-i)

The other variables are as specified in Section 4.1.3.1.

(ii) For rivet ductile failure mode (b):

$$P_{rp,b} = X_r \left[2J_p \sqrt{M_{r,p} f_{h,90} d_p} + \frac{L_p f_{ax}}{5.33} \right] 10^{-3} \quad (4.11)$$

The variables are as specified above.

4.1.3.5 Wood embedment strength

The embedment strength of wood for rivets, MPa, should be calculated as follows:

(i) For parallel-to-grain loading

- Yielding embedment strength:

$f_{hy,0}$ = 75.1ρ(1-0.0037d_l)10^{-3} for LVL

= 71.9ρ(1-0.0024d_l)10^{-3}

for glulam and sawn timber

where

ρ = wood design density at 12% moisture content, kg/m^3 (from Table H2.3 AS 1720)

d_l = rivet cross-section dimension bearing on the wood, parallel to grain

- Ultimate embedment strength:

$f_{hu,0}$ = 90.4ρ(1-0.0037d_l)10^{-3} for LVL

= 86.7ρ(1-0.0024d_l)10^{-3} for glulam and sawn timber

(ii) For perpendicular-to-grain loading

- Yielding embedment strength:

$f_{hy,90}$ = 49.9ρ(1-0.0037d_p)10^{-3} for LVL

= 35.9ρ(1-0.0024d_p)10^{-3} for glulam and sawn timber

- Ultimate embedment strength:

$f_{hu,90}$ = 60.2ρ (1-0.0037d_p)10^{-3} for LVL

= 43.3ρ(1-0.0024d_p)10^{-3} for glulam and sawn timber

The variables are as specified above.

4.1.3.6 Rivet moment capacity

The moment capacity of rivets should be taken as follows:

(i) For parallel-to-grain loading

- Yielding moment capacity:

$M_{ry,l}$ = 24,900 Nmm

- Ultimate moment capacity:

$M_{ru,l}$ = 30,000 Nmm

(ii) For perpendicular-to-grain loading

- Yielding moment capacity:

$M_{ry,p}$ = 12,450 Nmm

- Ultimate moment capacity:

$M_{ru,p}$ =15,000 Nmm

4.1.4 Wood Resistance: Parallel-to-Grain Wood Block Tear-Out

The analysis and design formulae for wood block tear-out strength in the following sections are based on a linearly elastic spring system, in which the applied load transfers from the main loaded wood block to the contact planes in conformity with the relative stiffness ratio of the wood block adjacent to each of the resisting planes.

The wood capacity is the sum of the load resisted by each plane when failure of one of these planes triggers failure of the joint. The analysis is based on three possible failure modes of wood block tear-out, as shown in Figure 4.6.

The design wood block tear-out depends upon rivet behaviour. The design block tear-out resistance of the wood is calculated as follows:

(i) c, $Q_{we,l}$

$\Phi_w Q_{we,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

(ii) Corresponding to the rivet yielding mode, $Q_{wy,l}$

$\Phi_w Q_{wy,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

where

$$\Phi_w Q_{w,l} = \Phi_w k_1 k_f n_p \min(P_{w,h}, P_{w,b}, P_{w,l}) \quad (4.12)$$

where

Φ_w = strength reduction factor for wood failure

= 0.7

k_1 = load duration factor (AS1720.1)

k_f = modification factor for joint position effect

= 0.9 for joint on edge grain of LVL

= 1.0 for joint on face grain of LVL or on edge/face grain of glulam and sawn timber

n_p = number of plates

= 1 for one-sided joint

= 2 for double-sided joint

$P_{w,h}$ = characteristic resistance for failure of head tensile plane, kN (Section 4.1.4.1)-wood block tear-out failure mode (a)

$P_{w,b}$ = characteristic resistance for failure of bottom shear plane, kN (Section 4.1.4.2)- wood block tear-out failure mode (b)

$P_{w,l}$ = characteristic resistance for failure of side lateral shear planes, kN (Section 4.1.4.3)- wood block tear-out failure mode (c)

The resistance calculation may necessitate iteration if failure mode (b) or (c) governs the wood strength. Thus, Equation (4.12) for $\Phi_w Q_{w,l}$ should be recalculated for the remaining planes (a second recalculation is required if one of these failure modes[failure mode (b) or (c)] occurs again) to determine whether the residual planes can resist higher load by defining no value for the terms related to the failed planes as follows:

(i) If failure mode (b) governs, use λ_2 and $\lambda_3 = 0$ in Sections 4.1.4.1 and 4.1.4.2 and do not consider Section 4.1.4.3

(ii) If failure mode (c) governs, use λ_1 and λ_3-1 = 0 in Sections 4.1.4.1 and 4.1.4.3 and do not consider Section 4.1.4.2.

The contribution of the side lateral shear planes to the total wood load-carrying capacity of the joint should be less than 30% of the total joint capacity, to limit splitting on these resisting planes before reaching the joint ultimate capacity. The maximum contribution of the side lateral shear planes is given by:

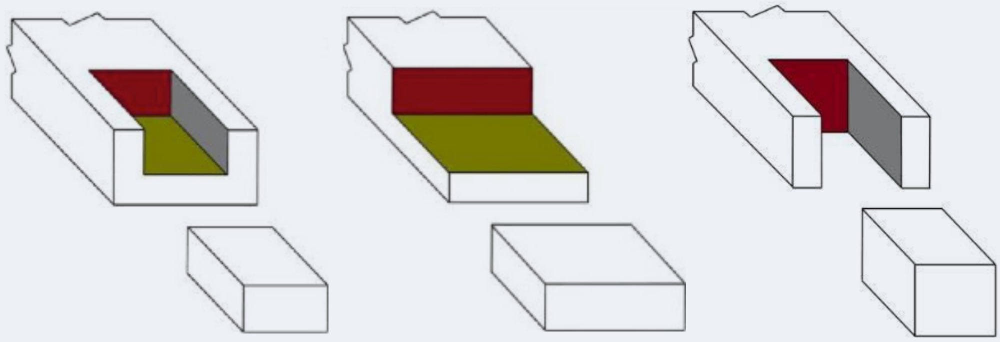


Figure 4.6: Different possible failure modes of wood block tear-out.

(i) If the wood capacity governed when all resisting planes contributed: $(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

(ii) If the wood capacity governed when the head and lateral resisting planes contributed: $(1+\lambda_2^{-1})^{-1} < 0.3$

4.1.4.1 Joint resistance governed by the head tensile plane failure – wood block tear-out failure mode (a)

The characteristic resistance for the failure of the joint triggered by the failure of the head tensile plane, $P_{w,h}$, kN, is given by:

$$P_{w,h} = X_t f_t A_{t,h} (1+\lambda^1+\lambda^2)\, 10^{-3} \text{, mode (a)} \qquad (4.13)$$

where

X_t = adjustment factor for tension strength parallel to grain (see Appendix A3 for details)

= 1.06 for LVL

= 1.19 for glulam

= 1.29 for sawn timber

f_t = member characteristic strength in tension parallel to grain, MPa

$A_{t,b}$ = area of head plane subjected to tensile stress, mm2

= $t_{ef,l}\, w_c$

where

$t_{ef,l}$ = effective wood thickness parallel to grain (Section 4.1.4.4)

w_c = a_2 (n_R-1)

where

a_2 = spacing across the grain, mm

$$\lambda_1 = 0.25\psi L_c (1-H)\left[\frac{A_{s,b}}{t_{ef,l} A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

$$\lambda_2 = 0.25\psi L_c (1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

where

$$\psi = \frac{G}{E}$$

where

E = wood modulus of elasticity parallel to grain, MPa (from tables in Section 1.3)

G = wood modulus of rigidity parallel to grain, MPa

L_c = a_2 (n_c-1)

where

a_1 = spacing along the grain, mm

$A_{s,b}$ = area of bottom plane subjected to shear stress, mm2

= w_c (L_c + a_{3t})

where

a_{3t} = loaded end distance, mm

$A_{s,l}$ = areas of side lateral planes subjected to shear stress, mm2

$2t_{ef,l}$ = (L_c + a_{3t})

$$H = \begin{cases} 0, \text{ If } a_{4c} \geq 1.25 w_c \\ 0.16(2.5 - 2a_{4,c} / w_c)^2, \text{ If } a_{4c} \leq 1.25 w_c \end{cases}$$

where

d_z = bottom distance, mm

= b/2- $t_{ef,l}$ for n_p = 2

= b- $t_{ef,l}$ for n_p =1

where b = member thickness, mm

$$H = \begin{cases} 0, \text{ If } a_{4c} \geq 1.25 w_c \\ 0.16(2.5 - 2a_{4,c} / w_c)^2, \text{ If } a_{4c} \leq 1.25 w_c \end{cases}$$

A_{4c} = unloaded edge distance, mm

4.1.4.2 Joint resistance governed by the bottom shear plane failure – wood block tear-out failure mode (c)

The characteristic resistance for the failure of the joint triggered by the failure of the bottom shear plane or by the failure in tension of the wood block adjacent to the bottom shear plane, $P_{w,b}$, kN, is given by:

$$P_{w,b} = (1 + \lambda_1^{-1} + \lambda_3) 10^{-3} \min = \begin{cases} X_s C_b f_s A_{s,b}, \text{Mode (a)} \\ X_t f_t w_c d_z, \text{Mode (c)} \end{cases} \quad (4.14)$$

where

X_s = adjustment factor for longitudinal shear strength (see Appendix A3 for details)

= 1.02 for LVL

= 0.96 for glulam

= 0.93 for sawn timber

f_s = member characteristic longitudinal shear strength, MPa,

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l} w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

$$C_b = \frac{a_1(n_c(n_c+1)/2-1)+a_{3t}}{n_c(L_c+a_{3t})}$$

The other variables are as specified in Section 4.1.4.1.

4.1.4.3 Joint resistance by the side lateral shear planes failure – wood block tear-out failure mode (c)

The characteristic resistance for the failure of the joint triggered by the failure of the lateral shear planes or by the failure in tension of the wood blocks adjacent to the lateral shear planes, $P_{w,l}$, kN, is given:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} X_s C_l\ f_s\ A_{s,l}, \text{Mode (a)} \\ 2X_t\ f_t t_{ef,l} a_{4c}\ , \text{Mode (b)} \end{cases} \quad (4.15)$$

where

$C_l \quad = k_e\, C_b$

where

$$k_e = \begin{cases} 1,\ \text{If } a_{4c} \geq 1.25 w_c \\ 0.8,\ \text{If } a_{4c} < 1.25 w_c \end{cases}$$

The other variables are as specified in Sections 4.1.4.1 and 4.1.4.2.

4.1.4.4 Parallel-to-grain wood effective thickness

The parallel-to-grain wood effective thickness is determined in accordance to rivet deformation as follows:

(i) Corresponding to the rivet elastic deformation (Figure 4.7a):

$t_{efe,l} \quad = C_{r,l} J_p\, L_p$

where

$C_{r,l} \quad = 0.90$, for $L_p = 28.5$ mm

$\quad = 0.85$, for $L_p = 53.5$ mm

$\quad = 0.80$, for $L_p = 78.5$ mm

For intermediate values of rivet penetration, L_p, use a linear interpolation to determine the value of the factor $C_{r,l}$.

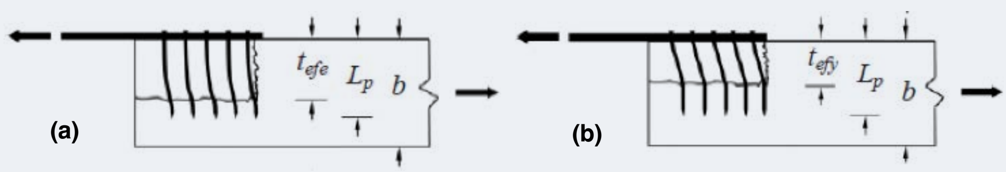


Figure 4.7: Effective wood thickness: (a) corresponding to rivet elastic deformation-brittle failure; (b) corresponding to rivet governing yielding mode-mixed failure.

(i) Corresponding to the rivet yielding mode (Figure 4.7b):

$$t_{efy,l} = \begin{cases} J_p \sqrt{\dfrac{M_{ry,l}}{f_{hy,0} d_l} + \dfrac{L_p^2}{2}}, \text{Rivet yielding mode (a)} \\ \\ 2J_p \sqrt{\dfrac{M_{ry,l}}{f_{hy,0} d_l}}, \text{Rivet yielding mode (b)} \end{cases} \quad (4.16)$$

The other variables are as specified in Section 4.1.3.1.

4.1.4.5 Multiple joints in tension parallel to grain

In the case where there is more than one joint acting in tension parallel to grain, such as in Figure 4.8, the wood block tear-out resistance of each joint can be derived by considering the edge distance as:

$a_{4c} = \min(a_{4c,a}, a_{4c,b})$

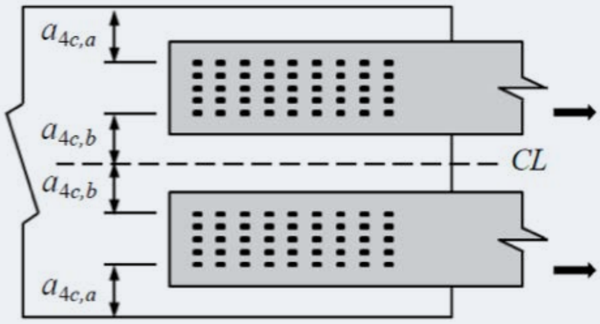


Figure 4.8: Multiple joints acting in tension parallel to grain.

4.1.4.6 Joints in both face and edge grains

If the rivets are required to be driven in both face and edge grains of the wood member, the following requirements should be satisfied to allow the top and side joints to work independently (Figure 4.9).

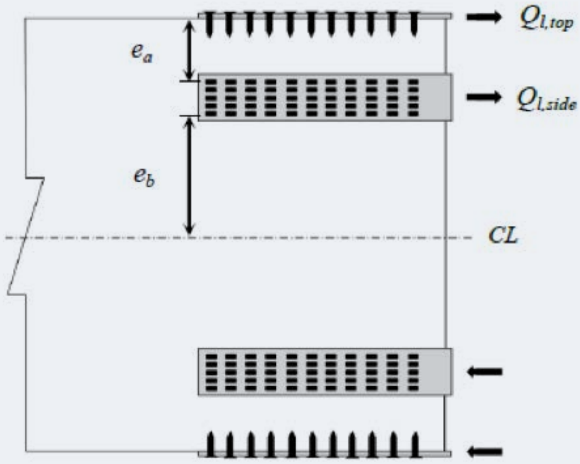


Figure 4.7: Effective wood thickness: (a) corresponding to rivet elastic deformation-brittle failure; (b) corresponding to rivet governing yielding mode-mixed failure.

(i) Minimum distance between the top of the member and the first rivet row of the side joint, e_a; 3 times the $t_{efe,l}$ of the top joint.

(ii) To determine the wood capacity of the side joint, the edge distance should be considered as;

$a_{4c} = \min(e_a - 3_{tefe,l}, e_b)$

where

e_b is distance between last rivet row of the side joint to centreline.

(iii) To determine the wood block tear-out capacity of the top joint. The bottom distance should be considered as;

$d_z \ = 2\, t_{efe,l}$

4.1.5 Wood Resistance: Perpendicular-to-Grain Wood Splitting

The analysis and design formulae for wood splitting strength in the following sections are based on two possible splitting failure modes: with crack width in member cross-section equal to (a) member thickness; or equal to (b) wood effective thickness on each side of the member (Figure 4.10). The wood capacity is the lesser of the resistance in these two possible splitting failure modes.

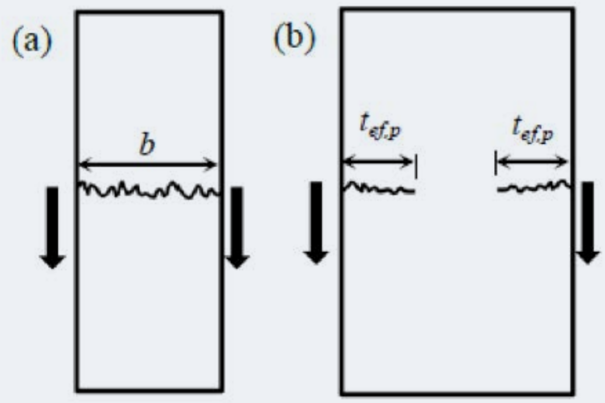


Figure 4 10: Crack width in member cross-section in different splitting failure modes: (a) entire member thickness; (b) tef,p on each side

The design splitting resistance of the wood is calculated as follows:

(i) Corresponding to the rivet elastic deformation, ΦwQwe,p ;

$\Phi_w Q_{we,p} = \Phi_w Q_{w,p}$ in which $t_{ef,p}$ equals to $t_{efe,p}$

(ii) Corresponding to the rivet yielding mode, ΦwQwy,p ;

$\Phi_w Q_{wy,p} = \Phi_w Q_{w,p}$ in which $t_{ef,p}$ equals to $t_{efy,p}$

where

$$\Phi_w\, Q_{w,p} = \Phi_w\, k_1\, g_{42}\, k_f\, n_p \min(P_{s,a}, P_{s,b}) \qquad (4.17)$$

where

Φ_w = strength reduction factor for wood failure
= 0.7

k_1 = load duration factor (as per AS 1720.1)

g_{42} = modification factor for interaction effect on a grid system specified in AS 1720.1 (see Section 4.1.5.3)
= 0.60 for multiple joints
= 1.0 for single joint

k_f = modification factor for joint position effect
= 0.55 for joint on edge grain of LVL
= 1.0 for joint on face grain of LVL or on edge/face grain of glulam and sawn timber

n_p = number of plates
= 1 for one-sided joint
= 2 for double-sided joint

$P_{s,a}$ = characteristic resistance for full width splitting – mode (a), (Section 4.1.5.1)

$P_{s,b}$ = characteristic resistance for partial width splitting – mode (b), (Section 4.1.5.2)

4.1.5.1 The characteristic full width splitting resistance – failure mode (a)

The characteristic resistance for the wood splitting failure of the whole member thickness, $P_{s,a}$, kN, is given by:

$$P_{s,a} = X_p \eta b C_{fp} \sqrt{\frac{h_e}{1-\frac{h_e}{h}}} 10^{-3} \tag{4.18}$$

where

X_p = adjustment factor for tension perpendicular to grain (see Appendix A3 for details)
= 1.23 for LVL
= 1.28 for glulam
= 1.31 for sawn timber

b = member thickness, mm

C_{fp} = member characteristic fracture parameter, N/mm1.5, (from tables in Section 1.3)

$$\eta = \frac{\min(\gamma h_e, a_{3c,L}) + (\gamma h_e, a_{3c,R}) + w_{net}}{2\gamma h_e}$$

γ = effective crack length coefficient for splitting mode (a)
= 4 for LVL
= 2.7 for glulam and sawn timber

$a_{3c,L}$ = minimum of unloaded end distance and half of the distance to adjacent joint on the left side, mm (Figure 4 11)

$a_{3c,R}$ = minimum of unloaded end distance and half of the distance to adjacent joint on the right side, mm (Figure 4 11)

w_{net} = net section of joint width
= $a_1 (n_R - 1) – 6.4n_R$

h_e = effective member depth, mm
= $h - a_{4c}$

where

h = member depth, mm

a = unloaded edge distance, mm

4.1.5.2 The characteristic partial width splitting resistance – failure mode (b)

The characteristic resistance for the wood splitting failure corresponding to $t_{ef,p}$ on each side of the member, $P_{s,b}$, kN, is given by:

$$P_{s,b} = X_p C_t f_{tp} t_{ef,p} \left[w_{net} + \min(\beta h_e, a3c, L) + \min(\beta h_e, a3c, R) \right] 10^{-3} \tag{4.19}$$

where

$$C_t = \begin{cases} 1.264\zeta^{-0.37}, \text{ If } \zeta < 1.9 \\ 1 \text{ If } \zeta \geq 1.9 \end{cases}$$

where

$$\zeta = \frac{a_{4c}}{a_2(n_c - 1)}$$

f_{tp} = characteristic strength in tension perpendicular to grain, MPa (from tables in Section 1.3)

$t_{ef,p}$ = wood effective thickness perpendicular to grain, mm (Section 4.1.5.4)

β = effective crack length coefficient for splitting mode (b)
= 2.4 for LVL
= 1.6 for glulam and sawn timber

The variables are as specified above.

4.1.5.3 Multiple joints in tension perpendicular to grain

In the case where there is more than one joint acting in tension perpendicular to grain, such as in Figure 4.11, the estimated wood splitting capacity of each joint is reduced by 40% using the g42 factor to take the effect of interaction between joints into account.

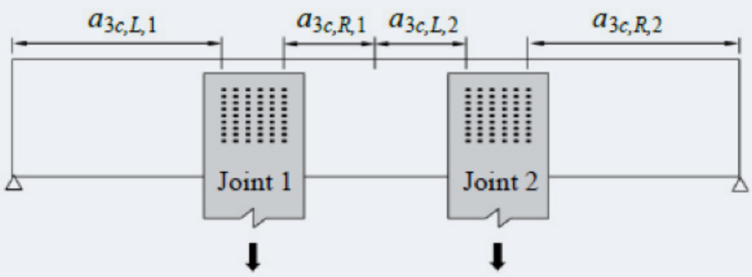


Figure 4.11: Multiple joints in tension perpendicular to grain.

4.1.5.4 Perpendicular-to-grain wood effective thickness

The perpendicular-to-grain wood effective thickness is determined as follows:

(i) Corresponding to the rivet elastic deformation (Figure 4.7 a):

$t_{efe,p} = C_{r,p} J_p L_p$

where

$C_{r,p}$

$= 0.85$, for $L_p = 28.5$ mm

$= 0.75$, for $L_p = 53.5$ mm

$= 0.65$, for $L_p = 78.5$ mm

For intermediate values of rivet penetration, L_p, use a linear interpolation to determine the value of factor $C_{r,p}$.

(ii) Corresponding to the rivet yielding (Figure 4.7b):

$$t_{efy,p} = \begin{cases} J_p \sqrt{\dfrac{M_{ry,p}}{f_{hy,90} d_p} + \dfrac{L_p^2}{2}} \\ 2J_p \sqrt{\dfrac{M_{ry,p}}{f_{hy,90} d_p}} \end{cases} \qquad (4.20)$$

The other variables are as specified in Section 4.1.3.1.

4.2 Withdrawal Resistance

The design withdrawal resistance (kN) from the side grains of a timber rivet joint is taken as follows:

$$\Phi_{ax} F_{ax} = \Phi_{ax} k_1 k_f n_R n_C P_{ax} \qquad (4.21)$$

where

Φ_{ax} = strength reduction factor for withdrawal resistance

= 0.6

k_f = modification factor for joint position effect

= 0.9 for joint on edge grain of LVL

= 1.0 for joint on face grain of LVL or on edge/face grain of glulam and sawn timber

P_{ax} = characteristic withdrawal resistance, kN

$= X_{ax} L_p f_{ax}$

where

X_{ax} = adjustment factor for characteristic withdrawal resistance (see Appendix A3 for details)

= 0.84 for LVL

= 0.61 for glulam

= 0.49 for sawn timber

L_p = rivet penetration length, mm

f_{ax} = withdrawal resistance per millimetre of penetration, N/mm

= $15.9\rho\, d_p(1\text{-}0.0037d_p)10^{-3}$ for LVL

= $11.5\rho\, d_p(1\text{-}0.0024d_p)10^{-3}$ for glulam and sawn timber

where

ρ = wood design density at 12% moisture content, kg/m³ (from Table H2.3, AS 1720)

d_p = rivet major cross-section dimension

= 6.4 mm

4.3 Joint Deflection

The deflection of the joint due to rivet slip, γ, mm, can be determined by:

For parallel-to-grain loading:

$$\delta_l = 4\left[1 - \sqrt{1 - \frac{N_l^*}{\phi_r Q_{ru,l}}}\right] \quad (4.22)$$

where

N^*_1 = serviceability design load, parallel to grain (kN), for deflection under (SLS)

= ultimate design load, parallel to grain (kN), for deflection under (ULS)

$\Phi_r\, Q_{ru,l}$ = design rivet ultimate resistance, parallel to grain, kN (Section 3.1.3)

For perpendicular-to-grain loading:

$$\delta_p = 5.5\left[1 - \sqrt{1 - 0.99\frac{N_p^*}{\phi_r Q_{ru,p}}}\right] \quad (4.23)$$

where

N^*_p = serviceability design load, perpendicular to grain (kN), for deflection under (SLS)

= ultimate design load, perpendicular to grain (kN), for deflection under (ULS)

$\Phi_r\, Q_{ru,p}$ = design rivet ultimate resistance, perpendicular to grain, kN (Section 3.1.3)

For loading at angle θ to grain:

$$\delta_\theta = \sqrt{\delta_l^2 + \delta_p^2} \quad (4.24)$$

5

Design Examples

This section provides a series of examples to illustrate the design process. For the selection of an appropriate rivet layout and to help start the design, reference capacity tables for joint samples under parallel and perpendicular to grain loadings are provided in Appendix A1.

Truss connection

The example outlines the design of an LVL truss connection that connects the bottom chord, post and strut (see Section 5.1). Each joint is designed separately and resists a load acting parallel to grain (Figure 5.1).

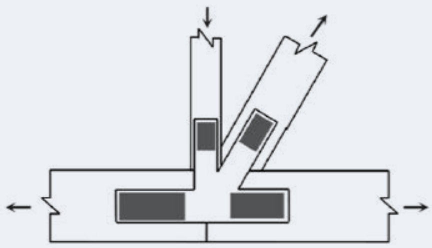


Figure 5.1: Truss connection.

Base connection

The example outlines the design of a base joint that connects a glulam column to the foundation (see Section 5.2). The joint resists load acting at an angle to grain, therefore requires calculation of the capacity for both parallel-to-grain and perpendicular-to-grain loads (Figure 5.2).

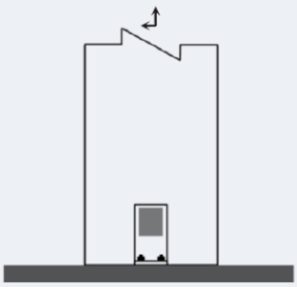


Figure 5.2: Base connection.

Moment connection

The example outlines the design of a connection that transfers moment between two LVL members in a beam (see Section 5.3). Each joint is identical and resists a parallel-to-grain load (Figure 5.3).

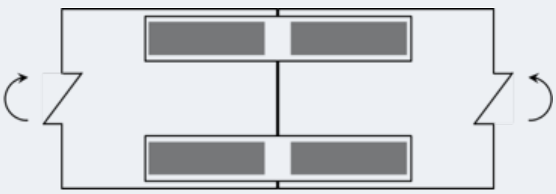


Figure 5.3: Moment connection.

AS 1170.1, Structural design actions, in Permanent, imposed and other actions. 2002, Australian Standard™.

AS 1720.1, Timber structures, in Part 1: Design methods. 2010, Standards Australia: Australia.

Buchanan, A., Timber design guide New Zealand Timber Industry Federation Inc. Wellington, New Zealand, 2007.

Hanger connection

The example outlines the design of a hanger joint that connects a glulam secondary beam to the primary beam (see Section 5.4). The connection acts as two identical joints resisting a load acting perpendicular to grain (Figure 5.4).

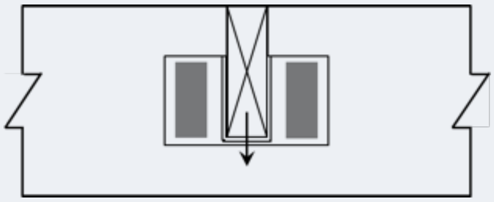


Figure 5.4: Hanger connection.

Shear wall connections

The example outlines the design of the connections in an LVL shear wall, including a hold-down and a floor– wall connection (see Section 5.5). Each joint in the hold-down connection is identical and resists a load acting parallel to grain. The two arrays of rivets in the floor–wall connection act as one joint and resist a load perpendicular to grain (Figure 5.5).

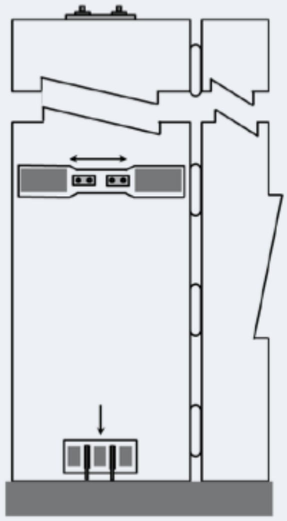


Figure 5.5: Shear wall connections.

5.1 Truss Connection

For a truss node to connect the bottom chord, post, and strut together, the rivet connection is arranged as shown in Figure 5.6. The rivet plates are installed on opposing faces of the dry wood members, which are of grade 11 Radiata Pine LVL.

5.1.1 Design Actions

It is assumed that after taking the effect of the load duration factor (k_1) into account, the critical load combination for the connection design is [1.2G, 1.5Q], as per AS 1170. The design loads acting on the joint are shown in Figure 5.6. Two strength limit states are of interest: rivet strength and wood strength. An efficient connection design can be achieved by decreasing the difference between the capacity of the wood and the rivets.

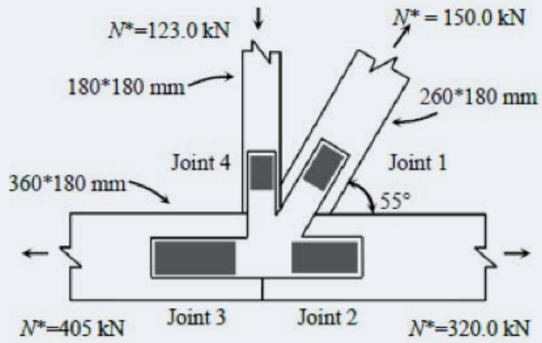


Figure 5.6: Design actions.

This example assumes that four standard 45 mm thicknesses of LVL have been laminated together in a factory with full indoor quality control of the secondary fabrication process. LVL of 45 mm thickness has sufficient over thickness tolerance to enable planning of the surface to glue and deliver a 180mm section. LVL of 90 mm thickness is typically produced with a 'minus' tolerance to fit inside framing so cannot be glued to make 180 mm.

5.1.2 Connection Geometry

Try 65 mm long rivets with the following configuration for each joint:

Side plate thickness: t_p = 10 mm

Note that the steel side plates need to be checked to verify that they have cross-section adequate for resisting tension and compression forces.

5.1.2.1 Joint-1

N^* = 150.0 kN

Number of rows of rivets parallel to direction of load: $n_R = 5$

Number of rivets per row: $n_C = 6$

Spacing along the grain: a_1 = 25 mm

Spacing across the grain: a_2 = 25 mm

End distance: a_{3t} = 100 mm

Edge distance: a_{4c} = 80 mm (based on member width)

5.1.2.2 Joint-2

N^* = 320.0 kN

Number of rows of rivets parallel to direction of load: $n_R = 7$

Number of rivets per row: $n_C = 8$

Spacing along the grain: a_1 = 25 mm

Spacing across the grain: a_2 = 25 mm

End distance: a_{3t} = 120 mm

Edge distance: a_{4c} = 105 mm (based on member width)

5.1.2.3 Joint-3

N^* = 405.0 kN

Number of rows of rivets parallel to direction of load: n_R= 8

Number of rivets per row: $n_C = 9$

Spacing along the grain: a_1 = 25 mm

Spacing across the grain: a_2 = 25 mm

End distance: a_{3t} = 120 mm

Edge distance: a_{4c} = 93 mm (based on member width)

5.1.2.4 Joint-4

N^* = 123.0 kN

Number of rows of rivets parallel to direction of load: $n_R = 5$

Number of rivets per row: $n_C = 5$

Spacing along the grain: $a_1 = 25$ mm

Spacing across the grain: $a_2 = 25$ mm

End distance: $a_{3t} = 75$ mm

Edge distance: $a_{4c} = 40$ mm (based on member width)

5.1.3 Connection Lateral Resistance

$$Q_s = Q_{s,l}$$

$$Q_{s,l} = \begin{cases} \phi_w Q_{we,l} \text{ if } \phi_w Q_{we,l} < \phi_r Q_{ry,l} & \text{(Brittle mode)} \\ \phi_r Q_{ry,l} \text{ if } \phi_w Q_{wy,l} < \phi_r Q_{ry,l} \leq \phi_w Q_{we,l} & \text{(Mixed mode)} \\ \phi_w Q_{wy,l} \text{ if } \phi_r Q_{ry,l} \leq \phi_w Q_{wy,l} < \phi_r Q_{ru,l} & \text{(Mixed mode)} \\ \phi_r Q_{ru,l} \text{ if } \phi_r Q_{ru,l} < \phi_w Q_{wy,l} & \text{(Ductile mode)} \end{cases}$$

5.1.3.1 Joint-1

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Step 1: Rivet capacity corresponding to yielding, parallel to grain, $\Phi_r Q_{ry,l}$

$\Phi_r Q_{ry,l}$ = $\Phi_r Q_{ry,l}$ in which $f_{h,0}$ and Mr,l equal to $f_{hy,0}$ and $M_{ry,l,}$ respectively

$\Phi_r Q_{ry,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

Φ_r = 0.8

k_1 = 0.77 for load combination [1.2G, 1.5Q]

k_f = 1.0 for edge grain of glulam

n_p = 2

n_R = 5

n_C = 6

Rivet failure – mode (a)
Determine $P_{rl,a}$ using Section 4.1.3.1(i):

$$P_{rl,a} = X_r \left[J_p f_{h,0} L_p d_l \left(\left(\sqrt{2 + \frac{4M_{r,l}}{f_{h,0} d_l L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

X_r = 0.93 for LVL

J_p = 1.0 (side plate factor)

$f_{h,0}$ = $f_{hy,0}$

= 75.1ρ (1-0.0037d_l) 10^{-3} for LVL (Section 4.1.3.3)

ρ = 620 kg/m^3 for grade 11 LVL

$f_{hy,0}$ = 46.0 MPa

L_p = $L_r - t_p - 3.2$

L_r = 65 mm

t_p = 10 mm

L_p = 51.8 mm

d_l = 3.2 mm

$M_{r,p}$ = $M_{ry,p}$

= 24900 Nmm

f_{ax} = $15.9\rho d_p(1-0.0037d_p)\ 10^{-3}$

d_p = 6.4

f_{ax} = 61.6 N/mm

Calculate $P_{rl,a}$:

$P_{rl,a}$ = 4.11 N

Rivet failure – mode (b)
Determine $P_{rl,b}$ using Section 4.1.3.1(ii):

$$P_{rl,b} = X_r\left[2J_p\sqrt{M_{r,l}f_{h,0}d_l} + \frac{L_p f_{ax}}{5.33}\right]10^{-3}$$

$P_{rl,b}$ = 4.12 kN

Therefore, the rivet yield capacity, parallel to grain,

$\Phi_r\, Q_{ry,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

$\Phi_r\, Q_{ry,l}$ = 151.9 (Rivet Yelding mode: a)

Step 2: Wood capacity, parallel to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l}$ = $\Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

= $\Phi_w\, n_p\, k_1\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

Φ_w = 0.7

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ = $X_t\, f_t\, A_{t,h}\, (1+\lambda_1+\lambda_2)10^{-3}$

X_t = 1.06 for LVL

f_t = 30 MPa for grade 11 LVL

$A_{t,h}$ = $t_{ef,l}\, w_c$

$t_{ef,l}$ = $t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

= 44.2 mm

w_c = $a_2\, (n_R$-1) = 100 mm

$A_{t,h}$ = $a_2\, (n_R$-1) = 4420 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

where

$$\psi = \frac{G}{E}$$

G = 550 MPa for grade 11 LVL

E = 11000 MPa for grade 11 LVL

γ = 0.05

L_c = $a_1\, (n_c$-1)= 125 mm

d_z = 45.8

H = 0.23

$A_{s,b}$ = 22500 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_1 = 0.215

$$\lambda_2 = 0.25\psi L_c(1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_2 = 0.209

Calculate Pw,h:

$P_{w,h}$ = 200.2 kN, mode (a)

Wood failure – mode (b)
Determine Pw,b using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} X_s C_b\ f_s\ A_{s,b}, \text{Mode (a)} \\ X_t\ f_t w_c\ d_z\ , \text{Mode (c)} \end{cases}$$

X_s = 1.02 for LVL

C_b = 0.44

f_s = 6 MPa for grade 11 LVL

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l} w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 0.974

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} 60588, \text{Mode (a)} \\ 14544\ , \text{Mode (c)} \end{cases}$$

$P_{w,b}$ = 405.6 kN, mode (a)

Wood failure – mode (c)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} 43821.6, \text{Mode (a)} \\ 9855032.9, \text{Mode (b)} \end{cases}$$

C_l = 0.36

Calculate $P_{w,l}$:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} 43821.6, \text{Mode (a)} \\ 9855032.9, \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 294.3 kN, mode (a)

Therefore, the wood capacity, parallel to grain, corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ = $\Phi_w\ n_p\ k_1\ k_f$ min ($P_{w,h}$, *Pw,b, Pw,l*)

$\Phi_w Q_{we,l}$ = $\Phi_w\ n_p\ k_1\ k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

= 341.7 kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.147 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_r Q_{ry,l}$ (brittle failure mode)

If $\Phi_w Q_{wy,l} < \Phi_w Q_{ry,l}$

224.2 ≥ 151.9 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_w Q_{ru,l}$. If so, then $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

$\Phi_w Q_{wy,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

$t_{ef,l}$ = $t_{efy,l}$

$$t_{efy,p} = J_p \sqrt{\frac{M_{ry,p}}{f_{hy,90} d_p} + \frac{L_p^2}{2}}$$, yielding mode (a)

t_{ef} = 38.9 mm

The recalculated wood capacity (by following the same design procedure as defined above):

$\Phi_w Q_{wy,l}$ = 208.2 kN (failure governed by head tensile plane, mode (a))

Note that if the yielding mode (b) was governing then the reduction of wood strength would be much higher.

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.139 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

208.2 ≥ 151.9 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ru,l}$. If so, then $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

Ultimate rivet capacity parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l}$ = $\Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

$f_{h,0}$ = $f_{hu,0}$

= $90.4\rho\,(1\text{-}0.0037d_l)\,10^{-3}$ for LVL (Section 4.1.3.3)

= 55.4 MPa

$M_{r,l}$ = $M_{ru,l}$

= 30,000 Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

= 178.5 kN (Rivet failure mode: a)

Check if $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

208.2 > 178.5 (unsatisfied)

Thus, $Q_{s,l} = \Phi_r Q_{ru,l}$ (ductile failure mode)

$Q_{s,l}$ = 178.5 kN

Check joint ultimate lateral resistance:

N^* ≤ Q_s

N^* = 150.0 kN

Q_s = $Q_{s,l}$

= 178.5 kN

150.0 ≤ 178.5, OK (Joint mode of failure: ductile)

Adopt 65 mm long rivets with an array of 5 rows by 6 columns, spacing 25 mm by 25 mm along and across the grain, and 100 mm end distance (see Figure 5.7).

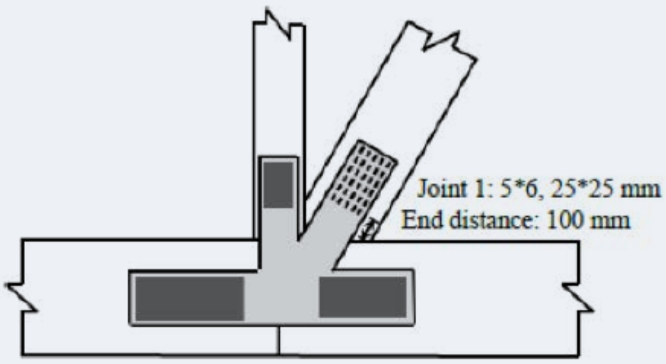


Figure 5.7: Connection configuration of joint 1.

5.1.3.2 Joint-2

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Rivet capacity corresponding to yielding parallel to grain, $\Phi_r Q_{ry,l}$

$\Phi_r Q_{ry,l}$ = $\Phi_r Q_{ry,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

$\Phi_r Q_{ry,l} = \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min $(P_{rl,a}, P_{rl,b})$

$P_{rl,a}$, $P_{rl,b}$ are calculated before.

Φ_r = 0.8

k_1 =0.77 for load combination [1.2G, 1.5Q]

k_f = 1.0 for edge grain of glulam

n_p = 2

n_R = 7

n = 8

$\Phi_r Q_{ry,l}$ = 0.8 x 0.77 x 1.0 x 1.0 x 2 x 7 x 8 min(4.11, 4.12)

= 283.5 kN (Yielding mode of failure: a)

Wood capacity parallel to grain corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l}$ = $\Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

= $\Phi_w\, n_p\, k_1\, k_f$ min $(P_{w,h}, P_{w,b}, P_{w,l})$

Φ_w = 0.7

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ = $X_t\, f_t\, A_{t,h}\, (1+\lambda_1+\lambda_2)10^{-3}$

X_t = 1.19 for glulam

F_t = 11 MPa for grade GL10 glulam

$A_{t,h}$ = $t_{ef,l}\, w_c$

$t_{ef,l}$ = $t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

= 44.2 mm

w_c =$a_2\, (a_2$-1) = 150 mm

$A_{t,h}$ =$a_2\, (a_2$-1) = 6630 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

where

$$\psi = \frac{G}{E}$$

G = 670 MPa for grade GL10 glulam

E = 10,000 MPa for grade GL10 glulam

Ψ = 0.067

L_c = a_1 (n_C-1) = 175 mm

d_z = 45.8

H = 0.23

$A_{s,b}$ = 44250 mm²

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_1 = 0.330

$$\lambda_2 = 0.25\psi L_c(1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_2 = 0.173

Calculate Pw,h:

$P_{w,h}$ = 317.0 kN, mode (a)

Wood failure – mode (b)
Determine Pw,b using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} X_s C_b\ f_s\ A_{s,b}, \text{Mode (a)} \\ X_t\ f_t w_c\ d_z\ , \text{Mode (c)} \end{cases}$$

X_s = 1.02

Xt = 1.06

C_b = 0.42

f_s = 6 MPa

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l}w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 0.524

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} 119347, \text{Mode (a)} \\ 53374\ , \text{Mode (c)} \end{cases}$$

$P_{w,b}$ = 519.6 kN, mode (a)

Wood failure – mode (c)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} X_s C_l\ f_s\ A_{s,l}, \text{Mode (a)} \\ 2X_t\ f_t t_{ef,l} a_{4c}\ , \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 467.4 kN, mode (a)

Therefore, the wood capacity parallel to grain corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ = $\Phi_w\ n_p\ k_1\ k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

= 341.7 kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.115 \quad < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ *(brittle failure mode)*

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

341.7 > 283.5 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Qr_{y,l}$. *If so, then* $Q_{s,l} = \Phi_r Q_{ry,l}$ *(mixed failure mode)*

Wood capacity parallel to grain corresponding to rivet yielding mode, $\Phi_w Q_{wy,l}$

$\Phi_w Q_{wy,l} = \Phi_w Q_{wy,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

$t_{ef,l} \quad = t_{efy,l}$

$t_{ef,l} \quad = 38.9$ mm

The recalculated wood capacity (by following the same design procedure as defined above):

$\Phi_w Q_{wy,l} = 327.5$ kN

Note that if the yielding mode (b) was governing then the reduction of wood strength would be much higher.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.106 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ *(brittle failure mode)*

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

327.5 ≥ 283.6 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ru,l}$. If so, then $Q_{s,l} = \Phi_r Q_{wy,l}$ (mixed failure mode)

Ultimate rivet capacity parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l} = \Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l} \quad = \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

$= 333.5$ kN (Rivet failure mode: a)

Check if $\Phi_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

327.5 ≤ 333.5 OK

$\Phi_{s,l} \quad = \Phi_w Q_{wy,l}$

$= 327.5$ kN

Check joint ultimate lateral resistance:

$N^* \quad \leq Q_s$

$N^* \quad = 320.0$ kN

$Q_s \quad = Q_{s,l}$

$= 327.5$ kN

320.0 ≤ 327.5, OK (Joint mode of failure: mixed)

Adopt 65 mm long rivets with an array of 7 rows by 8 columns, spacing 25 mm by 25 mm along and across the grain, and 120 mm end distance (see Figure 5.8).

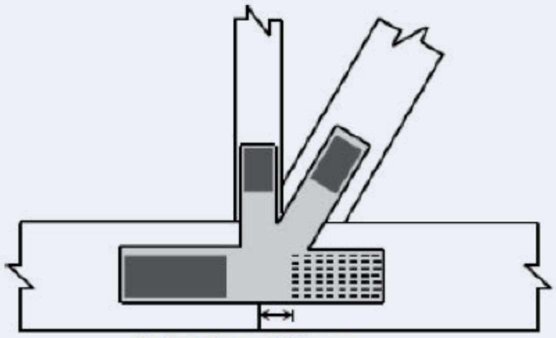


Figure 5.8: Connection configuration of Joint 2

5.1.3.3 Joint-3

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Rivet capacity corresponding to yielding parallel to grain, $\Phi_r Q_{ry,l}$

$\Phi_r Q_{ry,l}$ = $\Phi_r Q_{ry,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

$\Phi_r Q_{ry,l} = \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

$P_{rl,a}$, $P_{rl,b}$ are calculated for previous joints.

Φ_r = 0.8

k_1 = 0.77 for load combination [1.2G, 1.5Q]

k_f = 1.0 for edge grain of glulam

n_p = 2

n_R = 8

n = 9

$\Phi_r Q_{ry,l}$ = 0.8 x 0.77 x 1.0 x 1.0 x 2 x 8 x 9 min(4.11, 4.12)

= 364.6 kN (Yielding mode of failure: a)

Wood capacity parallel to grain corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l}$ = $\Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

= $\Phi_w\, n_p\, k_1\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

Φ_w = 0.7

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ = $X_t\, f_t\, A_{t,h}\, (1+\lambda^1+\lambda^2)10^{-3}$

X_t = 1.19 for glulam

F_t = 11 MPa for grade GL10 glulam

$A_{t,h}$ = $t_{ef,l}\, w_c$

$t_{ef,l}$ = $t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

= 44.2 mm

w_c =a_2 (n_R-1) = 150 mm

$A_{t,h}$ =a_2 (n_R-1) = 7735 mm^2

Ψ = 0.05

L_c = a_1 (n_C-1) = 200 mm

d_z = 23.3

L_c = a_1 (n_C-1) = 200 mm

H = 0.23

$A_{s,b}$ = 56,000 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_1 = 0.391

$$\lambda_2 = 0.25\psi L_c(1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

$A_{s,l}$ = 28,288 mm^2

F = 0.33

λ_2 = 0.136

Calculate $P_{w,h}$:

$P_{w,h}$ = 375.7 kN, mode (a)

Wood failure –mode (b)
Determine $P_{w,b}$ using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} X_s C_b\ f_s\ A_{s,b}, \text{Mode (a)} \\ X_t\ f_t w_c\ d_z\ , \text{Mode (c)} \end{cases}$$

X_s = 1.02

C_b = 0.42

f_s = 6 MPa

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l}w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 0.349

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} 143942, \text{Mode (a)} \\ 254877\ , \text{Mode (c)} \end{cases}$$

$P_{w,b}$ = 566.9 kN, mode (c)

Wood failure – mode (c)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} X_s C_l\ f_s\ A_{s,l}, \text{Mode (a)} \\ 2X_t\ f_t t_{ef,l} a_{4c}\ , \text{Mode (b)} \end{cases}$$

C_l = 0.34

Calculate $P_{w,l}$:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} 58862, \text{Mode (a)} \\ 261434, \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 657.2 kN, mode (a)

Therefore, the wood capacity parallel to grain corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ = $\Phi_w\, n_p\, k_1\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)
$\Phi_w Q_{we,l}$ = $\Phi_w\, n_p\, k_1\, k_f$ min (375.7, 566.9, 657.2)
= 405.0 kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4).
The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

0.089 < 0.3

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

If $\Phi_w Q_{we,l} < \Phi r Q_{ry,l}$

405.0 > 364.6 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_r Q_{ry,l}$ (mixed failure mode)

Wood capacity parallel to grain corresponding to rivet yielding mode, $\Phi_w Q_{wy,l}$

$\Phi_w Q_{wy,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

$t_{ef,l} = t_{efy,l}$

$t_{ef,l} = 38.9$ mm

The recalculated wood capacity (by following the same design procedure as defined above):

$\Phi_w Q_{wy,l} = 409.5$ kN

Note that if the yielding mode (b) was governing then the reduction of wood strength would be much higher.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.081 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

409.5 ≥ 364.6 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ru,l}$. If so, then $Q_{s,l} = \Phi_r Q_{wy,l}$ (mixed failure mode)

Ultimate rivet capacity parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l} = \Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l} = \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

$= 428.8$kN (Rivet failure mode: a)

Check if $\Phi_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

409.5 ≤ 428.8 OK

$\Phi_{s,l} = \Phi_w Q_{wy,l}$

$= 409.5$ kN

Check joint ultimate lateral resistance:

$N^* \leq Q_s$

$N^* = 405$ kN

$Q_s = Q_{s,l}$

$= 409.5$ kN

405 ≤ 409.5, OK (Joint mode of failure: mixed)

Adopt 65 mm long rivets with an array of 8 rows by 9 columns, spacing 25 mm by 25 mm along and across the grain, and 120 mm end distance (see Figure 5.9)

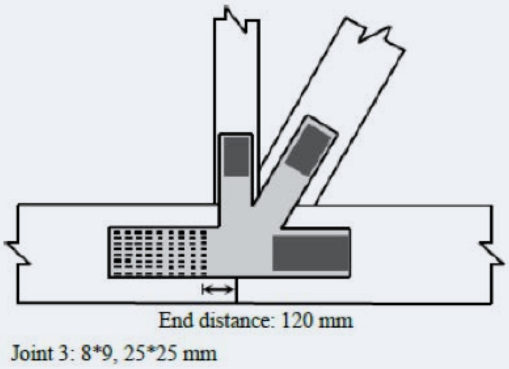


Figure 5.9: Connection configuration of Joint 3.

5.1.3.4 Joint-4

Note that the force is applied in such a way that the member is in compression, therefore, there is no need to check the wood block tear-out resistance.

Ultimate rivet capacity parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l}$ = $\Phi_r Q_{u,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

$f_{h,0}$ = $f_{hu,0}$

= 90.4ρ (1-0.0037d_l) 10^{-3} for LVL (Section 4.1.3.3)

= 55.4 MPa

$M_{r,l}$ = $M_{ru,l}$

= 30,000 Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

n_p = 2

n_R = 5

n_C = 5

$P_{rl,a}$ = 4.83

$P_{rl,b}$ = 4.84

= 0.8 x 0.77 x 1.0 x 1.0 x 2 x 5 x 5 x min (4.83, 4.84)

= 148.9 kN (Rivet failure mode: a)

Check if $\Phi_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

346.6 > 269.8 (unsatisfied)

Thus, $\Phi_{s,l} = \Phi_r Q_{ru,l}$ *(ductile failure mode)*

$\Phi_{s,l}$ = 269.8 kN

Check joint ultimate lateral resistance:

N^* $\leq Q_s$

N^* = 123 kN

Q_s = $Q_{s,l}$

Q_s = 148.9 kN

123 < 148.9, OK (Joint mode of failure ductile)

Adopt 65 mm long rivets with an array of 5 rows by 5 columns, spacing 25 mm by 25 mm along and across the grain, and 75 mm end distance (see Figure 5.10)

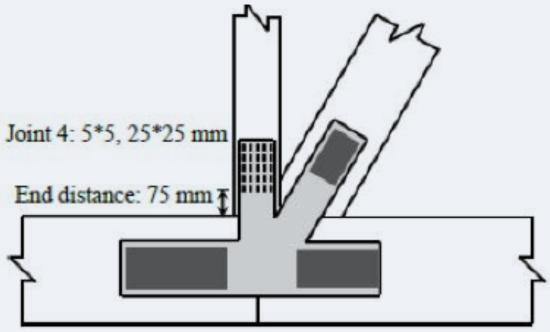


Figure 5.10: Connection configuration of Joint 4.

If ductile behaviour at the connection ultimate capacity is desirable, the failure mode of the connection can be improved from brittle/mixed to ductile by increasing the wood resistance with larger rivet spacing across and along the grain. Spreadsheets can be used to speed up computation and, once they are set up, adjustments in spacing, end and edge distances and capacities for a range of rivet lengths can be evaluated relatively quickly.

5.2 Base connection

For a base joint to connect the column to the foundation, the rivet connection is arranged as shown in Figure 5.11. The rivet plates are installed on opposing faces of the dry wood member, which is of GL10 Radiata Pine glulam.

5.2.1 Design Actions

It is assumed that after taking the effect of the load duration factor (k_1) into account, the critical load combination for the connection design is [0.9G, W_u], as per AS 1170. The design loads acting on the joint are shown in Figure 5.11. Two strength limit states are of interest: rivet strength and wood strength. An efficient connection design can be made by decreasing the difference between the capacity of the wood and the rivets.

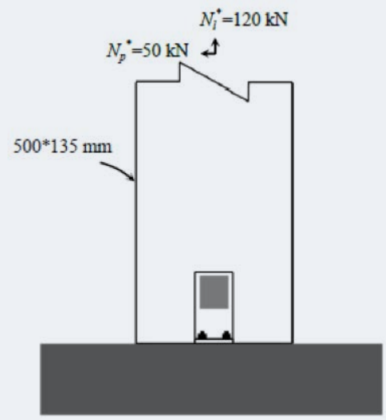


Figure 5.11: Design actions.

5.2.2 Connection Geometry

Try 65 mm long rivets with the following configuration:

Number of rows of rivets parallel to direction of load, $n_R = 8$ for tension force and $n_R = 9$ for shear force

Number of rivets per row, $n_C = 9$ for axial force and $n_C = 8$ for shear force

Spacing along the grain, $a_1 = 35$ mm Spacing across the grain, $a_2 = 25$ mm

End distance, a_{3t} (for tension force)/$a_{3c,L}$ (for shear force) $= 200$ mm

Upper end distance, $a_{3c,R} = 2{,}520$ mm (based on a column free height of 3,000 mm)

Edge distance, $a_{4c} = 163$ mm (based on member width)

Side plate thickness, $t_p = 10$ mm

The steel side plates should be checked to have a cross-section adequate for resisting tension and shear forces.

5.2.3 Connection Lateral Resistance

$Q_s = Q_{s,\theta}$

$= \min(\Phi_r Q_{ru,\theta}, Q_{s,l}/\cos\theta, Q_{s,p}/\sin\theta)$

$\theta = \tan^{-1}(N_p^*/N_l^*)$

$N_p^* = 50$ kN

$N_l^* = 120$ kN

$\theta = \tan^{-1}(50/120) = 22°$

Joint design lateral resistance parallel to grain,

$Q_s = Q_{s,l}$

$$Q_{s,l} = \begin{cases} \phi_w Q_{we,l} \text{ if } \phi_w Q_{we,l} < \phi_r Q_{ry,l} & \text{(Brittle mode)} \\ \phi_r Q_{ry,l} \text{ if } \phi_w Q_{wy,l} < \phi_r Q_{ry,l} \le \phi_w Q_{we,l} & \text{(Mixed mode)} \\ \phi_w Q_{wy,l} \text{ if } \phi_r Q_{ry,l} \le \phi_w Q_{wy,l} < \phi_r Q_{ru,l} & \text{(Mixed mode)} \\ \phi_r Q_{ru,l} \text{ if } \phi_r Q_{ru,l} < \phi_w Q_{wy,l} & \text{(Ductile mode)} \end{cases}$$

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Rivet capacity corresponding to yielding parallel to grain, $\Phi_r Q_{ry,l}$

$\Phi_r Q_{ry,l} = \Phi_r Q_{ry,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

$= \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

$\Phi_r = 0.8$

$k_1 = 1.14$ for load combination [0.9G, Wu]

$k_f = 1.0$ for edge grain of glulam

$n_p = 2$

$n_R = 8$

$n_C = 9$

Rivet failure – mode (a)
Determine P_{rl},a using Section 4.1.3.1(i):

$$P_{rl,a} = X_r \left[J_p f_{h,0} L_p d_l \left(\left(\sqrt{2 + \frac{4M_{r,l}}{f_{h,0} d_l L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

$X_r = 0.87$ for glulam

$J_p = 1.0$ (side plate factor)

$f_{h,90} = f_{hy,90}$

$= 71.9\rho\,(1\text{-}0.0024 d_p)\, 10^{-3}$ for glulam (Section 4.1.3.3)

$\rho = 470$ kg/m^3 for GL10 glulam

dl = 3.2 mm

$f_{hy,90} = 33.5$ MPa

$M_{r,p} = M_{ry,p}$

= 24,900 Nmm

$f_{ax} = 11.5\rho d_p(1\text{-}0.0024 d_p)\, 10^{-3}$

$f_{ax} = 34.1$ N/mm

Calculate $P_{rl,a}$:

$P_{rl,a} = 2.86$ N

Rivet failure – mode (b)
Determine $P_{rl,b}$ using Section 4.1.3.1(ii):

$$P_{rl,b} = X_r \left[2J_p \sqrt{M_{r,l} f_{h,0} d_l} + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

$P_{rl,a}$ = 3.13 kN

Therefore, the rivet yield capacity parallel to grain,

$\Phi_r Q_{ry,l}$ = $\Phi_r k_1 k_{12} k_f n_p n_R n_C$ min $(P_{rl,a}, P_{rl,b})$

$\Phi_r Q_{ry,l}$ = 329.3 (Rivet yielding mode: a)

Wood capacity parallel to grain corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l}$ = $\Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

= $\Phi_w n_p k_1 k_{12} k_f$ min $(P_{w,h}, P_{w,b}, P_{w,l})$

Φ_w = 0.7

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ = $X_t f_t A_{t,h} (1+\lambda_1+\lambda_2) 10^{-3}$

X_t = 1.19 for glulam

f_t = 11 MPa for grade GL10 glulam

$A_{t,h}$ = $t_{ef,l} w_c$

$t_{ef,l}$ = $t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

= 44.2 mm

w_c = $a_2 (n_R$-1) = 175 mm

$A_{t,h}$ = $a_2 (n_R$-1) =4420 mm^2

$$\lambda_1 = 0.25 \psi L_c (1-H) \left[\frac{A_{s,b}}{t_{ef,l} A_{t,h}} + \frac{0.4}{\psi L_c} \right]$$

where

$$\psi = \frac{G}{E}$$

Ψ = 0.067

L_c =$a_1(n_C$-1) = 200 mm

d_z = 23.3

L_c = $a_1(n_C$-1) = 280 mm

H = 0.54

$A_{s,b}$ = 84,000 mm^2

$$\lambda_1 = 0.25 \psi L_c (1-H) \left[\frac{A_{s,b}}{t_{ef,l} A_{t,h}} + \frac{0.4}{\psi L_c} \right]$$

λ_1 = 0.573

$$\lambda_2 = 0.25 \psi L_c (1-F) \left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c} \right]$$

λ_2 = 0.368

Calculate $P_{w,h}$:

$P_{w,h}$ = 196.5 kN, mode (a)

Wood failure – mode (b)
Determine $P_{w,b}$ using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3} \min = \begin{cases} X_s C_b f_s A_{s,b}, \text{Mode (a)} \\ X_t f_t w_c d_z, \text{Mode (c)} \end{cases}$$

X_s = 0.96 for glulam

C_b = 0.40

f_s = 3.7 MPa for grade GL10 glulam

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l} w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 0.642

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3} \min = \begin{cases} 119347, \text{Mode (a)} \\ 53374, \text{Mode (c)} \end{cases}$$

$P_{w,b}$ = 180.8 kN, mode (c)

Wood failure – mode (c)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3} \min = \begin{cases} X_s C_l f_s A_{s,l}, \text{Mode (a)} \\ 2X_t f_t t_{ef,l} a_{4c}, \text{Mode (b)} \end{cases}$$

C_l = 0.32

Calculate $P_{w,l}$:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3} \min = \begin{cases} 48237, \text{Mode (a)} \\ 188616, \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 256.2 kN, mode (a)

Therefore, the parallel-to-grain wood capacity corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ = $\Phi_w\, n_p\, k_1\, k_{12}\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

$\Phi_w Q_{we,l}$ = $\Phi_w\, n_p\, k_1\, k_{12}\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

= 292.1 kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4).
The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

Therefore, the parallel-to-grain wood capacity corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ = $\Phi_w\, n_p\, k_1\, k_{12}\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

$\Phi_w Q_{we,l}$ = 0.7 x 2 x 1.14 x 1.0 x 1.0 x min (196.5,180.8, 256.2)

= 288.6 kN (failure governed by bottom shear plane, mode (c))

Wood failure mode (c) governs failure, therefore the wood capacity, $\Phi_w Q_{we,l}$ should be recalculated from the remaining planes to determine whether the residual head tensile plane and lateral shear planes can resist higher load:

λ_1 = 0

λ_2 = 0.368

λ_3^{-1} = 0

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ $= X_t\, f_t\, A_{t,h}\,(1+\lambda_1+\lambda_2)10^{-3}$

$P_{w,h}$ $= 1.19 \times 11 \times 7736 \times (1+0.0+0.368) \times 10^{-3}$

$= 138.5$ kN, mode (a)

X_t $= 1.19$

f_t $= 11$

$A_{t,h}$ $= t_{ef,l}\, w_c$

$t_{ef,l}$ $= t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

$A_{t,h}$ $= a_2\,(n_R\text{-}1) = 7736$ mm^2

Wood failure – mode (c)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} X_s C_l\, f_s\, A_{s,l}, \text{Mode (a)} \\ 2X_t\, f_t t_{ef,l} a_{4c}\ , \text{Mode (b)} \end{cases}$$

C_l $= 0.32$

Calculate $P_{w,l}$:

$$P_{w,l} = (1+0.368^{-1}+0)10^{-3}\ \min = \begin{cases} 0.96\text{x}0.32\text{x}3.7\text{x}42438 \\ 2\text{x}1.19\text{x}11\text{x}44.2\text{x}163 \end{cases}$$

$$P_{w,l} = (1+0.368^{-1}+0)10^{-3}\ \min = \begin{cases} 48237, \text{mode (a)} \\ 188616, \text{mode (b)} \end{cases}$$

$P_{w,l}$ $= 180.6$ kN, mode (a)

Therefore, the parallel-to-grain wood capacity corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l}$ $= \Phi_w\, n_p\, k_1\, k_{12}\, k_f \min (P_{w,h}, P_{w,l})$

$\Phi_w Q_{we,l}$ $= 0.7 \times 2 \times 1.14 \times 1.0 \times 1.0 \times \min (138.5, 180.6)$

$= 220.6$ kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The residual head tensile plane and lateral shear planes resist lower load of 220.6 kN compared to wood capacity of 288.6 kN considering all resisting planes. Thus, $\Phi_w Q_{we,l} = 288.6$ kN.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

$0.190 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

$253.2 < 329.3$ OK

$Q_{s,l}$ $= \Phi_w Q_{we,l}$

$= 288.6$ kN

Joint design resistance, perpendicular to grain,

$$Q_s = Q_{s,p}$$

where

Check if $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$.

If so, then $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

Rivet capacity corresponding to yielding, perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ $= \Phi_r Q_{ry,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $h_{y,90}$ and $M_{ry,p}$, respectively

$= \Phi_r\, k_1\, k_{12}\, k_f\, n_p\, n_R\, n_C \min(P_{rp,a}, P_{rp,b})$

Rivet failure – mode (a)
Determine $P_{rp,a}$ using Section 4.1.3.2(i):

$$Q_{s,p} = \begin{cases} \phi_w Q_{we,p} \text{ if } \phi_w Q_{we,p} < \phi_r Q_{ry,p} & \text{(Brittle mode)} \\ \phi_r Q_{ry,p} \text{ if } \phi_w Q_{wy,p} < \phi_r Q_{ry,p} \leq \phi_w Q_{we,p} & \text{(Mixed mode)} \\ \phi_w Q_{wy,p} \text{ if } \phi_r Q_{ry,p} \leq \phi_w Q_{wy,p} < \phi_r Q_{ru,p} & \text{(Mixed mode)} \\ \phi_r Q_{ru,p} \text{ if } \phi_r Q_{ru,p} < \phi_w Q_{wy,p} & \text{(Ductile mode)} \end{cases}$$

$f_{h,90}$ $= f_{hy,90}$

$= 35.9\rho\,(1\text{-}0.0024 d_p)\, 10^{-3}$ for glulam (Section 4.1.3.3)

d_p = 6.4 mm

$f_{hy,90}$ = 16.6 MPa

$M_{r,p}$ $= M_{ry,p}$

= 12450 Nmm

Calculate $P_{rp,a}$:

$P_{rp,a}$ = 2.56 kN

Rivet failure – mode (b)
Determine $P_{rp,b}$ using Section 4.1.3.2(ii):

$P_{rp,b}$ = 2.29 kN

Rivet capacity corresponding to yielding, perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ $= \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rp,a}, P_{rp,b})$

= 0.8 x 1.14 x 1.0 x 2 x 9 x 8 x min (2.56, 2.29)

= 300.7 kN (Yielding mode of failure: b)

Wood design splitting resistance capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$

$\Phi_w Q_{we,p}$ $= \Phi_w Q_{w,p}$ in which $t_{ef,p}$ equals to $t_{efe,p}$

$= \Phi_w\, k_1\, g_{42}\, k_{12}\, k_f\, n_p \min(P_{s,a}, P_{s,b})$

Φ_w = 0.7

g_{42} = 1.0 for one joint

Determine characteristic full width splitting resistance, failure mode (a), $P_{s,a}$ using Section 4.1.5.1:

$$P_{s,a} = X_p \eta b C_{fp} \sqrt{\frac{h_e}{1-\frac{h_e}{h}}} 10^{-3}$$

X_p = 1.28 for glulam

γ = 2.7 for glulam

h_e = h - a_{4c}

h_e = 500-163 = 337 mm

w_{net} = a_1 (n_R - 1) – 6.4n_R

w_{net} = 222.4 mm

$a_{3c,L}$ = 200 mm

$a_{3c,R}$ = 2520 mm

η = 0.732

b = 135 mm

C_{fp} = 11.1 N/mm1.5 for GL10 Radiata Pine glulam

$P_{s,a}$ = 5.1 kN

Determine characteristic partial width splitting resistance, failure mode (b), $P_{s,b}$ using Section 4.1.5.2:

$$P_{s,b} = X_p C_t f_{tp} t_{ef,p} \left[w_{net} + \min(\beta h_e, a3c, L) + \min(\beta h_e, a3c, R) \right] 10^{-3}$$

$$\zeta = \frac{a_{4c}}{a_2(n_c - 1)}$$

ζ = 0.776

Ct = 1.388

f_{tp} = 1.19 MPa

$t_{ef,p}$ = $t_{efe,p}$ (Corresponding to rivet elastic deformation, Section 4.1.5.4)

= 39.2 mm

β = 1.6 for glulam

Calculate $P_{s,b}$

$P_{s,b}$ = 79.7 kN

Therefore, the wood design splitting resistance capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$:

$\Phi_w Q_{we,p}$ = $\Phi_w\, k_1\, k_f\, g_{42}\, n_p$ min ($P_{s,a}$, $P_{s,b}$)

= 0.7 x 1.14 x 1.0 x 1.0 x 2 x min (45.1, 79.7)

= 72.0 kN (governing failure, splitting width equal to member width, mode (b))

Check if $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

If $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$

72.0 < 300.7 OK

$Q_{s,p}$ = $\Phi_w Q_{we,p}$

= 72.0 kN

Ultimate rivet capacity at angle, θ=22°, to the grain, $\Phi_r Q_{ru,\theta}$

$$\phi_r Q_{ru,\theta} = \frac{\phi_r Q_{ru,l} \phi_r Q_{ru,p}}{\phi_r Q_{ru,l} \sin^2\theta + \phi_r Q_{ru,p} \cos^2\theta}$$

Ultimate rivet capacity, parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l}$ = $\Phi_r Q_{ru,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

$f_{h,0}$ = $f_{hu,0}$

= 86.7ρ (1-0.0024d_l) 10^{-3} for glulam (Section 4.1.3.3)

= 40.4 MPa

$M_{r,l}$ = $M_{ru,l}$

= 30,000 Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

= 444.9 kN (Rivet failure mode: a)

Ultimate rivet capacity, perpendicular to grain, $\Phi_r Q_{ru,p}$

$\Phi_r Q_{ru,p}$ = $\Phi_r Q_{r,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $f_{hu,90}$ and $M_{ru,p}$, respectively

$f_{h,90}$ = $f_{hu,90}$

= 43.3ρ (1-0.0024d_p)10^{-3} for glulam (Section 4.1.3.3)

= 20.0 MPa

$M_{r,p}$ = $M_{ru,p}$

= 15,000 Nmm

The rivet ultimate capacity, perpendicular to grain (by following the same design procedure as defined above):

$\Phi_r Q_{ru,p}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rp,a}$, $P_{rp,b}$)

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,p}$ = 354.8 kN (Rivet failure mode: b)

Therefore, ultimate rivet capacity at angle, θ=22°, to the grain, $\Phi_r Q_{ru,\theta}$:

$$\phi_r Q_{ru,\theta} = \frac{\phi_r Q_{ru,l} \phi_r Q_{ru,p}}{\phi_r Q_{ru,l} \sin^2\theta + \phi_r Q_{ru,p} \cos^2\theta}$$

$\Phi_r Q_{ru,p}$ = 429.7 kN

Check joint ultimate lateral resistance:

$N^* \leq Q_s$

$$N_\theta^* = \sqrt{N_l^2 + N_p^2}$$

$$N_\theta^* = \sqrt{120^2 + 50^2}$$

N^* = 130.0 kN

Q_s = $Q_{s,\theta}$

= min ($Q_{s,l}$/cosθ, $Q_{s,p}$/sinθ, $\Phi_r Q_{ru,\theta}$)

= min (288.6/cos22, 72.0/sin22, 429.7)

= min (311.3, 192.3, 429.7) = 192.3 kN

130.0 ≤ 192.3, OK *(Connection mode of failure: Brittle wood splitting – perpendicular to grain)*

Adopt 65 mm long rivets with an array of 8 rows by 9 columns, spacing 35 mm by 25 mm along and across the grain, and 200 mm end distance (see Figure 5.12).

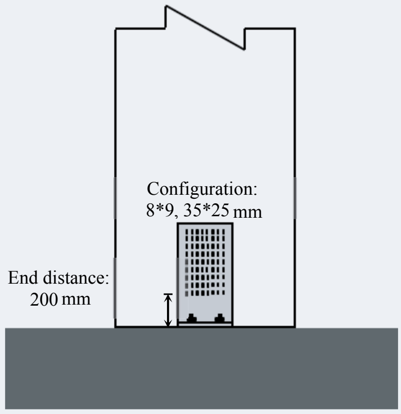


Figure 5.12: Connection configuration.

If ductile behaviour at the connection ultimate capacity is desirable, the failure mode of the connection can be improved from brittle/mixed to ductile by increasing the wood resistance with larger rivet spacing across and along the grain. Spreadsheets can be used to speed up the computation process, and once they are set up, adjustments in spacing, end and edge distances and capacities for a range of rivet lengths can be evaluated relatively quickly.

5.3 Moment Connection

To develop the moment in a beam splice, the rivet connection is arranged as shown in Figure 5.13. The rivet plates are installed on opposing faces of the dry wood member, which is of grade 11 Radiata Pine LVL. Shear is transferred by another scheme.

5.3.1 Design Actions

It is assumed that after taking the effect of the load duration factor (k_1) into account, the critical load combination for the connection design is [1.2G, 1.5Q], as per AS 1170. The design loads acting on the joint are shown in Figure 5.13. Two strength limit states are of interest: rivet strength and wood strength. An efficient connection design can be made by decreasing the difference between the capacity of the wood and the rivets.

Figure 5.13: Connection configuration.

5.3.2 Connection Geometry

Number of rows of rivets parallel to direction of load: $N_R = 5$

Number of rivets per row: $n_c = 10$

Spacing along the grain: $a_1 = 30$ mm

Spacing across the grain: $a_2 = 25$ mm

End distance: $a_{3t} = 150$ mm

Edge distance: $a_{4c} = 50$ mm (minimum of the edge distance and half the distance between adjacent joints)

Side plate thickness: $t_p = 10$ mm

The steel side plates should be checked to have a cross-section adequate for resisting tension and shear forces.

5.3.3 Connection Lateral Resistance

$Q_s = Q_{s,l}$

$$Q_{s,l} = \begin{cases} \phi_w Q_{we,l} \text{ if } \phi_w Q_{we,l} < \phi_r Q_{ry,l} & \text{(Brittle mode)} \\ \phi_r Q_{ry,l} \text{ if } \phi_w Q_{wy,l} < \phi_r Q_{ry,l} \leq \phi_w Q_{we,l} & \text{(Mixed mode)} \\ \phi_w Q_{wy,l} \text{ if } \phi_r Q_{ry,l} \leq \phi_w Q_{wy,l} < \phi_r Q_{ru,l} & \text{(Mixed mode)} \\ \phi_r Q_{ru,l} \text{ if } \phi_r Q_{ru,l} < \phi_w Q_{wy,l} & \text{(Ductile mode)} \end{cases}$$

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Rivet capacity corresponding to yielding – parallel to grain, $\Phi_r Q_{ry,l}$.

$\Phi_r Q_{ry,l}$ = $\Phi_r Q_{ry,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

$\Phi_r Q_{ry,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min $(P_{rl,a}, P_{rl,b})$

Φ_r = 0.8

k_1 = 0.77 for load combination [1.2G, 1.5Q]

k_f = 1.0 for face grain

n_p = 2

n_R = 5

n_C = 10

Rivet failure – mode (a)
Determine $P_{rl,a}$ using Section 4.1.3.1(i):

$$P_{rl,a} = X_r \left[J_p f_{h,0} L_p d_l \left(\left(\sqrt{2 + \frac{4M_{r,l}}{f_{h,0} d_l L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

X_r = 0.93 for LVL

J_p = 1.0 (side plate factor)

$f_{h,0}$ = $f_{hy,0}$

= 75.1(1-0.003dl) 10^{-3} for LVL (Section 4.1.3.3)

ρ = 620 kg/m^3 for grade 11 LVL

dl = 3.2 mm

$f_{hy,0}$ = L_r-t_p-3.2 MPa

L_p = 65 mm

$M_{r,l}$ = $M_{ry,l}$

= 24,900 Nmm

f_{ax} = 15.9ρ d_p(1-0.0024d_p)10^{-3}

= 61.6 N/mm

Calculate $P_{rl,a}$:

$P_{rl,a}$ = 4.11 kN

Rivet failure – mode (b)
Determine $P_{rl,b}$ using Section 4.1.3.1(ii):

$$P_{rl,b} = X_r \left[2J_p \sqrt{M_{r,l} f_{h,0} d_l} + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

$P_{rl,a}$ = 4.12 kN

Therefore, the rivet yield capacity parallel to grain,

$\Phi_r Q_{ry,l}$ $= \Phi_r\, k_1\, k_l\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

$= 0.8 \times 0.77 \times 1.0 \times 2 \times 5 \times 10 \min(4.11, 4.12)$

$\Phi_r Q_{ry,l}$ $= 253.0$ (Rivet Yielding mode: a)

Wood capacity, parallel to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

$= \Phi_w\, n_p\, k_1\, k_l \min(P_{w,h}, P_{w,b}, P_{w,l})$

Φ_w $= 0.7$

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ $= X_t\, f_t\, A_{t,h}\,(1+\lambda_1+\lambda_2)10^{-3}$

X_t $= 1.06$ for LVL

f_t $= 30$ MPa for grade 11 LVL

$A_{t,h}$ $= t_{ef,l}\, w_c$

$t_{ef,l}$ $= t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

$= 44.2$ mm

w_c $= a_2\,(n_R\text{-}1) = 100$ mm

$A_{t,h}$ $= a_2\,(n_R\text{-}1) = 4420\ \text{mm}^2$

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

where

$$\psi = \frac{G}{E}$$

G $= 550$ MPa for grade 11 LVL

E $= 11{,}000$ MPa for grade 11 LVL

Ψ $= 0.067$

L_c $= a_1\,(n_c\text{-}1) = 270$ mm

d_z $= 23.3$

H $= 0.54$

$A_{s,b}$ $= 42{,}000\ \text{mm}^2$

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_1 $= 0.378$

$$\lambda_2 = 0.25\psi L_c(1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_2 $= 0.427$

$A_{s,l}$ $= 2t_{ef,l}(L_c+a_{3t})$

$A_{s,l}$ $= 37{,}133\ \text{mm}^2$

F $= 0.36$

Calculate $P_{w,b}$:

$P_{w,h}$ $= 253.7$ kN, mode (a)

Wood failure – mode (b)
Determine $P_{w,b}$ using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3} \min = \begin{cases} X_s C_b f_s A_{s,b}, \text{Mode (a)} \\ X_t f_t w_c d_z, \text{Mode (c)} \end{cases}$$

X_s = 1.02 for LVL

C_b = 0.421

f_s = 6 MPa for grade 11 LVL

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l} w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 1.130

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3} \min = \begin{cases} 108214, \text{Mode (a)} \\ 74094, \text{Mode (c)} \end{cases}$$

$P_{w,b}$ = 354.0 kN, mode (c)

Wood failure – mode (a)
Determine $P_{w,l}$ using Section 4.1.4.3:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3} \min = \begin{cases} X_s C_l f_s A_{s,l}, \text{Mode (a)} \\ 2X_t f_t t_{ef,l} a_{4c}, \text{Mode (b)} \end{cases}$$

C_l = 0.337

Calculate $P_{w,l}$:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3} \min = \begin{cases} 76585, \text{Mode (a)} \\ 140556, \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 323.9 kN, mode (a)

Therefore, the wood capacity, parallel to grain, corresponding to rivet elastic deformation,

$\Phi_w Q_{we,l}$:

$\Phi_w Q_{we,l} = \Phi_w n_p k_1 k_f \min (P_{w,h}, P_{w,b}, P_{w,l})$

$\Phi_w Q_{we,l} = \Phi_w n_p k_1 k_f \min (253.7, 354.0, 323.9)$

= 273.4 kN (failure governed by head tensile plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})$-1 < 0.3

0.237 < 0.3

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

If $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$

273.4 ≥ 253.0 (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ry,l}$. If so, then $Q_{s,l} = \Phi_w Q_{ry,l}$ (mixed failure mode)

Wood capacity, parallel to grain, corresponding to rivet yielding mode, $\Phi_w Q_{wy,l}$

$\Phi_w Q_{wy,l} = \Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

$t_{ef,l} = t_{efy,l}$

$$t_{efy,p} = J_p \sqrt{\frac{M_{ry,p}}{f_{hy,90} d_p} + \frac{L_p^2}{2}}$$, yielding mode (a)

$t_{ef,l} = 38.9$ mm

The recalculated wood capacity (by following the same design procedure as defined above):

$\Phi_w Q_{wy,l} = 273.3$ kN

Note that if the yielding mode (b) was governing then the reduction of wood strength would be much higher.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})$-1 < 0.3

$0.208 < 0.3$

The connection does not need to be redesigned.

Check if $Q_{s,l} = \Phi_r Q_{ry,l}$ (brittle failure mode)

If $\Phi_w Q_{wy,l} < \Phi_r Q_{ry,l}$

$273.3 \geq 253.0$ (unsatisfied)

Thus, check if $\Phi_w Q_{wy,l} < \Phi_r Q_{ru,l}$. If so, then $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

Ultimate rivet capacity – parallel to grain, $\Phi_r Q_{ru,l}$

$\Phi_r Q_{ru,l} = \Phi_r Q_{r,l}$, in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

$f_{h,0} = f_{hu,0}$

$= 90.4\rho\,(1\text{-}0.0037 d_l)\, 10^{-3}$ for LVL (Section 4.1.3.3)

$= 55.4$ MPa

$M_{r,l} = M_{ru,l}$

$= 30{,}000$ Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l} = \Phi_r\, k_1\, k_l\, n_p\, n_R\, n_C \min(P_{rl,a}, P_{rl,b})$

$= 297.7$ kN (Rivet failure mode: a)

Check if $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

$273.3 \leq 297.7$ OK

Thus, $Q_{s,l} = \Phi_r Q_{ru,l}$ (ductile failure mode)

$Q_{s,l} = 283.9$ kN

Check joint ultimate lateral resistance:

$N^* \leq Q_s$

$N^* = M^*/0.410$

$M^* = 100$ kNm

$N^* = 243.9$ kN

$Q_s = Q_{s,l}$

$= 273.3$ kN

$243.9 \leq 273.3$, OK (Joint mode of failure: ductile)

Adopt 65 mm long rivets with an array of 5 rows by 10 columns, spacing 30 mm by 25 mm along and across the grain, and 150 mm end distance (see Figure 5.14).

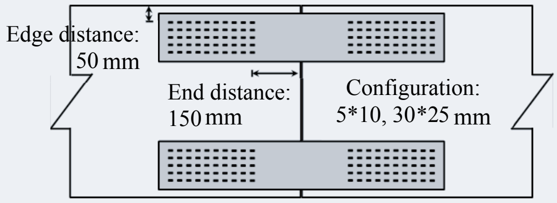


Figure 5.14: Connection configuration.

If ductile behaviour at the connection ultimate capacity is desirable, the failure mode of the connection can be improved from brittle/mixed to ductile by increasing the wood resistance with larger rivet spacing across and along the grain. Spreadsheets can be used to speed up computation and, once they are set up, adjustments in spacing, end and edge distances and capacities for a range of rivet lengths can be evaluated relatively quickly.

5.4 Hanger Connection

For a hanger joint to transfer load from a secondary beam to the primary beam, the rivet connection is arranged as shown in Figure 5.15. The secondary beam rivet plates are installed on opposing faces of the dry wood member, which is of grade GL 10 Radiata Pine glulam.

5.4.1 Design Actions

It is assumed that after taking the effect of the load duration factor (k_1) into account, the critical load combination for the connection design is [1.2G, 1.5Q], as per AS 1170. The design loads acting on the joint are shown in Figure 5.15. The design load acting on the secondary beams is assumed to be distributed evenly across the two joints in the hanger connection. Two strength limit states are of interest: rivet strength and wood strength. An efficient connection design can be made by decreasing the difference between the capacity of the wood and the rivets.

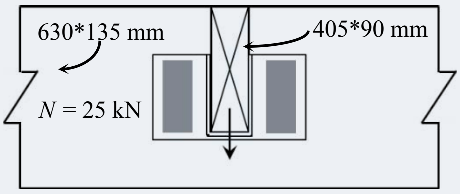


Figure 5.15: Design action.

5.4.2 Connection Geometry

Try 65 mm long rivets with the following configuration for each joint:

Number of rows of rivets parallel to direction of load: $N_R = 4$

Spacing along the grain: $a_1 = 30$ mm

Spacing across the grain: $a_2 = 60$ mm

Half the distance to the adjacent joint on the left side, end distance on the left side: $a_{3C,L} = 1{,}115$ mm (based on 2,600 mm spacing between secondary beams)

Half the distance to the adjacent joint on the right side, end distance on the right side: $a_{3c,R} = 95$ mm

Unloaded edge distance: $a_{4c} = 225$ mm

Loaded edge distance: $a_{4t} = 225$ mm

Side plate thickness: $t_p = 8$ mm

(The steel side plates need to be checked to have a cross-section adequate for resisting tension and shear forces).

5.4.3 Connection Lateral Resistance

$$Q_s = Q_{s,p}$$

where

$$Q_{s,p} = \begin{cases} \phi_w Q_{we,p} \text{ if } \phi_w Q_{we,p} < \phi_r Q_{ry,p} & \text{(Brittle mode)} \\ \phi_r Q_{ry,p} \text{ if } \phi_w Q_{wy,p} < \phi_r Q_{ry,p} \leq \phi_w Q_{we,p} & \text{(Mixed mode)} \\ \phi_w Q_{wy,p} \text{ if } \phi_r Q_{ry,p} \leq \phi_w Q_{wy,p} < \phi_r Q_{ru,p} & \text{(Mixed mode)} \\ \phi_r Q_{ru,p} \text{ if } \phi_r Q_{ru,p} < \phi_w Q_{wy,p} & \text{(Ductile mode)} \end{cases}$$

Check if $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$.

If so, then $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

Rivet capacity corresponding to yielding perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ $= \Phi_r Q_{ry,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $h_{y,90}$ and $M_{ry,p}$, respectively

$= \Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C \min(P_{rp,a}, P_{rp,b})$

Φ_r = 0.8

k_1 = 0.77 for load combination [1.2G, 1.5Q]

k_f = 1.0 for edge grain of glulam

n_p = 2

n_R = 4

n_C = 4

Rivet failure – mode (a)
Determine $P_{rp,a}$ using Section 4.1.3.2(i):

$$P_{rp,a} = X_r \left[J_p f_{h,90} L_p d_p \left(\left(\sqrt{2 + \frac{4M_{r,p}}{f_{h,90} d_p L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

X_r = 0.87 for glulam

J_p = 1.0

$f_{h,90}$ = $f_{hy,90}$

= $35.9\rho\,(1\text{-}0.0024 d_p)\, 10^{-3}$ for glulam (Section 4.1.3.3)

d_p = 6.4 mm

$f_{hy,90}$ = 16.6 MPa

$M_{r,p}$ = $M_{ry,p}$

= 12,450 Nmm

L_p = 51.8 mm

f_{ax} = $11.5\rho\, d_p(1\text{-}0.0024 d_p)\, 10^{-3}$

f_{ax} = 34.1 N/mm

Calculate $P_{rp,a}$:

$P_{rp,a}$ = 2.56 N

Rivet failure – mode (a)
Determine $P_{rp,b}$ using Section 4.1.3.2(ii):

$$P_{rp,b} = X_r \left[2J_p \sqrt{M_{r,p} f_{h,90} d_p} + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

$P_{rp,b}$ = 2.29 kN

Rivet capacity corresponding to yielding perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rp,a}$, $P_{rp,b}$)

= 0.8 x 0.77 x1.0 x 2 x 4 x 4 x min(2.56, 2.29) 10^{-3}

= 45.1 kN (Yielding mode of failure: b)

Wood capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$

$\Phi_w Q_{we,p}$ = $\Phi_w Q_{w,p}$ in which $t_{ef,p}$ equals to $t_{efe,p}$

= $\Phi_w\, k_1\, g_{42}\, k_f\, n_p$ min ($P_{s,a}$, $P_{s,b}$)

Φ_w = 0.7

g_{42} = 0.6 for multiple joints

Determine characteristic full width splitting resistance,
failure mode (a), $P_{s,b}$ using Section 4.1.5.1:

$$P_{s,a} = X_p \eta b C_{fp} \sqrt{\frac{h_e}{1 - \frac{h_e}{h}}} 10^{-3}$$

X_p = 1.28 for glulam

γ = 2.7 for glulam

h_e = h - a_{4c}

h_e = 630-225 = 405 mm

w_{net} = a_1 (n_R - 1) – 6.4n_R

w_{net} = 64.4 mm

$a_{3c,L}$ = 1,115 mm

$a_{3c,R}$ = 95 mm

$$\eta = \frac{\min(\gamma h_e, a_{3c,L}) + (\gamma h_e, a_{3c,R}) + w_{net}}{2\gamma h_e}$$

η = 0.573

b = 135 mm

C_{fp} = 11.1 N/mm$^{1.5}$ for Radiata Pine LVL

$P_{s,a}$ = 37.0 kN

Determine characteristic partial width splitting resistance,
failure mode (b), $P_{s,b}$ using Section 4.1.5.2:

$$P_{s,b} = X_p C_t f_{tp} t_{ef,p} \left[w_{net} + \min(\beta h_e, a3c, L) + \min(\beta h_e, a3c, R) \right] 10^{-3}$$

$$\zeta = \frac{a_{4c}}{a_2 (n_c - 1)}$$

ζ = 1.25

C_t = 1.164

f_{tp} = 1.19 MPa

$t_{ef,p}$ = $t_{efe,p}$ (Corresponding to rivet elastic deformation, Section 4.1.5.4)

= 39.2 mm

β = 1.6 for glulam

Calculate $P_{s,b}$:

$P_{s,b}$ = 56.1 kN

Therefore, the wood capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$:

$\Phi_w Q_{we,p} = \Phi_w\, k_1\, k_f\, g_{42}\, n_p \min(P_{s,a}, P_{s,b})$

$= 0.7 \times 0.77 \times 0.6 \times 1.0 \times 2 \times \min(37.0, 56.1)$

= 24.9 kN (governing failure, splitting with crack width equal to member thickness, mode (a))

Check if $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

If $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$

$24.9 < 45.1$ OK

$Q_{s,p} = \Phi_w Q_{we,p}$

= 24.9 kN

Check joint ultimate lateral resistance:

$N^* \leq Q$

N^* = 25 kN

$Q_s = Q_{s,p}$

= 24.9 kN

25 ≈ 24.9, OK (governing failure, splitting with crack width equal to member thickness, mode (a))

Adopt 65 mm long rivets with an array of 4 rows by 4 columns, spacing 30 mm by 60 mm along and across the grain, and 50 mm end distance (see Figure 5.16).

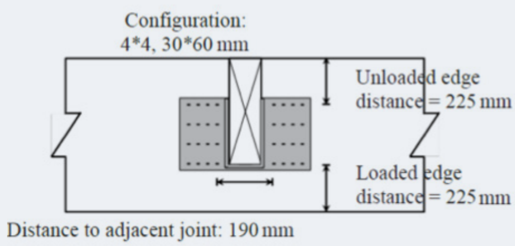


Figure 5.16: Connection configuration.

If ductile behaviour at the connection ultimate capacity is desirable, the failure mode of the connection can be improved from brittle/mixed to ductile by increasing the wood resistance with larger rivet spacing across and along the grain. Spreadsheets can be used to speed up computation and, once they are set up, adjustments in spacing, end and edge distances and capacities for a range of rivet lengths can be evaluated relatively quickl

5.5 Shear Wall Connections

For a shear wall to resist loads during a design earthquake, a rivet hold-down connection and floor–wall connection are arranged as shown in . The rivet plates are installed on opposing faces of the dry wood member, which is of grade 11 Radiata Pine LVL.

5.5.1 Design Actions

It is assumed that after taking the effect of the load duration factor (k_1) into account, the critical load combination for the connection design is [G, Ψ_cQ, E_u] as per AS 1170. The design loads acting on the joint are shown in Figure 5.17.

Two strength limit states are of interest: rivet strength and wood strength. To achieve a targeted system ductility at design drift under ultimate limit state (ULS), the slip of the rivets in the hold-down and floor–wall connections are required to be less than 2 mm and 3 mm, respectively. An efficient connection design can be made by decreasing the difference between the capacity of the wood and the rivets; however the deflection requirements may be critical for the rivet capacity.

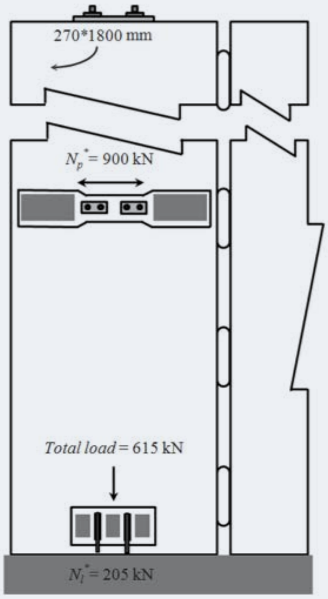


Figure 5.17: Design action.

The design load acting on the energy dissipaters in the hold-down connection is assumed to be distributed evenly across the three rivet joints. The rivet configuration in the floor–wall connection acts as one joint.

5.5.2 Hold-Down Connection

5.5.2.1 Connection geometry

Try 65 mm long rivets with the following configuration for each joint:

$N^* = 205.0$ kN

Number of rows of rivets parallel to direction of load: $N_R = 6$

Number of rivets per row: $n_C = 6$

Spacing along the grain: $a_1 = 30$ mm

Spacing across the grain: $a_2 = 30$ mm

Side plate thickness: $t_p = 8$ mm

(The steel side plates need to be checked to have a cross-section adequate
for resisting tension and shear forces).

End distance: $a_{3t} = 150$ mm

Edge distance: $a_{4c} = 100$ mm (minimum of edge distance and half the distance to an adjacent joint)

The rivet groups on either side of the energy dissipater are considered multiple joints acting parallel to grain and the connection is designed in accordance with Section 4.1.4.5.

5.5.2.2 Connection Lateral Resistance

$$Q_s = Q_{s,l} = \begin{cases} \phi_w Q_{we,l} \text{ if } \phi_w Q_{we,l} < \phi_r Q_{ry,l} & \text{(Brittle mode)} \\ \phi_r Q_{ry,l} \text{ if } \phi_w Q_{wy,l} < \phi_r Q_{ry,l} \leq \phi_w Q_{we,l} & \text{(Mixed mode)} \\ \phi_w Q_{wy,l} \text{ if } \phi_r Q_{ry,l} \leq \phi_w Q_{wy,l} < \phi_r Q_{ru,l} & \text{(Mixed mode)} \\ \phi_r Q_{ru,l} \text{ if } \phi_r Q_{ru,l} < \phi_w Q_{wy,l} & \text{(Ductile mode)} \end{cases}$$

Check if $\Phi_w Q_{we,l} < \Phi_r Q_{ry,l}$.

If so, then $Q_{s,l} = \Phi_w Q_{we,l}$ (brittle failure mode)

Rivet capacity corresponding to yielding parallel to grain, $\Phi_r Q_{ry,l}$

$\Phi_r Q_{ry,l}$ = $\Phi_r Q_{ry,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hy,0}$ and $M_{ry,l}$, respectively

$\Phi_r Q_{ry,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

Φ_r = 0.8

k_1 = 1.14 for load combination [0.9G, Wu]

k_f = 1.0 for edge grain of glulam

n_p = 2

n_R = 6

n_C = 6

Rivet failure – mode (a)
Determine $P_{rl,a}$ using Section 4.1.3.1(i):

$$P_{rl,a} = X_r\left[J_p f_{h,0} L_p d_l\left(\left(\sqrt{2+\frac{4M_{r,l}}{f_{h,0}d_l L_p^2}}\right)-1\right)+\frac{L_p f_{ax}}{5.33}\right]10^{-3}$$

Xr = 0.93 for LVL

Jp = 1.0

$f_{h,0}$ = $f_{hy,0}$

= 75.1ρ (1-0.0037d_l) 10^{-3} for LVL (Section 4.1.3.3)

ρ = 620 kg/m^3 for grade 11 LVL

d_l = 3.2 mm

$f_{hy,0}$ = 46.0 MPa

t_p = 8 mm

L_p = 53.8 mm

$M_{r,p}$ = $M_{ry,p}$

= 24,900 Nmm

f_{ax} = 15.9ρd_p(1-0.0037d_p) 10^{-3}

d_p = 6.4

f_{ax} = 61.6 N/mm

Calculate $P_{rl,a}$:

$P_{rl,a}$ = 4.22 N

Rivet failure – mode (b)
Determine $P_{rl,b}$ using Section 4.1.3.1(ii):

$$P_{rl,b} = X_r\left[2J_p\sqrt{M_{r,l} f_{h,0} d_l}+\frac{L_p f_{ax}}{5.33}\right]10^{-3}$$

$P_{rl,b}$ = 4.14 kN

Therefore, the rivet yield capacity parallel to grain,

$\Phi_r Q_{ry,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

$\Phi_r Q_{ry,l}$ = 271.8 (Rivet Yielding mode: b)

Wood capacity, parallel to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,l}$

$\Phi_w Q_{we,l}$ = $\Phi_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efe,l}$

= $\Phi_w\, n_p\, k_1\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

Φ_w = 0.7

Wood failure – mode (a)
Determine $P_{w,h}$ using Section 4.1.4.1:

$P_{w,h}$ = $X_t\, f_t\, A_{t,h}\, (1+\lambda_1+\lambda_1)10^{-3}$

X_t = 1.06 for LVL

f_t = 30 MPa for grade 11 LVL

$A_{t,h}$ = $t_{ef,l}\, w_c$

$t_{ef,l}$ = $t_{efe,l}$ (Corresponding to rivet elastic deformation, Section 4.1.4.4)

= 45.7 mm

w_c = $a_2\,(n_R$-1) = 150 mm

$A_{t,h}$ = $a_2\,(n_R$-1) = 6855 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

where

$$\psi = \frac{G}{E}$$

G = 550 MPa for grade 11 LVL

E = 11,000 MPa for grade 11 LVL

Ψ = 0.05

L_c = $a_1(n_C$-1) = 150 mm

d_z = 89.3

H = 0

$A_{s,b}$ = 45,000 mm^2

$$\lambda_1 = 0.25\psi L_c(1-H)\left[\frac{A_{s,b}}{t_{ef,l}A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

λ_1 = 0.369

$$\lambda_2 = 0.25\psi L_c(1-F)\left[\frac{A_{s,l}}{w_c A_{t,h}} + \frac{0.4}{\psi L_c}\right]$$

F = 0.220

$A_{s,l}$ = 27,419 mm^2

λ_2 = 0.156

Calculate $P_{w,h}$:

$P_{w,h}$ = 332.5 kN, mode (a)

Wood failure – mode (b)
Determine $P_{w,b}$ using Section 4.1.4.2:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} X_s C_b\ f_s\ A_{s,b}, \text{Mode (a)} \\ X_t\ f_t w_c\ d_z\ , \text{Mode (c)} \end{cases}$$

X_s = 1.02 for LVL

C_b = 0.42

f_s = 6.0 MPa for grade 11 LVL

$$\lambda_3 = \frac{t_{ef,l}(1-F)}{w_c(1-H)}\left[\frac{5\psi L_c A_{s,l} + t_{ef,l}w_c^2}{2.5\psi L_c A_{s,b} + w_c t_{ef,l}^2}\right]$$

λ_3 = 0.422

Calculate $P_{w,b}$:

$$P_{w,b} = (1+\lambda_1^{-1}+\lambda_3)10^{-3}\ \min = \begin{cases} 115668, \text{Mode (a)} \\ 425961\ , \text{Mode (c)} \end{cases}$$

C_l = 0.33

Calculate $P_{w,l}$:

$$P_{w,l} = (1+\lambda_2^{-1}+\lambda_3^{-1})10^{-3}\ \min = \begin{cases} 55375, \text{Mode (a)} \\ 290652, \text{Mode (b)} \end{cases}$$

$P_{w,l}$ = 545.5 kN, mode (a)

Therefore, the wood capacity, parallel to grain, corresponding to rivet elastic deformation, $\boldsymbol{\Phi}_w Q_{we,l}$:

$\boldsymbol{\Phi}_w Q_{we,l}$ = $\boldsymbol{\Phi}_w\, n_p\, k_1\, k_f$ min ($P_{w,h}$, $P_{w,b}$, $P_{w,l}$)

= 0.7 x 2 x 1.14 x 1.0 x min (332.5, 395.2, 454.6) = 530.7 kN
(failure governed by bottom shear plane, mode (a))

Wood failure mode (a) governs failure, therefore no recalculation is required (Section 4.1.4). The wood capacity involves all resisting planes.

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

0.103 < 0.3

The connection does not need to be redesigned.

Check if $Q_{s,l} = \boldsymbol{\Phi}_w Q_{we,l}$ (brittle failure mode)

If $\boldsymbol{\Phi}_w Q_{we,l} < \boldsymbol{\Phi}_r Q_{ry,l}$

530.7 ≥ 271.8 (unsatisfied)

Thus, check if $\boldsymbol{\Phi}_w Q_{wy,l} < \boldsymbol{\Phi}_r Q_{ry,l}$.

If so, then $Q_{s,l} = \boldsymbol{\Phi}_w Q_{ry,l}$ (mixed failure mode)

Wood capacity, parallel to grain, corresponding to rivet yielding mode, γwQwy,l

$\boldsymbol{\Phi}_w Q_{wy,l} = \boldsymbol{\Phi}_w Q_{w,l}$ in which $t_{ef,l}$ equals to $t_{efy,l}$

$t_{ef,l}$ = $t_{efy,l}$

$$t_{efy,p} = J_p \sqrt{\frac{M_{ry,p}}{f_{hy,90} d_p} + \frac{L_p^2}{2}}$$, yielding mode (a)

$t_{ef,l}$ = 26.0 mm

The recalculated wood capacity (by following the same design procedure as defined above):

$\boldsymbol{\Phi}_w Q_{wy,l}$ = 410.5 kN

Note that if the yielding mode (a) governs failure, therefore no recalculation is required (Section 4.1.4).

Check that the contribution of the lateral shear planes is less than 30% of the total joint capacity:

$(1+\lambda_2^{-1}+\lambda_3^{-1})^{-1} < 0.3$

0.2075 < 0.3

The connection does not need to be redesigned.

Check if Check if $Q_{s,l} = \boldsymbol{\Phi}_r Q_{ry,l}$ (brittle failure mode)

If $\boldsymbol{\Phi}_w Q_{wy,l} < \boldsymbol{\Phi}_r Q_{ry,l}$

410.5 ≥ 271.8 (unsatisfied)

Thus, check if $\boldsymbol{\Phi}_w Q_{wy,l} < \boldsymbol{\Phi}_r Q_{ru,l}$. If so, then $Q_{s,l} = \boldsymbol{\Phi}_w Q_{wy,l}$ (mixed failure mode)

Ultimate rivet capacity – parallel to grain, $\boldsymbol{\Phi}_r Q_{ru,l}$

$\Phi_r Q_{ru,l}$ = $\Phi_r Q_{r,l}$ in which $f_{h,0}$ and $M_{r,l}$ equal to $f_{hu,0}$ and $M_{ru,l}$, respectively

$f_{h,0}$ = $f_{hu,0}$

= 90.4ρ (1-0.0037d_l) 10^{-3} for LVL (Section 4.1.3.3)

= 55.4 MPa

$M_{r,l}$ = $M_{ru,l}$

= 30,000 Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,l}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rl,a}$, $P_{rl,b}$)

= 319.5 kN (Rivet failure mode: a)

Check if $Q_{s,l} = \Phi_w Q_{wy,l}$ (mixed failure mode)

If $\Phi_w Q_{wy,l} \leq \Phi_r Q_{ru,l}$

410.5 > 319.5 (unsatisfied)

Thus, $Q_{s,l} = \Phi_r Q_{ru,l}$ (ductile failure mode)

$Q_{s,l}$ = 319.5 kN

Check joint ultimate lateral resistance:

N^* $\leq Q_s$

N^* = 205 kN

Q_s = $Q_{s,l}$

Q_s = 319.5

205.0 < 319.5, ok (Joint mode of failure ductile)

Joint Deflection – parallel to grain

The joint deflection must not exceed 2 mm

δ_l $\leq$ 2 mm

$$\delta_l = 4\left[1 - \sqrt{1 - \frac{N_l^*}{\phi_r Q_{ru,l}}}\right]$$

$$\delta_l = 4\left[1 - \sqrt{1 - \frac{205}{319.5}}\right]$$

where

N^*_1 = ultimate design load parallel to grain, kN, for deflection under (ULS)

δ_l = 1.60 mm

1.60 $\leq$ 2 mm, OK

Adopt 65 mm long rivets with an array of 6 rows by 6 columns, spacing 30 mm by 30 mm along and across the grain, and 150 mm end distance spaced at 200 mm on either side of the energy dissipaters (see Figure 5.18).

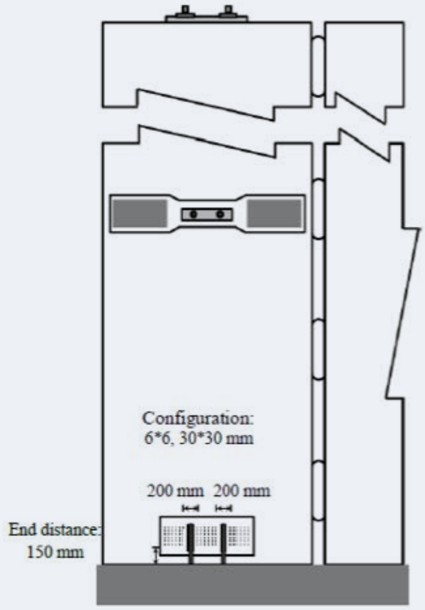


Figure 5.18: Connection configuration of the hold down connection.

5.5.3 Floor–Wall Connection

5.5.3.1 Connection geometry

Try 65 mm long rivets with the following configuration for each joint:

Side plate thickness: $t_p = 8$ mm

(The steel side plates need to be checked to have a cross-section adequate for resisting tension and compression forces).

$N^* = 1{,}200$ kN

Number of rows of rivets parallel to direction of load: $N_r = 8$

Number of rivets per row: $n_c = 30$ (as two 8*15 rivet joints either side of the connection)

Spacing along the grain: $a_1 = 30$ mm

Spacing across the grain: $a_2 = 30$ mm

Unloaded end distance on the left side: $a_{3C,L} = 3{,}450$ mm

Unloaded end distance on the right side: $a_{3c,R} = 3{,}450$ mm

Unloaded edge distance: $a_{4c} = 25$ mm

Loaded edge distance: $a_{4t} = 25$ mm

The two rivet groups are located along the load direction and are connected by the steel plate. There is possibility of splitting only at the unloaded edge of the steel plate. No splitting is possible between the two groups of rivets.

5.5.3.2 Connection lateral resistance

$Q_s = Q_{s,p}$

where

$$Q_{s,p} = \begin{cases} \phi_w Q_{we,p} \text{ if } \phi_w Q_{we,p} < \phi_r Q_{ry,p} & \text{(Brittle mode)} \\ \phi_r Q_{ry,p} \text{ if } \phi_w Q_{wy,p} < \phi_r Q_{ry,p} \leq \phi_w Q_{we,p} & \text{(Mixed mode)} \\ \phi_w Q_{wy,p} \text{ if } \phi_r Q_{ry,p} \leq \phi_w Q_{wy,p} < \phi_r Q_{ru,p} & \text{(Mixed mode)} \\ \phi_r Q_{ru,p} \text{ if } \phi_r Q_{ru,p} < \phi_w Q_{wy,p} & \text{(Ductile mode)} \end{cases}$$

if $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$.

If so, then $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

Rivet capacity corresponding to yielding perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ = $\Phi_r Q_{ry,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $h_{y,90}$ and $M_{ry,p}$, respectively

= $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min $(P_{rp,a}, P_{rp,b})$

Φ_r = 0.8

k_1 = 0.77 Load duration factor in AS 1720.1 for load combination [1.2G, 1.5Q]

k_f = 1.0 for LVL

n_p = 2

n_R = 8

n_C = 30

Rivet failure – mode (a)
Determine P_{rp},a using Section 4.1.3.2(i):

$$P_{rp,a} = X_r\left[J_p f_{h,90} L_p d_p\left(\left(\sqrt{2+\frac{4M_{r,p}}{f_{h,90} d_p L_p^2}}\right)-1\right)+\frac{L_p f_{ax}}{5.33}\right]10^{-3}$$

X_r = 0.93 for LVL

J_p = 1.0

$f_{h,90}$ = $f_{hy,90}$

= $49.9\rho\,(1\text{-}0.0037d_p)\,10^{-3}$ for LVL (Section 4.1.3.3)

d_p = 6.4 mm

ρ = 620 kg/m^3 for grade 11 LVL

$f_{hy,90}$ = 30.2 MPa

L_p = 28.8 mm

$M_{r,p}$ = $M_{ry,p}$

= 12,450 Nmm

f_{ax} = $15.9\rho d_p(1\text{-}0.0037d_p)\,10^{-3}$

f_{ax} = 61.6 N/mm

Calculate $P_{rp,a}$:

$P_{rp,a}$ = 4.89 kN

5.5.3.2 Connection lateral resistance

$Q_s = Q_{s,p}$

where

$$Q_{s,p} = \begin{cases} \phi_w Q_{we,p} \text{ if } \phi_w Q_{we,p} < \phi_r Q_{ry,p} & \text{(Brittle mode)} \\ \phi_r Q_{ry,p} \text{ if } \phi_w Q_{wy,p} < \phi_r Q_{ry,p} \le \phi_w Q_{we,p} & \text{(Mixed mode)} \\ \phi_w Q_{wy,p} \text{ if } \phi_r Q_{ry,p} \le \phi_w Q_{wy,p} < \phi_r Q_{ru,p} & \text{(Mixed mode)} \\ \phi_r Q_{ru,p} \text{ if } \phi_r Q_{ru,p} < \phi_w Q_{wy,p} & \text{(Ductile mode)} \end{cases}$$

if $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$.

If so, then $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

Rivet capacity corresponding to yielding perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ = $\Phi_r Q_{ry,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $h_{y,90}$ and $M_{ry,p}$, respectively

= $\Phi_r \, k_1 \, k_f \, n_p \, n_R \, n_C$ min ($P_{rp,a}$, $P_{rp,b}$)

Φ_r = 0.8

k_1 = 0.77 Load duration factor in AS 1720.1 for load combination [1.2G, 1.5Q]

k_f = 1.0 for LVL

n_p = 2

n_R = 8

n_C = 30

Rivet failure – mode (a)
Determine P_{rp},a using Section 4.1.3.2(i):

$$P_{rp,a} = X_r \left[J_p f_{h,90} L_p d_p \left(\left(\sqrt{2 + \frac{4M_{r,p}}{f_{h,90} d_p L_p^2}} \right) - 1 \right) + \frac{L_p f_{ax}}{5.33} \right] 10^{-3}$$

X_r = 0.93 for LVL

J_p = 1.0

$f_{h,90}$ = $f_{hy,90}$

= $49.9\rho\,(1\text{-}0.0037d_p)\,10^{-3}$ for LVL (Section 4.1.3.3)

d_p = 6.4 mm

ρ = 620 kg/m^3 for grade 11 LVL

$f_{hy,90}$ = 30.2 MPa

L_p = 28.8 mm

$M_{r,p}$ = $M_{ry,p}$

= 12,450 Nmm

f_{ax} = $15.9\rho d_p(1\text{-}0.0037d_p)\,10^{-3}$

f_{ax} = 61.6 N/mm

Calculate $P_{rp,a}$:

$P_{rp,a}$ = 4.89 kN

Rivet failure – mode (a)
Determine $P_{rp,b}$ using Section 4.1.3.2(ii):

$$P_{rp,b} = X_r\left[2J_p\sqrt{M_{r,p}f_{h,90}d_p} + \frac{L_p f_{ax}}{5.33}\right]10^{-3}$$

$P_{rp,b}$ = 3.46 kN

Rivet capacity corresponding to yielding perpendicular to grain, $\Phi_r Q_{ry,p}$

$\Phi_r Q_{ry,p}$ = $\Phi_r\, k_1\, k_f\, n_p\, n_R\, n_C$ min ($P_{rp,a}$, $P_{rp,b}$)

= 0.8 x 0.77 x 1.0 x 2 x 8 x 30 x min(4.86, 3.46) 10^{-3}

= 1024.2 kN (Yielding mode of failure: b)

Wood capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$

$\Phi_w Q_{we,p}$ = $\Phi_w Q_{w,p}$ in which $t_{ef,p}$ equals to $t_{efe,p}$

= $\Phi_w\, k_1\, g_{42}\, k_f\, n_p$ min ($P_{s,a}$, $P_{s,b}$)

Φ_w = 0.7

g_{42} = 1

Determine characteristic full width splitting resistance,
failure mode (b), Ps,a using Section 4.1.5.1:

$$P_{s,a} = X_p \eta b C_{fp}\sqrt{\frac{h_e}{1-\frac{h_e}{h}}}10^{-3}$$

X_p = 1.23 for LVL

γ = 2.7 for LVL

h_e = h - a4c

h_e = 1800-30 = 1770 mm

w_{net} = a_1 (n_R - 1) – 6.4n_R

w_{net} = 158.8 mm

$a_{3c,L}$ = 3450 mm

$a_{3c,R}$ = 3450 mm

$$\eta = \frac{\min(\gamma h_e, a_{3c,L}) + (\gamma h_e, a_{3c,R}) + w_{net}}{2\gamma h_e}$$

η = 0.499

b = 270 mm

C_{fp} = 16 N/mm$^{1.5}$ for LVL

$P_{s,a}$ = 863.2 kN

Determine characteristic partial width splitting resistance,
failure mode (b), Ps,b using Section 4.1.5.2:

$$P_{s,b} = X_p C_t f_{tp} t_{ef,p}\left[w_{net} + \min(\beta h_e, a3c, L) + \min(\beta h_e, a3c, R)\right]10^{-3}$$

$$\zeta = \frac{a_{4c}}{a_2(n_c - 1)}$$

$$n_{Cef} = n_C + \frac{L_{gap}}{a_2} - 1$$

(Just for defining the wood capacity)

L_{gap} = 900 mm

n_{Cef} = 59

ζ = 0.017

C_t = 5.678

f_{tp} = 1.45 MPa

$t_{ef,p}$ = $t_{efe,p}$ (Corresponding to rivet elastic deformation, Section 4.1.5.4)

= 40.3 mm

B = 2.4 for LVL

Calculate $P_{s,b}$:

$P_{s,b}$ = 2879.7 kN

Therefore, the wood capacity, perpendicular to grain, corresponding to rivet elastic deformation, $\Phi_w Q_{we,p}$:

$\Phi_w Q_{we,p}$ = $\Phi_w\, k_1\, k_f\, g_{42}\, n_p$ min ($P_{s,a}$, $P_{s,b}$)

= 0.7 x 0.77 x 1.0 x 1.0 x 2 x min (863.2, 2879.7)

=930.5 kN (governing failure, splitting with crack width equal to member thickness, mode (a))

Check if $Q_{s,p} = \Phi_w Q_{we,p}$ (brittle failure mode)

If $\Phi_w Q_{we,p} < \Phi_r Q_{ry,p}$

930.5 < 1024.2 OK

$Q_{s,p}$ = $\Phi_w Q_{we,p}$

= 930.5 kN

Check joint ultimate lateral resistance:

N^* ≤ Q

N^* = 900 kN

Q_s = $Q_{s,p}$

= 930.5 kN

900 ≤ 930.5, OK (governing failure, splitting with crack width equal to member thickness, mode (a))

Ultimate rivet capacity – perpendicular to grain, $\Phi_r Q_{ru,p}$

$\Phi_r Q_{ru,p}$ = $\Phi_r Q_{r,p}$ in which $f_{h,90}$ and $M_{r,p}$ equal to $f_{hu,90}$ and $M_{ru,p}$, respectively

$f_{h,90}$ = $f_{hu,90}$

= $60.2\rho(1\text{-}0.0037d_p)10^{-3}$ for LVL (Section 4.1.3.3)

$M_{r,p}$ = $M_{ru,p}$

= 15,000 Nmm

The recalculated rivet ultimate capacity (by following the same design procedure as defined above):

$\Phi_r Q_{ru,p}$ = 1199.5 kN (Rivet failure mode: b)

Joint deflection – perpendicular to grain

The joint deflection must not exceed 3 mm

δ_p ≤ 3 mm

$$\delta_p = 5.5\left[1 - \sqrt{1 - 0.99\frac{N_p^*}{\phi_r Q_{ru,p}}}\right]$$

$$\delta_p = 5.5\left[1 - \sqrt{1 - 0.99\frac{900}{1199.5}}\right]$$

δ_p = 2.71 ≤ 3 mm, OK

Adopt 65 mm long rivets with an array of 8 rows by 15 columns, spacing 30 mm by 30 mm along and across the grain, the two arrays will be spaced 900 mm apart with 30 mm loaded and unloaded edge distances, 3450 mm end distances on to the left and right side (see Figure 5.19).

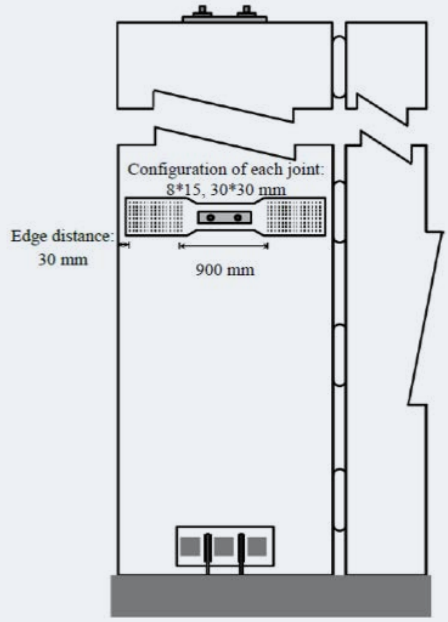


Figure 5.19: Connection configuration of the floor–wall connection.

References

AS 1170.1, *Structural design actions, in Permanent, imposed and other actions.* 2002, Australian Standard™.

AS 1720.1, *Timber structures, in Part 1: Design methods*. 2010, Standards Australia: Australia.

Buchanan, A., *Timber design guide New Zealand Timber Industry Federation Inc*. Wellington, New Zealand, 2007.

Buchanan, A.H. and J.C. Lai, *Glulam rivets in radiata pine*. Canadian Journal of Civil Engineering, 1994. 21(2): p. 340-350.

CSA086-01, *Engineering Design of Wood*. 2001, Canadian Standard Association: Ontario, Canada.

EN 1995-1-1:2004, *Eurocode 5-Design of Timber Structures-Part 1-1 in General rules and rules for buildings.* 2004, European Committee for Standardization: Brussels, Belgium.

Foschi, R.O. and J. Longworth, *Analysis and design of griplam nailed connections*. Journal of the Structural Division, 1975. 101(12): p. 2537-2555.

Foschi, R.O. and W.F.P. Laboratory, *Stress Analysis and Design of Glulam Rivet Connections for Perpendicular-to-grain Loading of Wood*. 1973: Western Forest Products Laboratory.

Madsen, B., *Behaviour of timber connections.* Canadian Journal of Civil Engineering, 2001. 28(3): p. 546.

McGowan, W. and B. Madsen, *A Rigid Field Joint for Glued-Laminated Construction*. Cooperative Study Report by the Forest Products Laboratory, Vancouver, and Research Committee of the Canadian Institute of Timber Construction, Vancouver: Forest Products Laboratory, 1965.

NDS, A.A., *National Design Specification (NDS), in National design specification for wood construction.* 2012, American Wood Council: Washington, DC.

Popovski, M., H.G. Prion, and E. Karacabeyli, *Seismic performance of connections in heavy timber construction.* Canadian Journal of Civil Engineering, 2002. 29(3): p. 389-399.

Racher, P., *Mechanical timber joints – General, in Timber Engineering Volume 1, Lecture C1.* 1994: Centrum Hout, Almere, The Netherlands.

Smith, I. and G. Foliente, *Load and Resistance Factor Design of Timber Joints: International Practice and Future Direction*. Journal of Structural Engineering, 2002. 128(1): p. 48-59.

Stahl, D.C., R.W. Wolfe, and M. Begel, *Improved analysis of timber rivet connections*. Journal of Structural Engineering, 2004. 130(8): p. 1272-1279.

Williams, C., *Timber rivets*. NZ Timber Design Journal, 2006. 16(2): p. 1-5.

WoodSolutions Technical Design Guide *#5: Timber service life design*. 2007, Forest & Wood Products Australia, Melbourne, Australia.

Zarnani, P., *Load-carrying capacity and failure mode analysis of timber rivet connections.* 2013, ResearchSpace@ Auckland.

A

Appendix A - Case Studies, Reference Capacity Tables, Adjustment Factors

A1 Case Studies

A1.1 Carterton Event Centre

Timber rivets were used for the first time in New Zealand in 2011, in the timber truss connections of the Carterton Event Centre (Figure 1.4). The trusses were constructed using LVL members supplied by Juken New Zealand Ltd. The auditorium trusses in the building were up to 24.6m long and 4.8m high, so each was delivered to the site in two pieces. The riveted connection used easily managed components that allowed the required mid-span splice connection to be completed without specialist lift equipment.

Use of the rivets allowed the fabricator, McIntosh Timber Laminates, to save more than $30,000 on this project when compared to the detailed bolted connection option. The rivets also allowed for adjustments on-site that would not have been possible with bolted fastenings. The timber rivets were found very user friendly; both in the work shop and onsite, with significantly less visual impact and material cost compared to the conventional fasteners.

A1.2 Trimble Building in Christchurch

In another project by TimberLab Solutions Ltd. (formerly McIntosh Timber Laminates Ltd.), the use of rivets in the connections of the structure and energy dissipating system of the building (Figure 1.5) demonstrates the advantages of this timber fastener.

The Trimble building in south-west Christchurch was damaged by the September 2010 and February 2011 earthquakes. The new building was a design-build project, which was undertaken originally by Mainzeal and Opus, commencing in February 2012.

The building holds more than 6,000 m^2 of office space over two levels and utilises LVL Post tension frames in one direction and post tension walls in the other to resist seismic loads (Refer to WoodSolutions Design Guide for information of post tension timber beams and walls). The LVL for this building was supplied by Carter Hold Harvey Ltd.

The principal structural engineer from Opus and the structural design team leader for the Trimble project found that the compact timber rivets provided high strength and stiff connections to take significant seismic loads.

Table A2.1: Reference design capacity (kN) for a double-sided joint, loaded parallel to grain, using Radiata Pine LVL grade 11 (spacing a_1=25 mm; a_2=25 mm).

Rivet Length, L_r (mm)	Member Thickness, *b* (mm)	Rivets per a row n_C	Number of rows n_R				
			6	8	10	12	14
40	90	6	199	265	331	397	464
		8	264	353	441	530	618
		10	241	327	415	504	593
		12	206	278	351	426	501
		14	186	249	314	380	447
	135	6	199	265	331	397	464
		8	234	321	409	497	586
		10	256	350	444	540	636
		12	267	362	460	559	658
		14	283	384	487	591	695
65	135	6	278	371	464	557	650
		8	316	482	543	659	775
		10	363	482	607	733	861
		12	408	533	665	800	957
		14	462	593	735	881	1029
	180	6	278	371	464	557	650
		8	347	471	597	725	854
		10	397	534	675	817	961
		12	469	619	775	935	1096
		14	533	694	864	1040	1217
90	180	6	253	337	428	521	625
		8	337	449	562	674	786
		10	421	562	702	842	983
		12	505	657	816	980	1147
		14	590	708	870	1039	1212
	225	6	253	337	428	521	615
		8	337	449	562	674	786
		10	421	449	562	674	786
		12	505	674	842	1011	1179
		14	590	780	963	1153	1146

Notes:

(1) Member depth is assumed to be two times the joint depth, $h = 2 \times a_2(n_R - 1)$.

(2) The joint is located at the centre of the member.

(3) End distance is considered based on minimum requirements.

(4) The value of k_1, k_{12} and k_t factors are assumed to be equal to one.

(5) Steel plate is considered to be 10 mm thick.

(6) Ultimate design capacity of 40, 65 and 90 mm long rivets under ductile failure is calculated as 2.76, 3.87 and 4.09 kN per rivet, respectively.

Table A1.2: Reference design capacity (kN) for a double-sided joint, loaded parallel to grain, using Radiata Pine glulam GL8 (spacing a_1=25 mm; a_2=25 mm).

Rivet Length, L_r (mm)	Member Thickness, *b* (mm)	Rivets per a row n_C	Number of rows n_R				
			6	8	10	12	14
40	90	6	85	116	148	180	212
		8	96	130	165	201	236
		10	81	109	138	167	197
		12	71	95	120	145	170
		14	75	90	109	132	155
	135	6	97	134	171	208	245
		8	128	175	222	270	318
		10	136	185	235	285	336
		12	129	175	221	269	316
		14	121	163	206	250	294
65	135	6	117	160	203	247	291
		8	131	176	222	268	315
		10	147	193	241	291	341
		12	169	218	270	324	378
		14	160	203	249	297	347
	180	6	126	172	220	267	314
		8	146	196	248	300	353
		10	167	221	278	336	394
		12	198	259	323	389	455
		14	226	291	361	433	506
90	180	6	156	213	271	329	387
		8	170	226	285	344	404
		10	184	240	300	360	422
		12	205	261	322	385	449
		14	231	286	349	414	481
	225	6	163	222			
		8	180	240			
		10	197				
		12	224				
		14	255				

Notes:

(1) Member depth is assumed to be two times the joint depth, $h = 2 \times a_2(n_R\text{-}1)$.

(2) The joint is located at the centre of the member.

(3) End distance is considered based on minimum requirements.

(4) The value of k_1, k_{12} *and* k_f *factors are assumed to be equal to one.*

(5) Steel plate is considered to be 10 mm thick.

(6) Ultimate design capacity of 40, 65 and 90 mm long rivets under ductile failure is calculated as 2.08, 2.62 and 2.98 kN per rivet, respectively.

Table A1.3: Reference design capacity (kN) for a double-sided joint, loaded perpendicular to grain, of Radiata Pine LVL grade 11 (spacing a_1=25 mm; a_2=25 mm).

Rivet Length, L_r (mm)	**Member Thickness, *b* (mm)**	**Rivets per a row n_C**	**Number of rows n_R**				
			6	8	10	12	14
40	90	6	24	26	29	31	33
		8	30	32	33	34	36
		10	33	34	36	37	
		12	36	37	39	40	41
		14	39	40	41	42	43
	135	6	24	26	29	31	33
		8	32	34	36	39	41
		10	39	41	44	46	48
		12	47	49	51	54	56
		14	54	56	59	61	63
65	135	6	40	42	45	47	50
		8	45	47	49	52	54
		10	50	52	54	56	58
		12	54	56	58	59	61
		14	58	60	62	63	65
	180	6	41	45	49	53	57
		8	54	58	62	66	70
		10	67	69	72	74	77
		12	72	75	77	79	82
		14	78	80	82	84	86
90	180	6	53	57	60	63	67
		8	60	63	66	69	72
		10	67	69	72	74	77
		12	72	75	77	79	82
		14	78	80	82	84	86
	225	6	53	58	63	65	73
		8	69	74	80	85	90
		10	83	86	90	93	96
		12	91	93	96	99	102
		14	97	100	103	105	108

Notes:

(1) Member depth is assumed to be two times the joint depth, $h = 2 \times a_2(n_C-1)$.

(2) The joint is located at the centre of the member.

(3) End distances on the left and right sides of the joint are assumed to be large enough that they do not affect the joint wood capacity.

(4) The value of k_1, g_{42} and k_f factors are assumed to be equal to one.

(5) Steel plate is considered to be 10 mm thick.

(6) Ultimate design capacity of 40, 65 and 90 mm long rivets under ductile failure is calculated as 2.72, 3.23 and 3.44 kN per rivet, respectively.

Table A1.4: Reference design capacity (kN) for a double-sided joint, loaded perpendicular to grain, using Radiata Pine glulam GL8 (spacing a_1=25 mm; a_2=25 mm

Rivet Length, L_r (mm)	Member Thickness, *b* (mm)	Rivets per a row n_C	Number of rows n_R				
			6	8	10	12	14
40	90	6	15	17	19	21	23
		8	20	22	23	25	27
		10	24	26	28	29	30
		12	27	28	29	31	32
		14	29	30	31	32	33
	135	6	15	17	19	21	23
		8	20	22	23	25	27
		10	24	26	2S	30	32
		12	28	30	32	34	36
		14	32	34	36	38	40
65	135	6	6	29	33	36	40
		8	33	37	39	41	43
		10	38	40	42	44	46
		12	41	42	44	46	48
		14	43	45	47	48	50
	180	6	26	29	33	36	40
		8	33	37	40	43	47
		10	41	44	47	51	54
		12	48	51	55	58	61
		14	55	59	62	65	67
90	180	6	34	38	42	47	51
		8	43	47	52	55	5S
		10	50	53	55	58	61
		12	54	57	59	61	64
		14	58	60	62	65	67
	225	6	34	38	42	47	51
		8	43	47	52	56	60
		10	52	57	61	65	70
		12	62	66	70	75	79
		14	71	75	78	81	84

Notes:

(1) Member depth is assumed to be two times the joint depth, $h = 2 \times a^2(n_C\text{-}1)$.

(2) The joint is located at the centre of the member.

(3) End distances on the left and right sides of the joint are assumed to be large enough that they do not affect the joint wood capacity.

(4) The value of k_1, g_{42} and k_f factors are assumed to be equal to one.

(5) Steel plate is considered to be 10 mm thick.

(6) Ultimate design capacity of 40, 65 and 90 mm long rivets under ductile failure is calculated as 1.62, 2.16 and 2.27 kN per rivet, respectively.

A3 Adjustment Factors

A3.1 Adjustment Factor for Tension Parallel to Grain

X_t = adjustment factor for tension parallel to grain

$$= \left(\frac{f_{t,m}}{f_{t,k}}\right)\left(\frac{Q_{wk,l}}{Q_{wm,l}}\right) = \left(\frac{1-1.645COV_{w,l}}{1-1.645COV_t}\right)$$

= 1.06 for LVL

= 1.19 for glulam

= 1.29 for sawn timber

where

COV_t = coefficients of variation for the conducted tensile strength material property tests, parallel to grain

= 0.12 for LVL

= 0.24 for glulam

= 0.30 for lumber (estimated by adding half the difference between LVL and glulam)

$COV_{w,l}$ = maximum coefficients of variation for the conducted connection wood strength tests, parallel to grain

= 0.09 for LVL

= 0.17 for glulam

= 0.21 for lumber (estimated by adding half the difference between LVL and glulam)

m and *k* indices stand for the mean and characteristic values, respectively.

A3.2 Adjustment Factor for Longitudinal Shear

X_s = adjustment factor for longitudinal shear

$$X_S = \left(\frac{f_{s,m}}{f_{s,k}}\right)\left(\frac{Q_{wk,l}}{Q_{wm,l}}\right) = \left(\frac{1-1.645COV_{w,l}}{1-1.645COV_{tl}}\right)$$

= 1.02 for LVL

= 0.96 for glulam

= 0.93 for lumber

where

COV_s = coefficients of variation for the conducted longitudinal shear strength material property tests

= 0.10 for LVL

= 0.15 for glulam

= 0.18 for lumber (estimated by adding half the difference between LVL and glulam)

A3.3 Adjustment Factor for Tension Perpendicular to Grain

X_p = adjustment factor for tension perpendicular to grain

$$X_P = \left(\frac{f_{tp,m}}{f_{tp,k}}\right)\left(\frac{Q_{wk,p}}{Q_{wm,p}}\right) = \left(\frac{1-1.645COV_{w,p}}{1-1.645COV_{tp}}\right)$$

= 1.23 for LVL

= 1.28 for glulam

= 1.31 for lumber

where

COV_{tp} = coefficients of variation for the conducted tensile strength material property tests, perpendicular to grain

= 0.18 for LVL

= 0.25 for glulam

= 0.29 for lumber (estimated by adding half the difference between LVL and Glulam)

where

$COV_{w,p}$ = maximum coefficients of variation for the conducted connection wood strength tests, perpendicular to grain

= 0.08 for LVL

= 0.15 for glulam

= 0.19 for lumber (estimated by adding half the difference between LVL and glulam)

A3.4 Adjustment Factor for Rivet Characteristic Resistance

$X_r = 1\text{-}1.645COV_r$

= 0.93 for LVL

= 0.87 for glulam

= 0.84 for lumber

where

COV_r = maximum coefficients of variation for the conducted connection rivet strength tests

= 0.04 for LVL

= 0.08 for glulam

= 0.10 for lumber (estimated by adding half the difference between LVL and glulam)

A3.5 Adjustment Factor for Characteristic Withdrawal resistance

$X_{ax} = 1\text{-}1.645COV_{ax}$

= 0.84 for LVL

= 0.61 for glulam

= 0.49 for lumber

where

COV_{ax} = coefficients of variation for the conducted rivet withdrawal strength tests

= 0.10 for LVL

= 0.24 for glulam

= 0.31 for lumber (estimated by adding half the difference between LVL and glulam.

B

Appendix B - Notation

The symbols and letters used in this Design Guide are listed below:

A_{tb}	effective tensile area of bottom wood block
A_{th}	area of head plane subjected to tensile stress
A_{tl}	effective tensile area of lateral wood blocks
$A_{t,b}$	area of head plane subjected to tensile stress
$A_{s,l}$	areas of side lateral planes subjected to shear stress
A_{sb}	area of bottom plane subjected to shear stress
A_{sl}	area of lateral planes subjected to shear stress
a	unloaded edge distance
a_1	minimum parallel-to-grain rivet spacing
a_2	minimum perpendicular-to-grain rivet spacing
a_{3t}	loaded rivet end distance for parallel-to-grain loading
a_{3c}	unloaded rivet end distances
a_{4t}	loaded rivet edge distance
a_{4c}	unloaded rivet edge distance
b	timber thickness
C_{fp}	fracture parameter
C_t	coefficient depending on unloaded edge distance and joint length
C_l	factor accounting for embedding and withdrawal interaction effect
COV_{tp}	coefficients of variation for the conducted tensile strength material property tests – ..... perpendicular to grain
COV_r	maximum coefficients of variation for the conducted connection rivet strength tests
COV_{ax}	coefficients of variation for the conducted rivet withdrawal strength tests
d_l	rivet cross-section dimension bearing on the wood – parallel to grain
d_p	rivet cross-section dimension bearing on the wood – perpendicular to grain
d_z	bottom distance
E	modulus of elasticity – parallel to grain
f_{ax}	withdrawal resistance per millimetre of penetration
f_h	wood embedment strength
$f_{h,u}$	wood ultimate embedment strength
$f_{h,y}$	wood yielding embedment strength
$f_{h,0}$	wood embedment strength – perpendicular to grain
$f_{h,90}$	wood embedment strength – perpendicular to grain
f_s	member characteristic longitudinal shear strength
f_t	wood mean strength in tension – parallel to grain
f_{tp}	wood mean strength in tension – perpendicular to grain
g_{42}	modification factor for interaction effect on a grid system specified in AS 1720.1
G	modulus of rigidity – parallel to grain

h	member depth
h_e	effective member depth
J_p	side plate factor
k_f	modification factor for joint position effect
k_1	duration of load, (timber) specified in AS 1720.1
k_4	moisture condition, (timber) specified in AS 1720.1
k_6	temperature (timber) specified in AS 1720.1
k_{12}	stability factor, (timber) specified in AS 1720.1
L_p	rivet penetration depth
L_r	rivet length
M_r	moment capacity of rivet
$M_{r,l}$	moment capacity of rivets – parallel to grain
$M_{r,p}$	moment capacity of rivets – perpendicular to grain
$M_{r,u}$	ultimate moment capacity of rivet
$M_{r,y}$	yielding moment capacity of rivet
N	applied load
N^*_1	serviceability design load, parallel to grain, for deflection under (SLS)
n_p	number of plates
n_C	number of rivet columns
n_R	number of rivet rows parallel to direction of the load
P	applied load
P_{ax}	characteristic withdrawal resistance
P_r	fastener capacity
$P_{r,yld}$	yield capacity of fastener
$P_{r,ult}$	ultimate capacities of fastener
$P_{s,a}$	characteristic resistance for full width splitting – mode (a)
$P_{s,b}$	characteristic resistance for partial width splitting
P_w	load-carrying capacity of wood
$P_{w,b}$	maximum load causing failure on bottom plane
$P_{w,h}$	maximum load causing failure on head plane
$P_{w,l}$	maximum load causing failure on lateral planes
$P_{rl,a}$	characteristic strength, parallel to grain, for rivet failure mode (a)
$P_{rl,b}$	characteristic strength, parallel to grain, for rivet failure mode (b)
$P_{w,tefe}$	load-carrying capacities of wood corresponding to tef,e
$P_{w,tefy}$	load-carrying capacities of wood corresponding to tef,y
$P_{rp,a}$	characteristic strength, perpendicular to grain, for rivet failure mode (a)
$P_{rp,b}$	characteristic strength, perpendicular to grain, for rivet failure mode (b)
$P_{c,ult}$	connection ultimate resistance
Q_s	joint design lateral resistance
$Q_{we,l}$	design wood block tear-out resistance, parallel to grain, corresponding to rivet elastic deformation
$Q_{ry,l}$	design rivet yielding resistance – parallel to grain

$Q_{wy,l}$	design wood block tear-out resistance, parallel to grain, corresponding to rivet yielding mode
$Q_{ru,l}$	design rivet ultimate resistance – parallel to grain
$Q_{re,l}$	design rivet resistance corresponding to the rivet elastic deformation – parallel to grain
$Q_{ru,l}$	design rivet resistance corresponding to the rivet ultimate deformation – parallel to grain
$Q_{we,p}$	design wood block tear-out resistance, perpendicular to grain, corresponding to rivet elastic deformation
$Q_{ry,p}$	design rivet yielding resistance – perpendicular to grain
$Q_{wy,p}$	design wood block tear-out resistance, perpendicular to grain, corresponding to rivet yielding mode
$Q_{ru,p}$	design rivet ultimate resistance – perpendicular to grain
$Q_{re,p}$	design rivet resistance corresponding to the rivet elastic deformation – perpendicular to grain
$Q_{ru,p}$	design rivet resistance corresponding to the rivet ultimate deformation – perpendicular to grain
N^*	design force
t_{ef}	wood effective thickness
$t_{ef,e}$	wood effective thickness for brittle failure mode
$t_{ef,l}$	effective wood thickness – parallel to grain
$t_{ef,y}$	wood effective thickness for mixed failure mode
t_p	side plate thickness
w_{net}	net section of joint width
X_r	adjustment factor for characteristic resistance
X_t	adjustment factor for tension strength – parallel to grain
X_p	adjustment factor for tension – perpendicular to grain
X_s	adjustment factor for longitudinal shear
X_{ax}	adjustment factor for characteristic withdrawal resistance
β	effective crack length coefficient for partial width splitting
γ	effective crack length coefficient for full width splitting
δ	deflection of the joint due to rivet slip
ϕ	capacity factor
ϕ_w	capacity factor of wood
ϕ_r	capacity factor of rivet
ζ	factor depending on unloaded edge distance and joint
θ	angle
ρ	wood design density at 12% moisture content

Discover more ways to build your knowledge of wood

If you need technical information or inspiration on designing and building with wood, you'll find WoodSolutions has the answers. From technical design and engineering advice to inspiring projects and CPD linked activities, WoodSolutions has a wide range of resources and professional seminars.

www.woodsolutions.com.au
Your central resource for news about all WoodSolutions activities and access to more than three thousand pages of online information and downloadable publications.

Technical Publications
A suite of informative, technical and training guides and handbooks that support the use of wood in residential and commercial buildings.

WoodSolutions Tutorials
A range of practical and inspirational topics to educate and inform design and construction professionals. These free, CPD related, presentations can be delivered at your workplace at a time that suits you.

Seminars and Events
From one day seminars featuring presentations from leading international and Australian speakers to international tours of landmark wood projects, WoodSolutions offer a range of professional development activities.

What is WoodSolutions?
Developed by the Australian forest and wood products industry for design and building professionals, WoodSolutions is a non-proprietary source of information from industry bodies, manufacturers and suppliers.

Wood
Solutions™
design and build
woodsolutions.com.au

35

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Nov 2016

Please check woodsolutions.com.au/publications to ensure you have the latest edition

Floor Diaphragms in Timber Buildings

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Wood Solutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-33-1

Acknowledgments

Authors: Daniel Moroder, Prof Stefano Pampanin, Prof Andrew Buchanan

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: 2013
Revised: November 2016

Contents

1

Introduction

Floor diaphragms transfer horizontal forces to the lateral load-resisting system. Horizontal loads, such as wind, applied to the façade will be transferred as line loads to the edges of the diaphragms. Horizontal loadings cause inertia forces to develop within the flooring system that have to be carried to the frames, walls or lift shafts and stairway cores.

Differences in the behaviour of the lateral load-resisting systems (deformed shape, discontinuous geometry, difference in stiffness) induce additional transfer forces in some diaphragms. Roof and floor diaphragms also carry gravity loads and they link all vertical structural elements together. It is of paramount importance that diaphragms maintain their force transferring and linking behaviour before, during and after any horizontal loadings.

The connection to the lateral load-resisting system can be the weak link in diaphragm design. As discussed in the next section, this force transfer can be compromised by displacement incompatibilities typical of jointed-ductile systems.

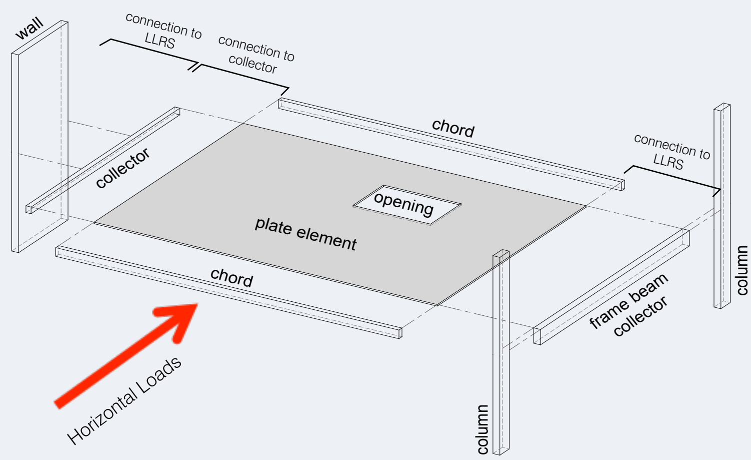


Figure 1.1: Definitions of single diaphragm components.

In this Guide, the horizontal action is considered to be caused by horizontal loads such as wind load.

The first part of the Guide presents the terminology, concept and design of timber diaphragms with their connections to the lateral load-resisting system (LLRS).

The second part reviews a design example of a timber–concrete diaphragm and its connections to the LLRS. The diaphragm is subjected to the wind load applied perpendicular to its long side.

For the calculation of timber concrete composite diaphragms, engineers prefer a grillage method with the use of analysis software. Further information on this grillage method or equivalent concrete truss method can be found in Strut and Tie Seminar Notes[1].

An equivalent truss model is recommended for calculating timber diaphragms. Further information can be found in An equivalent truss method for the analysis of timber diaphragms[2].

New alternative connections and further details including their behaviour in experimental test and also cost comparison can be found in Design of Floor Diaphragms in Multi-Storey Timber Buildings[3], and Seismic design of floor diaphragms in post-tensioned timber buildings[4].

2

Terminology

Diaphragms can be made from many different materials – plywood panels, stressed-skin panels, timber concrete composite floors, cross-laminated timber (CLT), solid floor panels, structural insulated panels – but their main components can be grouped as follows (see Figure 1.1):

- plate element
- chords
- collectors/struts
- connections to the lateral load-resisting system.

The simplest method to design diaphragms is the horizontal steel girder analogy, where the web is made by the plate element and the flanges consist of the chords. The plate element with possible openings transfers the horizontal shear forces. Several single floor elements may have to be linked together and forces carried around openings or re-entrant corners. The resultant shear forces have to be collected and conveyed to the lateral load resisting system via the collectors or struts. The connection of the collector to the lateral load-resisting system has to be designed properly, as it is an essential part of the load path into the foundations.

As shown in Figure 2.1 and according to Malone and Rice[5], the following terminology is suggested:

- Strut: receives shear from one side only
- Collector: receives shear from both sides
- Chord: perpendicular to the applied load and receives axial tension and compression forces.

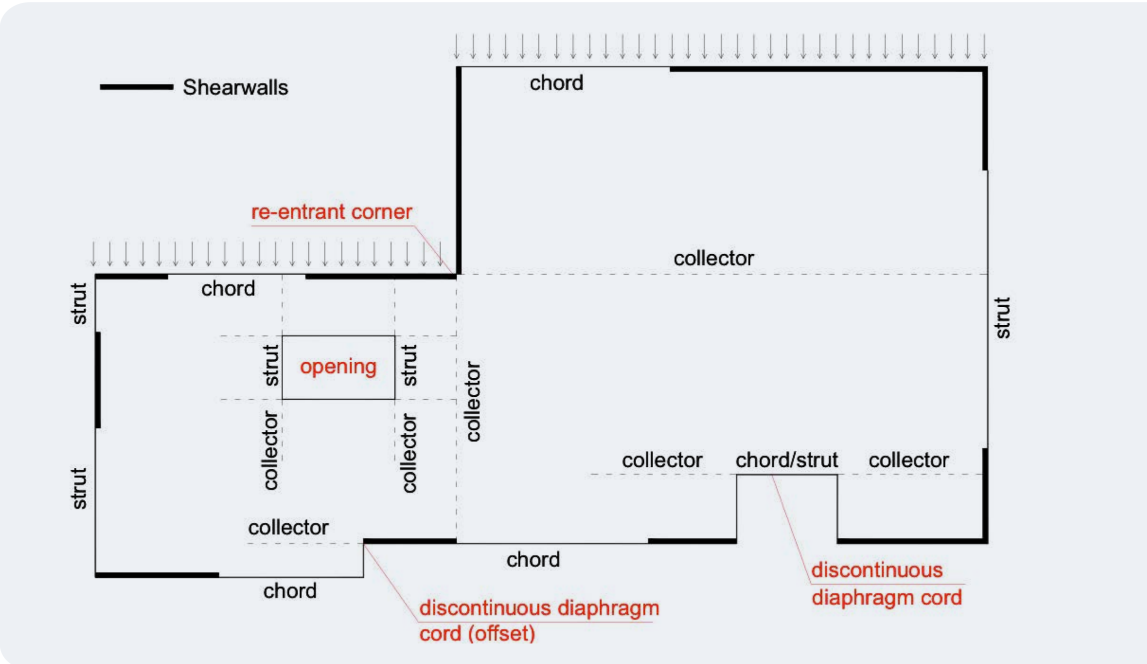


Figure 2.1: Irregular floor geometry with typical diaphragm elements.

Openings are an unavoidable feature of floor plans, as staircases, lift shafts and channels for services need to go along the height of a building. The position of these openings has to be chosen carefully, as they influence the behaviour of a building in multiple ways. First, it influences the load path in the diaphragm, as the shear forces might have to be carried around it. Bigger openings also increase the flexibility of diaphragms, which influences the behaviour of the structure and the load distribution into the lateral load-resisting system. In certain positions, openings also cause a separation of the diaphragm, as the forces cannot be transferred appropriately and the plane element cannot act as a unit.

In this situation, often the concept of sub-diaphragms or transfer diaphragms is introduced[5,6]. These are portions of the main diaphragm and are used to transfer or anchor higher shear stresses (from openings, re-entrant corners or concentrated loads) into the remaining diaphragm or the lateral load-resisting system. Often, closer nail spacing and thicker framing members are adopted, but the sub-diaphragms are essentially designed as regular diaphragms.

3

Displacement Incompatibilities

Displacement incompatibilities within diaphragms or between diaphragms and the LLRS can damage structures. Hence, design and detailing are essential to consider displacement incompatibilities within diaphragms or between diaphragms and the LLRS[7]. The displacement incompatibilities normally associated to concrete and steel structures can also be observed in traditional and innovative timber structures[8].

Experimental testing has shown that the flexibilities of timber members and steel fasteners can, in many cases, accommodate the required displacements without compromising the diaphragm behaviour.

With careful design, well-designed timber diaphragms can easily undergo horizontal loadings without any damage. In the case of TCC floors or any floors with concrete topping, some additional detailing may be necessary as detailed in Section 13.

4

Diaphragm Design for Wind Action

Wind loads are obtained from AS 1170.2. Wind pressures applied to the façade (perpendicular pressure or wind friction) are then simply transferred to the diaphragms according to their tributary areas.

5 Load Paths in Diaphragms

For regularly shaped floor geometries, i.e. rectangular, without big openings or re-entrant corners, timber-only diaphragms can be designed by using the girder analogy. This implies that the shear is taken by the web (diaphragm sheeting) and the bending is taken by the flanges (diaphragm chords). Even though the girder analogy may not be strictly appropriate for deep beams with anisotropic materials, tests have shown that flange stresses are smaller than using that approach, providing a conservative design.9,10 Furthermore, shear stresses develop uniformly over the web, instead of the parabolic shape found in steel girder webs.

Diaphragms can be designed as simply supported or continuous beams, providing that the span-to-depth ratio is greater than 2. For aspect ratios smaller than 1, the girder analogy is quite conservative, as the sheeting and joists contribute substantially in the bending resistance. Considering the high depth of the diaphragm, the chord forces will be small.11 Different authors provide an upper limit for the span-to-depth-ratio, as the diaphragm may become too flexible. If floors are running over internal supports and the different diaphragm parts on each side are connected, they can be analysed as a continuous beam.

As shown in Figure 5.1, diaphragms with simple plan geometries can be calculated by considering a girder analogy, where the bending is taken by the chord elements in form of tension or compression forces and the uniform shear is taken by the plate element.

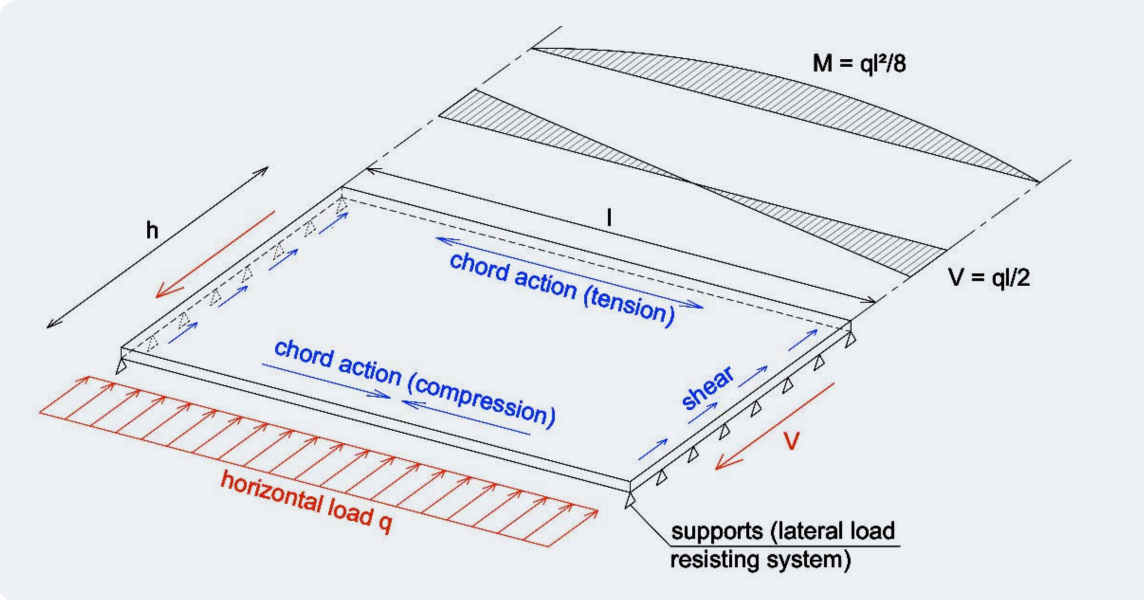


Figure 5.1: Girder analogy for diaphragms.

The tension and compression forces in the chords (T and C) can be calculated as follows, where the terms are shown in Figure 5.1:

$$T = -C = \frac{M}{h} = \frac{ql^2}{8h} \quad (5\text{-}1)$$

The shear flow along the edges of the diaphragm is:

$$v = \frac{V}{h} = \frac{ql}{2h} \quad (5\text{-}2)$$

where:

q = the lateral load applied to a horizontal diaphragm

l = the span of horizontal diaphragm

h = the width of horizontal diaphragm

V = the shear force applied to a diaphragm

M = the design moment of diaphragm.

The loading of the diaphragms is normally considered as distributed uniformly along the length of the diaphragm. This is the wind load transferred from the façade, or the inertia forces generated by the mass of the floor itself. In the case of multi-storey structures, vertical offsets can introduce concentrated loads. Openings, re-entrant corners, offsets or concentrated horizontal forces will disturb the shear flow and locally higher stresses might arise.

6

Flexible and Rigid Diaphragms

The distribution of forces into the lateral load-resisting system depends on the flexibility of the diaphragm, i.e. rigid or flexible diaphragms. A diaphragm is considered to be flexible if its deformation is more than twice the average inter-story drift at that level.

In the case of rigid diaphragms, the transferred forces depend on the stiffness of the diaphragm with respect to the global stiffness of the lateral load-resisting system. In the likely case that the centres of stiffness and mass are not coincident, torsional effects have to be taken into account.

To define whether the diaphragm is rigid or flexible, the deflection calculation for both the vertical LLRS and the diaphragm is required. For flexible diaphragms (Figure 6.1) the load can be determined by using a tributary area approach.

Depending on the geometry, timber diaphragms often behave somewhere between these two extreme cases. It remains the designer's choice of which approach to use: to calculate both to obtain an envelope; to use a beam on elastic support approach (beam and elastic support have the stiffness characteristics of the diaphragm and lateral load-resisting system respectively); or to use a finite element method.

The beam-on-elastic-support approach is valid as long as the stiffness of the diaphragm and the lateral load-resisting system are similar. The finite element method allows study of the shear stress distribution over the whole plate element, but requires much more effort in modelling and running the simulation.

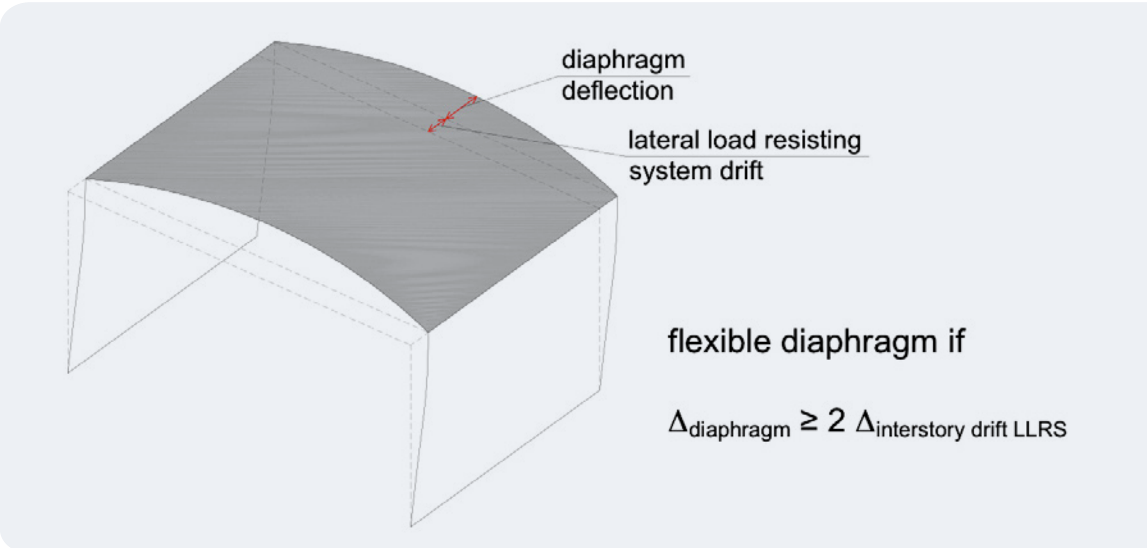


Figure 6.1: Flexible diaphragm.

7

Design of Structural Elements in Diaphragms

The structural elements in a timber diaphragm consist of the plate element, the chords and the collector/strut beams.

Depending on the chosen setup, the plate element consists either of sheeting panels and framing members or thicker solid type panels. They all have to be joined to act as a single unit. For a traditional timber joist floor with panels of particleboard or plywood, the strength of the diaphragm depends on the amount of nailing and the presence of blocking (connection of the panel edges to adjacent panels). Blocked diaphragms have a stiffness and strength two to three times higher than the unblocked equivalent so, for multi-storey timber buildings, such floors should always have solid blocking along all the sheet edges. The buckling of the sheeting is normally prevented by edge connections.

The capacity checks necessary to design a diaphragm should be performed in accordance with the relevant timber code and should include:

- the shear capacity of the nails
- the shear capacity of the panel
- the out-of-plane buckling of the panel.

Nail spacing and sheeting thickness is normally dictated from the stress values at the supports of the diaphragm. As an alternative, the sheeting panels can be glued or screwed to the framing elements; this can provide a higher shear capacity and stiffer connections.

Chord, collector and strut beams have to transfer tension and compression forces and need to be designed accordingly. If they consist of single jointed elements, continuity has to be provided by adequate ductile connections. Chords should be spliced as far as possible away from the point of maximum moment. Often these elements also work under gravity loads and internal stresses have to be combined with the lateral forces. Since the direction of wind forces is arbitrary, chords also act as collectors – and vice versa – depending on the loading direction.

8

Horizontal Deflection of Diaphragms

There are several reasons why the horizontal stiffness of a diaphragm needs to be calculated. The vertical elements supporting or attached to a diaphragm need to maintain their load carrying capacity to guarantee structural integrity, and should not be damaged due to excessive deformation. Furthermore, the distribution of the in-plane loads to the lateral load-resisting system is a function of the stiffness of the diaphragm as discussed above. Finally, the dynamic period of the diaphragm can interfere with the dynamic behaviour of the structure and cause higher modes effect.

The horizontal deflection of diaphragms is the sum of the following single contributions:

Δ_1 = flexural deflection of the diaphragm considering the chords acting as a moment resisting couple

Δ_2 = deflection due to shear in the panels

Δ_3 = deflection of the diaphragm due to fastener slip

Δ_4 = deflection of the diaphragm due chord connection deformation.

Equations for the determination of these contributions are provided in NZS 3603.[12]

$$\Delta_1 = \frac{5ql^3}{192EAB^2} \quad (8\text{-}1)$$

$$\Delta_2 = \frac{ql}{8GBt} \quad (8\text{-}2)$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad (8\text{-}3)$$

$$\Delta_4 = \frac{\sum \delta_s x}{2B} \quad (8\text{-}4)$$

where:

q = lateral load applied to a horizontal diaphragm

l = span of horizontal diaphragm

E = elastic modulus of chord member

A = section area of the chord

B = distance between diaphragm chord member

G = shear modulus of the diaphragm sheathing

t = thickness of the diaphragm sheathing

m = number of the sheathing panels along the length of the edge chord

e_n = fastener slip resulting from the shear force V

a = aspect ratio of each sheathing panel given in the NZS 3603[12]

x = distance of the splice from the origin

δ_s = splice slip in the chord.

The fastener slip en is calculated for the maximum unit shear force at the support. Since all diaphragms should be designed as elastic, the slip only depends on the slip modulus and spacing of the fasteners[3]. Equation (8-3) can be modified where different fasteners are used for the panel-to-panel, panel-to-chord and panel-to-collector connections.

Given the variety of wooden construction materials and means of connections, the possible diaphragm setup and the limited amount of experimental tests, the deflection values only provide a rough approximation of the diaphragm deformation. The verification of a certain deflection limit, however, is to be set by the designer with knowledge of the affect deflection will have on the surrounding structure.

The deflection given above is only applicable to simply supported blocked diaphragms with chord beams. To account for diaphragm irregularities such as variable loads, openings, re-entrant corners, changes in diaphragm depth or staggered fastener layouts, these equations can be integrated over parts of the diaphragm[7].

The presence of openings, varying nailing pattern or non-uniform forces should be considered when calculating the horizontal diaphragm deflection. This can be done by modifying the basic deflection equation by changing the coefficients of the single contributions according to basic beam theory or by integrating the equation over segments of the diaphragm. If more precise results are required, finite element analysis might have to be considered.

The design of floors where the diaphragm action is taken by the concrete topping should be in accordance with the relevant concrete code. Special provisions regarding displacement incompatibilities will be given later.

9

Connection Between Single Timber Floor Elements

To connect two single floor panels together, the following alternative connection systems are suggested (see Figure 9.1):

a) nailing (and gluing) of adjacent panels

b) wooden strip in recess between panels with screws or nails

c) inclined fully threaded screws, or regular screws at 90° between joists

d) nailing (and gluing) of panel to the next joist

e) double inclined screws in shear between solid panels

f) tongue and groove with double inclined fully threaded screws.

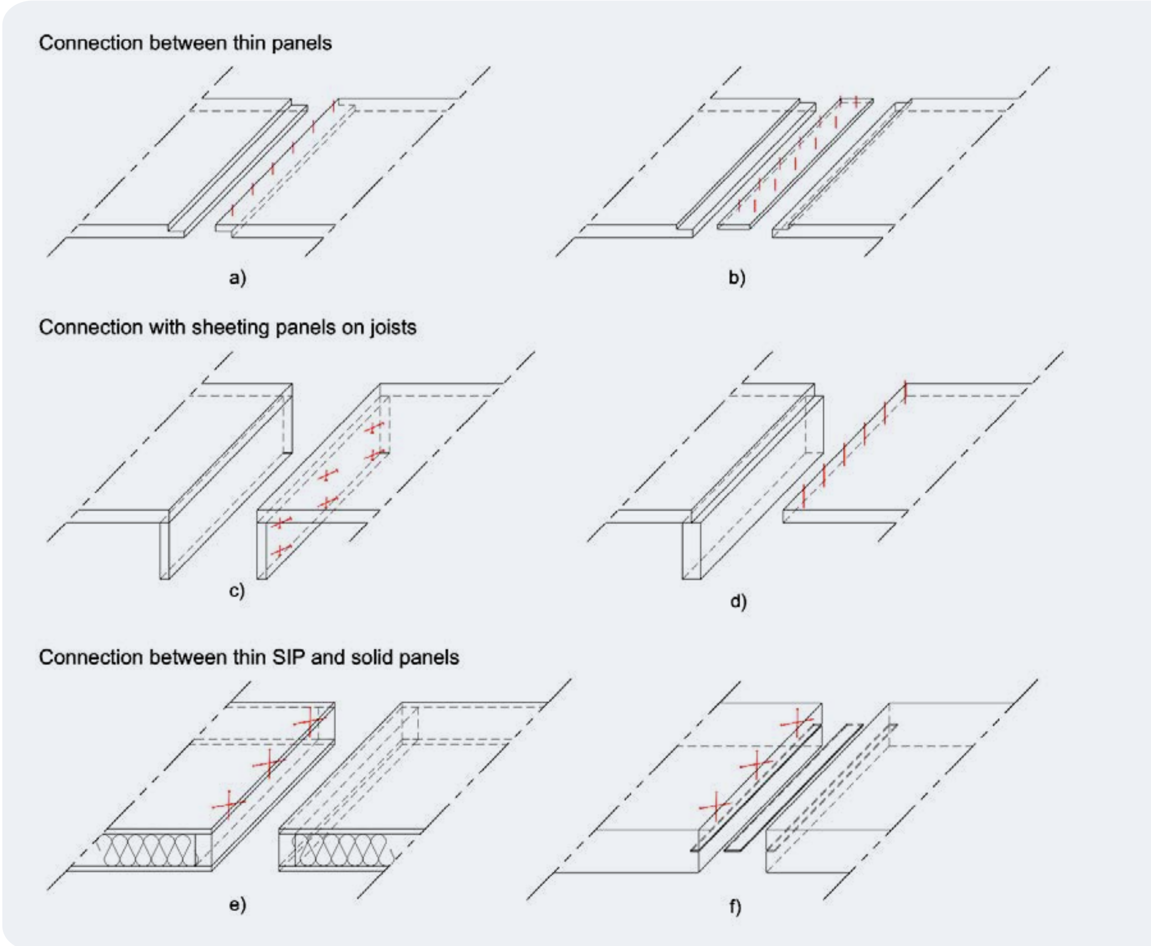


Figure 9.1: Connection details between floor elements.

These connections are generally valid and are to be designed to guarantee adequate shear transfer. Because of the displacement incompatibilities mentioned before, special detailing for floor joints close to the beam–column joints may be necessary.

Gluing must be considered carefully. A nailed joint or screwed joint with glue will become much stronger, but also much more brittle and less deformable under extreme loads, so there will be many cases where glue should not be used.

10

Connection Between Single Timber Floor Elements

Because of the variety of building geometries, lateral load-resisting systems, floor assemblies and the available types of connection, no unique detail solution can be given. Key aspects to consider for the connection design are the kind of required force transfer (horizontal shear only or combined with gravity forces) and the type of diaphragm (timber only or concrete topping).

While the connections mainly have to transfer the horizontal forces deriving from the diaphragm action, out-of-plane forces of the lateral load-resisting system have to be considered as well (see Figure 10.1). These forces can be wind suction at leeward walls, inertia forces on the façade, or dragging forces from the constraint of vertical elements to move with the rest of the structure under a certain drift.

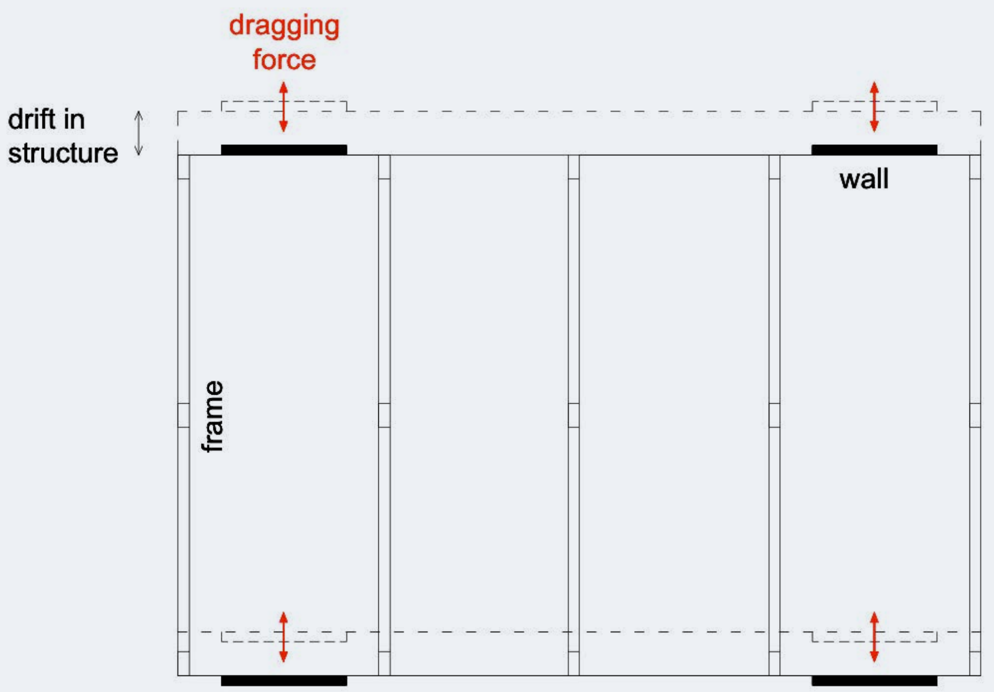


Figure 10.1: Dragging forces on walls from drift in structure with north–south forces resisted by frames.

11 Connection Between Timber Diaphragms and Gravity Frames

11.1 Gravity and Shear Forces

In this section, timber-only floors running perpendicular to the lateral and gravity frames are considered. The floor elements have to transfer vertical gravity forces and horizontal shear forces to the beam, which acts as a collector or strut. For floors sitting between the beams, gravity loads can be transferred by a timber corbel, a pocket in the main beam or steel hanger brackets. The horizontal shear forces can be transferred directly by nailing or screwing the sheeting panels to the beam (see Figure 11.1).

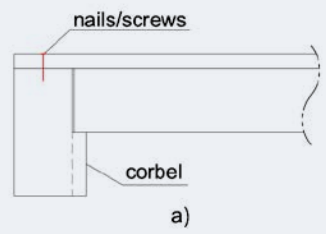


Figure 11.1: Suggested floor-to-frame connections (floor joists flush with beam): a) floor joist on corbel; b) floor joist in pocket; c) steel bracket/hanger.

Where the floors sit on the beams, gravity forces are transferred by direct contact. Shear forces can be transferred by using fully threaded screws at 45° angle or by connecting the sheeting to blocking elements, which are again joined to the beam by screws or steel plate elements (see Figure 11.2).

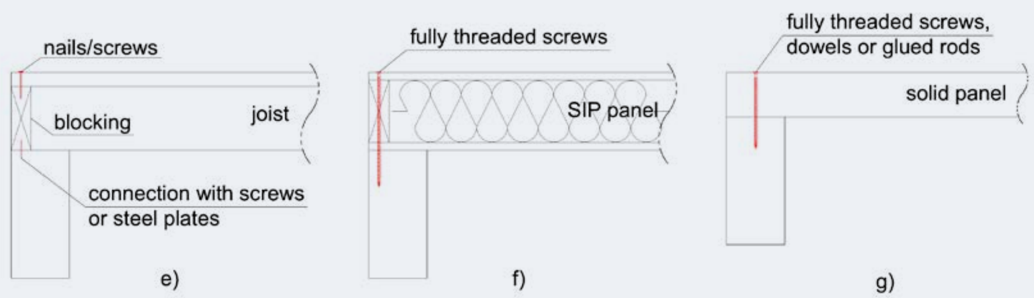


Figure 11.2: Suggested diaphragm to frame connections (floor joists on top of beam): e) floor joist sitting on beam – additional blocking required; f) SIP panel on beam; g) solid timber floor on beam.

11.2 Out-of-Plane Rocking

Where the horizontal load acts perpendicular to the frame, the whole building will undergo a certain drift (depending on the lateral load-resisting system in this direction) and hence the frame will have to rotate out of plane. As indicated in Figure 11.3, it is suggested to leave a construction gap between the floor elements and the beams (also useful for construction tolerance and variances in ambient conditions). This will allow the beam to rotate, without damaging the timber floor or the connection to it. The connection between the beam and the floor also needs to transfer the dragging force to rotate the frame out of plane.

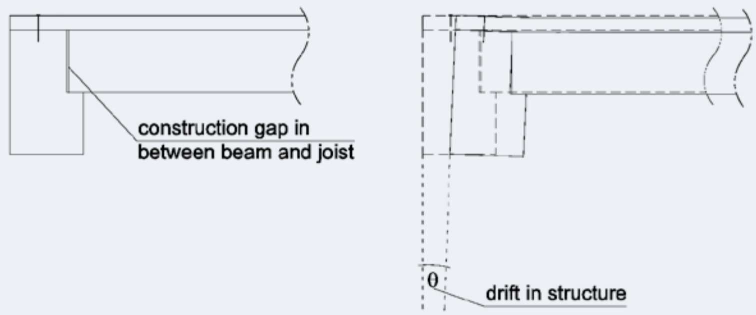


Figure 11.3: Construction gap between a timber floor and supporting beam to allow for rotation: a) undeformed state; b) deformed state.

11.3 Frame Elongation

The formation of gaps at the beam–column joint produces frame elongation. The diaphragm must be able to extend. This behaviour has to be allowed for without a brittle tearing of the plate element, as it would cause permanent damage and compromise the shear transfer. The flexibility of the timber elements and the low stiffness of the steel connections allow for two simple design solutions for engineered timber floors:

Solution 1: Concentrated gap (see Figure 11.4, blue details):

As the required deformation in the floor level occurs only at the beam–column joint, a joint between two adjacent floor panels should be positioned accordingly. This joint needs special detailing, whereas other panel joints can be designed normally.

For floor setups with sheeting panels and slender joists, only the lower part of the joist should be connected, so that the joist can bend along its height, but still guarantees shear transfer (see Figure 11.5a). If a different floor setup is used where the joist are too stiff, special steel elements can be used. These should allow the panels to move apart from each other, but still transfer shear forces (an example is shown in Figure 11.5b). Appropriate gaps in the floor finishing and the wall linings have to be provided to allow these deformations to occur.

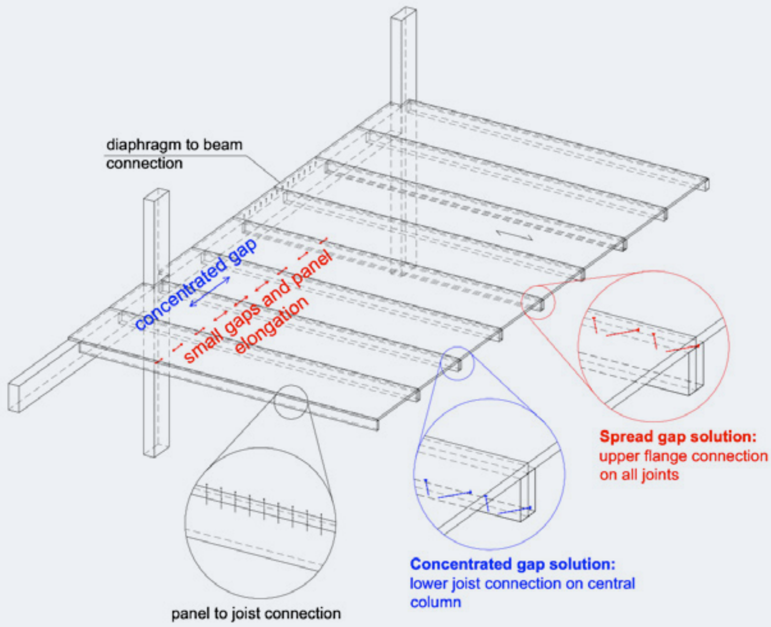


Figure 11.4: Sample design for a concentrated floor gap (blue) and spread gaps and panel elongation (red). Solutions for a timber-only engineered floor.

Solution 2: Spread floor gaps and panel elongation (see Figure 11.4, red details):

As an alternative to a concentrated gap at each beam location, detailing for uniformly spread gaps can be used. The required deformation will be accommodated by a number of small panel gap openings and the elongation of the sheeting panel itself. This implies that the panel is relatively flexible in the direction perpendicular to the span direction).

Two to three floor elements each side of the interested beam–column joint should be connected to each other by means of metallic connectors such as nails or screws (like an upper joist connection shown in Figure 11.5c). The connection needs to guarantee full shear transfer between the elements, but should be flexible enough to allow for a small displacement. Small gaps will hence open in several panel joints and the sheeting panels will elongate. The sum of all contributions will make up the required displacement, as demonstrated in recent testing.

Site gluing to connect floor elements should be avoided, as it results in a stiff and brittle connection that cannot accommodate the required deformations. Furthermore, the panels close to the beam–column joint(s) should not be connected to the beam to transfer diaphragm forces, as this would prevent the development of floor gap openings and panel elongations further away from the area of interest.

The floor finishing should be chosen to be elastic enough to follow the formation of the spread gaps.

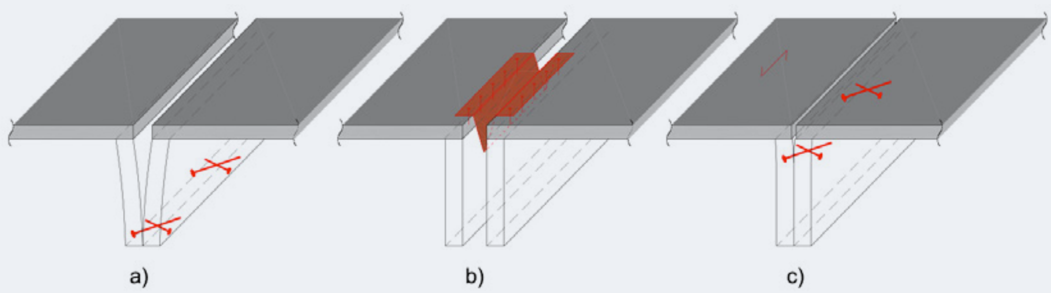


Figure 11.5: a) Lower flange connection; b) connection with thin steel plate; c) upper flange connection.

12

Connections between Timber Diaphragms and Walls

For wall structures, the diaphragm and gravity forces are transferred via the collector/strut beam to the lateral load-resisting system (see Figure 12.1). The most appropriate connection detail to link the collector beam to the walls depends on the span direction of the floor. For floor elements running parallel to the wall, only horizontal forces have to be transferred – otherwise gravity forces have to be taken as well.

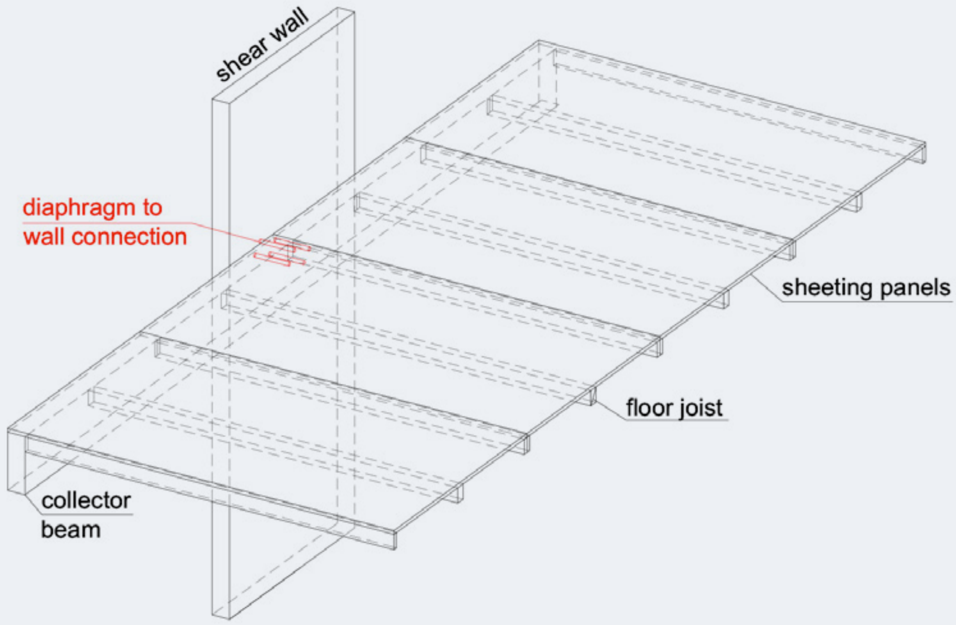


Figure 12.1: Scheme of a typical diaphragm-to-wall connection.

To allow for the required uplift and rotation of the walls, a discrete connection placed at the centre of the wall should be used. Ideally, a single dowel-type connector with a vertical slotted hole would overcome all displacement incompatibilities by still transferring the horizontal forces from the diaphragm. However, a single dowel is not usually suitable because of the magnitude of the forces, possible splitting of wood with large diameter dowels and the difficulty of providing slotted holes in timber.

If only horizontal forces have to be transferred to the wall, connections with steel plates and dowels placed in slotted holes can be used. The plate itself can be fixed by screws, nails, rivets or bolts to the timber elements. Where gravity forces also have to be conveyed to the wall, a vertical restraint is necessary. This solution can be achieved by simply connecting the timber beam and wall together with dowel-type connectors. While a single big diameter dowel is an attractive solution, little is known regarding its embedment strength. As an alternative, a ring of closely spaced dowels will approximate a hinge.

Table 12.1 summarises four connection details and their properties (refer also to Figure 12.2).

Table 12.1: Possible wall to collector beam connections.

Connection type	*Force transfer*	*Displacement incompatibilities*		*Comments*
Big pin connection	Horizontal shear and gravity	Rotation is allowed	Uplift is not allowed	The embedment strength and behaviour of large diameter dowels is not well known
Slotted steel plate with rivets	Horizontal force only	Rotation is allowed	Uplift is allowed	This connection allows for all displacement incompatibilities. Lots of steelwork required
Ring of dowels	Horizontal shear and gravity	Rotation is partially allowed*	Uplift is not allowed	Simple solution, the flexibility of the connection allows for some rotation
Steel profile with slotted holes	Horizontal force only	Rotation is allowed	Uplift is allowed	Possible problems due to friction

** Given the possibility of using oversized holes in the timber and relatively flexible dowel connection, the rotation of the wall normally can be accommodated for limited drift ratios.*

If uplift of the walls is not allowed by the connection and the collector/strut beam is also attached to other vertical elements, such as columns (see Figure 12.3a), the beam and the diaphragm will both need to bend. This can be tolerated if the collector beam is flexible enough (i.e. because of its small section or a long span to the next vertical restraint). The additional re-entering force resulting from bending should be considered when designing the wall.

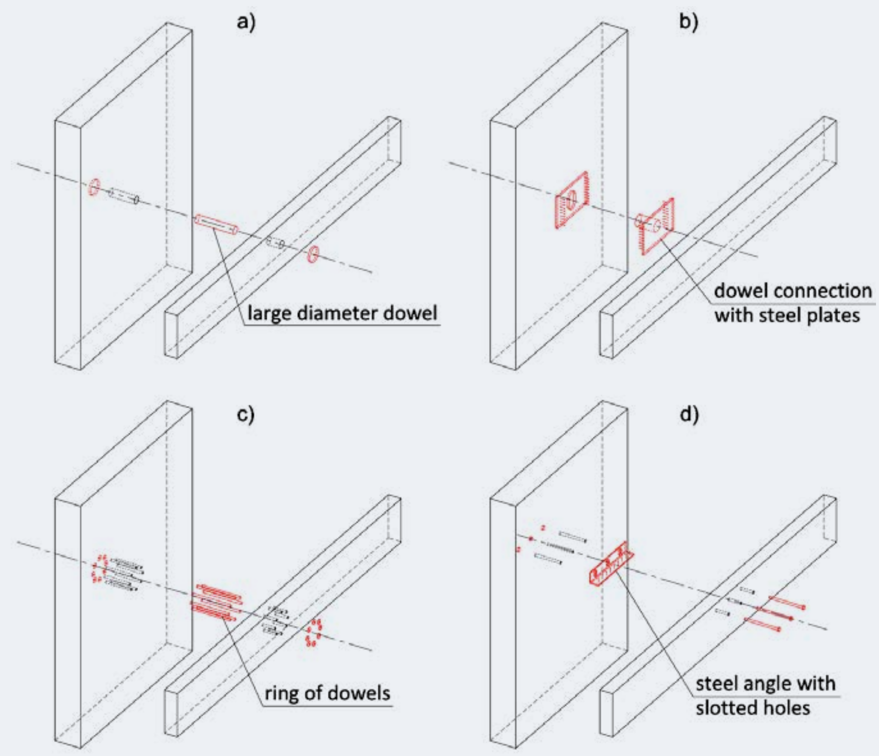


Figure 12.2: Suggested diaphragm-to-wall connection details: a) large diameter dowel connection (timber–timber); b) dowel connection (steel–steel); c) multiple dowel connection (timber–timber); and (d) steel angle with slotted holes.

For multiple dowel connection (ring of dowels), the additional moment coming from the rotational restraint should be checked in the beam and in the wall design. Because of the oversized hole in the timber elements, the compact geometry and the relatively small connection stiffness, this connection should almost behave as a hinge.

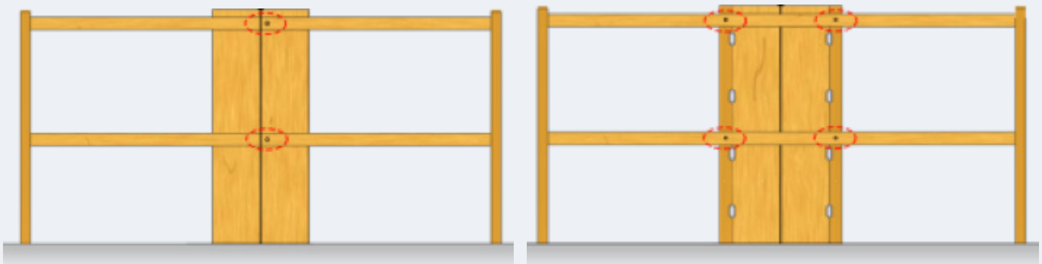


Figure 12.3: a) Single wall; and b) wall with external columns.

Where gravity forces have to be transferred to the wall and the uplift of the collector beam has to be avoided, a wall configuration with external columns as shown in Figure 12.3b can be used. Under horizontal loading, the wall would rock but the columns would only follow rotation without any uplift. In this way, gravity and horizontal forces can be transferred directly to the columns by avoiding any vertical displacement incompatibility. The connection only has to accommodate the rotation of the columns. The horizontal force transfer from the columns to the wall and the buckling restrain of the columns itself must be considered appropriately.

12.1 Out-of-Plane Rocking

As in frame structures, the connections between the floor diaphragm, the collector beam and the wall itself have to transfer not only the shear flow from diaphragm action, but also the drag force from the out-of-plane deformation of the wall. This will occur when the horizontal load act perpendicular to the walls and the whole structure deforms in the out-of-plane direction of the walls. Construction gaps between the floor and the walls must accommodate the displacement incompatibility.

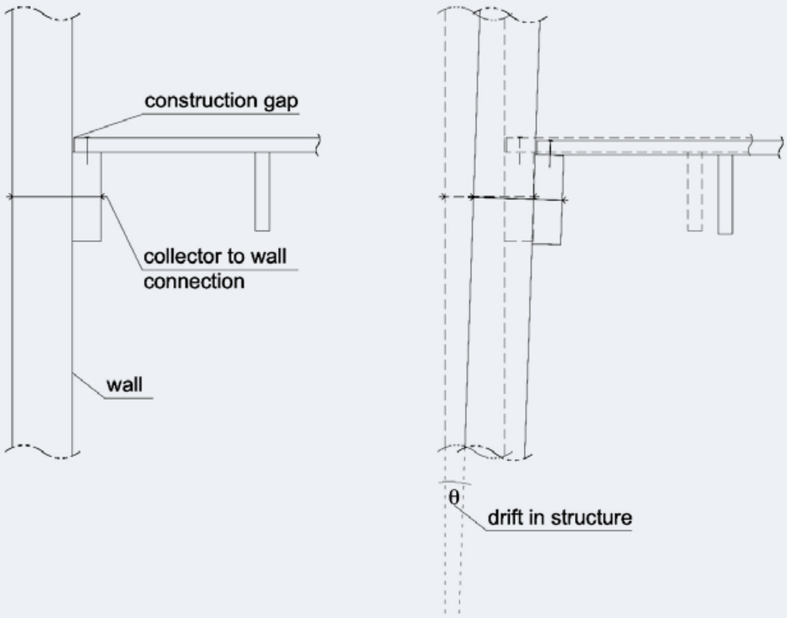


Figure 12.4: Construction gap between the floor and the wall to allow for rotation: a) undeformed state; b) deformed state.

13

Connection Details for Timber Concrete Composite (TCC) Floors

As a result of the low tensile strength of concrete, tearing forces due to frame elongation and bending forces due to uplift and rotation of the walls tend to crack the diaphragm topping. If these cracks become larger, the force transfer is interrupted and the diaphragm action compromised[13]. It is essential to design the diaphragm with its connections accordingly.

For frame structures with TCC floors, the displacement incompatibility required from the beam-column-gap opening can be accommodated similarly to the concentrated floor gap solution already described for timber diaphragms. As suggested in Figure 13.1, the concrete should be pre-cracked along the line of the beam–column joint. Unbonded rebars should be placed across the crack, designed to deform elastically in case of gap opening and to provide shear transfer via dowel action.

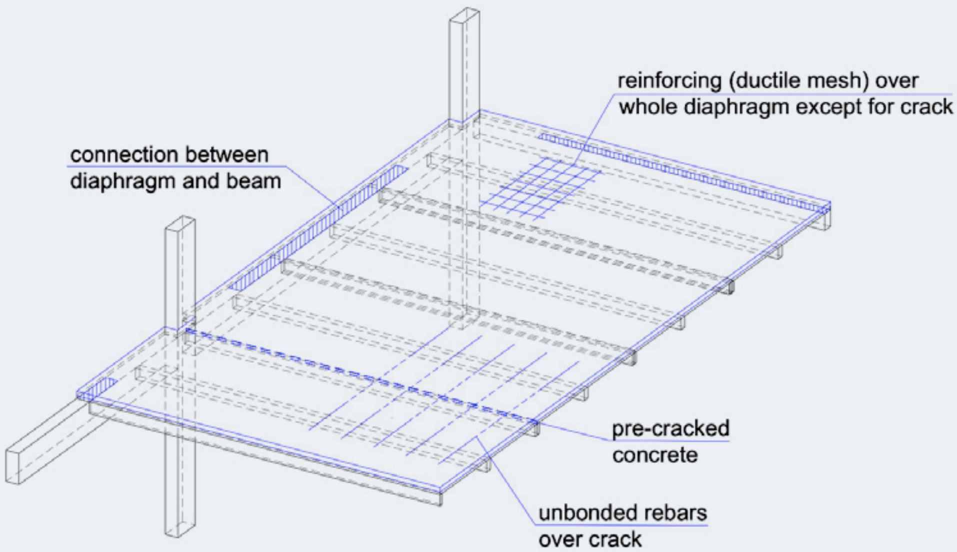


Figure 13.1: Suggested detailing for a TCC floor in a frame system.

The diaphragm has to be tied appropriately to the collector beams. One way to do this is shown in Figure 13.3. The force transfer from the diaphragm to the beam should be guaranteed in the central portion of the beams, leaving it unconnected close to the beam–column joints (in the disturbed areas shown in Figure 13.2). In this way, frame elongation will not compromise the force transfer, which starts away from the disturbed areas where the displacement incompatibility is attenuated. This is especially important on external beams and columns, as no concentrated gap opening can be guaranteed.

A different solution to avoid the frame elongation problem on a multi-bay frame consists in connecting the diaphragm only to one bay and letting the diaphragm slide over the remaining beams. This solution, however, might result in high shear forces at the connection between the diaphragm and the beam, and requires proper detailing to allow for the sliding of the diaphragm in respect to all other elements.

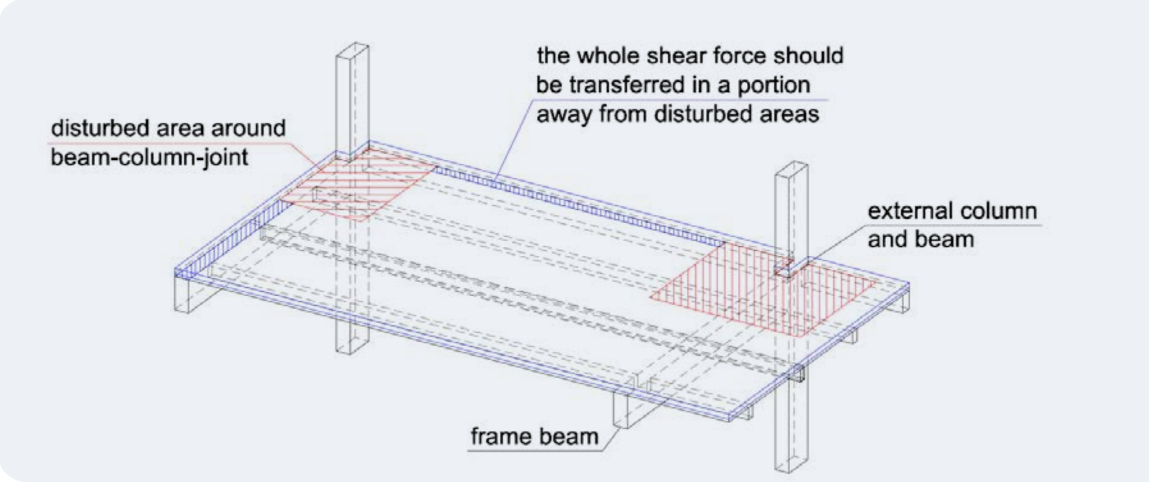


Figure 13.2: Shear transfer between the concrete topping and beams.

If the floor gap opening occurs along a collector beam or tie back, care is needed as cracking of the concrete can compromise the force transfer. Ideally, the pre-crack should be placed away from any connection to the beams.

Figure 13.3 shows a suggested connection between the concrete topping and the collector or frame beam. The diaphragm shear is introduced to the beam via notched connections used for the TCC design (see WoodSolutions Technical Design Guide #30: *Timber Concrete Composite Floors*). If the concrete topping is connected to the beam directly, the beam has to be designed as a composite section. As an alternative, an edge joist from the TCC floor can be connected to the frame beam via a timber-timber connection. Starter bars are required by the code and have to tie the collector/strut beam to the diaphragm as well as carrying the shear in case of a crack along the interface.

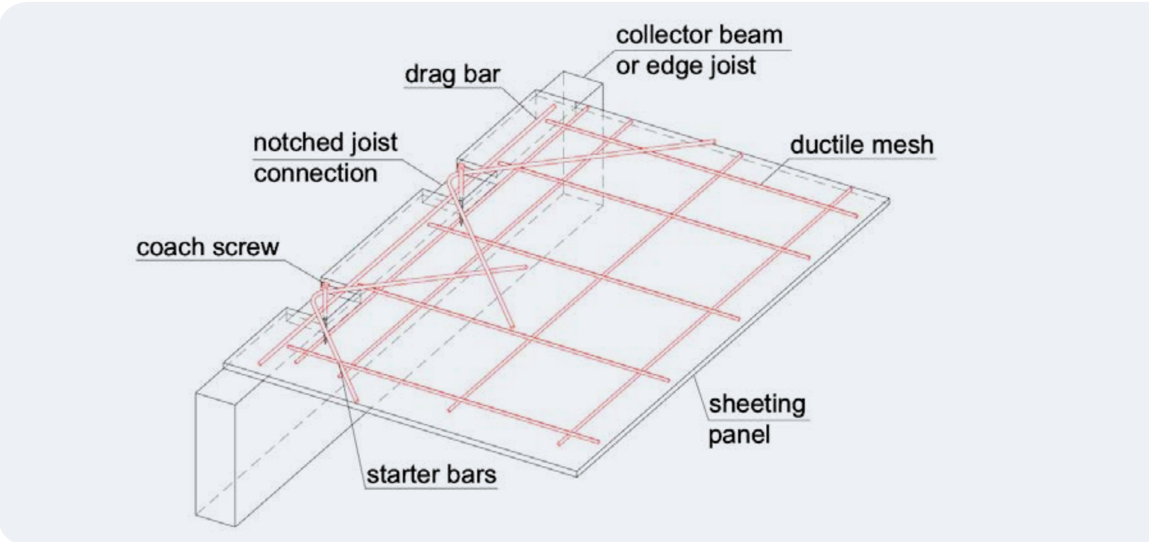


Figure 13.3: Suggested connection between the concrete topping and timber beams.

In wall structures with TCC floors, the force transfer occurs between the wall and the collector beam and is unaffected by the presence of the concrete topping. The connection between the diaphragm topping and the collector/strut beam should be designed as described for frame structures.

If no slotted solution can be adopted for the wall connection, an eventual out of plane bending of the beam has to be considered. Again, if the span to the next vertical restraint is large enough, the bending should be accommodated in the concrete topping without excessive cracking. The additional re-centring force in the wall should be considered, as it might give substantial contribution.

The use of concrete diaphragms in structures that undergo beam elongations cause several complications and special detailing is required[13,14]. The suggestions provided in this Guide have not been fully tested and should be applied with proper engineering judgment.

14

Design of TCC Floor Diaphragm

To design the TCC diaphragm, a strut and tie model as per Section 7 “Strut and Tie Modelling” of the AS 3600 Australian Concrete Code has been adopted. For this design example, only the design for the wind load applied perpendicularly to the long side of the building has been carried out. For wind loads perpendicular to the short side of the building, only a conceptual strut and tie model is shown.

The uniformly distributed loads on the windward and leeward façades (Figure 14.1 and Figure 14.2) for a wind load have been applied on a four metre grid. The resultant forces are transferred by a collector beam running on the inner side of the staircase into the post-tensioned walls. The 100 mm concrete topping is reinforced with a ductile mesh (Ø6.75 mm Grade 500 rebars on 200 mm centres with a resulting reinforcement area of 179 mm² per metre width).

Wind loads at ULS:

$$p = \left(p_{windward} + p_{\text{internal}}\right)h \qquad (14\text{-}1)$$

$$p_1 = \left(0.35 \, {}^{kN}\!/\!_{m^2} + 0.21 \, {}^{kN}\!/\!_{m^2}\right)3.6m = 2.0 \, {}^{kN}\!/\!_{m} \text{ load on windward façade} \qquad (14\text{-}2)$$

$$p_2 = \left(0.25 \, {}^{kN}\!/\!_{m^2} + 0 \, {}^{kN}\!/\!_{m^2}\right)3.6m = 0.9 \, {}^{kN}\!/\!_{m} \text{ load on leeward façade} \qquad (14\text{-}3)$$

The loads applied to the edge of the diaphragm are carried over compression struts into the collector beam. Several chord beams along the depth of the diaphragm are taking the tension forces, in this way the forces can be kept relatively low.

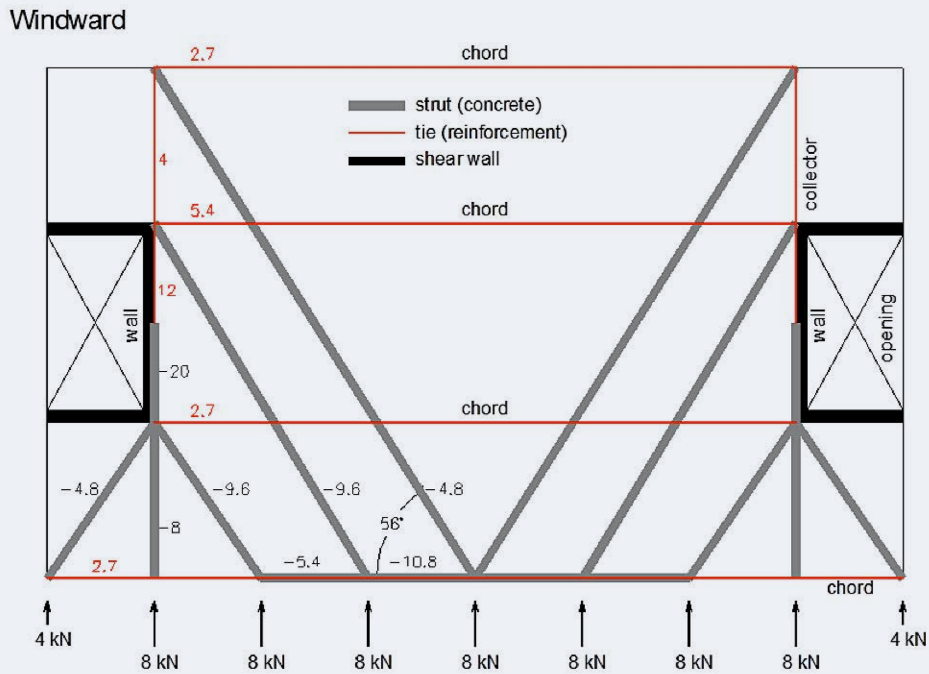


Figure 14.1: Strut and tie model for wind loads on the windward façade.

On the leeward façade, the wind loads are first carried over the tension ties into the diaphragm. From there, compression struts carry the forces into the collector beams.

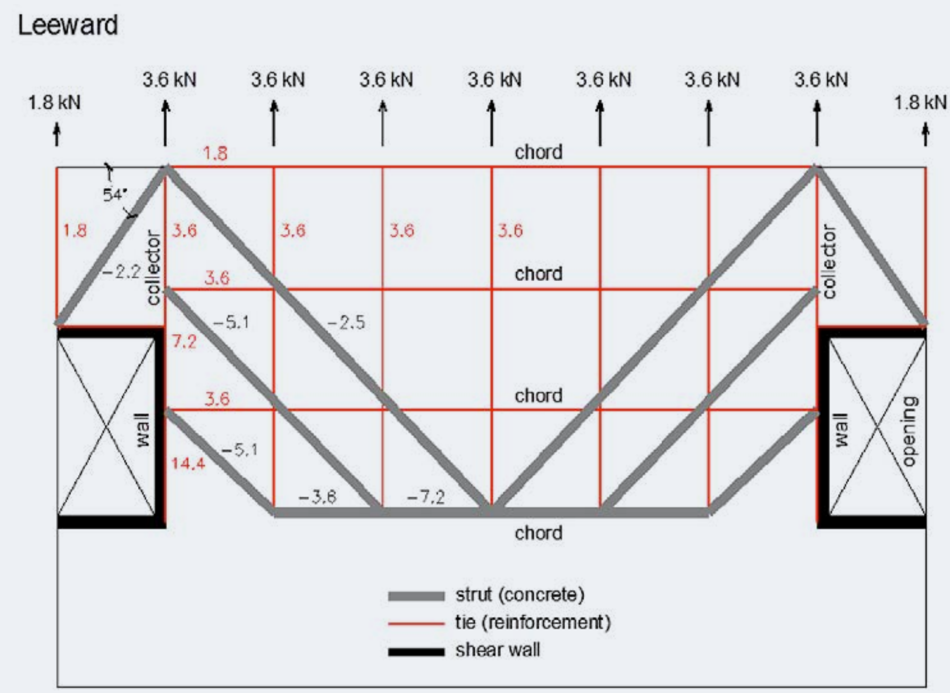


Figure 14.2: Strut and tie model for wind loads on the leeward façade.

There is no unique strut and tie model for a given geometry and load scenario. A minimum of experience is required to set up a well-balanced model, as incomplete or not well elaborated models might lack of equilibrium at the nodes, undergo excessive deformation or require load redistribution because after concrete cracking.

The conceptual strut and tie model for a wind load perpendicular to short sit of the building is depicted in Figure 14.3.

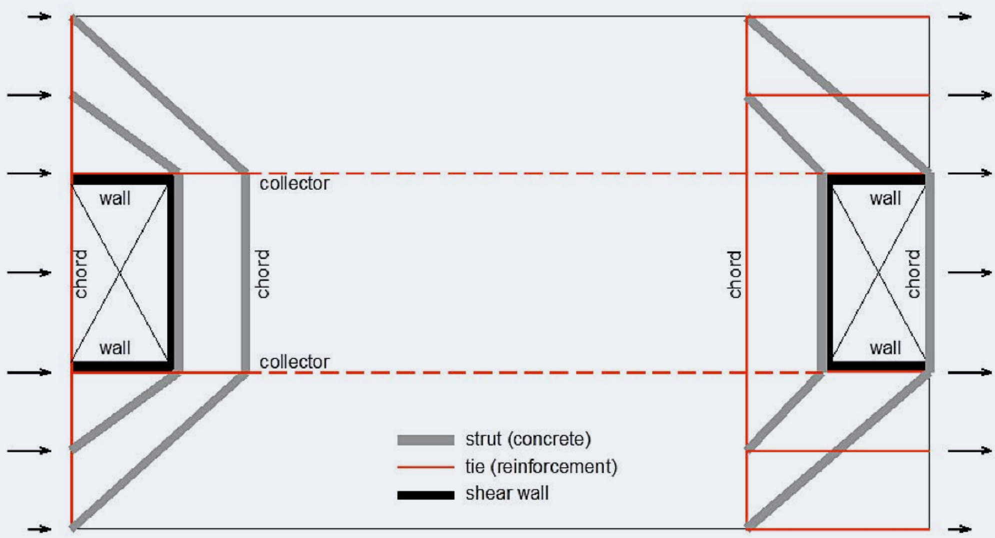


Figure 14.3: Strut and tie model for wind load perpendicular to the short side of the building.

14.1 Tension Ties

The concrete topping is reinforced by a D500DL72 ductile mesh (reinforcing in both direction made of Grade 500 reinforcing bars), which also satisfies the minimum reinforcement for crack control for shrinkage and temperature effects as per Clause 9.4.3 in AS 3600:

$$A_{s,\min} = 75\% \cdot \left(1.75 - 2.5\sigma_{cp}\right) bD \cdot 10^{-3} = 0.75 \cdot 1.75 \cdot 100mm \cdot 1000mm = 131mm^2 \qquad (14\text{-}4)$$

where

$A_{s,min}$ = minimum reinforcement area

σ_{cp} = average intensity of effective pre-stress (0 MPa in this case)

b = width of the diaphragm (taken as 1 m)

D = depth of the concrete topping.

As shown in Figure 14.1 and Figure 14.2, the maximum force in the ties is 12+14.4 = 26.4 kN along the collector beam running parallel to the wall. Along the collector beam, two additional Ø10 Grade 300 reinforcing bars are placed.

$$F_{nt} = \phi_{st} A f_y = 0.8 \cdot \frac{10^2 \cdot \pi}{4} \cdot 300 = 48kN \geq F^* = 26.4kN \therefore OK \qquad (14\text{-}5)$$

where:

F_{nt} = nominal tension capacity of steel tie

A = area of the tension reinforcement

ϕ_{st} = capacity factor for tension struts (0.8).

All other ties have forces of maximum 5.4 kN; therefore, a single leg of the ductile mesh with a Ø6.75 mm Grade 500 rebar provides enough strength to transfer the tension forces.

$$F_{nt} = \phi_{st} A f_y = 0.8 \cdot \frac{6.75^2 \cdot \pi}{4} mm^2 \cdot 500 \, N/_{mm^2} = 14.3kN \geq F^* = 5.4kN \qquad (14\ 6)$$

To guarantee the force transfer in the tension ties, the reinforcing bars and the ductile mesh have to be placed with the required overlapping as provided by the code or the manufacturer.

14.2 Compression Struts

The design strength of a concrete strut, neglecting the reinforcement, is:

$$F_{nc} = \phi_{st} \beta_s 0.9 f'_{conc} A_{conc} \qquad (14\text{-}7)$$

where

F_{nc} = nominal compression capacity of concrete strut

f'_{conc} = compressive strength of concrete

A_{conc} = cross sectional area at one end of the strut, considering the thickness as the depth of the diaphragm slab (see AS 3600 Australian Concrete Code for more detail)

β_s = efficiency factor for concrete struts

ϕ_{st} = capacity factor for compression struts (0.6).

Considering the maximum force in a strut of only 20kN and a thickness of the slab of 100 mm, all struts are easily verified. Detailed verifications of the struts and the nodal areas are left to the reader.

The reinforcement plan for the concrete diaphragm is shown in Figure 14.4. Appropriate overlapping of the reinforcing bars and the mesh has to be guaranteed.

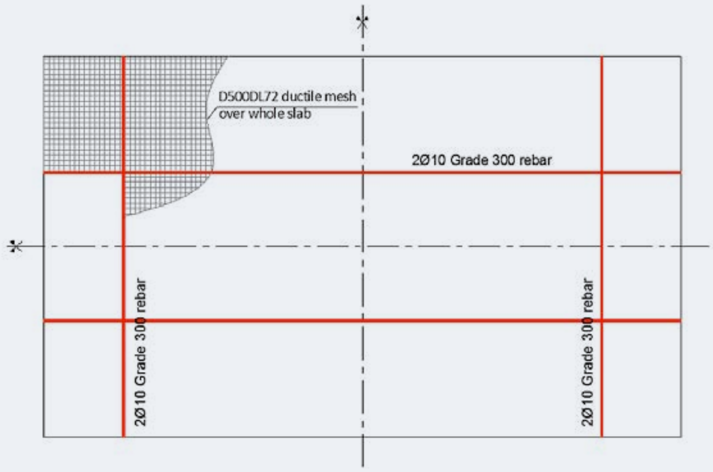


Figure 14.4: Reinforcement plan.

14.3 Connection of the Collector to Walls

The force transfer between the diaphragm and the wall can be realized in different ways. Two design solutions, which are also compatible with the gravity force transfer, are shown. Since the uplift and rotation of the wall is negligible for this design, no special detailing for the connection between the floor and the lateral load-resisting system is required.

Solution 1– Direct Connection between Concrete Slab and Wall

Solution 1 consists in a diaphragm force transfer via coach screws fixed directly to the LVL post-tensioned wall. These are then integrated in the concrete slab when it is cast into place. The floor elements are sitting on corbels or fixed by steel hangers that are directly connected to the wall. The design of the latter is not shown here.

The diaphragm force to be transferred into the wall is 46.4 kN (26.4 kN from the ties and 20 kN from the strut). This force is transferred from the concrete topping to the wall through 6 Ø12 coach screws (Figure 14.5).

According to clause C4.2 of AS 1720.1:2010, and considering an effective timber thickness of $b_{eff} = 2 \times t_p = 192$ mm and embedment strength of $f'_{pj} = 17$ MPa, the connection capacity is as follows:

$$t_p \geq 8D = 8 \cdot 12mm = 96mm \quad (14\text{-}8)$$

$$Q_{sk} = Q_{skp} = \min\begin{Bmatrix} b_{eff} f'_{pj} D/2 \\ 15 f'_{pj}\sqrt{D^3} \end{Bmatrix} = \min\begin{Bmatrix} 19.6kN \\ 10.6kN \end{Bmatrix} = 10.6kN \quad (14\text{-}9)$$

where:

t_p = penetration length of fastener

Q_{sk} = characteristic capacity for a laterally loaded single bolt in a joint system

Q_{skp} = system capacity for fasteners loaded perpendicular to the grain

f'_{pj} = characteristic value for bolts bearing perpendicular to grain

The second member in the connection is the concrete slab, which can be considered as stiff; hence, the factor for side plates (k_{16}) can be taken as 1.2.

$$N_{dj} \geq N^*$$
$$N_{dj} = \phi k_1 k_{13} k_{16} k_{17} n Q_{sk} = 0.8 \cdot 1.14 \cdot 1.0 \cdot 1.2 \cdot 1.0 \cdot 6 \cdot 10.6kN = 69.6kN \geq N^* = 46.4kN \quad (14\text{-}10)$$

where

N_{dj} = design capacity for joints under direct load

k_1 = 1.14 for wind loads as per Clause 2.4.1.1.

k_{13} = 1.0 factor for end grain effects

k_{16} = 1.2 factor for side plates

k_{17} = 1.0 factor for multiple fastener effect.

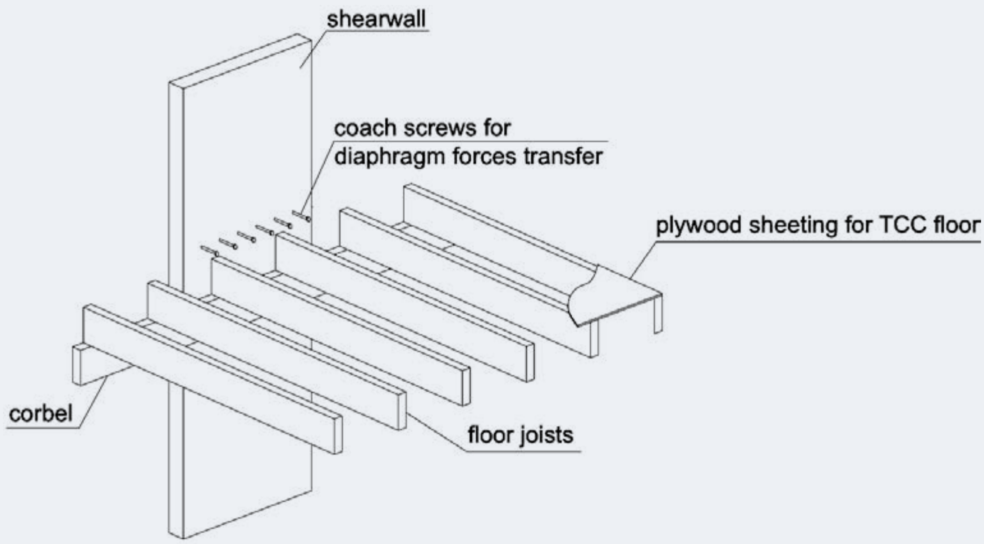


Figure 14.5: Diaphragm force transfer over coach screws.

The strength of the coach screws embedded in the concrete can be checked according Australian Code AS 2327.1:2003 Composite structures. Part 1: Simply supported beams.

Solution 2 – Connection between Concrete Slab and Wall via Strut/Collector Beam

An alternative solution consists in a transverse LVL beam running parallel to the wall, fixed with bolts to it. The diaphragm forces are transferred via notched connections from the concrete topping into the timber beam (Figure 14.6). For the gravity loads, the floor joists are connected to the transverse beam by steel hangers. The connection of the beam to the wall is designed for the combination of gravity and horizontal wind loads.

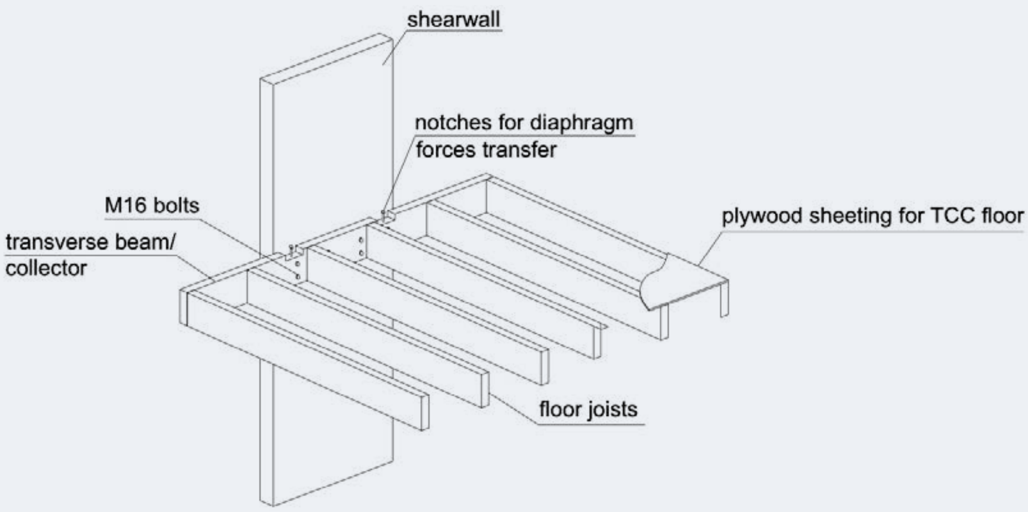


Figure 14.6: Transverse beam with notches for the diaphragm force transfer.

The factored gravity load is:

$$p = 1.2G + 1.5\psi_a Q = 6.7\, {}^{kN}\!/_{m^2} \qquad (14\text{-}11)$$

and hence the gravity force on the beam considering a tributary area approach is:

$$F = p \cdot A = 6.7\, {}^{kN}\!/_{m^2} \cdot \frac{4m}{2} \cdot 7m = 94kN \qquad (14\text{-}12)$$

From the wind load a horizontal force of 46.4 kN has to be transferred into the wall. This leads to a resultant force and respective angle of:

$$R^* = \sqrt{46.4^2 + 94^2} = 105\ \text{kN} \qquad (14\text{-}13)$$

$$\vartheta = \arctan\frac{94}{46.4} = 63.7^\circ \qquad (14\text{-}14)$$

M16 bolts are used to connect the beam to the wall. A joint group JD3 is assumed for the connection in LVL elements. The thickness of the connected members is 90 mm for the beam and 225 mm for the wall. Since the Australian Timber Code AS 1720.1:2010 does not provide the situation of a force transferred on an angle to the grain in between two members running at 90° to each other, as a conservative approach the resultant force is applied perpendicularly to the beam.

Considering b_{eff} = 2 x 90 mm = 180 mm and f'_{pj} = 17 MPa the connection capacity is:

$$Q_{sk} = Q_{skp} = \min\begin{Bmatrix} b_{eff} f'_{pj} D/2 \\ 15 f'_{pj}\sqrt{D^3} \end{Bmatrix} = \min\begin{Bmatrix} 24.5kN \\ 16.3kN \end{Bmatrix} = 16.3kN \qquad (14\text{-}9)$$

A connection with 8 M16 bolts is chosen to transfers the load from the beam into the wall:

$$N_{dj} \geq R^*$$
$$N_{dj} = \phi k_1 k_{16} k_{17} n Q_{sk} = 0.8 \cdot 1.14 \cdot 1.0 \cdot 1.0 \cdot 8 \cdot 16.3kN = 119 \geq R^* = 105kN \qquad (14\text{-}10)$$

A different way to connect the collector beam to the wall could be by using inclined fully threaded screws; this is, however, not covered here.

To transfer the diaphragm force from the concrete topping into the beam, two notched trapezoidal connections are provided (Figure 14.7). More information on the design of these connections can be found in the WoodSolutions Technical Design Guide #30:*Timber Concrete Composite Floor Systems*.

$$2N_{dj} = 152.4 > Q^* = 64.4\text{kN} \qquad (14\text{-}15)$$

$$2N_{dj} = \phi k_1 k_4 k_6 Q_k = 0.8 \times 1.14 \times 1.0 \times 1.0 \times 83.5\text{kN} = 76.2\text{kN} \qquad (14\text{-}16)$$

$$Q_k = 0.95 \times 90 - 2 = 83.5\text{kN} \qquad (14\text{-}17)$$

where:

Q_k = characteristic strength of the TCC connection in shear

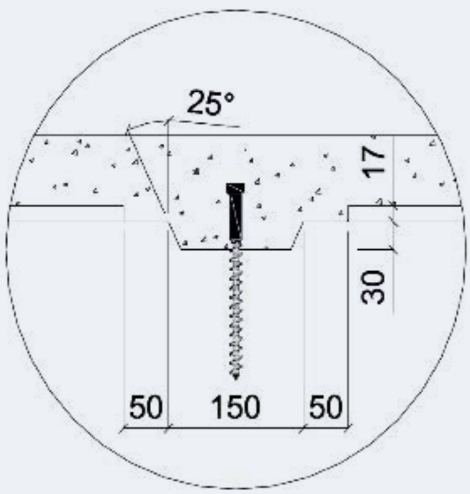


Figure 14.7: Trapezoidal notch.
Source: WoodSolutions Technical Design Guide #30:Timber Concrete Composite Floor Systems.

References

1. Bull, D.K. and R. Henry, *Strut and Tie. Seminar Notes. Technical Report No.R57.*, in *Bulletin of the New Zealand Society for Earthquake Engineering*. 2014, The New Zealand Concrete Society. Canterbury Earthquake Royal Commission 2012.
2. Moroder, D., et al. An equivalent truss method for the analysis of timber diaphragms. in Proceedings of the Tenth Pacific Conference on Earthquake Engineering -Building an Earthquake-Resilient Pacific. 2015. Sydney, Australia.
3. Moroder, D., et al., *Design of Floor Diaphragms in Multi-Storey Timber Buildings*. International Network on Timber Engineering Research. Bath, England, 2014.
4. Moroder, D., et al., Seismic design of floor diaphragms in post-tensioned timber buildings, in World Conference on Timber Engineering. 2014: Quebec City, Canada.
5. Malone, R.T. and R.W. Rice, The Analysis of Irregular Shaped Structures - Diapghragms and Shear Walls. 2012: McGraw Hill.
6. Diekmann, E.F., *Diaphragms and Shear Walls, in Wood Engineering and Construction Handbook*, K.F. Faherty and T.G. Williamson, Editors. 1995, McGraw-Hill: New York. p. 8.1-8.68.
7. Malone, R.T. and R. Rice, The Analysis of Irregular Shaped Structures Diaphragms and Shear Walls. 2011: McGraw Hill Professional.
8. Vogt, T., J. Hummel, and W. Seim. Timber framed wall elements under cyclic loading. in 12th World conference on timber engineering, Auckland. 2012.
9. Smith, P.C., D.J. Dowrick, and J.A. Dean, Horizontal timber diaphragms for wind and earthquake resistance. Bulletin of the New Zealand Society for Earthquake Engineering, 1986. 19(2): p. 135-142.
10. ATC, *Guidelines for the design of horizontal wood diaphragms*. Vol. ATC 7. 1981, Berkeley, California: Applied Technology Council.
11. Prion, H.G.L., *Shear Walls and Diaphragms*, in *Timber engineering*, S. Thelandersson and H.J. Larsen, Editors. 2003, J. Wiley: New York. p. 383-408.
12. Standards New Zealand, Timber Structures, in NZS 3602:1993. 1993, Standards New Zealand: New Zealand.
13. Bull, D.K., *Understanding the complexities of designing diaphragms in buildings for earthquakes.* Bulletin of the New Zealand Society for Earthquake Engineering, 2004. 37(2): p. 70-88.
14. Standards New Zealand, 1170.5 Supp - Structural Design Actions Part 5: Earthquake Actions - New Zealand - Commentary. 2004b: Wellington, New Zealand.

Australian Standards

AS 1720.1*, Timber structures*, in Part 1: *Design methods*. 2010, Standards Australia: Australia.

AS 1170.2:2002 *Structural Design Actions Part 2: Wind Actions*. 2002, Standards Australia, Australia.

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

AS 3600 *Concrete structures*, 2009, Standards Australia, Australia.

WoodSolutions Design Guide

WoodSolutions Technical Design Guide #30, *Timber Concrete Composite Floor System.* WoodSolutions, 2015, Melbourne, Australia.

A

Appendix A - Notations

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

a	aspect ratio of each sheathing panel given in the NZS 3603.[12]
A	section area of the chord
A	area of the tension reinforcement
A_{conc}	cross sectional area at one end of the strut, considering the thickness as the depth of the diaphragm slab as per AS 3600
$A_{s,min}$	minimum reinforcement area
b	width of the diaphragm (taken as 1 m)
B	distance between diaphragm chord member
b_{eff}	effective width of member in joint assembly
D	depth of the concrete topping
e_n	fastener slip resulting from the shear force V
F	gravity force on the beam
F_{nt}	nominal tension capacity of steel tie
F_{nc}	nominal compression capacity of concrete strut
F^*	design action in tension
f'_{conc}	compressive strength of concrete
f'_{pj}	characteristic value for bolts bearing perpendicular to grain
f_y	yield strength of steel
G	shear modulus of the diaphragm sheathing
h	diaphragm width
l	length of diaphragm
k_1	modification factors for duration of load given in AS 1720.1
k_{13}	nail connector factor for end grain effects given in AS 1720.1
k_{16}	nail connector factor for plywood or metal side plates given in AS 1720.1
k_{17}	nail connector factor for multiple fastener effect given in AS 1720.1
m	thickness of the diaphragm sheathing
M	design moment of diaphragm
N_{dj}	design capacity for joints under direct load
N^*	design action for joints under direct load
P	wind load at ULS
P_1	wind load on windward façade
P_2	wind load on leeward façade
p	factored gravity load
q	uniformly distributed horizontal load
Q_k	characteristic strength of the TCC connection in shear
Q_{sk}	characteristic capacity for a laterally loaded single bolt in a joint system
Q_{skp}	system capacity for fasteners loaded perpendicular to the grain

R^*	resultant force
t	thickness of the diaphragm sheathing
t_p	penetration length of fastener
V	shear force applied to a diaphragm
v	shear flow along the edges of the diaphragm
x	distance of the splice from the origin
β_s	efficiency factor for concrete struts
σ_{cp}	average intensity of effective pre-stress (0 MPa in this case)
δ_s	splice slip in the chord
Δ_1	deflection due to bending
Δ_2	deflection due to shear in the panels
Δ_3	deflection due to the connection elements
Δ_4	deflection of the diaphragm due chord connection deformation
$\Delta_{diaphram}$	diaphragm deflection at mid-span
$\Delta_{interstory\ drift\ LLRS}$	lateral-ing system drift
ϕ_{st}	capacity factor for tension struts (0.8)
ϑ	angle resultant force transferred into the wall

Discover more ways to build your knowledge of wood

If you need technical information or inspiration on designing and building with wood, you'll find WoodSolutions has the answers. From technical design and engineering advice to inspiring projects and CPD linked activities, WoodSolutions has a wide range of resources and professional seminars.

www.woodsolutions.com.au

Your central resource for news about all WoodSolutions activities and access to more than three thousand pages of online information and downloadable publications.

Technical Publications

A suite of informative, technical and training guides and handbooks that support the use of wood in residential and commercial buildings.

WoodSolutions Tutorials

A range of practical and inspirational topics to educate and inform design and construction professionals. These free, CPD related, presentations can be delivered at your workplace at a time that suits you.

Seminars and Events

From one day seminars featuring presentations from leading international and Australian speakers to international tours of landmark wood projects, WoodSolutions offer a range of professional development activities.

What is WoodSolutions?

Developed by the Australian forest and wood products industry for design and building professionals, WoodSolutions is a non-proprietary source of information from industry bodies, manufacturers and suppliers.

Wood Solutions™
design and build
woodsolutions.com.au

36

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Oct 2016
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Engineered Woods and Fabrication Specification

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image courtesy of Carter Holt Harvey Woodproducts

WoodSolutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-34-8

Acknowledgments

Authors: TDA

The research and development forming the foundation of this Design Guide as well as its preparation and production was proudly made possible by the shareholders and financial partners of the Structural Timber Innovation Company Ltd.

First published: 2015
Revised: October 2016

These materials are published under the brand WoodSolutions by FWPA. This guide has been reviewed and updated for use in Australia by TDA NSW.

IMPORTANT NOTICE

WoodSolutions Australia is a registered business division of Forest and Wood Products Australia Limited.

Contents

1

Introduction

This Guide provides a summary of fabrication and installation specifications of engineered wood products, i.e. laminated veneer lumber (LVL) and glulam.

It provides recommendations for different steps of the timber structure supply chain, including storage, handling and transportation, erection and assembly. It includes insect and mould preventions and moisture design considerations.

This Guide is based on information contained within the Structural Timber Innovation Company, EXPAN guides and timber manufacturing and supply companies. It is considered general information only and should not take precedence over manufacturers' literature and specifications.

2

Fabrication Specifications

2.1 General

Fabrication should be in accordance with recognised sound practice, using adequate plant and equipment under the supervision of qualified personnel, and must be adequate and uniform with correct damping sequences and contact pressures.

2.2 Timber

Timber should have characteristic strengths and stiffness properties. For glulam members manufactured to AS/NZS 1328, they should have stress grades of GL8, GL10, GL12, GL13, GL17, GL18 or GL21* as given in AS 1720.1. Alternatively, manufacturers or suppliers of the Glulam may have specific characteristic strengths and stiffness of glulam members.

Characteristic strengths and stiffness for Laminated Veneer Lumber (LVL) should be as per the LVL manufacturers or suppliers.

GL21 should be as per the manufacturer's detail.

2.3 Moisture Content

The moisture content of glulam or LVL members should be between 8% and 15%.

2.4 Adhesive Used for Manufacturing and Fabrication

Adhesive components used in fabrication of timber elements should be stored, mixed, handled, spread and cured in accordance with the adhesive manufacturers' instructions. For service classed 1 and 2, a minimum bond of type II should be used. A minimum bond of type I is required for service class 3 (exposed exterior use). Refer to AS 4364 for further information on bond types.

2.5 Appearance of Finished Members

2.5.1 Glulam

The appearance of glulam should meet the requirements of grade A, B or C from AS/NZS 1328.2, or as required by the specifier. Refer to Table 2.1 for descriptions of each of these grades. Special finishes, such as band-sawn, should be nominated by the specifier and by direct negation with the manufacturer or supplier.

Table 2.1: Glulam appearance grades.

Grade	***Intended Use***	***Requirement***
A	This grade is intended for use in applications where appearance of the member is important and clear or painted finishes are to be used.	Filled and sanded finish. All surface voids to be plugged or filled.
B	This grade is intended for use in applications where surface appearance is important but a machine-planed finish is acceptable.	Machine-planed finish with occasional skips, blemishes and voids.
C	This grade is intended for use in applications where appearance is unimportant.	All blemishes and voids are acceptable.

2.5.2 Laminated Veneer Lumber (LVL)

Generally LVL has a sanded surface but, because the knot holes are not filled, the appearance of LVL is similar to the requirements of 'DD' plywood. Refer to the plywood standard AS 2269.0 for further information.

Other appearance specifications for LVL elements are possible, but only by direct negation with the manufacturer or supplier.

2.6 Protection of Finished Members

Where timber elements are to be exposed to the weather during their erection and/or are to be the final decorative surface, all members should be weather protected with a minimum of one coat of weather-approved sealer.

End grain should be sealed with a minimum of two coats of sealer or end capped. All surfaces need to be maintained in good order. All surfaces to be coated should be clean, dry and free from mould, fungi, etc.

Where LVL is not exposed to the weather, there is no need to use weather sealer.

3

Timber Storage, Transportation and Handling

This Section outlines the specific requirements for the on-site storage, transportation and handling of LVL and glulam. Strict adherence to these requirements will ensure that the finished product performs to specifications.

Engineered wood products must be stored properly and handled with care to assure optimum performance. Care must be taken during loading, unloading and transportation, as well as when they are in storage and on the construction site, to protect them from damages.

Engineered wood products may be supplied with various forms of protection, depending on their final application. This protection should be commensurate with the end use of the products.

3.1 Loading and Unloading

The wood products should be loaded, unloaded and secured during transport by means that will not damage the edges, surfaces or packaging.

Timber beams are commonly loaded and loaded using forklifts. For more stability and safety, it is recommended to place the sides of the beams flat on the forks, rather than the bottoms of the beams, as depicted in Figure 3.1. In the case of extremely long timber beams, two or more forklifts maybe need to lift the beams in unison to avoid flex.

To avoid damage to structural timber members, only fabric webbing slings should be used to lift or secure timber products. Chains and wire slings are not recommended. Where chains and wire ropes are used, adequate corner protection is required. Slings should be located carefully to ensure balanced support or a spreader bar should be used. Guy lines should be used to control the members during lifting. Timber members should be lifted on edge wherever possible and spreader bars of suitable length used on long members to eliminate the possibility of overstressing the member and the risk of damage during lifting.

Figure 3.1: Loading and unloading timber beams using a forklift.

3.2 Timber Storage

Engineered wood products should be kept dry on-site and protected from direct exposure to the weather. The following recommendations for storage are made to ensure that the full benefits of engineered wood products as a dry, straight and true material are available at the time of installation:

1. Stack on level bearers to keep flat and straight.
2. Stack well clear of the ground using timber blocking, skids or rack systems for good ventilation. as shown in Figure 3.2.
3. Store in a level, well-drained covered storage site to keep dry prior to installation during storage and transport. If covered storage is not available, the members should be covered with suitable non- transparent plastic or tarpaulins.
4. Forklift damage should be prevented. If the ground is not level in the storage area, reduce forklift speed to avoid bouncing the load.
5. The cover should be placed to preclude moisture while maintaining good air circulation in and around the members with fillets placed between each layer.
6. Bearers and fillets should be placed vertically in line to support engineered wood products evenly and flat.
7. Glulam members that are supplied individually wrapped should be placed on the dunnage with the wrapping material edge or seal face down as shown in Figure 3.3.
8. Always store I-joist beams vertically and level – never flatwise, as shown in Figure 3.4.

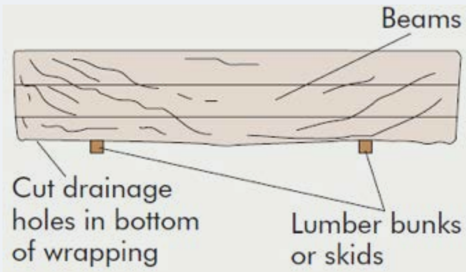


Figure 3.2: Stacking timber clear off the ground by skids.

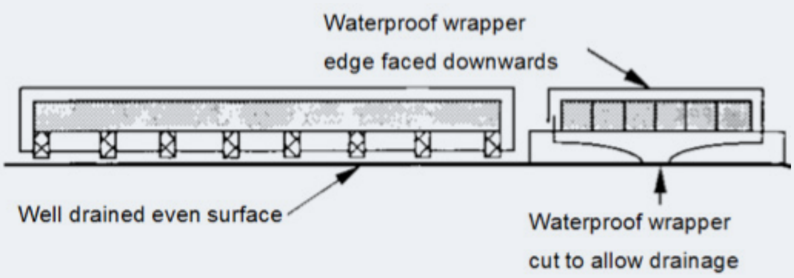


Figure 3.3: Site storage of glulam.

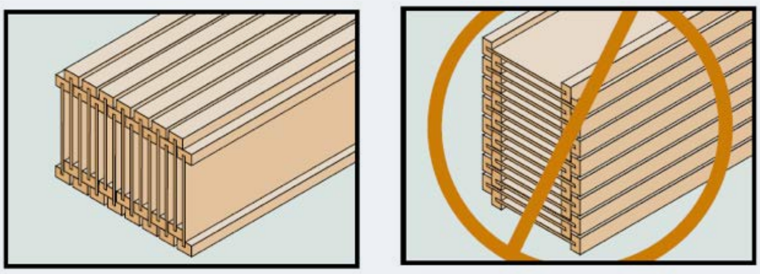


Figure 3.4 Storage of I-joist beams.

Engineered wood products should be stored on level bearers at maximum 1800 mm centres at least 75 mm clear of the ground, well ventilated and away from any source of ignition.

The bearers, such as skids or blocks, need to be located so that self-weight is uniformly supported to avoid distortion. Strips or blocks as spacers need to be used, in line vertically, between components to avoid dirt or water being trapped between timber faces.

If the elements are wrapped, they need to be kept wrapped. The wrapping protects them from moisture, soiling, sunlight and scratches. For long-term storage, cutting slits in the bottom of the wrapping will allow ventilation and drainage of the entrapped moisture.

Exposure to rain can lead to swelling and staining. Exposure to sunlight can darken timber quite quickly.

3.2.1 Fire During Construction

To minimise construction fire risk, consideration needs to be given to the storage location of timber elements prior to their installation. As far as possible, program the delivery of combustible materials to minimise the time they are stored on-site.

Where significant volumes of combustible building materials are to be kept on-site, they should be stored in a secure area at least 10 metres away from any buildings or partially constructed buildings, and any location where hot works are undertaken.

Where there are no reasonably practicable alternatives and combustible building materials have to be stored within or close to the building under construction, the area used for storage should:

- have controlled access
- not be in an area where hot works are being carried out
- be in either an area covered by the site fire detection system or included on the route of regular fire checks
- have firefighting equipment close by
- be protected from ignition sources, where reasonably practicable, by fire preventative covers (e.g. fire retardant, fire resistant, or non-combustible sheeting).

For more information refer to WoodSolutions Technical Design Guide #20: *Fire Precautions During Construction of Large Buildings.*

3.3 Transportation and Handling

Engineered wood products (EWP) should be kept dry during transportation by individual wrapping or as a truck load tarped to protect the products from weather and wheel spray.

Engineered wood products should be stacked or supported so that they are not subject to permanent bending or twisting that will affect the final intended shape.

Care should be taken to prevent damage to the finished surfaces in all handling; such treatment may cause damage to the surfaces and edges and possibly structural damage. Causes of mechanical damage to surfaces of engineered wood products occur from dropping, jarring, crowbarring or dragging the products, or running into them with lifting equipment.

3.4 Erection

The safety of all erection operations should be the responsibility of the specialist contractor. Design of the lifting system should be agreed upon during the design process. A detailed method statement for erection should be developed in agreement with the structural engineer.

The specialist contractor may be required to supply an erection supervisor to oversee the installation of all timber elements. Cranage, scaffolding and erection equipment should be provided by the specialist contractor.

The building contractor must liaise with the structural engineer for the project to determine the propping requirements that may or may not be required. These requirements will be dependent on the design philosophy adopted by the engineer around managing the creep deflections associated with moisture. After installation, exposure to sun and rain for normal periods of construction is not a cause for concern. (Add detail regards to finishes, maintenance, etc)

4

Mould Prevention

4.1 Occurrence

Moulds and algal growths occur on both treated and untreated timber and on timber that is seasoned. Their presence is related to temperature, humidity, wetting and the presence of atmospheric mould spores. Both sapstain and moulds can be black. Some moulds are green, but algae are associated with surface water as compared to high moisture content timber. Mould growth can spread rapidly in favourable circumstances, especially warm humid weather.

4.2 Effect of Moulds

Moulds and algae are a surface effect and do not penetrate the wood structure. Sapstain and decay fungi penetrate the wood and actively grow within the wood section.

The engineered wood product is not weakened by their presence even for lengthy periods. Moulds do not result in damage to the wood structure or loss of strength during normal construction periods. With appropriate treatment, mould and algae are not a decay hazard. In wet situations, e.g. decking, moulds or algae can be slippery, which becomes a safety issue.

Where weather exposure becomes extensive, causing high moisture contents for long periods, fungal decay may become a hazard and may require investigation. Where this happens, it may become associated with fungal attack and some elements close to the ground may require investigation.

4.3 Termination

Below about 18% moisture content, mould development will cease. Enclosing the structure and allowing it to dry out will eliminate the presence of mould, although there may be discolouration. Providing provision for floors to drain in wet weather is strongly recommended, as is early close in for the structure. Any flowering parts of the mould will fall or brush off and will have no effect on dry cavities. Similarly, dry out eliminates surface algae.

4.4 Action

Surface mould on timber elements during normal construction does not require specific action by the builder unless it is to be used in a decorative application. Investigation may be required if building is interrupted and the engineered wood product is exposed for exceptional periods. Where concern is raised, specialist investigation for the presence of significant decay fungi may be required. It is not possible to generalise on how long an exceptional period is, as local conditions and actual exposure will affect this.

4.5 Remediation

Although moulds do not affect the performance of the timber, the application of proprietary products can restore the appearance of the product.

5

Moisture Protection – Rain During Construction

Timber products are manufactured under controlled environment to ensure they are dry. The moisture content depends upon humidity, exposure to wetting and drying conditions during construction and service life. Wetting during construction may lead to temporary elevated moisture content and dimensional changes in timber products.

Once covered, LVL and I-joist will ultimately dry and re-equilibrate to the ambient humidity conditions, even if there is a weather sealant applied. However, due to exposure to normal and excessive moisture exposures during distribution, storage and construction, some dimensional changes affecting serviceability may remain after redrying. These dimensional changes include cupping, bowing or expansion to dimensions beyond the specified tolerance of the product.

Where appearance is critical, timber elements and fastenings must be protected from moisture both during construction and in service. Where appropriate, building practices should be followed that minimise moisture exposure and facilitate products to re-equilibrate to dry conditions, and moisture increase taking place under normal construction situations should have no adverse effect on the performance of timber products. For more information on protection from weathering during service life, refer to WoodSolutions Design Guide #5: *Timber Service Life Design – Design Guide for Durability.*

If it is not possible to fully protect the timber element during construction, then some surface remedial work may be required to remove water stains and construction damage.

Where appearance is not critical, untreated timber can withstand some rain wetting during construction or occasional wetting in service without significant structural degradation; however, water staining and mould growth is possible. Timber must return to the dry condition (below 15% moisture content is considered dry for structural purposes) before installation with moisture sensitive materials, such as wall linings, floor coverings coatings or adhesives.

Methods of protection may be a sealant coating or impervious covering/ wrapping to minimise moisture uptake during construction. Protective timber sealant coating can retard moisture ingress during normal construction periods of around six weeks. Should this period be extended, or should there be signs that the sealant is no longer performing, renewal of the protective sealant may be necessary. Should the engineered wood product be cut, checked, bolted or otherwise worked on, renewal of the protective sealer to the exposed or unsealed timber will be required.

Where an impervious wrapper is used, it should be placed with the edge on the underside if possible and should be slit on the underside to allow moisture to escape, as shown in Figure 5.1. Where supports or intersecting members damage the protection, moisture ingress should be prevented.

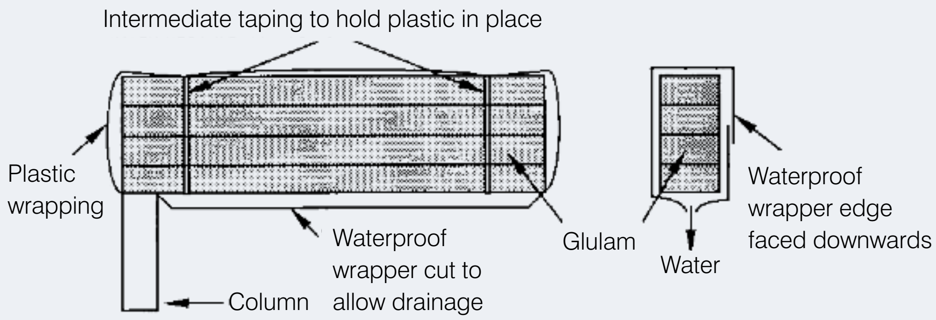


Figure 5.1: Protection of glulam in final position.

5.1 Iron Stain

Iron stain is an unsightly blue–black or grey discoloration that can occur on LVL in wet conditions on site, as shown in Figure 5.2.

Figure 5.2: Iron stain on LVL members in wet conditions on site. (Image courtesy of Nelson Pine Ltd)

The discoloration is caused by a chemical reaction between extractives in the wood and iron in steel products, such as nails, screws and other fasteners and appendages. This often occurs the first morning after rain or dew, when water enables the extractives and iron to meet and react.

If the timber products are kept dry (indoors), no discoloration will occur. Steel used in contact with timber products must not corrode. This can be accomplished by using stainless steel or by coating the steel. Coatings for fasteners, such as galvanising (zinc) or ceramic coatings, give a wide range of performance; whereas, stainless steel is the best choice for fasteners, particularly screws.

Where traces of iron are left on wood from cutting or slicing, cleaning the surface with steel wool, wire brushes, or iron tools; using finishes stored in rusty containers; and using iron-containing or iron-contaminated finishes may also cause discoloration. Iron dust from metalworking and even plant fertilisers can also be sources of iron.

Iron staining can be removed by diluted oxalic acid as the oxalic acid reacts with iron to form a colourless chemical complex. After treating wood with oxalic acid, thoroughly wash the surface with fresh, warm water to remove excess acid. If all sources of iron are not removed or protected from corrosion, staining will occur again.

References

Australian Standards

AS 1328 *Glued laminated structural timber*. 1998, Standards Australia, Australia.

AS 1720.1 *Timber structures, in Part 1: Design methods*. 2010, Standards Australia, Australia.

AS 4364 *Timber, in Bond performance of structural adhesives.* 2010, Standards Australia, Australia.

AS 2269.0 *Plywood—Structural, Part 0: Specifications.* 2012, Standards Australia, Australia.

WoodSolutions Technical Design Guides

WoodSolutions Technical Design Guide #20: *Fire Precautions During Construction of Large Buildings.* 2014, WoodSolutions, Melbourne, Australia.

WoodSolutions Technical Design Guide #5: *Timber Service Life Design – Design Guide for Durability*. 2012, WoodSolutions, Melbourne, Australia.

.

37c

Wood Solutions™
design and build
woodsolutions.com.au

This guide was revised in Dec 2019
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Mid-rise Timber Buildings
Commercial and Education
Class 5, 6, 7, 8 and 9b (including Class 4 parts)

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image by Boris Iskra, TPS Solutions (Aust) Pty Ltd

Wood Solutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-72-0

Researchers

Paul England
EFT Consulting
PO Box 1288, Upwey, VIC 3158
paul.england@eftconsult.net

Boris Iskra
TPC Solutions (Aust) Pty Ltd
Level 2, 394 Little Bourke Street
Melbourne VIC 3000

Published: December 2019

Table of Contents

Introduction

Mid-rise timber buildings are typically 4 to 8 storeys high

The National Construction Code Volume One, Building Code of Australia 2019 (NCC), allows the use of timber construction systems under the Deemed-to-Satisfy (DTS) Provisions for all buildings up to 25 metres in effective height ('mid-rise construction', see Figure 0.1).

The DTS provisions cover both traditional 'lightweight timber framing' and 'massive timber' products such as Cross-laminated Timber (CLT) and Laminated Veneer Lumber (LVL) in conjunction with the use of appropriate non-combustible fire-protective coverings – termed 'fire-protected timber' in the NCC – and appropriate compliant automatic sprinkler systems. With mid-rise timber construction design, fire and sound are two of the major considerations: appropriate fire-resisting construction is critical to providing acceptable levels of fire safety, while sound or acoustic performance is essential because of its daily impact on occupant amenity and quality of life.

This Guide applies to Class 5 to 8 and 9b Commercial Buildings or parts of buildings and Class 4 parts. It aims to assist in providing specific advice on fire safety and, to a limited degree for the majority of these building types, acoustic performance and is specifically written for use by designers, specifiers, builders, regulatory and certifying authorities. It is set-out according to a simple step-by-step process as presented in Figure 0.2. The steps are then used as the basis for headings throughout the main body of this Guide. Details on the scope and other important aspects of the Guide are set out below.

Scope

This Guide explains how to achieve the targeted fire and sound Performance Requirements in the National Construction Code (NCC) for the following mid-rise timber buildings using the Deemed-to-Satisfy pathway for fire-protected timber first introduced in the 2016 edition of the NCC, with further developments included in the 2019 edition:

- Class 5 – Office Buildings
- Class 6 – Shops, Restaurants, Bars etc.
- Class 7 – Carparks and Storage / Wholesale Facilities
- Class 8 – Laboratory, Manufacturing / Processing Goods or Produce
- Class 9b – Assembly Buildings including Education Facilities

The treatment of a Class 4 part of a building (a dwelling in a Class 5, 6, 7, 8 or 9 building) is also addressed in this Guide.

Low-rise timber buildings

are typically 1 to 3 storeys high depending on the class of building

Mid-rise timber buildings

have an effective height of not more than 25 metres

Typically, they are 4-8 storeys high (the maximum number of storeys depends on the floor-to-floor height)

High-rise timber buildings

have an effective height greater than 25 m.

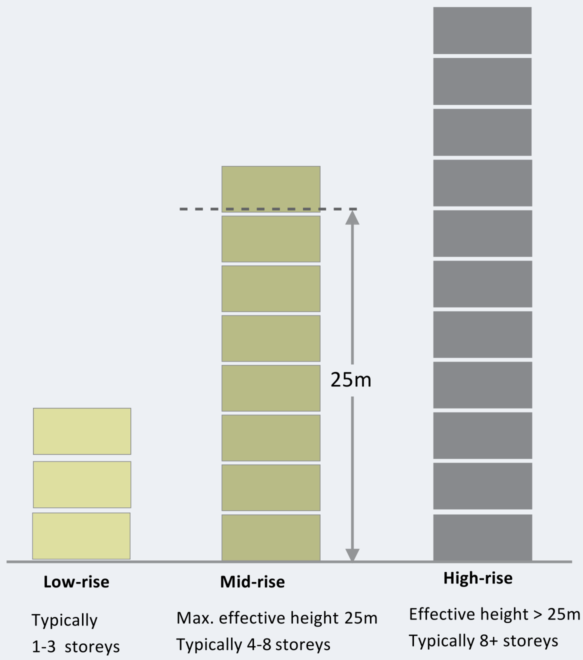


Figure 0.1: Comparison of low, mid and high-rise buildings

The guide provides some advice relating to good practice where the NCC leaves off in areas of increasing interest to users

This Guide specifically focuses on:

- fire-resisting construction (including fire-protected timber provisions) of wall, floor and ceiling elements for office buildings, retail premises, carparks and educational facilities
- additional fire safety measures required for mid-rise timber buildings.

In addition, this Guide provides advice on good practices to facilitate compliance, ease of maintenance and enhancements to the minimum NCC prescriptive provisions relating to fire and sound.

This Guide does not deal with all aspects of fire safety and sound insulation. Nor does it provide advice on which specific wall, floor and related systems should be used as there are many suppliers of proprietary systems and the intention is to encourage innovation. Generic details are provided for demonstration purposes. Before adopting these details, designers should check the availability of appropriate Evidence of Suitability with the material suppliers and, if necessary, modify the details accordingly.

Design Process for Sound- and Fire-Resisting Construction

Although national, some NCC provisions vary by State. It is vital to know the applicable provisions

Regulatory Differences between States and Territories

This Guide focuses on the NCC requirements of the 2019 edition. From time-to-time, State and Territory-based NCC amendments or other State legislation may vary requirements. Users of this Guide should make themselves aware of any differences and should develop a full understanding of the resulting implications. This Guide should be used on this basis.

Timber Construction Options for Mid-rise Timber Buildings

General Construction Options for Timber Buildings

A number of timber system options are available for the construction of mid-rise timber buildings with a range of possible options shown in Figure 0.2. Note: Under the NCC DTS provisions for mid-rise buildings only fire-protected timber building systems are permitted, where an element is required to be of non-combustible construction or of masonry or concrete construction.

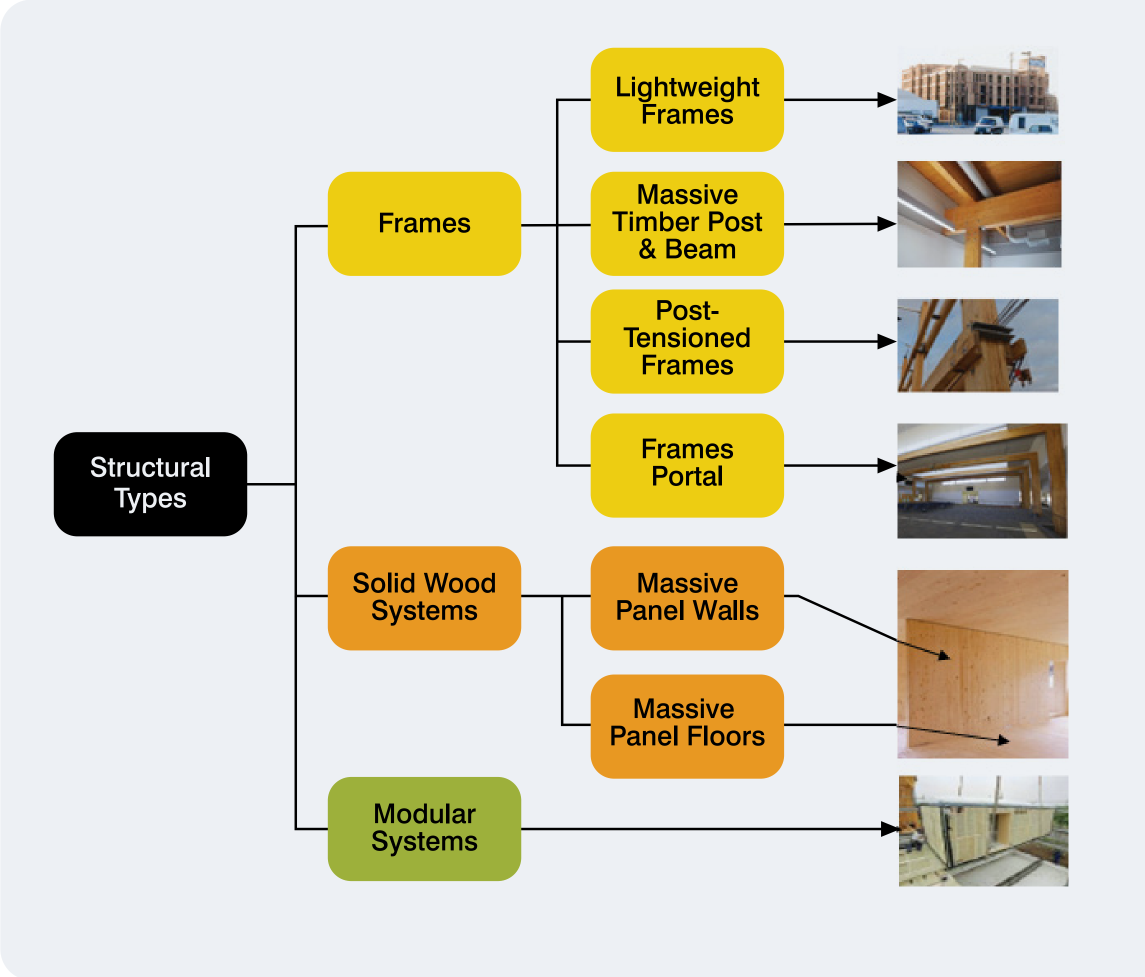
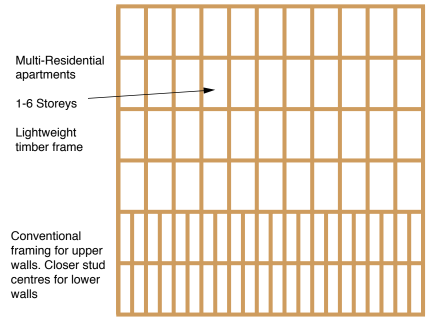


Figure 0.2: Summary of structural timber construction options.

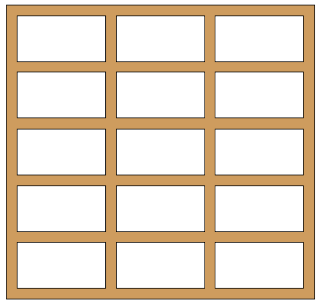
Note: No exposed structural timber is currently permitted under Deemed-to-Satisfy Provisions for mid-rise buildings where an element or component must be of non-combustible construction.

The most appropriate system is influenced by the function and floor plan of the building.

- In general, apartment (Class 2) and hotel/motel (Class 3) buildings tend to have sole-occupancy units (SOUs) or individual rooms with quite closely spaced walls. In effect they form a 'honeycombed' structure with many individual load paths and, as such, the use of lightweight timber-framed systems (up to around six storeys) combined with fire and sound-rated plasterboard is an efficient form of construction. Alternatively, a solid wood system, or a mixture of solid and lightweight timber construction might be considered.



- Commercial and educational buildings by contrast generally require larger open-plan spaces for either work amenity or flexibility in fit-out and as such these buildings are often constructed utilising a post and beam approach for the overall structure and non-loadbearing lightweight partition walls as needed. Typically, the main columns and beams might be constructed using glued-laminated timber (Glulam) with floors being either lightweight prefabricated cassettes or solid massive panel floor plates.



There are significant efficiency, speed and cost benefits in using timber structural systems compared to alternative material such as reinforced concrete. These include:

- Reduced on-site construction infrastructure (preliminary costs) such as fixed cranes, site accommodation, storage areas, scaffolding and edge protection, hoists, etc.
- Direct savings from faster methods of construction compared to traditional steel and concrete structures due to:

  – increased scope for off-site prefabrication and panelisation, and

  – lighter and more easily manoeuvred and installed materials.
- Reduced foundation requirements due to a lighter above-ground structure.
- Significantly reduced on-site costs and Work, Health and Safety (WHS) issues, particularly with a shift to more prefabricated solutions.
- Increased ability to commence follow-on trades earlier in the construction process, reducing the overall construction time.
- Increased accessibility of the construction site and significantly lower impacts of noise and site activities on local neighbourhoods (less truck movements); a major benefit for suburban multi-residential developments.

Detailed information on the specific construction cost benefits of timber systems in different Class buildings can be found in the following WoodSolutions Technical Guides:

Further information on the cost benefits of using timber can be found in the Rethinking Series of WoodSolutions Design Guides

#26 Rethinking Office Construction – Consider Timber – a material cost comparison of a typical office building

#27 Rethinking Apartment Building Construction – Consider Timber – a material cost comparison of a typical apartment building

#28 Rethinking Aged Care Construction – Consider Timber – a material cost comparison of typical aged care accommodation

#29 Rethinking Industrial Shed Construction – Consider Timber – a material cost comparison of a typical industrial shed.

Fire-protected Timber Options for Mid-rise Timber Buildings

The NCC prescriptive Deemed-to-Satisfy (DTS) solution for mid-rise buildings requires non-combustible fire-protective coverings to be applied to timber elements.

Refer Section 4.3

Whichever timber construction option is selected, the prescriptive Deemed-to-Satisfy (DTS) solutions for mid-rise timber buildings require timber members to be fire-protected, where an element is required to be of non-combustible construction or, concrete or masonry construction.

The 'general timber' requirements that apply for fire-protected timber are:

- the building element must be protected to achieve the required FRL, and
- a non-combustible fire-protective covering must be applied to the timber that achieves a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

The NCC permits a 'relaxation' to the general requirements in the case of fire-protected massive timber panels if the following additional criteria are satisfied:

- the timber panel is at least 75 mm thick, and
- any cavities between the surface of the timber and the fire-protective covering and between timber elements within the fire protective covering are filled with non-combustible materials.

If both these conditions are satisfied it is still necessary for the fire-protected timber member to achieve the required FRL and have a non-combustible fire-protective covering. However, the thickness of the fire-protective coverings, based on the covering's RISF performance and required FRL, can be modified depending on the application (e.g. internal SOU wall, external wall).

The basis for allowing specific provisions for massive timber panels is that timber with a large cross-section can achieve high fire-resistance due to the formation of a char layer that protects the timber core and allows it to continue to support an imposed load or maintain a fire separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for longer than lighter forms of construction; and by not permitting any concealed spaces between the massive timber members, or between the timber and fire-protective coverings, the risk of fire spread is addressed.

Further details relating to fire-protected timber are provided in Section 4.3.

The different timber construction systems generally use one or more of a range of different sawn or engineered timber products, including:

- Sawn timber – softwood (MGP) and hardwood (F- and A-grade).
- Engineered timber – particleboard, plywood, Oriented Strand Board (OSB), Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL), I-beams, fabricated floor and roof trusses, Glued Laminated Timber (Glulam) and Cross-laminated Timber (CLT).

Depending on the dimensions of the element and configuration, some of these construction systems may satisfy the requirements that allow the massive timber provisions to apply.

Timber-framed Construction

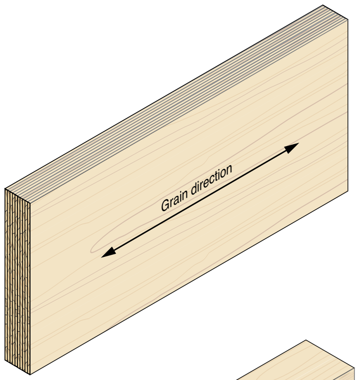
Lightweight timber-framed construction systems use commonly available structural timber framing products assembled into lightweight systems such as wall frames, floor and roof trusses, and prefabricated cassette floor modules.

Sawn Timber Products

Sawn timber products include seasoned structural softwood (MGP10, 12 & 15) or seasoned structural hardwood (typically F17 or F27). Typical thicknesses are 35 and 45 mm and depths include: 70, 90, 120, 140, 190, 240* and 290* mm.

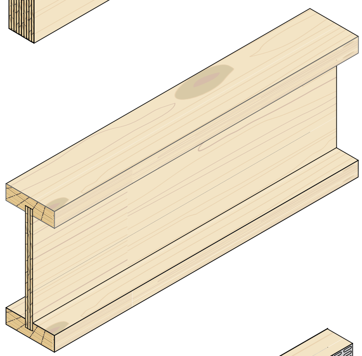
* available on order

Laminated Veneer Lumber (LVL)



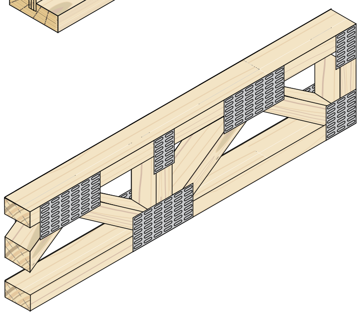
Lightweight framing elements are available in Laminated Veneer Lumber (LVL), a widely used softwood engineered wood product, available in all the standard framing sizes. LVL is manufactured by bonding together rotary peeled or thin sliced wood veneers under heat and pressure. As LVL is typically used in a beam or stud application, the grains of the veneers are all oriented in the same direction. LVL is typically manufactured in slabs 1200 mm wide, known as billets, which are then cut into the commonly available framing member depths required. LVL is typically manufactured in lengths up to 12 metres in 0.3 metres increments.

I-Beams



I -Beams are lightweight, high-strength, long-span structural timber beams. They typically comprise top and bottom flanges of LVL or solid timber – which make the distinct shape. The flanges are separated by a vertical web, usually manufactured from structural plywood, Oriented Strand Board (OSB) or light gauge steel. Typical depths are: 200, 240, 300, 360 and 400 mm and lengths are available up to 15 metres.

Parallel Chord Trusses

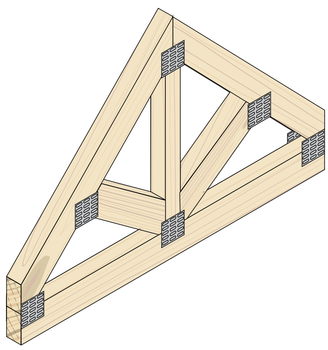


Parallel Chord Trusses are similar to I-beams in that they have top and bottom chords (flanges) of LVL or solid timber but instead of solid webs, web struts are used.

The struts may be either timber or light gauge steel and are secured to the chords typically with nailplates. The struts may be diagonal (more common for steel struts) or a mix of vertical and diagonal (more common with timber struts).

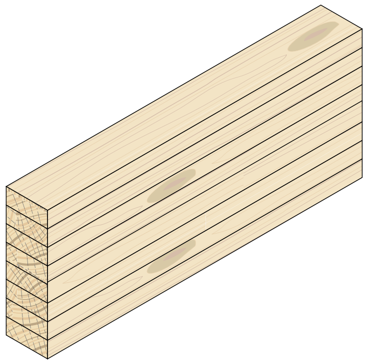
Figure 0.3: Hybrid floor system with parallel-chord trusses supported from a steel I-beam.

Roof Trusses



Timber roof trusses provide an engineered roof frame system designed to carry the roof or roof and ceiling, usually without the support of internal walls. The characteristics of a roof depend on the purpose of the building it covers, the available roofing materials and the wider concepts of architectural design and practice. Light truss roofs are formed from sawn or LVL timber elements connected with nailplates or other mechanical fixings designed and supplied by frame and truss manufacturers.

Glued-laminated Timber (Glulam)



Glued-laminated Timber (Glulam) consists of a number of strength graded, kiln-dried laminations face bonded and finger-jointed together with adhesives. Elements can be manufactured to practically any length, size or shape: beams are often manufactured with a built-in camber to accommodate dead load deflection or curved for aesthetic appeal.

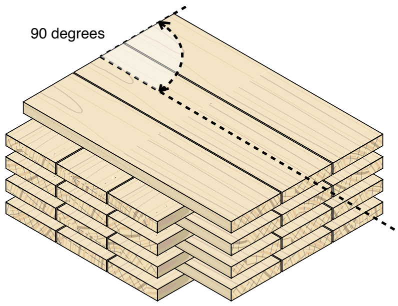
A range of GL Grades are produced in Australia or imported depending on the timber species used in manufacture: GL10 (Cypress), GL13 (Radiata Pine, Oregon), GL17 (Slash Pine, Merbau), GL18 (Tas Oak, Vic Ash), GL21 (Spotted Gum) – the GL descriptor refers to the element's Modulus of Elasticity (E), i.e. GL10 describes a Glulam member that has an E-value of 10GPa.

A wide range of depths are available in increments from 90 mm to over 1,000 mm; and thicknesses from 40 mm to 135 mm; with 65 mm and 85 mm being two commonly used. Lengths up to 18 metres are available in 0.3 metre increments from traditional suppliers and up to 27 metres in length from specialist manufacturers.

Solid Massive Wood Panel Construction Systems

Solid wood panel construction systems utilise massive timber engineered wood panels such as Laminated Veneer Lumber, Cross-laminated Timber (CLT), or Glue/nail-laminated Mass Timber panels, in minimum panel thicknesses of 75 mm when used in accordance with the NCC DTS provisions. Solid wood panels can be used to form complete floors, walls and roofs and construction methods have more in common with precast concrete panels than timber framing; except that timber panels are much lighter, more easily worked and easier to erect.

Cross-laminated Timber (CLT)



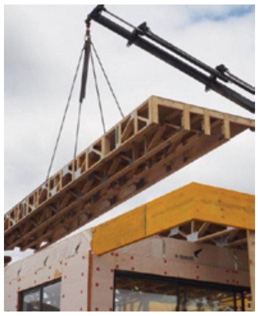
Cross-laminated Timber (CLT) utilises individual planks of timber 12-45 mm thick and 40-300 mm wide face-glued together (and edge-glued in some instances), each layer at 90° to its neighbouring lamella; effectively 'jumbo plywood'. CLT panels are typically 57 mm – 320 mm thick and made up of 3, 5, 7 or 8 layers depending on application. Panels are available in 2.2 to 2.95 metres wide and up to 11.9 metres in length (dictated mainly by transport restrictions).

More detailed information on CLT can be found at the WoodSolutions website or in *Technical Guide #16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)*, which introduces the use of CLT in construction and provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

Prefabrication and Modular Wood Construction Systems

A major benefit in utilising timber structural systems in mid-rise construction is the ability to prefabricate off-site and manufacture frames or cassettes, panelised elements or full volumetric modules to minimise the on-site construction requirements and costs.

Prefabricated Cassette Floor Systems



Prefabricated Cassette floor systems utilise a range of timber structural products, typically for flooring (particleboard, plywood or OSB panels) and for floor joists and bearers (sawn timber, LVL, OSB beams, floor trusses or I-beams). Cassettes tend to be around 3 metres wide and up to around 12 metres long (due to travel restrictions). Cassette floor systems are highly effective in mid-rise construction as they are extremely fast to install and far safer for on-site workers, dramatically reducing 'fall-from-height' risks for workers.

With mid-rise construction, where effective acoustic separation is required, a Timber/Concrete Composite Cassette Floor might be considered; the concrete screed adding mass for acoustic and vibration control as well as acting compositely with the timber components for improved structural performance. For more detailed information on this refer to *WoodSolutions Technical Guide #30, Timber Concrete Composite Floor Design Guid*e.

Panelised Elements

Fully panelised elements involve the total off-site manufacture of all components including; structural members, insulation, sarking, plumbing and electrical fittings, window and/or door installation and internal lining installation (and external cladding if appropriate).

Modular Wood Construction Systems

Modular wood construction systems (volumetric modules) utilise either light-frame systems (mainly) or solid wood panel systems. The main principle is that the entire volumetric box consisting of walls, floor and ceiling, as well as inner lining and all services are assembled in a factory and transported to the construction site for erection. To assemble the modules on top of each other a male-female connector arrangement is often used. The size of the modules is generally limited by transportation restrictions with maximum dimensions of around 4.2 metres wide, up to 13 metres long and 3.1 metres high. The overall building height is generally the same as for light-frame systems, around 6–7 storeys. Modular systems are used in a range of applications including apartments, student housing, hotels and aged care facilities.

Hybrid Construction

Mid-rise timber buildings may also use a 'mix' of materials – hybrid construction – to achieve cost-effective, practical and robust solutions.

For example, a common configuration is to use concrete construction for below ground structures, such as car parks and basements to reduce the risks to timber elements associated with groundwater and for ground floor construction to provide a physical separation from the ground as part of a termite management strategy (where required).

Other forms of hybrid construction may use timber and other structural materials within the same element of construction, for example:

- concrete toppings to timber floors with shear connections to use composite action
- floors constructed with a mix of steel beams and timber beams to manage a mix of spans as shown in Figure 0.3 (LVL timber beams are increasingly used to replace steel beams for this application).

Robust Structural Design

The structural design of mid-rise timber buildings must comply with the relevant NCC requirements, including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1(a)(iii).

Further guidance is provided in *WoodSolutions Technical Design Guide #39 Robustness in Structures.*

Options for fire precautions during construction should be considered as part of the overall design process

Fire Precautions during Construction

Mid-rise timber buildings, when complete, provide a high level of safety because of the combination of automatic fire sprinklers and fire-protected timber, among other things.

While the use of timber significantly reduces a number of risks during construction, the fire risk can be increased as a result of the increased volumes of unprotected timber. *WoodSolutions Technical Design Guide #20 Fire Precautions during Construction of Large Buildings* provides advice relating to fire precautions during construction to help building professionals and organisations with responsibilities for fire safety on a construction site reduce the risk of fire.

Other Design Considerations

Designers need to take account of a broad range of design considerations to ensure that a building is fit-for-purpose and complies with all requirements of the NCC and other legislation. These include:

- structural design (for safety and serviceability)
- weatherproofing
- safe access and egress
- light and ventilation (including condensation control)
- energy efficiency
- durability (including termite management)
- design in bushfire-prone and flood-prone areas.

Some sources of information on these matters are referenced in the Appendices of this Guide.

1

Step 1 – High-Level NCC Design Issues (Schematic Design)

The National Construction Code (NCC) is the regulatory framework for determining the minimum design and construction requirements for buildings in Australia. This Step covers a selection of high-level design issues relating to fire-resisting and sound-insulating construction.

1.1 Determine the Type of Construction Required

Refer NCC Volume One A6 for details of all classes of building

The NCC contains mandatory performance requirements that apply to 10 primary classes of building that are determined by the building's purpose. The classes directly relevant to this Guide are:

- **Class 5 buildings** – are office buildings used for professional or commercial purposes.
- **Class 6 buildings** – are shops or other buildings used for the sale of goods by retail or the supply of services direct to the public, including
  - an eating room, café, restaurant, milk or soft-drink bar; or
  - a dining room, bar area that is not an assembly building, shop or kiosk part of a hotel or motel; or
  - a hairdresser's or barber's shop, public laundry, or undertaker's establishment; or
  - a market or sale room, showroom, or service station.
- **Class 7 buildings** – are storage-type building that includes one or more of the following sub-classifications
  - Class 7a – carpark.
  - Class 7b – buildings that are used for storage, or display of goods or produce for sale by wholesale.
- **Class 8 buildings** – are process-type buildings that include the following:
  - A laboratory.
  - A building in which the production, assembling, altering, repairing, packing, finishing, or cleaning of goods or produce for sale takes place.
- **Class 9b buildings** – are assembly buildings including a trade workshop or laboratory in a primary or secondary school.
- **Class 4 building (part)** – is a dwelling in a Class 5, 6, 7, 8 or 9 building.

The Class of Building in conjunction with the building height, expressed in terms of the rise in storeys, and the maximum size of fire components are used to determine the type of construction required.

Refer NCC Volume One C1.2 for calculation of rise in storeys

The rise in storeys is the sum of the greatest number of storeys at any part of the external walls of the building and any storeys within the roof space:

- above the finished ground next to that part; or
- if part of the external wall is on the boundary of the allotment, above the natural ground level at the relevant part of the boundary.

The maximum size of fire compartments (floor area, volume) is defined in the NCC Clause C2.2

Type C construction is applicable to most low-rise buildings. It is the least fire-resisting form of construction and places few fire-related restrictions on the use of structural timber members.

Type B construction, while not requiring as high FRLs as Type A construction, applies similar constraints to the use of timber.

Type A construction is the most fire-resisting and the prescriptive solutions within the NCC have, in the past, imposed severe limitations on the use of timber through the prescription of masonry and concrete construction and non-combustibility for elements required to achieve a prescribed Fire Resistance Level (FRL).

Table 1.1 shows the required types of construction specified by the NCC.

Table 1.1: Types of construction required by NCC Volume One.

Rise in storeys	Multi-residential		Office	Retail	Car Park/ Storage	Factory/ Laboratory	Hospitals/ Public assembly
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
4 or more	**A**	**A**	**A**	**A**	**A**	**A**	**A**
3	**A***	**A***	**B**	**B**	**B**	**B**	**A**
2	**B***	**B***	**C**	**C**	**C**	**C**	**B**
1	**C**	**C**	**C**	**C**	**C**	**C**	**C**

** Refer low-rise concessions (e.g. Specification in C1.1 Clause 3.10 and Clause 4.3).*

Design parameters such as the class of building, rise in storeys, effective height and type of construction should be confirmed with the building surveyor/certifier.

1.2 Determine NCC Compliance Pathway

1.2.1 NCC Compliance Pathway

To comply with the NCC the relevant Performance Requirements must be satisfied, as demonstrated by means of the Assessment Methods specified in the NCC. There are two pathways that can be followed (or a combination of the two).

- For a Deemed-to-Satisfy Solution, it is necessary to provide Evidence of Suitability to show that the Deemed-to-Satisfy Provisions have been met.
- For a Performance Solution, specific building solutions are developed for a building that may vary from the Deemed-to-Satisfy Provisions.

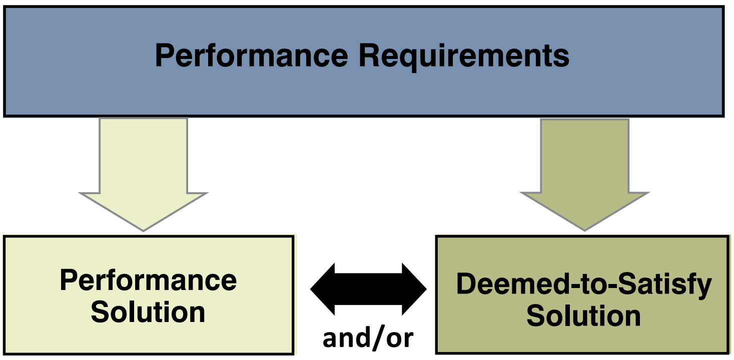


Figure 4: Pathways for demonstrating compliance with NCC performance requirements.

The construction systems and details in this Guide are based on the Deemed-to-Satisfy (DTS) Solution pathway for mid-rise timber buildings which was first introduced into the 2016 edition of the NCC for Class 2, 3 and 5 buildings and extended to all classes of buildings in the 2019 edition. This does not prevent designers using Performance Solutions, but it should be ensured that any variations from the DTS Solution do not adversely affect the fire safety strategy for the building. Further guidance is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-Rise Buildings*.

1.2.2 NCC Compliance Options for Timber Buildings

In the context of this Guide, timber buildings are defined as buildings where timber is the predominant material in the structure. There are still opportunities to use timber for some structural and non-structural applications in buildings using other materials for the primary structure.

Table 1.2 summarises options for complying with the NCC Performance Requirements for Class 2 to 9 buildings with further details provided below. DTS Solutions are available for the building configurations shaded in green (light, mid and dark green) with this Guide being directly applicable to the applications shaded in mid green. At the time of writing, Performance Solutions are required to be developed for the areas shaded in blue.

Mid-rise Class 4,5,6,7,8 and 9b buildings are addressed in this Guide.

Table 1.2: Design options for timber buildings.

Rise in storeys or effective height	**Multi-residential**		**Office**	**Retail**	**Car Park/ Storage**	**Factory/ Laboratory**	**Hospitals/ Public assembly/ Schools**
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
Effective height greater than 25m	High	High	High	High	High	High	High
8[EH]	Mid	Mid	Mid	Mid	Mid	Mid	Mid
7	Mid	Mid	Mid	Mid	Mid	Mid	Mid
6	Mid	Mid	Mid	Mid	Mid	Mid	Mid
5	Mid	Mid	Mid	Mid	Mid	Mid	Mid
4	Mid[1]	Mid[1]	Mid	Mid	Mid	Mid	Mid
3	Low[1]	Low[1]	Mid	Mid	Mid	Mid	Mid
2	Low[1]	Low[1]	Low	Low	Low	Low	Mid
1	Low	Low	Low	Low	Low	Low	Low

EH: Effective height of not more than 25 metres

Note 1: Refer to Technical Design Guide #02 to check if low-rise timber concessions apply.

Low	Deemed-to-Satisfy Solution – Guide #02 or 03
Mid	Deemed-to-Satisfy – Guide #37R for Class 2 and 3 buildings
Mid	Deemed-to-Satisfy – Guide #37C for Class 5, 6, 7, 8 and 9b commercial and educational buildings (this Guide) and Guide #37H for Class 9 healthcare buildings.
High	Performance Solution

Low-rise timber buildings

There are relatively few fire-related restrictions on the use of structural timber members in Buildings of Type C construction irrespective of the Class of Building under the DTS Solution pathway and for domestic housing.

The NCC Volume One Deemed-to-Satisfy Solution pathway includes concessions that facilitate the use of timber-framed construction for Class 2 and 3 buildings up to a rise in storeys of 3 and, in limited cases, up to 4 storeys. Guidance in relation to construction of these low-rise options is provided in the following WoodSolutions Technical Design Guides:

Refer WoodSolutions Design Guides #01, #02 and #03 for low-rise options.

#01 Timber-framed Construction for Townhouse Buildings Class 1a – information about the fire safety and sound insulation performance requirements in the NCC for Class 1a attached buildings.

#02 Timber-framed Construction for Multi-residential Low-rise Buildings Class 2 and 3 – information about the fire and sound performance requirements in the NCC for Class 2 and 3 low-rise buildings.

#03 Timber-framed Construction for Commercial Low-rise Buildings Class 5, 6, 9a & 9b – information about the fire performance requirements in the NCC for Class 5, 6, 9a and 9b buildings

These buildings would normally be designed following the Deemed-to-Satisfy Solution pathway with Performance Solutions being used to address minor variations and/or unusual design circumstances.

The construction systems and details in this Guide are based on the Deemed-to-Satisfy (DTS) Solution pathway for mid-rise timber buildings as detailed in the 2019 edition of the NCC.

This does not prevent designers using Performance Solutions but variations from the DTS Solution should ensure the fire safety strategy for the building is not adversely affected. Further guidance in relation to this is provided in *WoodSolutions Design Guide #38 Fire Safety Engineering Design of Mid-Rise Buildings.*

Check with the regulatory authority that the building effective height does not exceed 25 metres if applying the mid-rise fire-protected timber solution

Mid-rise timber buildings

Mid-rise buildings are of Type A or B construction up to an effective height of not more than 25 metres. The use of timber structural members under the NCC prescriptive pathway is restricted for mid-rise buildings unless the option to use fire-protected timber in conjunction with automatic fire sprinklers is adopted. This Guide addresses Class 5, 6, 7, 8 and 9b buildings applying these design principles and provides guidance on the application of the NCC provisions for a Class 4 part of a building.

Guidance on the technical derivation of the mid-rise fire-protected timber solution for Class 2 and 3 buildings is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-rise Timber Buildings* which may assist with the development of a Performance Solution.

The NCC defines effective height as *"the vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units)"*. If there is any doubt as to whether a building's effective height does not exceed 25 metres, it is recommended that the effective height is checked with the relevant authorities.

High-rise buildings

All high-rise timber buildings (effective height greater than 25m) need to follow the Performance Solution pathway.

1.2.3 Overview of the Deemed-to-Satisfy Solution for Mid-rise Timber Buildings

The NCC 2019 includes Deemed-to-Satisfy Provisions that allow the construction of mid-rise timber buildings. The main features of the mid-rise timber building DTS Solutions are:

- the building has an effective height of not more than 25 metres
- the building has a sprinkler system, other than a FPAA101D or FPAA101H system, complying with Specification E1.5 of the NCC throughout
- fire-protected timber complying with Specification C1.13a of the NCC can be used for loadbearing timber elements, non-loadbearing timber walls required to achieve an FRL and for elements of construction required to be non-combustible
- cavity barriers are provided in accordance with Specification C1.13 of the NCC, and
- any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible.

These fire safety precautions aim to provide a robust building solution on the following basis:

Automatic sprinkler suppression system: Objective is to suppress a fire before the structure is threatened and greatly reduce the risk to people and property.

Fire-protected timber (NCC prescribes FRLs AND non-combustible fire protective coverings): Objective is to prevent or delay ignition of the timber structural members so that the response to an enclosure fire will be similar to non-combustible elements, masonry or concrete during the growth period and prior to fire brigade intervention.

Cavity barriers: Objective is to prevent uncontrolled spread of fire through cavities in the low probability events of either failure of the protective covering or fire starting within the cavity.

Non-combustible insulation: Objective is to minimise the risk of fire spread through cavities by removing a potential source of fuel, i.e. combustible insulating materials.

1.2.4 Performance Solution Options

For high-rise timber buildings, the Performance Solution pathway must be adopted. Refer to the following WoodSolutions Technical Guides for further information.

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT) – introduces the use of CLT in construction, outlining the history, environmental performance and mechanical properties. Also provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

#38 Fire Safety Engineering Design of Mid-rise Timber Buildings – reference source describing methods and supporting data that can be used for the fire safety engineering design of mid-rise timber buildings based on the research undertaken to develop and justify the changes to the NCC 2016.

1.2.5 Evidence of Suitability

The NCC requires every part of a building to be constructed in an appropriate manner to achieve the performance requirements using materials and construction that are fit for the purpose for which they are intended, including safe access for maintenance.

The NCC Volume One specifies requirements for Evidence of Suitability in Clause A5.2 but there are the following additional specific requirements that apply to certain aspects of fire safety under NCC prescriptive requirements:

- NCC Clause A5.4 Fire-Resistance of Building Elements
- NCC Clause A5.5 Fire Hazard Properties
- NCC Clause A5.6 Resistance to the Incipient Spread of Fire.

In most instances, for the materials and systems considered in this Guide, the Evidence of Suitability for the fire resistance or Resistance to the Incipient Spread of Fire (RISF) of an element of construction will be in a report from an Accredited Testing Laboratory.

If a Performance Solution is proposed, compliance should be demonstrated using the procedures prescribed in Clauses A5.4 and A5.6 of the NCC as appropriate for fire-protected timber elements.

1.3 Determine Schematic Building Layout

Evidence of Suitability for fire resistance and Resistance to the Incipient Spread of Fire should be a report from an Accredited Testing Laboratory as prescribed in the NCC

1.3.1 Mixed Class Buildings

The NCC DTS Solution, using fire-protected timber in conjunction with automatic fire sprinklers, can also be applied to mixed class buildings, provided the different classes are adequately fire separated (refer Step 4) and the entire building is protected by an automatic fire sprinkler system complying with NCC Volume One Specification E1.5, other than a FPAA101D or FPAA101H system.

This provides added flexibility for the design of new buildings and facilitates the recycling of existing buildings. For example, a fire-protected timber retail level (Class 6) could be constructed above existing concrete-framed carpark levels minimising the increase in foundation loads as shown in Figure 1.2.

Fire-protected timber can be used in conjunction with other forms of construction in mixed class buildings

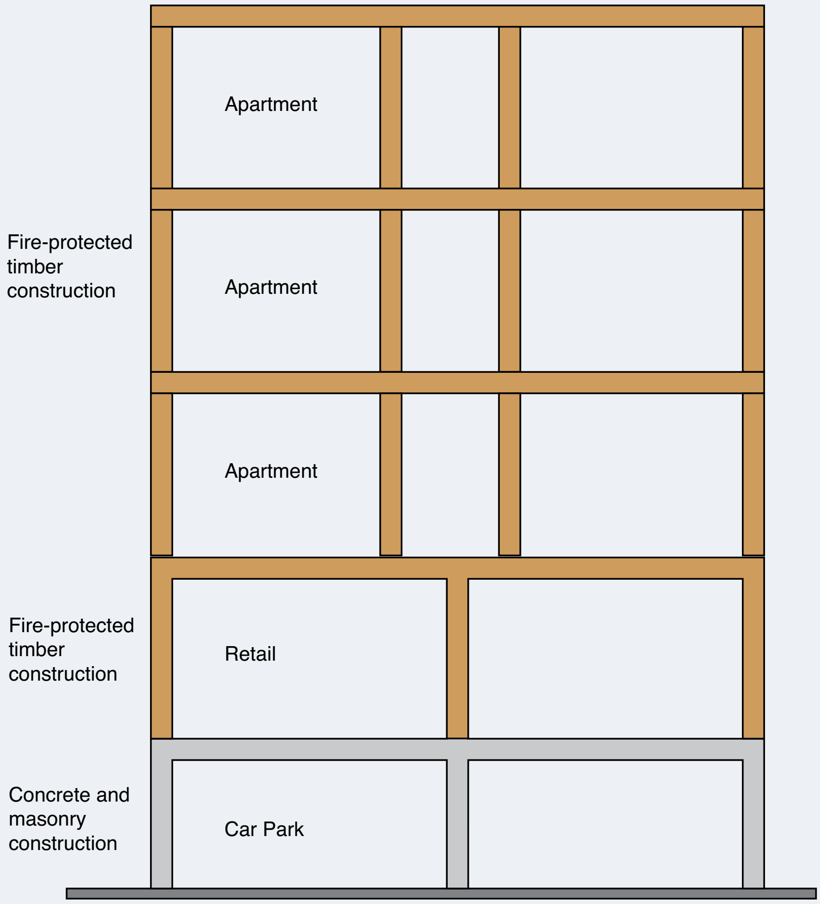


Figure 1.2: Multi-class and mixed forms of construction.

Note: Sprinkler protection required throughout entire building development.

1.3.2 Check Compliance with Fire Compartment Size Limits

The NCC does not nominate a maximum fire compartment size for Class 2, 3 and 4 buildings or parts of buildings but it does require the bounding construction of Sole Occupancy Units (SOUs) to be of fire-resistant construction.

For Class 5 to 8 and 9b buildings maximum compartment sizes are prescribed in Clause C2.2 of the NCC and are summarised in Table 1.3 below for the mid-rise buildings considered in this Guide.

Table 1.3: NCC Fire Compartment Sizes for mid-rise Commercial and Education Buildings.

Building Class	Type A Construction		Type B Construction	
	Floor Area m²	Volume m³	Floor Area m²	Volume m³
5	8,000	48,000	5,500	33,000
9b				
6	5,000	30,000	3,500	21,000
7				
8				

Note: There are concessions permitted by the NCC for large isolated buildings that allow relaxations to the above limits under certain circumstances (refer NCC Clause C2.3)

Individual floors are normally substantially less than the maximum floor areas specified in Table 1.3 and under these circumstances each floor will generally comprise a separate fire compartment unless relaxations are permitted (e.g. non-required, non-fire isolated connections as permitted in Clause D1.2 of the NCC).

The fire safety strategy should be specified at the start of the project and refined as the design progresses

1.3.3 Determine Schematic Fire Safety Design Strategy

The preliminary specification of a fire safety strategy for a building is important since it may impact significantly on the building layout. This is applicable irrespective of the compliance pathway chosen (Performance Solution or DTS Solution) since, even within the DTS pathway, options have to be selected that affect the building layout, detailed design and on the use of the building through it's life cycle.

The schematic fire safety design strategy should at the preliminary stage provide as a minimum:

- A summary of the fire safety objectives.
- Building uses that the design needs to address.
- Occupant characteristics that the design addresses.
- Approach to demonstrating compliance with the NCC (Performance Solution, DTS Solution or a combination).
- Where a DTS Solution is specified it is still necessary to provide details of the DTS options selected as detailed below.

With a DTS solution critical decisions still need to be made such as the application of concessions

> Schematic drawings and brief descriptions as appropriate indicating:
  - design requirements for automatic fire sprinkler systems
  - design requirements for detection and alarm systems
  - general layout showing fire / smoke resistant compartmentation and structural elements
  - active smoke control measures if provided
  - means of egress during a fire emergency including travel distances to exits, discharge of exits, door operations, etc
  - evacuation strategy and associated emergency warning and intercom system (EWIS)
  - means to alert the fire brigade and equipment to facilitate fire brigade intervention
  - any other fire protection measures.

> An implementation plan stating who is responsible for ensuring compliance and measures that will be in place to facilitate compliance (e.g. inspection schedule).

> Protection measures required during construction.

> Fire safety management measures after completion to ensure ongoing effectiveness of the fire safety strategy through the life of the building.

This preliminary strategy should be regularly reviewed and updated with further details added as the design develops.

Consider building services throughout the design process

1.3.4 Determine Building Services and Preliminary Layout

The preliminary selection of building services and service locations should be considered when determining the general building layout and provision allowed for safe maintenance, modification and addition of services without compromising fire safety and sound separation.

Typical matters for consideration include:

- selection of HVAC systems (centralised v localised systems)
- locate service shafts to minimise nuisance noise and facilitate fire compartmentation
- minimise service penetrations through fire barriers
- group service penetrations together and select treatments that minimise the risk of fire spread to cavities.
- avoid the need for hot works (e.g. welding, grinding) on services at the position of penetrations through fire-resistant elements
- provide for maintenance and additional services
- select services and materials that minimise the need for hot works (e.g. welding, grinding, soldering).

Consider fire safety during construction throughout the design process

1.4 Safe Design

Safe design principles should be applied throughout the design process and consider the entire building life cycle. Some general guidance in relation to mid-rise timber buildings is given in this section.

1.4.1 Building Life Cycle

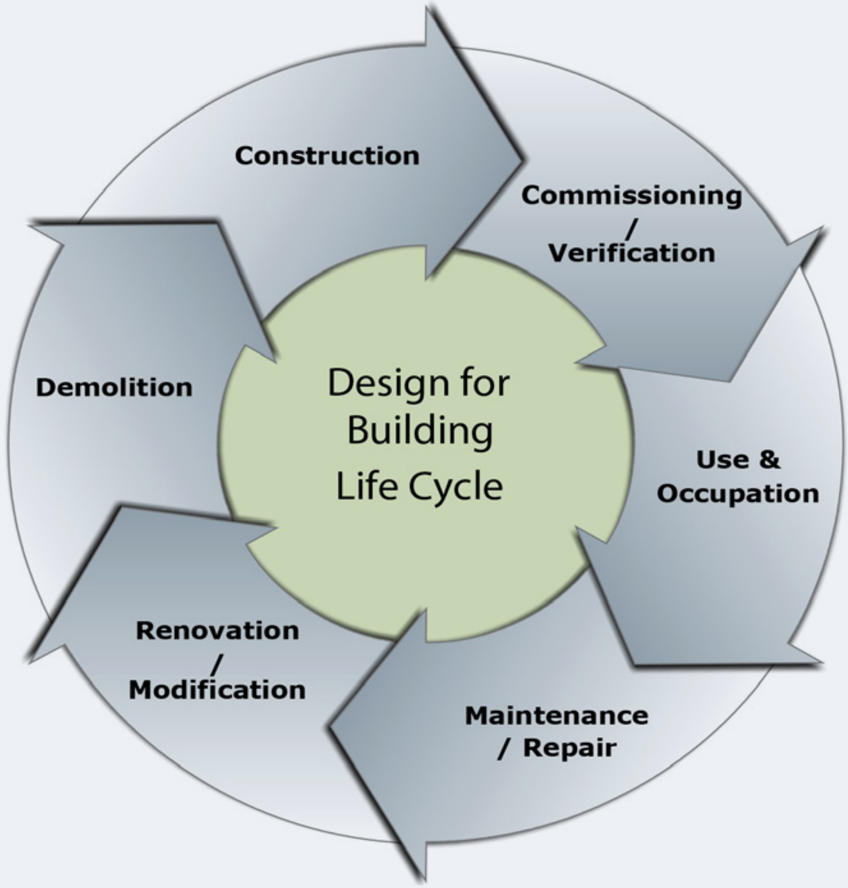


Figure 1.3: Typical building life cycle.

It is important to consider the impacts of design decisions on all phases of the of the building's life cycle.

For example, the NCC Deemed-to-Satisfy Provisions may require a fire safety feature to be incorporated into a building. During the design process it is necessary to determine:

- how the provision can be installed/constructed safely to achieve its required performance
- how the feature will be commissioned, and its performance verified
- that the feature will not present a hazard during occupation of a building
- how the feature can be maintained and repaired safely
- measures to be taken to ensure the feature does not present a hazard during renovation/ modification or demolition and to ensure that the performance of the feature is not compromised during the renovation/modification process.

Many of these matters lie outside the scope of the NCC but they are addressed through State and Territory Building Acts and regulations and Workplace Health and Safety (WHS) legislation.

For further details on how to address WHS requirements refer to the Safe Work Australia Code of Practice on Safe Design of Structures

1.4.2 Responsibilities for Safe Design

While this Guide focuses on NCC 2019 requirements relating to Deemed-to-Satisfy solutions for mid-rise timber buildings, the NCC provides a uniform set of technical provisions for the design and construction of buildings and other structures throughout Australia. The NCC does not regulate matters such as the roles and responsibilities of building practitioners and maintenance of fire safety measures that fall under the jurisdiction of the States and Territories.

State and Territory building legislation is not consistent in relation to these matters, with significant variations with respect to:

- registration of practitioners
- mandatory requirements for inspections during construction
- requirements for maintenance of fire safety measures.

Workplace Health and Safety (WHS) legislation requires safe design principles to be applied. A *Code of Practice – Safe Design of Structures* published by Safe Work Australia provides guidance to persons who design structures that will be used, or could reasonably be expected to be used, as a workplace. It is prudent to apply these requirements to all buildings as they are generally a workplace for people doing building work, maintenance, inspections and the like.

The Code defines safe design as: *"The integration of control measures early in the design process to eliminate or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed."*

It indicates that safe design begins at the start of the design process when making decisions about:

- the design and its intended purpose
- materials to be used
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal
- what legislation, codes of practice and standards need to be considered and complied with.

The Code also provides clear guidance on who has health and safety duties in relation to the design of structures and lists the following practitioners:

- architects, building designers, engineers, building surveyors, interior designers, landscape architects, town planners and all other design practitioners contributing to, or having overall responsibility for, any part of the design
- building service designers, engineering firms or others designing services that are part of the structure such as ventilation, electrical systems and permanent fire extinguisher installations
- contractors carrying out design work as part of their contribution to a project (for example, an engineering contractor providing design, procurement and construction management services)
- temporary works engineers, including those designing formwork, falsework, scaffolding and sheet piling
- persons who specify how structural alteration, demolition or dismantling work is to be carried out.

In addition, WHS legislation places the primary responsibility for safety during the construction phase on the builder.

The design team in conjunction with owners/operators and the builder have a responsibility to document designs, specify and implement procedures that will minimise risks to health and safety throughout the life of the structure being designed.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides further guidance

1.4.3 Applying Safe Design Principles

A key element of safe design is consultation to identify risks, practical mitigation measures and to assign responsibilities to individuals/organisations for ensuring the mitigation measures are satisfactorily implemented.

This approach should be undertaken whichever NCC compliance pathway is adopted and applies to all forms of construction.

Some matters specific to fire safety are summarised below:

- The NCC and associated referenced documents represent nationally recognised standards for fire safety for new building works.
- The NCC's limited treatment of fire precautions during construction focuses on manual fire-fighting, egress provisions and fire brigade facilities. Additional precautions are required to address WHS requirements such as fire prevention and security. Refer to Section 1.4.4 and *WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings,* for further information.
- Minimising service penetrations through fire-resisting construction.
- Grouping of service penetrations through fire-resisting walls with safe access for installation, inspection and maintenance.
- Detailed design of fire safety measures to optimise reliability and facilitate safe installation, maintenance and inspection where practicable. Special attention should be given to protection of service penetrations and cavity barriers.
- Documentation of procedures and allocation of responsibilities for determining Evidence of Suitability for fire safety measures.
- Documentation of procedures and allocation of responsibilities for the verification and commissioning of all fire safety installations.
- Provision of specifications and drawings of all fire safety measures within the building, Evidence of Suitability, commissioning results and requirements for maintenance and inspection to the owner as part of the fire safety manual. (Note: Some State and Territory legislation contains minimum requirements for inspection of fire safety measures).
- The fire safety manual should also provide information on how to avoid compromising fire safety through the life of a building (e.g. preventing disconnection of smoke detectors or damage to fire resisting construction).

1.4.4 Fire Precautions during Construction

Refer NCC Volume One Cl E1.9 for NCC precautions during construction

Fires may occur on building construction sites due to the nature of the works.

Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Mid-rise timber buildings complying with the NCC 2019 Deemed-to-Satisfy Provisions offer a safe and economical building option. The addition of the fire-protective coverings plays an important role in providing this fire safety and, due to the construction sequencing, there may be a period where the timber is not fully protected and/or automatic fire sprinkler protection is not fully operational. During this period timber buildings are at their highest risk from construction fires.

The builder and design team need to consider fire precautions during construction. The scope of the NCC is limited to specifying minimum requirements for fire hydrants, hose reels and extinguishers and egress provisions (NCC Clause E1.9).

Consider fire safety during construction throughout the design process

Addressing WHS requires a broad holistic approach that considers the building layout and site layout throughout the construction process to minimise the fire risk at a time when the building could be at its most vulnerable. Typical matters that should be considered include:

- progressive installation of services
- progressive installation of fire-protective grade covering of timber members and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works
- security provisions (to address arson)
- safe access for maintenance of equipment and minimising the down time of fire safety equipment during maintenance
- detailing service penetration and construction interfaces to minimise the risk of cavity fires during installation.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

2

Step 2 – Define NCC Design Requirements for Sound, Thermal Resistance, Damp & Weatherproofing and Structural Tests

Timber building systems can be designed to meet the regulatory requirements of the National Construction Code Volume One (NCC). From a performance perspective, the NCC sound provisions tend to govern the choice of timber building systems more so than the fire provisions due to the lightweight nature of these systems. However, there are no mandatory sound transmission performance requirements for Class 5, 6, 7, 8 and 9b buildings. It should be kept in mind that the NCC Provisions are minimum requirements and further consideration may be given to whether or not the NCC DTS Provisions will meet the expectations of the building occupants.

2.1 Utilising the Deemed-to-Satisfy Provisions for Sound

Part F5 of the NCC is concerned with safeguarding *'occupants from illness or loss of amenity as a result of undue sound being transmitted'* and primarily addresses Class 2, 3 and 9c buildings. Therefore, there are no mandatory sound transmission NCC Performance Requirements for Class 5, 6, 7, 8 and 9b buildings.

However, in large open plan workspaces, consideration should be given to the way occupants will work and interact and the effects of generated sound on their working environment. In office areas, this would include workstations, private meeting rooms, board rooms through to kitchen areas and would consider levels to enable the ability to hold "comfortable" conversations without adversely impacting on adjacent spaces.

A holistic acoustic design would consider:

- sound insulation between spaces (e.g. quiet room and boardroom)
- internal room spaces
- noise from building services/plant
- vibration, and
- external noise.

It should be noted that there are no NCC requirements for sound ratings of external walls, but in some parts of Australia there may be state planning regulations or local government requirements for external wall sound rating. For information on the sound performance of common timber-framed external walls, refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise;* and for Cross Laminated Timber internal walls, floors and for service isolation refer to *WoodSolutions Technical Design Guide #44 CLT Acoustic Performance.*

2.2 The Next Step

As there are no mandatory NCC minimum sound-insulation requirements, the next step is to:

- go to Step 3 to find out about possible options for consideration for improving and/or upgrading sound performance; or
- go to Step 5 to select timber building systems that will comply with minimum NCC sound requirements.

Once sound-insulation requirements are satisfied, go to Step 4 Fire Design Requirements.

2.3 Other Design Considerations

There are other design considerations that need to be taken into account in meeting NCC requirements. The following are not covered in detail in this Guide but are listed as requiring consideration.

2.3.1 Thermal Resistance (R-value)

NCC 2019 Volume One, Section J provides the energy efficiency requirements that a building, including its services, must achieve in order to address the annual greenhouse gas emissions of buildings. The NCC 2016 Amendment 1 edition may be used until the 30 April 2020 after which time the NCC 2019 Section J provisions apply.

The energy efficiency provisions can be met via the NCC 2019 Verification (modelling) Methods, including the Additional Requirements (e.g. floor edge insulation, building sealing) contained in Specification JVa or complying with the Deemed-to-Satisfy Provisions. These provisions will vary based on a range of factors including: the building's location (climate zone), direction of heat flow, level of external wall/window ratio and shading, size and performance of external glazing/windows and form of construction of the external building fabric.

The thermal resistance of timber building elements is dependent upon the level of installed insulation (i.e. thickness), number and thickness of sheet lining layers (plasterboard, flooring), element construction (e.g. incorporating furring channels) and overall thickness of the building element.

Guidance on timber wall and floor/ceilings can be found in WoodSolutions' *R-values for Timber-framed Building Elements.*

2.3.2 Damp and Weatherproofing

The requirements for the damp and weatherproofing of buildings are provided in NCC Volume One Part F1. The intent is to protect the building from external (rain) and internal water (e.g. laundry overflow) and the accumulation of internal moisture in a building causing unhealthy conditions for occupants and potential damage to building elements.

Key areas of consideration include:

- External walls. There are currently no Deemed-to-Satisfy Provisions in the NCC for weatherproofing and therefore suppliers of weatherproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). It is important that installed weatherproofing membranes/ systems are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate (i.e. water barrier) through to the structural timber building elements.
- Internal wet areas (e.g. bathroom) must be waterproofed in accordance with the NCC requirements (F1.7) and have adequate overflow systems (e.g. floor waste) in place to deal with the possibility of wastewater overflow.

Roof coverings. For the purposes of this Guide, and as required by the NCC, roof coverings must be of non-combustible materials (e.g. concrete, metal, terracotta) and be fixed in accordance with, and comply with, the relevant Standard as specified in the NCC Clause F1.5.

Note: *The drawings in this Guide have either omitted damp, weatherproofing and waterproofing details or provided indicative details only. Specific details may vary with climatic conditions and in many instances the only compliance pathway is a Performance Solution which may yield solutions that vary from project to project. Care should be taken to ensure that the Performance Solutions for damp, weatherproofing and waterproofing do not conflict with NCC fire safety requirements.*

2.3.3 Structural Tests

NCC Specification C1.8 describes structural tests for fire-resisting, lightweight wall construction that bounds lift, stair and service shafts, fire-isolated passageways and ramps as well as external and internal walls. The test methods and criteria for compliance are stated in relation to materials, damage, deflection (under static pressure and impact) and surface indentation.

Lightweight wall systems do not require testing if designed and constructed in accordance with the relevant design and loading standards specified in the NCC Part B1 Structural Provisions.

3

Step 3 – Improve and Upgrade Sound Performance

Although not an NCC 2019 requirement for Class 5, 6, 7, 8 and 9b buildings, sound performance between floor levels and between adjacent rooms (e.g. boardroom and workstations) can have a positive benefit for building occupants. Sound performance can often be improved by simple attention to the form and spatial arrangement of the building design. Attention to flanking noise is another important way to improve sound performance. End users may wish to enhance the sound performance of their buildings and, as a result, this Step in the Guide focuses on ways to improve and upgrade sound performance.

3.1 Attention to Building Design to Reduce Sound Transmission

Aspects of the form and spatial design of a building that can be adapted to improve sound performance are dealt with under the following headings.

3.1.1 Floor Layout

Check that the floor layout is beneficial rather than detrimental to sound transmission. Service rooms, including photocopying/collation rooms and kitchens, create extra sound compared the typical work environment. Adequate sound insulation between spaces will assist in enhancing the acoustic environment for occupants.

3.1.2 Windows

Windows normally have lower sound insulation than the walls around them and can be used to improve the building occupant's comfort level from external noise. To improve the acoustic performance of window, consider one or more of the following:

- use thicker glass or double glazing
- use fixed glazing in lieu of opening windows (this may also require sound-insulated ventilation)
- locate windows so that they do not face noisy areas
- reduce the area of windows in the facade
- fill voids between the wall frame and window frame with an appropriate acoustic sealant
- use acoustic sealing strips/gaskets around the edges of open-able sashes.

3.1.3 Doors

As with windows, doors tend to be a weak link in wall systems. Where sound control is desired, solid core doors should be used and be treated with soft acoustic gaskets at interfaces with door jambs. Threshold closers at the bottom of the door or air seals will also help reduce sound transmission. In most cases, achieving the required sound rating will involve the use of gaskets and seals. If a fire door is required, many suppliers have systems to enhance the acoustic performanace of fire-door systems.

3.1.4 Services

The location and detailing of building services are two of the most important considerations in controlling sound transmission in residential buildings.

Generally, services and service penetrations should be located on internal walls or dedicated sound resisting service shafts. In all instances, service pipes should be located away from noise-sensitive parts of the building (e.g. meeting rooms).

3.1.5 External Walls

There are no NCC requirements for sound ratings of external walls, but in some parts of Australia there may be state planning regulations or local government requirements for external wall sound rating. For information on the sound performance of common timber-framed external walls, refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

3.2 Addressing Flanking Noise

The ability to insulate against sound moving from one location in building to the next depends not only on insulating individual wall and floor elements, but also on stopping noise from jumping or transferring from one building element to the next or, worse still, moving through the building in an uncontrolled way. As a result, the effectiveness of sound-insulated construction is concurrently dependent on addressing flanking noise. Flanking noise refers to sound passing around rather than through wall/ floor elements, causing sound to unexpectedly manifest itself in unwanted places.

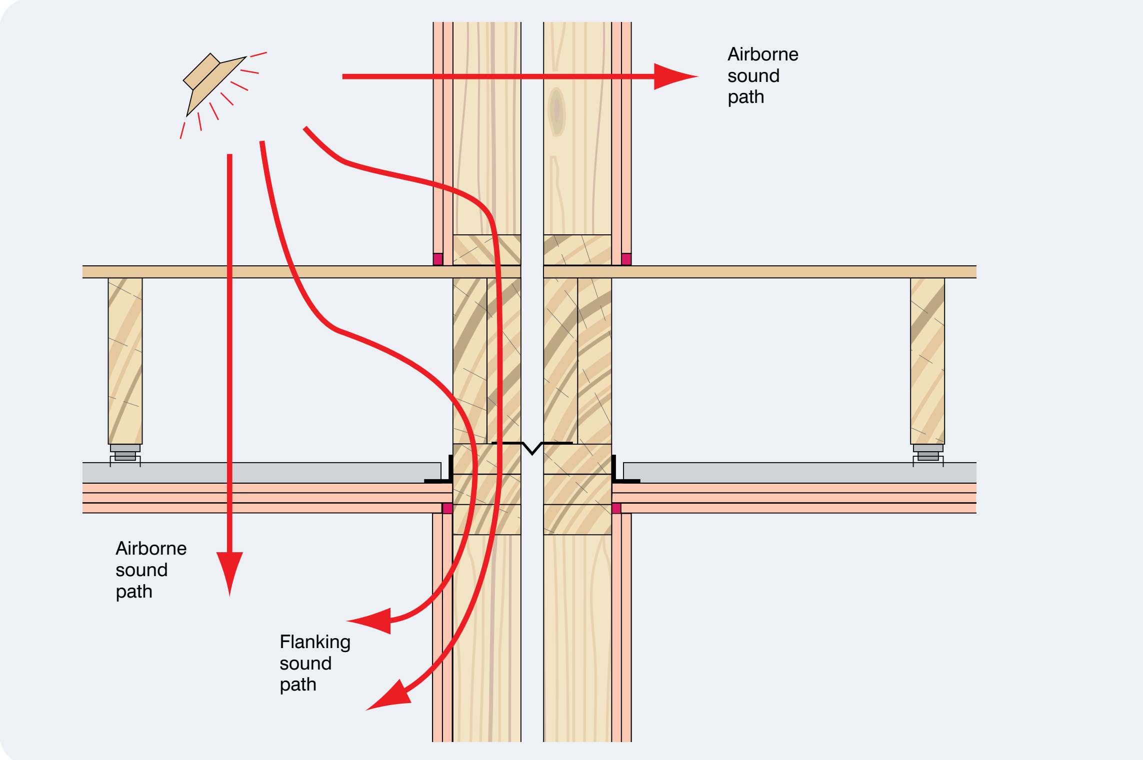


Figure 3.2: Flanking and airborne noise pathways – elevation view.

There are no minimum requirements addressing flanking noise in the NCC's Deemed-to-Satisfy Provisions, though there is an onus on designers and builders to address flanking noise in order to ensure that laboratory-tested wall and floor elements perform to their full potential in the field.

This Guide's approach is to consider reducing flanking noise paths wherever possible. The content is the result of careful thought, taking into account issues such as the limits on what could be achieved in reducing flanking because of their effect on fire and structural integrity. Even though direct reference to reducing flanking noise has not been made, many of the details incorporate elements within them.

An example of reducing flanking noise can be seen in the standard detail for floor joist and flooring over bounding walls where the joist and flooring are not continuous (Figure 3.2).

.

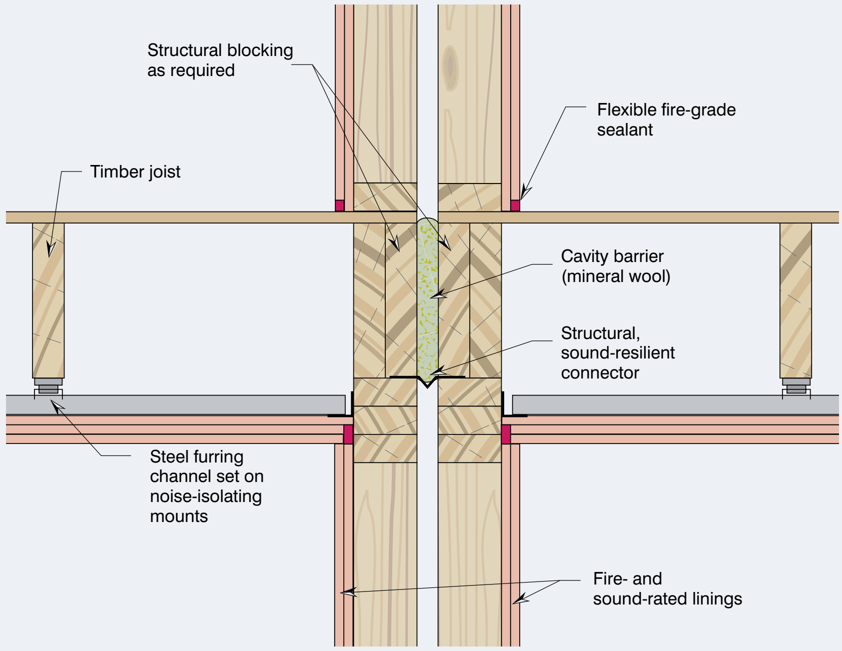


Figure 3.3: Discontinuous floor joist and floor sheeting – elevation view.

There are two main approaches used for addressing flanking noise in timber-framed buildings:

- Limit the ability of the noise to migrate from one element to another, e.g. dampening and isolation at junctions between elements (Figure 3.3).
- Limit the noise getting into wall/floor element, e.g. carpet, floating floors (Figure 3.4).

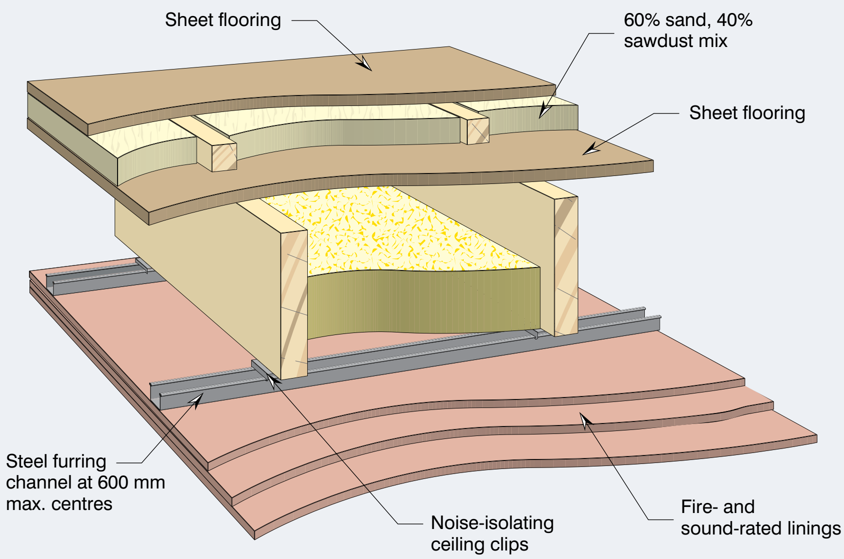


Figure 3.4: Acoustic isolating pad to reduce flanking noise.

In addition to these, timber-framed construction details orientated to improving flanking sound are provided in Section 3.3 and include:

- discontinuous elements at walls, floors and ceilings
- cavities within sound rated elements blocked or travel path increased to reduce noise
- introduced isolating elements, e.g resilient mats or brackets
- platform flooring discontinuous over double stud walls.

3.3 Strategies for Upgrading Sound Performance in Construction

Building occupants often desire higher sound performance than the NCC's minimum requirements. This is especially the case for impact sound and the related issue of vibration from footsteps, water movement through pipes, water hammer and sources such as washing machines, air conditioning units and dishwashers. Other scenarios not dealt with in the NCC include acoustic requirements for home entertainment areas, noise transfer within a dwelling and noise from outside the building (e.g. busy roads, trains, aircraft noise). Options for upgrading typical construction are provided below. Using a combination of options is more likely to give the best performance.

Isolating one side of a bounding construction from the other (e.g. using double stud cavity wall construction). This is also known as decoupling and can be useful in reducing both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.

Avoiding rigid connections between the opposing sides of isolated (decoupled) elements. This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at floor or ceiling level.

Using absorptive materials to fill wall and floor cavities (non-combustible glass fibre or mineral wool) can reduce airborne sound transmission.

Sealing sound leaks at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

For information for the upgrade of external walls refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

3.3.1 Walls

Extra mass on the walls – the addition of mass is a simple yet effective way to improve sound performance in timber construction. In its simplest form, it involves adding extra layers of material such as plasterboard to the outer layer of the sound-rated wall system.

Use a 90 mm rather than 70 mm wall studs – The deeper the wall, the better its sound performance. This is particularly the case where trying to improve C_{tr} scores (being the modification factor for low frequency bass noise applied to R_w scores). The simplest way to do this is to use 90 mm deep studs instead of 70 mm deep studs in a double stud wall system.

Upgrade batts in the wall/floor – There are many types and grades of non-combustible insulation batts in the marketplace. Sound insulation specific batts are best and high-density materials tend to outperform low-density materials. Always refer to the supplier's documented recommendations; some systems require insulation or linings to affect different frequencies and therefore may have differing advice.

3.3.2 Floors

Extra mass on the ceilings – adding mass is a simple yet important way to improve sound performance in timber-framed construction. At its simplest manifestation, this involves adding extra layers of material such as plasterboard to the sound-rated ceiling system.

Extra mass on floors – the addition of mass on floors is an effective way to address impact noise (e.g. footsteps). The additional mass can be in the form of additional layers of sheet flooring.

3.4 The Next Step

The strategies and methods shown in this Step of the Guide may involve specialist proprietary systems that go beyond the scope of this publication. As a result, the next step is:

- Go to proprietary system suppliers and ask for advice on how to integrate their systems with those discussed in this Guide. As part of this, care must be taken to ensure that the fire performance of systems in this Guide are not compromised in any way;
- Go to Step 4 to find out about fire-resisting construction requirements so that these requirements can be considered in tandem with sound requirements before selecting the appropriate timber construction system in Step 5.
- Go to Step 5 to select timber construction that will comply with minimum NCC fire requirements.

4

Step 4 – Define NCC Fire Design Requirements (Design Development)

Designing fire-resisting construction involves a process of understanding how the NCC's Performance Requirements translate into the more objective and measurable Deemed-to-Satisfy Solutions for mid-rise timber buildings, prior to finalising the building layout and selecting timber construction systems that meet these requirements.

4.1 Utilising the Deemed-to-Satisfy Solutions for Fire Design

Section C of the NCC Volume One is concerned with safeguarding people if a building fire occurs. Specific attention is given to evacuating occupants, facilitating the activities of emergency services personnel, avoiding the spread of fire between buildings, and protecting other property from damage as a result of fire.

The NCC details Deemed-to-Satisfy (DTS) Solutions that satisfy the Performance Requirements under:

Part C1 – Fire-resistance and stability

Part C2 – Compartmentation and separation

Part C3 – Protection of openings

These Parts deal with a wide range of issues but it is primarily the fire-resistance of building elements and provisions that relate specifically to mid-rise timber buildings that are dealt with in this Guide. To this end, only the more relevant clauses from Parts C1, C2 and C3 are discussed in more detail below together with the following provisions that apply specifically to mid-rise timber buildings.

- Protection of the building with an automatic fire sprinkler system complying with Specification E1.5 of the NCC (other than a FPAA101D or FPAA101H system)
- fire-protected timber complying with Specification C1.13a of the NCC used for loadbearing internal walls, loadbearing fire walls and for elements of construction required to be non-combustible
- any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible
- cavity-barriers provided in accordance with Specification C1.13 of the NCC.

The NCC Deemed-to-Satisfy Provisions that facilitate the construction of mid-rise timber buildings are shown in Figure 4.1.

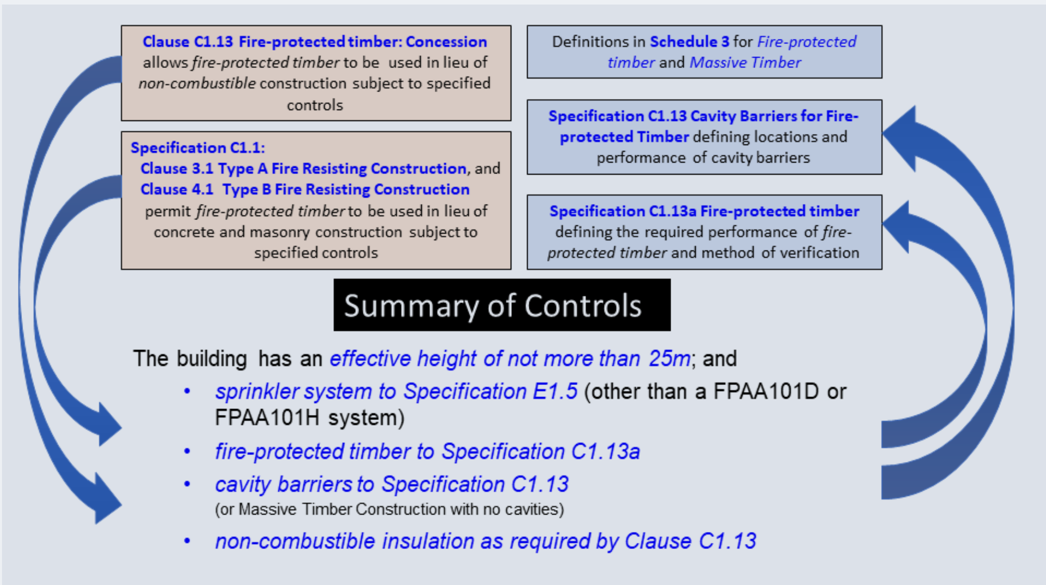


Figure 4.1: Mid-rise timber buildings overview of NCC DTS provisions.

4.2 Automatic Fire Sprinklers

A key fire safety feature for mid-rise timber buildings is the requirement to provide automatic fire sprinkler systems in accordance with NCC Specification E1.5 throughout the building (other than a FPAA101D or FPAA101H system), including any parts of the building that are not of timber construction. This requirement, in conjunction with other fire safety measures, is expected to reduce the risk from fires in mid-rise timber buildings below that in other forms of construction complying with the minimum NCC requirements that do not incorporate sprinkler systems.

4.2.1 Sprinkler Design Standards permitted by NCC Specification E1.5

Under Vic Spec E1.5.2 the sprinkler protection requirements are modified. Refer NCC Vic Spec E1.5.2 for projects constructed in accordance with the Victorian Acts and Regulations

Specification E1.5 allows sprinkler systems to be designed in accordance with:

- AS 2118.1: Automatic Fire Sprinkler Systems – General Requirements
- AS 2118.4: Automatic Fire Sprinkler Systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height
- AS 2118.6: Combined sprinkler and hydrant systems in multi-storey buildings.

AS 2118.4 is limited to accommodation (residential) buildings not exceeding four storeys in height, therefore, most mid-rise timber building sprinkler systems will be designed to comply with AS 2118.1 or AS 2118.6.

Note: FPAA101D and FPAA101H systems are not currently permitted to be used in conjuction with fire-protected timber systems in accordance with the NCC DTS provisions.

4.2.2 Designing Fire Sprinkler Systems to Improve Their Effectiveness

There are opportunities during the design process to incorporate features that can enhance the effectiveness of an automatic sprinkler system and simplify ongoing maintenance. A few examples are described below.

Fast Response heads

Both AS 2118.1 and 2118.6 allow the use of appropriately listed fast response heads. These heads have a more rapid response than standard heads and are therefore more likely to either suppress a fire or limit the fire to a smaller size thus reducing the risk to occupants within the building. Therefore, where appropriate, fast response heads should be specified.

Monitored valves

The reliability of fire sprinkler systems can be enhanced by providing monitored components, such as main stop valves and subsidiary stop valves. Monitored stop valves on each floor, for example, enables sprinkler protection to be maintained throughout the remainder of the building while work is undertaken on part of the sprinkler system. If the valve is left closed when the work is completed, the building owner/operator can be alerted to ensure the error is quickly corrected. This minimises the time periods and extent of areas where sprinkler protection is unavailable. The progressive installation of monitored valves during construction can be used as part of the strategy to address fires during construction by facilitating the progressive commissioning of the sprinkler system.

False ceilings

If sprinkler pipes are run above a ceiling system that is required to have Resistance to the Incipient Spread of Fire (RISF), the ceiling will need to be penetrated to accommodate sprinkler heads; potentially compromising the fire performance of the ceiling if the sprinkler system fails to operate successfully.

The need to penetrate fire-protective coverings can be avoided by providing a false ceiling and running the pipes below the RISF ceiling so that the sprinkler heads and piping need only penetrate the non-fire-resisting false ceiling.

This detail also provides flexibility for lighting systems, air-conditioning and other services.

Selection of materials and pipe connections

The use of materials and pipework connections that minimise the need for hot works on site and reduce the time the sprinkler system is not in operation during maintenance should be considered.

Protection of voids/concealed spaces

Concealed spaces within fire-protected timber elements greater than 200 mm deep generally require protection in accordance with AS 2118.1 and AS 2118.6. Where these voids include elements such as beams, the void depth is measured from the soffit of the beam.

Where open web beams (trusses) or similar elements are included in the cavity, consider providing sprinkler protection in non-insulated spaces where the distance between a ceiling and the bottom chord is less than 200 mm because open webs will not obstruct the sprinkler discharge to the same extent as solid beams.

4.2.3 Hazard Classes of Occupancies for Sprinkler System Design

Sprinkler Systems are classified based on hazard classes of occupancy which are dependent upon the expected rate of heat release together with the fuel loading and burning characteristics of materials in a fire compartment.

The following major classifications apply which include sub-classes;

- Light Hazard Occupancies
- Ordinary Hazard Occupancies
- High Hazard Occupancies

The Hazard Class can have an impact on the required water supply and as a consequence the building layout and costs. The Sprinkler Hazard Class(es) for a building should therefore be identified at an early stage of the project to determine if the existing water supply is adequate or whether pumps and water tanks are required to supplement the existing water supply.

4.3 Fire-Protected Timber Requirements

The NCC defines fire-protected timber as fire-resisting timber building elements that comply with Specification C1.13a.

4.3.1 Fire-Protected Timber – General Requirements

Specification C1.13a applies the following General Requirements to fire-protected timber:

- the building element must be protected to achieve the required FRL
- a non-combustible fire-protective covering must be applied to the timber; it must achieve a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

Note: The NCC Clause C1.9(e) permits some materials, including plasterboard and fibre-reinforced cement sheeting, to be used wherever a non-combustible material is required.

To adequately specify or check Evidence of Suitability of a fire-protected timber element, three items of information are required:

- Fire-resistance Level (FRL) – determined from an AS 1530.4 test or an equivalent or more severe test.
- Resistance to the Incipient Spread of Fire (RISF) – determined from an AS 1530.4 test or an equivalent or more severe test.
- Results from a non-combustibility test in accordance with AS 1530.1 – for materials not deemed non-combustible by the NCC.

(i) FRL is the grading period in minutes for the following three criteria expressed in the order listed below separated by forward slashes (/).

- structural adequacy – ability of a loadbearing element to support an applied load
- integrity – ability of an element of construction to resist the passage of flames and hot gases from one space to another
- insulation – ability of the surface of an element of construction, on the non-fire side of the element, to maintain a temperature below the specified limits.

  For example, if an FRL of 120/60/30 is specified, the element would need to satisfy the structural adequacy criteria for 120 minutes, the integrity criteria for 60 minutes and the insulation criteria for 30 minutes. A dash means that there is no requirement for that criterion, i.e. an FRL of 90/–/– means that only the criterion of structural adequacy applies for 90 minutes.

(ii) The RISF in relation to a fire-protective covering means the covering's ability to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits.

(iii) A material is classified as non-combustible if flaming is not observed and specified temperature rise limits are not exceeded when a sample of material is exposed to the heating conditions specified in AS 1530.1 or it is 'deemed' non-combustible in accordance with NCC Clause C1.9(e).

To facilitate a consistent approach to specifying the performance of fire-protected timber the following format is recommended.

Fire-Protected Timber – FRL120/120/120:RISF45:NC

This means that the element must satisfy the structural adequacy, integrity and insulation requirements for 120 minutes; the RISF criteria for 45 minutes and the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

While individual test/assessment reports from NATA Accredited Testing Laboratories can be used as Evidence of Suitability, it may be more practical for Accredited Testing Laboratories to provide consolidated reports stating the performance in the above format.

Further information relating to the test procedures to determine the FRL and RISF are provided in Appendix A.

NCC Spec C1.13a includes some Deemed-to-Satisfy fire-protective covering systems based on fire-protective grade plasterboard

Cavities are permitted within fire-protected timber elements that, without adequate measures in place, can allow fire spread through concealed spaces. The risk of fire spread from enclosure fires to the cavities is substantially reduced by the requirement for an RISF45 applied to the fire-protective covering, among other things. There is a small residual risk of fire spread to the cavity from an enclosure fire or a fire start within a cavity due to hot works, for example. The risk of fire spread via concealed spaces is further reduced by the provisions for cavity barriers and requirements for wall/ceiling cavity insulation, if present, to be non-combustible.

Specification C1.13a deems two layers of 13 mm fire-protective grade plasterboard fixed in accordance with manufacturer's system requirements to achieve equivalent performance to an RISF45:NC fire-protective covering.

The timber-framed wall system in Figure 4.2 with two layers of 13 mm fire-protective grade plasterboard either side of a cavity between studs could be classified as fire-protected timber FRL90/90/90:RISF45:NC if Evidence of Suitability (as required by the NCC) is provided for the loadbearing wall system to verify that it achieves an FRL of 90/90/90 under similar or more severe load conditions.

This evidence would normally be an AS 1530.4 fire test report from an Accredited Testing Laboratory. The RISF45 for two layers of 13 mm thick fire-protective grade plasterboard does not require further verification since it is Deemed-to-Satisfy the 45-minute requirement and plasterboard is also deemed non-combustible by Clause C1.9(e) of the NCC.

If fire-protective coverings satisfy the NCC Deemed-to- Satisfy Provisions for RISF and non-combustibility, it is only necessary to provide Evidence of Suitability to verify the required FRL

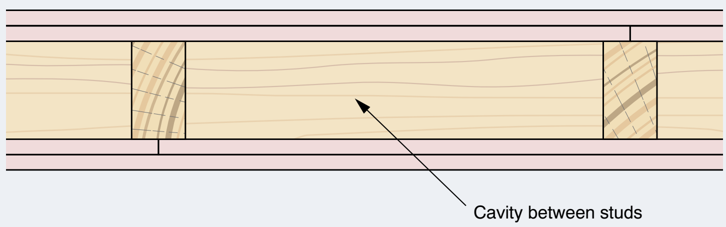


Figure 4.2: Horizontal section through typical FRL90/90/90:RISF45:NC timber stud wall showing allowable cavity.

Ensure the RISF is not compromised by service penetrations, doors and other openings and connections

The primary objective for the inclusion of the non-combustibility requirement for the fire-protective covering is so that the reaction of the fire-protected timber to external and enclosure fires is comparable to elements of construction that are non-combustible: such as reinforced concrete or steel protected with non-combustible materials.

The primary objective for the specification of RISF45 is to reduce the risk of the timber structural elements being ignited prior to burn-out of the contents or fire brigade intervention, in the unlikely event of the automatic fire sprinkler system failing. To achieve this, it is necessary that the RISF performance is not compromised by the presence of building service penetrations and openings for doors and windows. Refer to the relevant sections in this Step for further details on how the RISF performance can be maintained through appropriate penetration fire stopping systems, cavity barriers and lining of openings.

4.3.2 Relaxations for Massive Timber Panels

Subject to compliance with specific requirements, the RISF criteria can be 'relaxed' for massive timber panels in recognition of the higher inherent fire-resistance and mitigation of the risk of cavity fires

The NCC permits the General Requirements for fire-protected timber to be 'relaxed' if both the following additional criteria are satisfied:

- the minimum thickness of timber panels is not less than 75 mm
- there are no cavities between the surface of the timber and the fire-protective covering system or between timber members.

This 75 mm dimension relates to the minimum dimension of the dressed/finished timber member. In most instances, massive timber elements will have minimum dimensions much greater than 75 mm to meet the structural adequacy and integrity criteria of AS 1530.4.

Typical examples of massive timber installations satisfying the conditions are shown in Figure 4.3.

The rationale for allowing the 'concession' for massive timber is that it is reasonable to reduce the performance of the fire-protective covering, subject to maintaining the required FRL, because the consequences of ignition of timber structural members are significantly reduced:

- Timber with a large cross-section can achieve high fire-resistance levels due to its relatively high inherent fire resistance allowing it to continue to support an imposed load or maintain a fire separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for a greater period than lightweight construction.
- By not permitting any concealed spaces between the timber and fire-protective coverings or between timber members, the risk of fire spread through concealed cavities is mitigated.

If the massive timber conditions are met, the following requirements can be adopted for fire-protected timber in lieu of the General Requirements:

- The building elements must be protected to achieve the required FRL and have a non-combustible fire-protective covering applied to the timber which achieves the Modified Resistance to the Incipient Spread of Fire (MRISF) of not less than the values stated in Table 4.1 when tested in accordance with AS 1530.4.
- The Modified Resistance to the Incipient Spread of Fire (MRISF) is determined in accordance with Clause 3 of NCC Specification C1.13a. Further information relating to the test procedures to determine the Fire Resistance and the MRISF are provided in Appendix A.

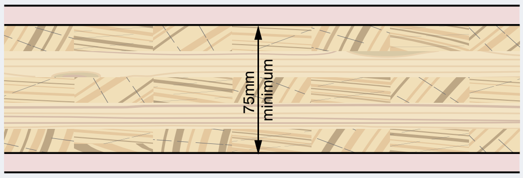
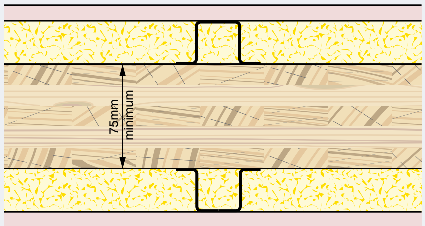
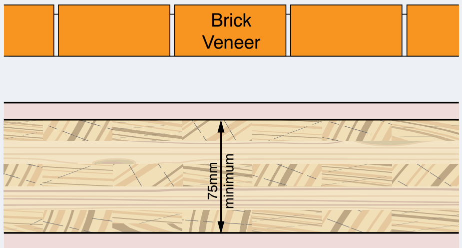
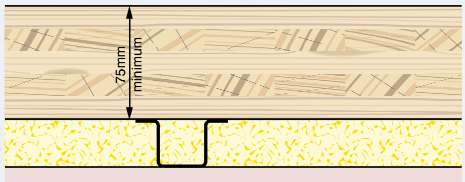
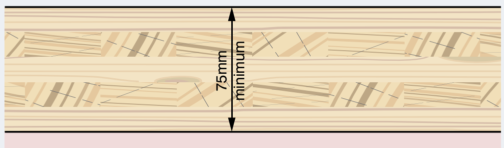
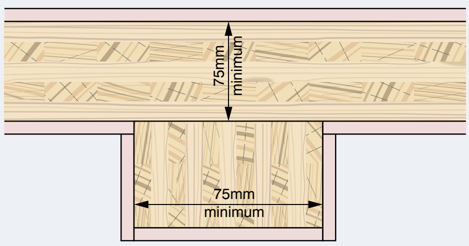
Description	Schematic section
Massive Timber Wall Panels	
Fire-protective covering direct fix to massive timber panel.	
Multi-layer fire-protective covering system direct fix to massive timber panel.	
External brick veneer wall – Note massive timber is faced on both sides with fire-protective coverings.	
Massive Timber Floor Panels	
Multi-layer fire-protective covering system direct fixed to the underside of the massive timber panel.	
Fire-protective covering direct fix to massive timber panel.	
Profiled ceiling fire-protective covering system follows the floor profile maintaining contact with massive timber element.	

Figure 4.3: Typical massive timber panel details for which the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria may be applied.

To facilitate a consistent approach to specifying the performance of fire-protected massive timber the following format is recommended.

Fire-Protected Timber – FRL90/90/90:MRISF30:NC

This means that the element must satisfy:

- the structural adequacy, integrity and insulation requirements for 90 minutes
- the Modified Resistance to the Incipient Spread of Fire criteria for 30 minutes
- the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

Table 4.1: Minimum Fire-protective covering requirements – Massive timber.

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)	Minimum Deemed-to-Satisfy Fire-protective Grade Plasterboard
Inside a fire-isolated stairway or lift shaft	20 minutes	1 layer x 13 mm thick
External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	45 minutes	2 layers x 13 mm thick
All other applications	30 minutes	1 layer x 16 mm thick

Table 4.1 also includes Deemed-to-Satisfy fire-protective grade plasterboard minimum requirements if fixed in accordance with the manufacturer's system requirements in order to achieve the required FRL of the element for massive timber.

For example, if a non-loadbearing wall system is required to achieve an FRL of -/60/60, an appropriate specification for a massive timber element would be:

Fire-Protected Timber – FRL–/60/60:MRISF30:NC

If there is appropriate Evidence of Suitability to show a massive timber element can achieve an FRL of -/60/60 when protected by 16 mm fire-protective grade plasterboard, then no further evidence is required since the 16 mm thick plasterboard is Deemed-to-Satisfy the MRISF30 requirement and the plasterboard is also deemed to be non-combustible.

4.3.3 Fire Resistance Requirements for Mid-Rise Class 5 to 8 and 9b Buildings

Refer Section 1.1 for determination of the Type of Construction Required

Three-storey Class 5 to 8 buildings are required to be of Type B construction and three-storey Class 9b buildings are required to be of Type A construction. Above 3-storeys Class 5 to 8 and 9b buildings are required to be of Type A construction.

Having determined the Type of Construction for the building, it is possible to determine the fire-protected timber requirements for various wall, floor, ceiling and other building elements which varies between the different building Classes.

A typical mid-rise timber commercial building layout is shown in Figure 4.3 and Figure 4.4. Typically post and beam type construction using engineered timber products such as LVL, Glulam or CLT is adopted to achieve the necessary spans normally required for commercial buildings although these elements may be supplemented by lightweight timber frame elements such as pre-fabricated floor elements.

The below ground carpark levels are of reinforced concrete to provide protection against water penetration and termites.

The required FRLs vary with the Class of building and are summarised in Table 4.2 for Type A construction and Table 4.3 for Type B construction.

Check Support for another part requirements (NCC Spec. C1.1 Cl. 2.2) early in design process since it may impact on the required FRLs of some elements of construction

It should be noted that fire protection requirements for a support for another part in Spec C1.1 Clause 2.2 of the NCC can require the structural adequacy component of an FRL to be increased in certain situations where a structural element provides support for another structural element required to have a higher FRL if located in the same fire compartment

For detailed requirements for external walls refer to Section 4.4.

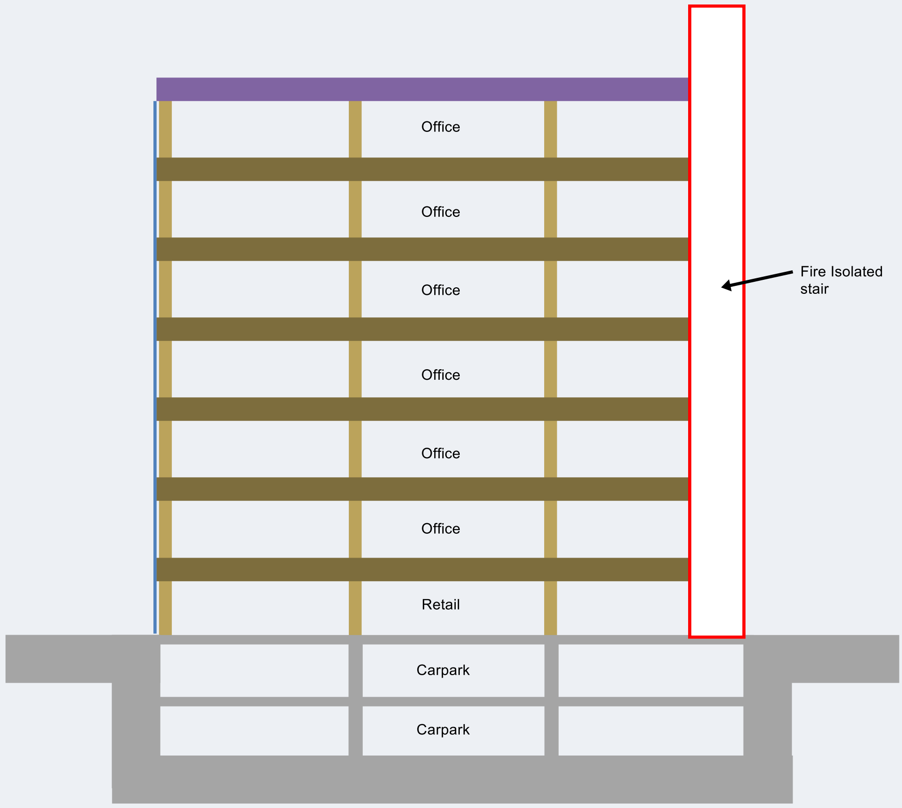


Figure 4.3: Typical schematic section through a mid-rise commercial building.

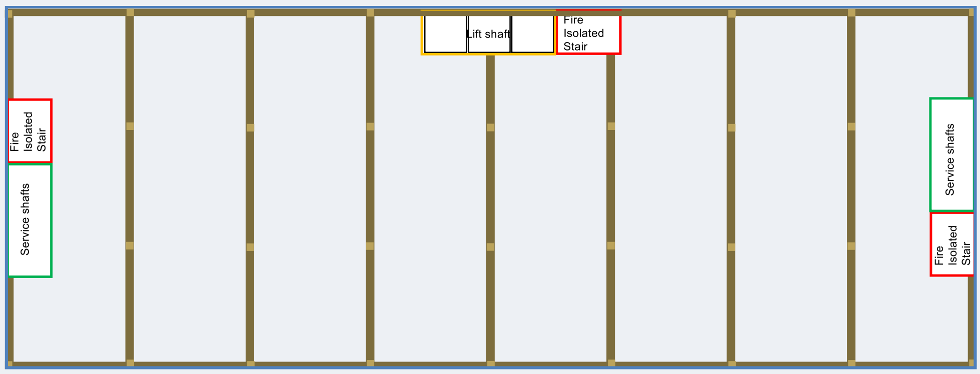


Figure 4.4: Plan of a typical commercial building floor.

Table 4.2: FRLs for mid-rise Class 5 to 8 and 9b buildings of Type A construction.

Symbol	Description	FRL – Structural Adequacy /Integrity/Insulation (minutes)		
		Class 5, 7a or 9b	Class 6	Class 7b or 8
	External wall – maximum[1]	120/120/120	180/180/180	240/240/240
	Fire stair shaft	120/120/120	180/120/120	240/120/120
	Service Shaft	120/90/90	180/120/120	240/120/120
	General loadbearing elements	120/-/-	180/-/-	240/-/-
	Lift Shaft walls	120/120/120	180/120/120	240/120/120
	Common Walls and Fire Walls	120/120/120	180/180/180	240/240/240
	Door to fire Stair	–/60/30	–/60/30	–/60/30
	Fire Door to service shaft	–/60/30	–/60/30	–/60/30
	Lift door	–/60/–	–/60/–	–/60/–
	Fire doors to services risers[2]	–/60/30	–/60/30	–/60/30
	Floors	120/120/120	180/180/180	240/240/240
	Roofs	120/ 60/ 30	180/ 60/ 30	240/ 90/ 60

Note 1: Required FRLs for external walls vary with distance from fire source feature. Further information is provided in Section 4.4

Note 2: Riser doors may not require an FRL if service penetrations are fire stopped at floor level.

Note 3: Specification C1.1 Clause 3.3 of the NCC permits a reduction in the required FRLs in Class 5 and 9b buildings for floors to 90/90/90 and roofs to 90/60/30 if the live load on the floor below does not exceed 3kPa

Note 4: Fire Protection requirements for a support for another part (Spec C1.1 Clause 2.2 of the NCC) can require the structural adequacy component of an FRL to be increased

If live loads do not exceed 3kPa in Class 5 and Class 9b buildings, FRLs may be reduced to 90/90/90 for floors and 90/60/30 for roofs (refer NCC Spec C1.1 Cl. 3.3

Table 4.3: FRLs for mid-rise Class 5 to 8 and 9b buildings of Type B construction

Symbol	Description	FRL – Structural Adequacy /Integrity/Insulation (minutes)		
		Class 5, 7a or 9b	Class 6	Class 7b or 8
	External wall – maximum[1]	120/120/120	180/180/180	240/240/240
	Fire stair shaft	120/120/120	180/120/120	240/120/120
	Common Walls and Fire Walls	120/120/120	180/180/180	240/240/240
	General loadbearing columns and walls that are not required barriers	120/–/–	180/–/–	240/–/–
	Lift Shaft walls	120/120/120	180/120/120	240/120/120
	Door to fire Stair	–/60/30	–/60/30	–/60/30
	Lift door	–/60/–	–/60/–	–/60/–

Note 1: Required FRLs for external walls vary with distance from fire source feature. Further information is provided in Section 4.4

Note 2: Fire Protection requirements for a support for another part (NCC Spec. C1.1, Clause 2.2) can require the structural adequacy component of an FRL to be increased

4.3.4 Resistance to the Incipient Spread of Fire Requirements for Class 5 to 8 and 9b Buildings

If the requirements for general timber systems are applicable, the minimum Resistance to the Incipient Spread of Fire (RISF) performance required from the fire protective coverings is 45 minutes. The NCC includes a DTS solution comprising 2 layers of 13mm fire protective grade plasterboard. In most Class 5 to 8 and 9b building applications using timber-framed construction these minimum requirements will be exceeded to achieve the required FRL for the element.

For massive timber systems the NCC permits concessions to the General Requirements for fire-protected timber if both the following additional criteria are satisfied:

- the minimum thickness of timber panels is not less than 75 mm
- there are no cavities between the surface of the timber and the fire-protective covering system or between timber members.

Instead of applying the RISF criteria, Modified Resistance to the Incipient Spread of Fire (MRISF) criteria apply which, amongst other things, apply limiting temperatures of 300°C instead of 250°C to inner surface of the fire protective covering (refer Appendix A for further details). In addition, the minimum required MRISF times vary with the exposure of the element with the greatest protection required for external walls close to boundaries or adjacent buildings. The inherent fire resistance of some massive timber elements may be such that the required FRL could be achieved without fire protective coverings but it is still required to comply with the minimum fire protective covering requirements to, amongst other things, limit the potential increase in fire severity from exposed timber elements.

Refer NCC Volume One Specification C1.1 for required FRLs and Specification C1.13a for MRISF requirements

The minimum requirements for fire protective coverings are summarised in Table 4.4

Application	General timber systems (minimum DTS requirements)		Massive timber systems (minimum DTS requirements)1	
	RISF (minutes)	DTS Fire-protective Grade Plasterboard	MRISF (minutes)	DTS Fire-protective Grade Plasterboard
Inside a fire-isolated stairway or lift shaft	45	2 layers x 13 mm	20	1-layer x 13 mm
External walls < 1m from allotment boundary or <2 m from a building			45	2-layers x 13 mm
All other applications			30	1-layer x 16 mm

Note 1 Massive Timber requirements can only be applied if both the minimum element thickness and cavity restrictions are satisfied. If these conditions are not fully satisfied for an element, the General Requirements must be applied.

4.4 External Walls/Building Façades

In addition to maintaining loadbearing capacity when subjected to fires within a building, the external walls also need to address the risk of fire spread via the building façade under the following scenarios:

- Fire spread from adjacent buildings (or the fire source feature as defined in the NCC) to the subject building. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by means of specification of minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Fire spread from the subject building (or the fire source feature as defined in the NCC) to adjacent buildings. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by specifying minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.

- Fire spread from an external fire source adjacent to the façade other than adjacent structures; including balcony fires. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Vertical fire spread between openings from a fully developed fire within the subject building. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.

The measures described above are considered in more detail in the following Sections.

4.4.1 Fire-Protected Timber Requirements for External Walls

Refer NCC Volume One Specification C1.1 for required FRLs and Specification C1.13a for MRISF requirements

The FRLs required for external walls are nominated in NCC Specification C1.1 and are dependent on the building use (Class of Building), Type of Construction and proximity to the boundary (fire source feature) or other buildings.

The requirements for Class 5 to 8 and 9b buildings of Type A and B construction are summarised in Table 4.5 and Table 4.6.

Table 4.5: FRLs for external walls of mid-rise Class 5 to 8 and 9b buildings of Type A construction

Distance from fire source feature	Loadbearing (Y/N)	FRL – Structural Adequacy /Integrity/ Insulation (minutes)		
		Class 5,7a or 9b	Class 6	Class 7b or 8
<1.5 m	Y	120/120/120	180/180/180	240/240/240
	N	–/120/120	–/180/180	–/240/240
≥1.5 and <3 m	Y	120/90/90	180/180/120	240/240/180
	N	–/90/90	–/180/120	–/240/180
≥3 m	Y	120/60/30	180/120/90	240/180/90
	N	–/–/–	–/–/–	–/–/–
External columns	Y	120/–/–	180/–/–	240/–/–

Table 4.6: FRLs for external walls of mid-rise Class 5 to 8 buildings of Type B construction

Distance from fire source feature	Loadbearing (Y/N)	FRL – Structural Adequacy /Integrity/ Insulation (minutes)		
		Class 5,7a or 9b	Class 6	Class 7b or 8
<1.5 m	Y	120/120/120	180/180/180	240/240/240
	N	–/120/120	–/180/180	–/240/240
≥1.5 and <3 m	Y	120/90/60	180/120/90	240/180/120
	N	–/90/60	–/120/90	–/180/120
≥3 and <9 m	Y	120/30/30	180/90/60	240/90/60
	N	–/–/–	–/–/–	–/–/–
≥9 and <18 m	Y	120/30/–	180/60/–	240/60/–
	N	–/–/–	–/–/–	–/–/–
≥18 m	Y	–/–/–	–/–/–	–/–/–
	N	–/–/–	–/–/–	–/–/–
External columns <18 m	Y	120/–/–	180/–/–	240/–/–

Even though non-loadbearing external walls do not require an FRL if more than 3 metres from a fire-source feature, the fire-protective coverings must be applied since the external wall is required to be non-combustible. This is to address the risk of external fires on balconies or external areas adjacent to the building and the risk of vertical fire spread through openings if a fully developed fire occurs.

Refer NCC Volume One Clause C2.6(b)(iii)

4.4.2 Vertical separation of openings in external walls

The NCC DTS Provisions for external walls require vertical separation of openings to be addressed for Type A buildings to reduce the risk of fire spreading between floors if a fully developed fire occurs.

This can be achieved by providing spandrel panels or horizontal projections but the NCC waives these requirements if an automatic fire sprinkler system is provided in accordance with NCC Specification E1.5. This recognises that early suppression or control of an internal fire by an automatic fire sprinkler system is an effective means of minimising the risk of fire spread between floors via the façade provided fire-protected timber or non-combustible construction is adopted.

Overcoming the need to provide additional vertical separation by, for example, spandrel panels simplifies construction and provides greater design flexibility

4.4.3 External Wall/Facade Systems

External walls form the building façade and the NCC requires them to serve several functions in addition to addressing fire safety, including:

- structural performance (for safety and serviceability)
- weather resistance (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation.

The external face of the wall may form part of the fire-protective covering, e.g. brick veneer construction, or may cover a fire-protective covering to prevent water penetration and serve other non-fire related functions (e.g. rain screen). In both cases, the NCC requires the external walls to be of non-combustible construction and therefore all these coverings must be non-combustible.

Typical details of brick-veneer construction or fixing of non-fire-resistant coverings, such as rain screens, are shown in Section 5.

If the building design specifies combustible cladding systems, the performance pathway could be adopted subject to it being possible to demonstrate compliance of the wall system and building with the relevant NCC performance requirements.

NCC Verification Method CV3, in conjunction with Verification Methods CV1 and CV2, defines an appropriate method which requires, amongst other things:

- the external wall system to achieve the EW classification in accordance with AS5113,
- enhancements to automatic sprinkler protection.

4.5 Cavity Insulation Requirements

Combustible cavity insulation can facilitate ignition of cavity fires and the rapid spread of fire through cavities. Therefore, if cavity insulation is provided within fire-protected timber elements, it is required to be non-combustible.

Typical solutions include mineral fibre or glasswool insulation with very low organic binder contents. It is therefore important to check that Evidence of Suitability in the form of a current AS 1530.1 report from a NATA Accredited Testing Laboratory is available for the specific products selected.

4.6 Cavity Barrier Requirements

Cavity barriers are barriers placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements that comply with Specification C1.13.

They are required to be provided by the following clauses as part of a Deemed-to-Satisfy solution:

- Clause C1.13
- Clause 3.1d(iii) of Specification C1.1
- Clause 4.1e(iii) of Specification C1.1.

The spread of fire, smoke and hot gases to other parts of the building is limited by cavity barriers in conjunction with other measures such as the use of non-combustible cavity insulation.

The risk of fire spread via cavities and voids in designs that use massive timber is addressed by prohibiting designs that incorporate cavities and voids and hence the level of protection to the timber element can be reduced under certain circumstances.

4.6.1 Determining the Positions of Cavity Barriers

Cavity barriers are required at the following positions unless massive timber construction has been adopted (timber panels not less than 75mm thick and no cavities):

- junctions between fire-resisting floor/ceiling assemblies and fire-resisting walls
- junctions between fire-resisting floor/ceiling assemblies and fire-resisting external walls
- junctions between fire-resisting walls and external walls
- around the perimeters of door and window openings in fire-resisting construction
- horizontal barriers must be provided at each floor level up to a maximum distance of 5 metre centres
- vertical barriers must be provided in walls up to a maximum distance of 10 metre centres.

In many commercial buildings massive timber construction is likely to be employed using, for example, post and beam construction and massive timber floor panels. With careful detailing and direct fixing of the fire protective coverings cavity barriers may not be required.

The following details relate to applications where cavity barriers are required, for example, lightweight timber-frame construction or hybrid systems where lightweight elements are used in conjunction with massive timber elements.

Typical positions of cavity barriers are shown for a representative commercial building of timber-frame construction in Figures 4.5 to 4.7. Table 4.7 is a key to these Figures describing the position and types of interface being protected. Typical cavity barrier details are shown in Section 5.

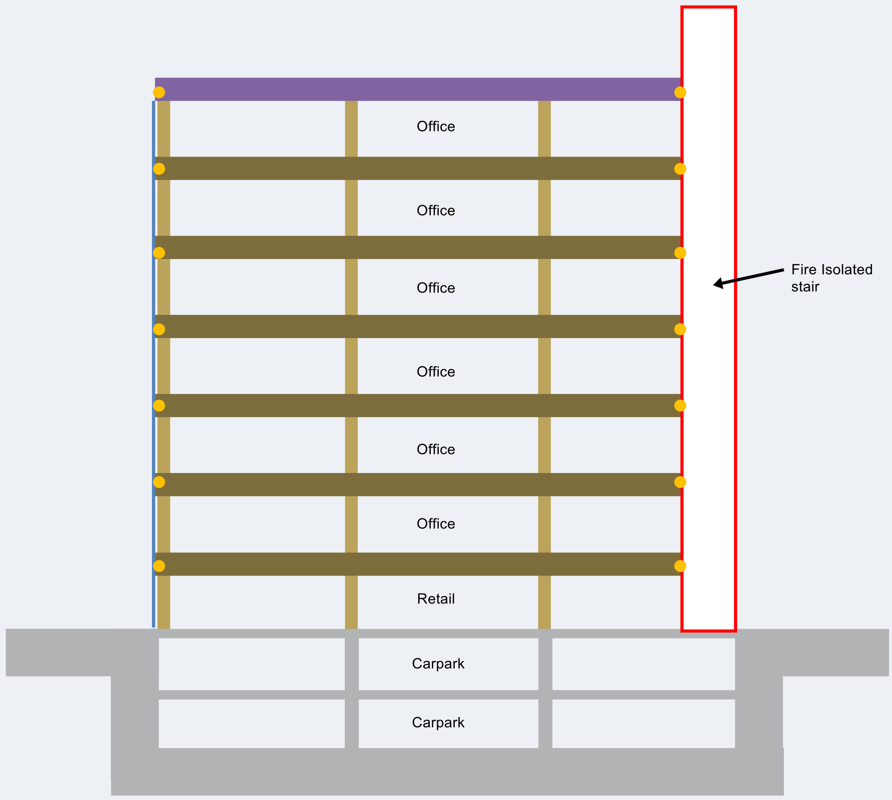


Figure 4.5: Vertical section of a commercial building showing typical cavity barrier positions.

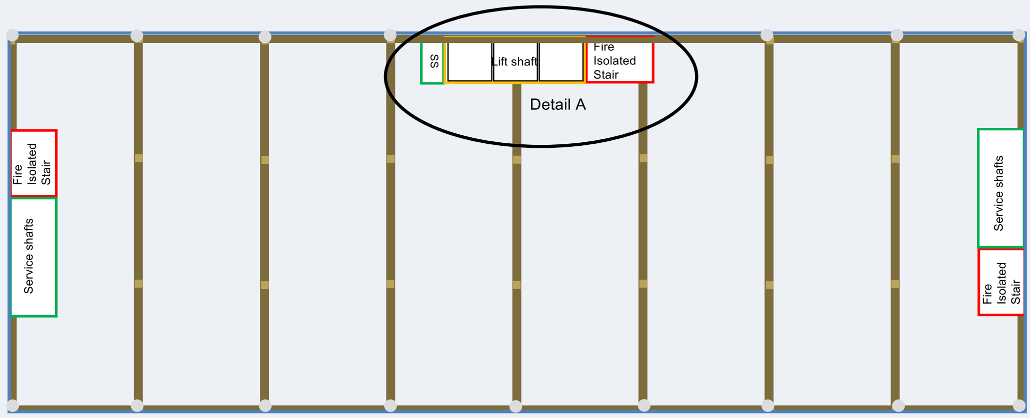


Figure 4.6: Floor plan section of a commercial building showing cavity barriers in external walls at maximum 10m centres (refer Figure 4.7 for details around the cores and typical window details)

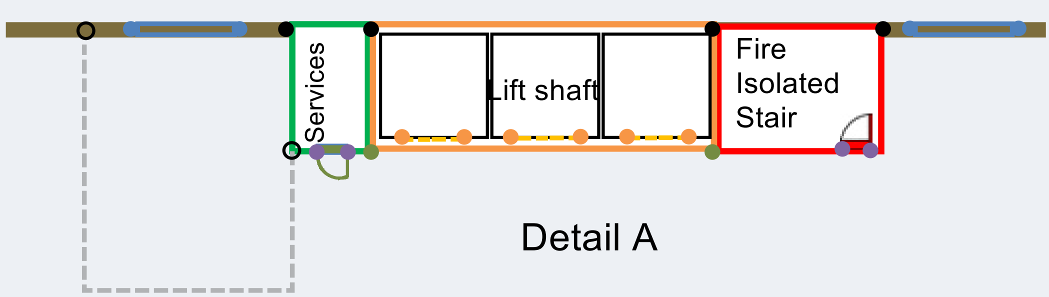


Figure 4.7: Floor plan section of a commercial building core showing cavity barriers around shafts, windows in external walls and interfaces with non-fire resistant walls

Symbol	Description	Comments
●	Horizontal cavity barriers around perimeter of floors	If the floor to floor height is greater than 5 m intermediate horizontal barriers in walls would be required
●	Cavity barriers in fire-protected timber walls	Vertical cavity barriers are required at maximum 10 m centres
●	Cavity barriers around perimeter of non-fire-resisting doors and windows	Required to prevent entry of fire into cavity when non-fire-resisting elements fail
●	Interface of fire-resisting walls with external walls	Can be incorporated as part of a standard detail
●	Interface of shafts with other fire-resistant shafts and walls	Can be incorporated as part of a standard detail
●	Interface with fire doors	Normally part of the standard detail for installation since the doorset is required to maintain the fire resistance of the wall
●	Interface with lift doors	In some instances, it may be more practical to interface with other forms of construction around lift doors
○	Interface between non-fire-resisting wall and fire resisting walls	Continuity of the fire protective coverings should be maintained at the point of penetration

If cavities are provided within floor / ceiling systems having large areas, consideration should be given to providing supplementary cavity barriers within timber floor spaces to restrict fire spread through the cavity. If sub-division of cavities is provided as far as practicable it should be compatible with functional areas and any smoke control systems provided.

4.6.2 Specifying Cavity Barrier Requirements for Building Elements

Essentially there are two levels of performance required for cavity barriers prescribed by the NCC.

- Cavity barriers with FRLs of -/45/45 for building elements with FRLs not exceeding 90/90/90.
- Cavity barriers with FRLs of -/60/60 for building elements with FRLs greater than 90/90/90

For each case, the NCC prescribes Deemed-to-Satisfy Provisions based on minimum thicknesses of timber or non-combustible mineral fibre in the direction of heat flow as summarised below in Table 4.8.

Table 4.8: NCC prescribed Deemed-to-Satisfy solutions for cavity barriers.

Prescribed solution options	Fire-protected timber FRL	
	-/60/60 or -/90/90	-/120/120, -/180/180, -/240/240
FRL for cavity barrier	-/45/45	-/60/60
Timber required minimum thickness*	45 mm	60 mm
Mineral wool required minimum thickness*	45 mm	60 mm

* Minimum thickness measured in the direction of heat flow - refer Appendix B.

For fire-protected timber with large cavities, which may occur in floor and roof cavities for example, it may be more practical to construct cavity barriers from plasterboard supported from timber framing (refer Figure 5.48)

4.6.3 Interfacing fire protected timber with external façade systems

In commercial building applications it is common to use glazed curtain walling systems secured to floor plates particularly where the building is more than 3m from the allotment boundary since non-loadbearing external elements do not require an FRL. In these applications any openings between the floor plate and the curtain walling systems need to be protected to maintain the same FRL as that required for the floor system. These perimeter seals are sometimes also referred to as cavity barriers, but the performance levels and Deemed-to-Satisfy solutions provided in Specification C1.13 for cavity barriers within timber frame construction should not be used since amongst other things higher FRLs are required at the perimeter of a floor plate.

If a combustible façade is proposed to be used using verification method CV3 as Evidence of Suitability, the cavity barriers forming part of the external façade system are required to be evaluated by one of the full-scale test methods nominated by AS 5113 and alternative cavity barrier systems cannot be substituted.

4.7 Lift Shafts

Some designs of timber buildings adopt a hybrid approach and incorporate concrete or masonry shafts. Where this approach is adopted it is important that the potential for differential movement between the timber structure and shaft be considered when detailing connections and interfaces. When designing lift shafts, the lift supplier should be involved at an early stage to ensure the shaft will satisfy their design requirements and applicable regulations.
The remainder of this Section addresses the fire safety performance of lift shafts of fire-protected timber construction.

When designing lift shafts, it is important to involve the lift supplier at an early stage to ensure the shaft will satisfy their design requirements and applicable regulatory requirements.

4.7.1 Timber-framed Lift Shaft Construction

Table 4.9 shows the NCC requirements that are applicable to timber-framed lift shafts in mid-rise commercial timber buildings.

EWFA RIR 37401400 is available on the WoodSolutions website

Table 4.9: Requirements for fire-protected timber-framed lift shafts.

Criteria	Required Performance		
	Class 5, 7a or 9b	**Class 6**	**Class 7b or 8**
FRL for loadbearing lift shaft walls	120/120/120	180/120/120	240/120/120
FRL for non-loadbearing lift shaft walls	–/120/120	–/120/120	–/120/120
RISF for lift shaft walls	45 mins		
FRL Lift landing doors	–/60/–		

Note1: The wall FRL and RISF requirements are applicable from both inside and outside the shaft.

To minimise sound transmission to adjoining areas, double stud construction may be employed and/or an independent support structure provided within the shaft.

The fire-resistance of lift landing door assemblies should be determined by fire tests in a representative wall construction type. At the time of preparation of this Guide Evidence of Suitability for lift landing doors directly fixed to timber-framed wall assemblies was unable to be obtained.

A practical way to address this is to transition the shaft wall construction around the door opening to a form of non-combustible construction having FRLs with which the performance of the lift door has been verified.

An example of transitioning to a steel shaft wall system from a fire-protected timber wall shaft is shown in Figures 4.8 and 4.9 - refer to Section 4.10 and Section 5 for further details. These interface details have been assessed by an Accredited Testing Laboratory (EWFA Regulatory Information Report (RIR) 37401400), which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the lift landing doors up to the lesser of 120/120/120 or the FRL of the element.

Evidence of Suitability for the specific proprietary lift door, steel stud shaft wall and timber shaft wall, in accordance with Clause A5.2 and A5.4 to A5.6 as appropriate of the NCC, should be submitted to the relevant regulatory authority in addition to RIR 37401400.

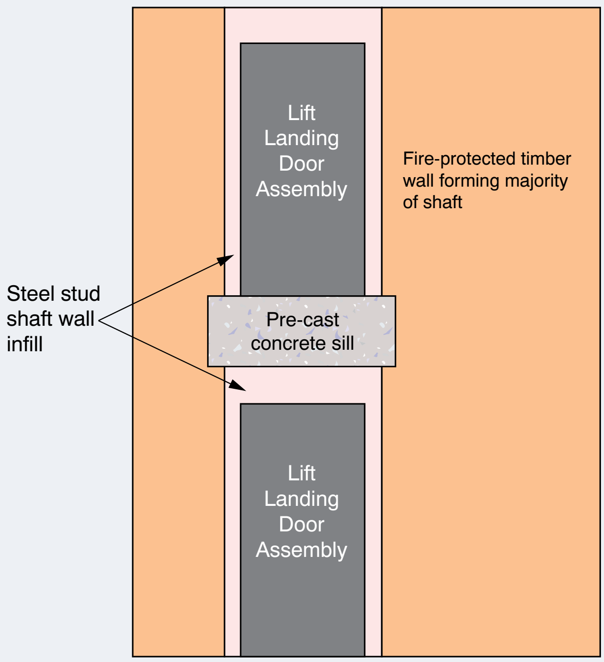


Figure 4.8: Elevation showing wall transition around lift landing doors.

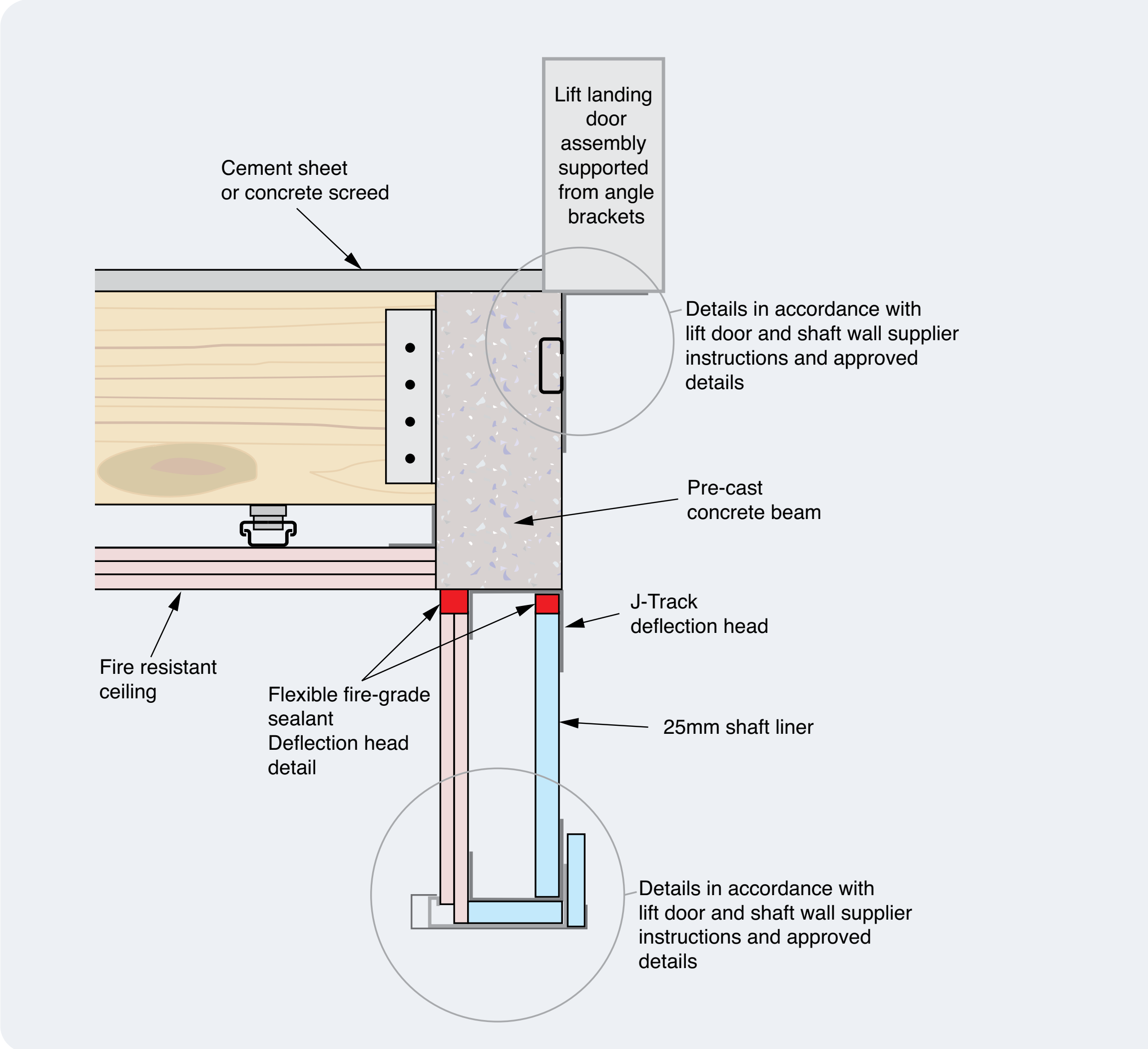


Figure 4.9: Generic detail for sill and head mounting.

4.7.2 Massive Timber Lift Shaft Construction

Table 4.10 shows the NCC requirements that are applicable to massive timber lift shafts in mid-rise commercial timber buildings.

Table 4.10: Requirements for fire-protected timber lift shafts if using massive timber.

Criteria	Required Performance		
	Class 5, 7a or 9b	Class 6	Class 7b or 8
FRL for loadbearing lift shaft walls	120/120/120	180/120/120	240/120/120
FRL for non-loadbearing lift shaft walls	–/120/120	–/120/120	–/120/120
MRISF for lift shaft walls	30 mins outside face; 20 mins inner face		
FRL Lift landing doors	–/60/–		

If utilising massive timber construction, the MRISFs are reduced from 30 to 20 minutes within the lift shaft. This relaxation reflects the lower probabilities of severe fires occurring within these areas, but a basic level of protection is retained to address the small potential of fires occurring within these areas; where fire may spread to evacuation paths which could be quickly compromised due to rapid fire spread in the early stages of a fire. The outer faces still require an MRISF of 30 minutes. This configuration is shown in Figure 4.10.

To minimise sound transmission to adjoining areas, double skin construction may be employed and/or an independent support structure provided within the shaft for a single skin option. If double skin construction is employed it should be noted that the NCC does not permit an unfilled cavity between the massive timber skins when using the massive timber provisions. If unfilled double-skin construction is preferred, there is still an option to use the General Requirements (timber-framed construction) rather than the massive timber requirements. The General Requirements require the inner and outer faces to achieve a RISF of 45 minutes. This can be achieved by applying two layers of 13 mm thick fire-protective grade plasterboard to both the inner and outer faces of the shaft.

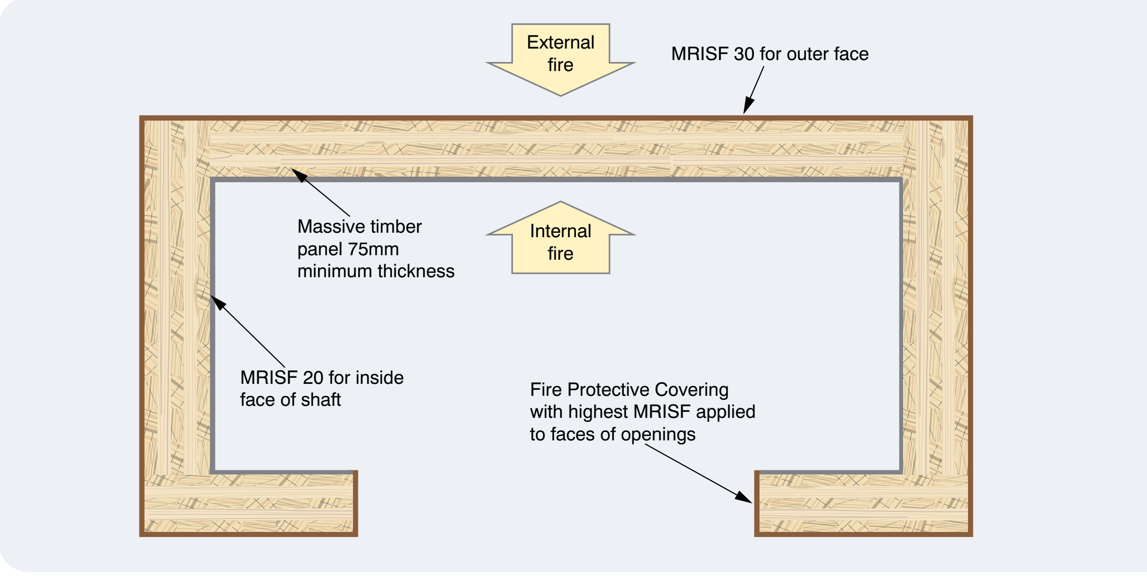


Figure 4.10: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

EWFA Regulatory Information Report (RIR) 37401400 is available on the WoodSolutions website

4.8 Fire Isolated Stairs and Passageways

The FRL, RISF or MRISF required for Fire-Isolated Stairs and Passageways are the same as those required for lift shafts described in Section 4.7 without the complication of lift landing doors.

Fire doors to fire-isolated stairs or passageways are required to achieve an FRL of –/60/30. Several proprietary fire door systems have been tested when mounted in timber construction. Installation details for fire doors capable of achieving the required FRLs should be obtained from the supplier as they may vary. Figure 4.11 shows a typical interface detail with a fire-protected timber wall. These interface details have been assessed by an Accredited Testing Laboratory (EWFA RIR 37401400) which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the fire doors up to the lesser of 120/120/120 or the FRL of the element. Evidence of Suitability for the specific proprietary door, and timber shaft wall, in accordance with Specification A5.4 of the NCC, should be submitted to the relevant regulatory authority in addition to EWFA RIR 37401400.

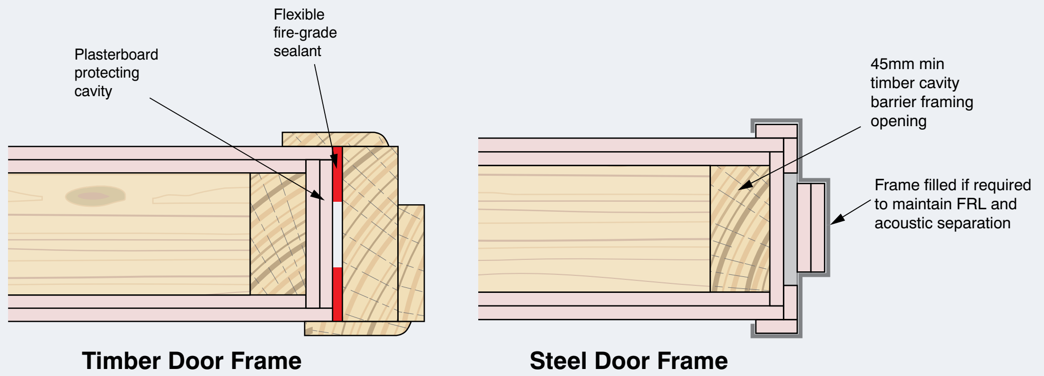


Figure 4.11: Typical fire door installation details.

4.8.1 Timber Stairways Concession

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway constructed from fire-protected timber in accordance with Specification C1.13a subject to:

- timber having a finished thickness of not less than 44 mm
- an average timber density of not less than 800 kg/m^3 at a moisture content of 12%
- the building being protected throughout by a sprinkler system complying with Specification E1.5, other than FPAA101D and FPAA101H systems, that is extended to provide coverage within the fire-isolated enclosure, and
- the underside of flights of stairs and landings at or near the level of egress or direct access to a car park being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres.

Fires starting in fire-isolated stairs are rare. When they do occur, they generally involve stored or introduced materials and often the cause is malicious. Even though it is not permitted to store goods in fire-isolated stairs and passageways, areas under the lowest flight of stairs form a convenient dry area for temporary storage. These areas may also not be secured, further increasing the risk of malicious fire starts.

While it could be argued that the extension of the sprinkler system to fire-isolated stairs and passageways addresses this issue, as an additional precaution, the underside of the lower stairs where combustibles can be stored are required to be protected by a fire-protective covering of 13 mm fire-protective grade plasterboard.

Section 5 provides further details of the requirements for timber stairways.

Careful planning and design of building services and distribution paths at all stages of the design process can greatly simplify construction and subsequent maintenance

4.9 Building Services

4.9.1 Selection of Building Services and Distribution Paths

The building services and associated cable and pipe runs need to be selected and refined during the design process to ensure the installation of the services and associated fire protection is efficient and reliable, with access to ensure the systems can be maintained or expanded without compromising fire safety.

Key principles for consideration with respect to fire safety and acoustics are:

a) Minimise service penetrations through fire-protected timber construction and fire-resisting construction.

   This can be achieved by measures such as self-contained air-conditioning systems serving each fire compartment (where practicable), and false ceilings and wall facings allowing services to run behind the non-fire-rated facing without penetrating the fire-resisting elements.

b) If service penetrations through fire-resisting construction cannot be avoided, the services should penetrate shaft or service duct walls rather than fire-resisting walls or floors separating occupied areas as far as practicable.

   This reduces the acoustic impact as well as limiting the consequences if a penetration protection system fails; as smoke and fire spread will initially be limited to the service ducts.

c) Where practical, shafts, service risers and service ducts should be readily accessible from public parts of the building to facilitate maintenance and inspection, but access hatches/panels or doors should normally be secured to prevent unauthorised access and tampering with fire protection systems.

d) If service penetrations through fire-protected timber construction cannot be avoided they should be grouped together and penetrate framed out openings that are then fire stopped with proprietary systems such as non-combustible batts, board or pillow systems.

   This approach substantially reduces the risk of fire spread to cavities at a point of weakness and ignition if hot works are being undertaken on the services.

e) Services and connection details should be designed to avoid or minimise the need for hot works.

f) For fire services, such as sprinkler systems, the time they will be unavailable during maintenance and modification should be minimised.

In some applications these principles may conflict with other principles and project needs but with careful design, suitable solutions can be developed in most cases. For example, the use of CPVC piping for sprinkler systems can reduce hot works but if the pipework is needed to be adjusted the system will be unavailable while the glue sets. A better option to avoid hot works and minimise down time may be to use mechanical joiners for metal pipes.

It can be relatively straight forward to apply the above principles to typical mid-rise commercial buildings where one or more structural cores are used to manage lateral resistance. These cores are commonly used to house lift and stair shafts, service risers and kitchens and toilets are either included in the core or located next to the core.

It is then relatively easy to distribute power, lighting communications and install the sprinkler system piping for a whole floor of the building above a false ceiling so that it is not necessary to penetrate the fire protective coverings or fire resistant elements except for floor to floor penetrations at the building core positions. A schematic of this arrangement is shown in Figure 4.12.

In residential parts of buildings (e.g. a class 4 part of a commercial building or a mixed use building containing apartments), each residential SOU is effectively a fire compartment and includes bathrooms and kitchens and, in many instances, it is impractical to consolidate services such as drain, waste and vent (DWV) pipes around the central core and therefore service shafts are distributed around the floor. For apartment buildings, the use of self-contained heating, ventilation and air-conditioning (HVAC) systems tends to be preferred. For further information on mid-rise residential buildings refer to:

WoodSolutions Technical Design Guide #37R Mid-rise Timber Buildings Multi-Residential Class 2 and 3

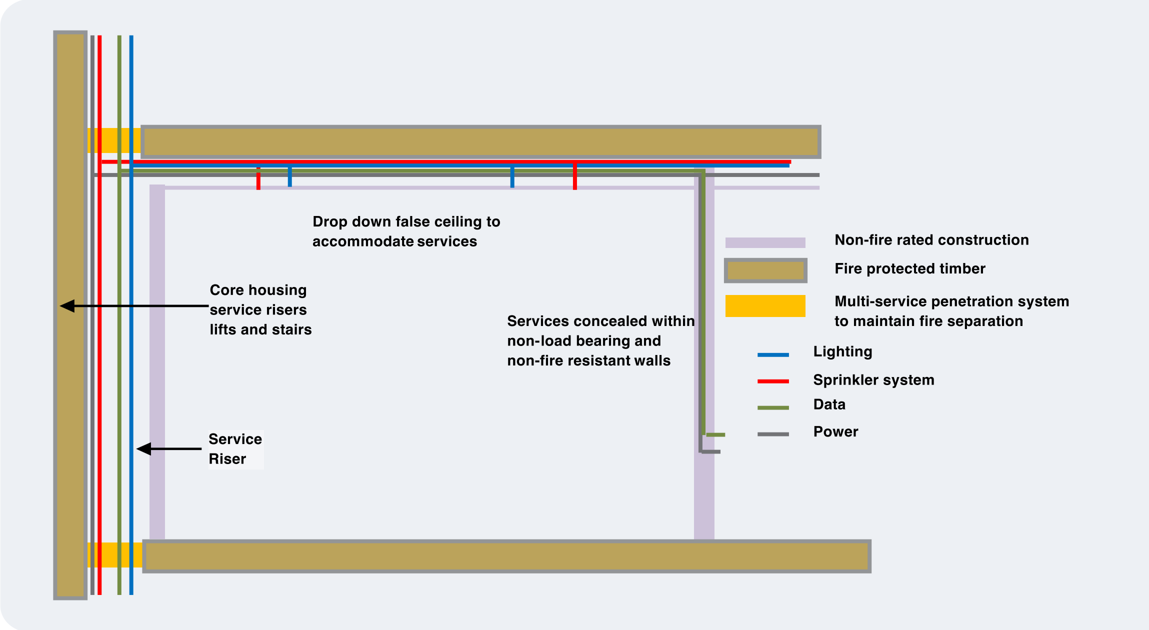


Figure 4.12: Schematic showing distribution of services from a Central Core

4.9.2 Service Shaft Construction

The requirements for fire-protected timber service shafts used for ventilation, pipes, garbage or similar purposes are summarised in Table 4.10 for buildings of Type A construction.

Shafts must also be enclosed at the top and the bottom with a floor/ceiling system of the same Fire Resistance Levels and Resistance to the Incipient Spread of Fire ratings as the walls; except where the top of the shaft is extended beyond the roof, or the bottom of the shaft is of non-combustible construction laid directly on the ground.

Table 4.10: Requirements for fire-protected service shafts in mid-rise timber buildings.

Criteria	Required Performance		
	Class 5, 7a or 9b	Class 6	Class 7b or 8
FRL for loadbearing service shaft walls	120/90/90	180/120/120	240/120/120
FRL for non-loadbearing service shaft walls	–/90/90	–/120/120	–/120/120
RISF for service shaft walls	45 minutes		
MRISF for service shaft walls (Massive Timber)	30 minutes		

In many instances it is more practical to construct non-loadbearing shafts from laminated board systems or steel shaft wall construction in lieu of fire-protected timber construction.

If the shaft wall is non-loadbearing and of non-combustible construction, and the Specification E1.5a concessions are applicable, the FRL of the shaft can be reduced to –/45/45 with service penetrations required to achieve an FRL of –/45/15.

Details on how to construct shafts in timber-framed construction and how to interface fire-protected timber walls with laminated board shafts or steel shaft wall construction are given in Section 5.

4.9.3 Protection of Service Penetrations

The NCC requires service penetration systems to comply with AS 4072.1 and AS 1530.4. For services penetrating fire-protected timber elements there is an added complication that the Resistance to the Incipient Spread of Fire (RISF) or Modified RISF criteria have also to be satisfied in addition to the integrity and insulation criteria applied to the non-fire side.

Further explanations of the test procedures are provided in Appendix A.

Typical solutions to address RISF performance criteria include:

- boxing out openings with plasterboard or other non-combustible board achieving the required RISF, MRISF and FRL performance
- filling the area around the service penetration with non-combustible mineral fibre insulation
- transitioning to a different wall type where service penetrations are required.

Examples are provided in Section 5.

4.10 Interfacing With Other Forms of Construction

There can be advantages in adopting hybrid forms of construction in buildings. For example, ground floor and basement areas may be constructed from concrete to minimise the risk of water penetration, minimise potential damage in flood-prone areas or address the risk of termites.

The relatively lighter weight of timber structures also makes timber construction ideally suited to the upward extension of existing buildings facilitating infill developments and recycling existing buildings. For example, it may be possible to add apartments above existing retail buildings without having to undertake extensive foundation works.

4.10.1 Separation of Different Classes of Buildings

The NCC addresses the separation of classifications within a building in Clauses C2.8 and C2.9.

Refer NCC Clause C1.3 for determining the type of construction required for a multiple classification building

For different classifications on the same storey, parts having different classifications should be separated by a fire wall having the higher FRL of the two, in accordance with Specification C1.1.

For different classifications in different storeys in a Type A building (most mid-rise buildings), the floor between the adjoining parts must have an FRL not less than that prescribed by Specification C1.1 for the lower storey.

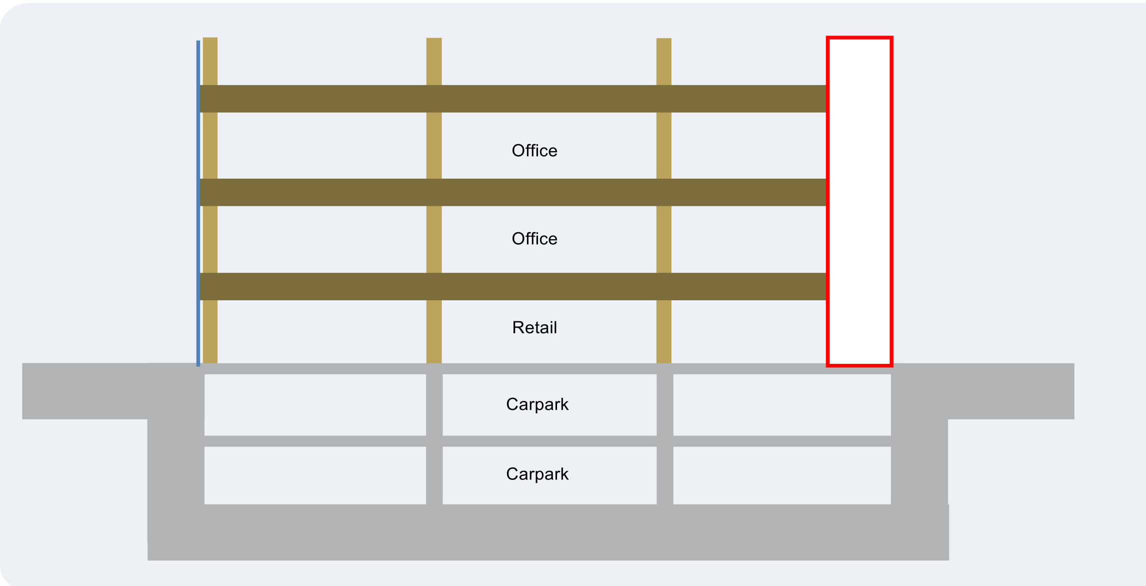


Figure 4.13: Example of multi-class building.

A typical building layout is shown in Figure 4.13 with concrete framed construction for the carpark below timber retail and office levels. For the fire-protected timber concession to apply, the whole building must be sprinkler protected in accordance with NCC Specification E1.5 excluding FPAA101D and FPAA101H systems.

Retail use is assigned to Class 6 buildings. From Table 3 of Specification C1.1, the floor separating the retail and office levels would require an FRL of 180/180/180.

4.11 Special Fire Issues

In constructing mid-rise timber buildings, special issues arise as buildings become larger and more complicated. Although this Guide does not attempt to provide information to suit all circumstances, information is provided where there is relevance to timber construction practices.

Refer NCC Volume One Clause C1.13 for the Fire-protected timber Concession

4.11.1 Fire Precautions During Construction

Fires may occur on building construction sites due to the nature of the works. Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Timber construction covered with fire-protective linings is a safe and economical building system. The fire-protective coverings play an important role in providing this fire safety but, due to the construction sequencing, there may be a period where the timber is not protected. This is when timber buildings are at their highest risk from construction fires.

The NCC requires a suitable means of fire-fighting to be installed in a building under construction to allow initial fire attack by construction workers and for the fire brigade.

A building under construction that is less than 12 metres in effective height must have one fire extinguisher to suit Class A, B and C fires, as defined in AS 2444, and electrical fires provided at all times on each storey adjacent to each exit, or temporary stairway or exit.

After the building has reached an effective height of 12 metres, the following additional measures must be operational:

- the required fire hydrants and fire hose reels must be operational in at least every storey that is covered by the roof or the floor structure above, except the two uppermost storeys
- any required booster connections must be installed.

In this instance, 'required' means satisfying the NCC performance requirements in the complete building using either the performance or Deemed-to-Satisfy pathways.

As the scope of the NCC does not fully address Workplace Health and Safety (WHS) issues, and the NCC prescribes minimum levels of compliance, builders and building owners need to consider what is actually required for the building site. Typical matters that should be considered include:

- progressive installation of fire-fighting services
- progressive installation of fire-protective grade covering of timber members (i.e. installation of fire-protective coverings) and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

4.11.2 Bushfire-prone Areas

The requirements for commercial buildings to address the risk of bushfires vary between States and Territories and may fall under different jurisdictions to standard building works. The need to consider bushfire exposures should be determined early in the design processes and addressed accordingly.

The NCC requires external walls to be of non-combustible construction in mid-rise buildings and the fire-protected timber provisions requires timber elements to be protected by non-combustible fire-protective coverings providing a good basis for the building to resist bushfire attack.

4.11.3 Lightweight Construction Structural Requirements – Specific Applications

The NCC requires elements that have Fire Resistance Levels (FRLs), or that form a lift, stair shaft, an external wall bounding a public corridor, non-fire-isolated stairway or ramp, to comply with Specification C1.8, if they are made out of lightweight materials such as timber-framing faced with plasterboard.

Specification C1.8 defines a structural test for lightweight construction and, in most parts, is directly related to the performance of the linings used. Appropriate Evidence of Suitability should be obtained from suppliers of lining materials used to verify compliance during the design phase.

4.11.4 Robust Structural Design

The NCC, under Part B1 Structural Provisions (BV2), provides a verification method for structural robustness as a means of verifying compliance with performance requirement BP1.1(a)(iii). The Verification Method states:

Compliance with BP1.1(a)(iii) is verified for structural robustness by -

(a) assessment of the structure such that upon the notional removal in isolation of -

- (i) any supporting column; or
- (ii) any beam supporting one or more columns; or
- (iii) any segment of a loadbearing wall of length equal to the height of the wall, the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high-risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The structural design of mid-rise timber buildings must comply with these requirements and the design guidance provided in *WoodSolutions Design Guide #39 Robustness in Structures* to ensure the building is adequately robust in the event of localised failure of elements during a fire.

5

Step 5 Integrate Architectural, Structural and Building Services Designs (Detailed Design)

This step brings together the content of the previous Sections to develop an integrated design. An office building with basement carparking and retail at ground level is used to demonstrate the process including interfacing a fire-protected timber building with other forms of construction and parts of a building with a different Class.

A key focus of this Step is coordinating the various design disciplines so that:

- Timber elements and protection systems are optimised to satisfy the NCC requirements in a practical and cost-effective manner by focusing on the synergies between elements designed to satisfy the following criteria:
  - fire-protected timber
  - sound transmission and insulation
  - thermal resistance
  - weatherproofing
  - structural tests for lightweight construction.
- Interfaces between building services and the structure, fire-protected timber elements and acoustic barriers are designed:
  - to minimise building service penetrations through fire-protected timber elements and acoustic barriers as far as practical
  - such that where services have to penetrate fire-protected timber elements the fire safety performance of the element is not compromised, and fire separation is maintained
  - so that if services have to penetrate acoustic barriers the positions are selected to minimise negative impacts on amenity
  - so that service penetration systems can accommodate any differential movement between elements
  - to allow for maintenance and additions/modifications to the building services.
- Structural design is efficient and robust.
- Other fire safety principles for mid-rise buildings are satisfactorily implemented including:
  - cavity barriers
  - automatic fire sprinkler systems.
- Other design requirements are addressed such as termite management and resistance to ground water/moisture penetration.

5.1 Optimising the Performance of Elements of Construction

Elements of construction in a modern building may have to serve several functions including:

- restricting fire spread
- limiting sound transmission from adjacent enclosures (and in some instances external noise)
- limiting heat loss and/or heat gain through external elements
- weather resistance of external facades and roofs
- impact resistance to reduce the risk of damage to lightweight construction.

The elements also need to achieve levels of durability appropriate for the application. Further advice on durability is provided in: *WoodSolutions Timber Design Guide #5 Timber service life design – Design guide for durability.*

Efficient designs can be achieved by selecting combinations of materials and configurations that work together to satisfy the design objectives summarised in the following Sections.

Typical examples include:

- Cavity barriers required by the NCC Deemed-to-Satisfy for mid-rise timber buildings to reduce the risk of fire spread through concealed spaces can also be used to minimise flanking noise transmission around the perimeters of elements of construction and reduce heat loss via leakage through the structure.
- Non-combustible cavity insulation will:
  - reduce the risk of fire spread through cavities
  - reduce sound transmission through elements of construction
  - reduce heat loss and/or gain through external walls.

5.1.1 Fire-protected Timber

Fire-protected timber has timber structural members protected by non-combustible fire-protective coverings. The fire-protective coverings:

- prevent or delay the ignition of the timber members so that the response to an enclosure fire will be similar to non-combustible elements such as masonry or concrete during the growth period and prior to fire brigade intervention
- ensure the fire-protected timber element achieves the Fire Resistance Level (FRL) prescribed for the particular element.

Any insulating materials provided within cavities must be non-combustible to reduce the risk of fire spread through cavities and voids.

The NCC contains some Deemed-to-Satisfy Solutions for fire-protective grade plasterboard coverings but there are many opportunities for the use of optimised proprietary systems. For example, combinations of high-performance non-combustible fire resisting claddings and mineral fibre insulation could provide lighter weight, more cost effective options.

The NCC DTS solutions recognise that massive timber panels have a relatively high inherent fire resistance and, if there are no concealed cavities or voids, the risk of fire spread through concealed spaces will be substantially reduced or removed. Therefore, provided the minimum dimensions prescribed for massive timber panels are satisfied and there are no internal cavities and voids, the NCC allows some relaxations to the requirements for fire-protective coverings (refer Section 4.3).

Note: The use of timber blocks and other combustible fire protection systems such as intumescent paints in lieu of non-combustible fire-protective coverings is not permitted under the NCC DTS solutions for mid-rise timber buildings due to the potential increase in risk of fire spread to the structural element as the combustible materials are consumed.

5.1.2 Cavity Barriers

The primary objective of cavity barriers is to prevent uncontrolled spread of fire through cavities in the low probability the protective covering fails or fire starts within the cavity.

The NCC provides Deemed-to-Satisfy solutions using solid timber or mineral fibre but also specifies FRLs for cavity barriers encouraging the development of proprietary systems optimised for specific applications.

Careful detailing can provide opportunities for efficient design.

Typical examples include:

- in a single leaf, timber-framed stud wall, the top and bottom plates can be dimensioned such that they can act as cavity barriers
- if a cavity is filled with non-combustible mineral fibre insulation to achieve a nominated R-value or enhanced acoustic separation, the mineral fibre may also satisfy the requirements for a cavity barrier.

Battening wet areas protects fire- and sound-rated walls from compromise due to bath and shower installation and can also be used to reduce service penetrations through fire-protected timber elements

5.1.3 Sound Transmission and Insulation

In timber construction, airborne and impact sound requirements are primarily achieved using one or more of the following principles:

- **Increasing mass (e.g. increasing the thickness of wall linings).** This can be particularly useful in reducing airborne sound transmission. For instance, like fire-grade linings, the greater the number of layers, the greater the increase in R_w (Note: extra factors are involved in increasing R_w+C_{tr}).
- **Isolating one side of a wall from the other** (e.g. using double stud cavity wall construction). This is also known as decoupling (discontinuous construction) and can reduce both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.
- **Avoiding rigid connections between the opposing sides of isolated (decoupled) elements.** This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at changes in floor level (Figure 5.2).
- **Using absorptive materials to fill wall and floor cavities** (glass fibre or mineral wall) can reduce airborne sound transmission. The NCC requires absorptive material to be non-combustible.
- **Sealing sound leaks** at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

There are also simple techniques that can be incorporated into the building design that can dramatically improve the sound performance of timber wall and floor/ceiling systems. The following systems provide examples that can be used to enhance sound performance of walls and floors.

Wall Systems

Batten out walls in wet area. In wet area construction, fire/sound rated walls can be compromised where bath and shower base units need to be recessed into the wall. A simple means of achieving this is to batten out the wall (after fire/sound resisting linings have been applied) and then provide an additional lining over the top (Figure 5.1). The bath can then be installed into the batten space without affecting the fire- and sound-rated wall. In such instances, it is best to have at least 35 mm batten space and to place insulation into the cavity. This arrangement also substantially reduces the risk of compartmentation being compromised during refurbishment activities. For example, if the additional lining boards are removed or replaced, the fire-protective covering can be left in place, maintaining the required fire separation.

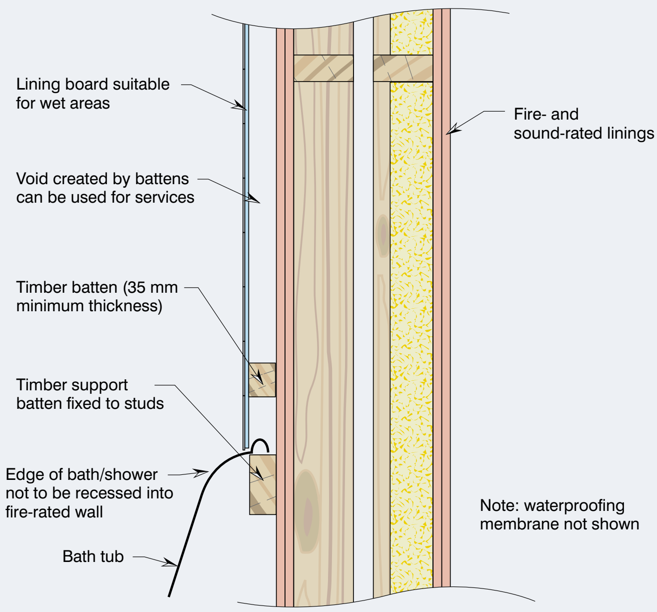


Figure 5.1: Batten detail for wet area walls – elevation view.

Floor Systems

Floor joists parallel to sound rated wall. By running floor joists parallel rather than perpendicular to the sound rated wall, the ability of impact sound from the floor being transferred across the wall to the adjoining SOU is lessened (Figure 5.2).

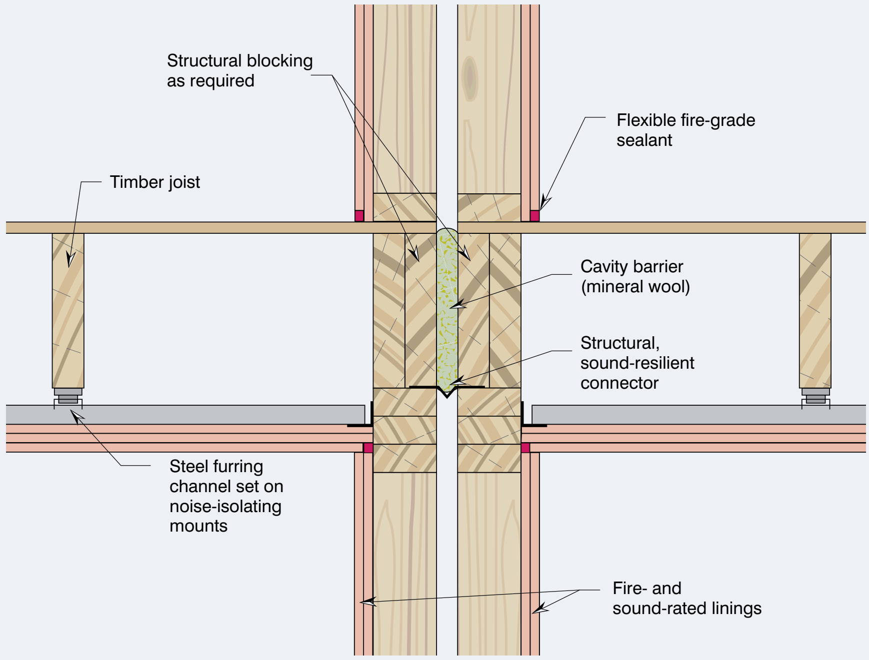


Figure 5.2: Joists running parallel to bounding wall – elevation view.

Upgrade sound-resilient ceiling mounts. Ceiling mounts are commonly used to prevent noise that gets into the floor from coming out through the ceiling below. They help reduce sound transfer between the bottom of the floor joist and the ceiling lining. To improve performance, some ceiling mounts now provide an isolating and damping effect (Figure 5.3). They typically force the sound energy through a rubber component that deforms slightly under load as the sound passes from the joist to ceiling sheet. Therefore, sound-resilient mounts are not all the same and different systems have different performance.

Time spent choosing the right sound-resistant ceiling mount can pay dividends.

Evidence of suitability must be provided to show that the required FRL of a ceiling system can be achieved using the acoustic resiliant mounts.

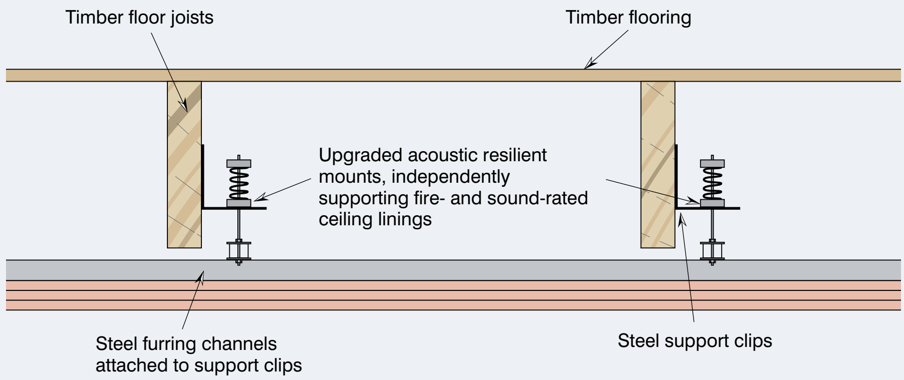


Figure 5.3: Upgraded sound-resilient ceiling mounts – elevation view.

Increase mass of the top layer of floor systems. Increasing the mass of the top surface of the acoustic floor system is one of the best ways to improve acoustic performance. There are three common ways – concrete topping, sand or additional floor sheets.

Quantifying the improvement is difficult as the acoustic performance is aimed at improving the low frequency performance of the floor, a phenomena not measured by tested systems. It is suggested that the base floor system be designed to comply with the NCC's sound requirements, and the additional floor mass provides enhanced performance unless evidence of suitability is available to quantify the improvement.

When height is added to a floor, consideration of the effect this has in other areas (such as wet areas, corridors, stairs, doors and windows) is needed at the planning stage.

Sand used to increase mass in timber floors. This increases the mass of the upper layer of the floor element. The air spaces between the sand particles help reduce the vibration and energy created by impact sound from footfalls. Typically, this is achieved by placing 45 mm battens directly over a normal acoustic floor system at typical 450 or 600 mm centres (dependent on floor sheet spanning capacity). A dry sand layer, or dry sand mixed with sawdust is placed between the battens and levelled just below the surface of the final floor sheet. The final floor sheet is fixed in the normal manner, and desired floor covering placed on this (Figure 5.4).

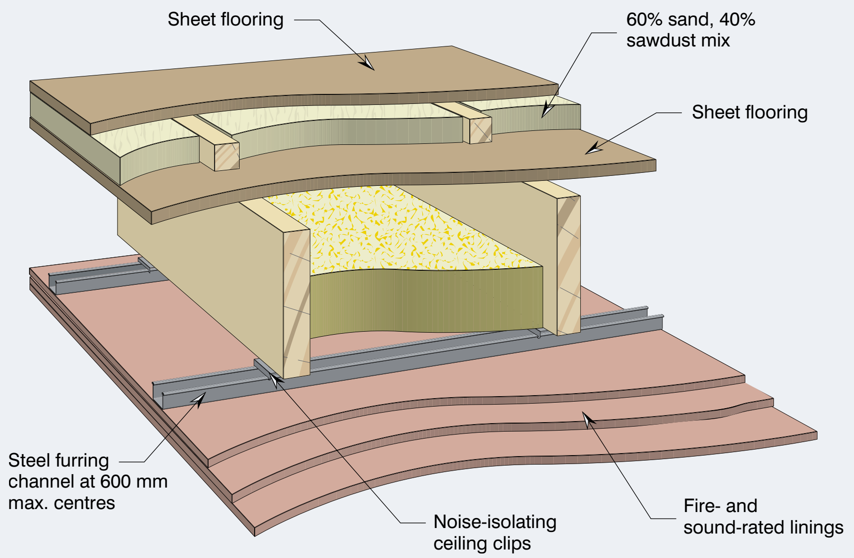


Figure 5.4: Adding mass to floor system through the use of sand top layer.

Concrete topping. This increases the sound performance of the floor system, and typically can be achieved with a 35 to 45 mm thick layer of concrete placed over an isolating acoustic mat. Care is required to turn the isolating acoustic mat up at the perimeter of the topping adjacent to the wall, otherwise the effect of the topping is negated (Figure 5.5).

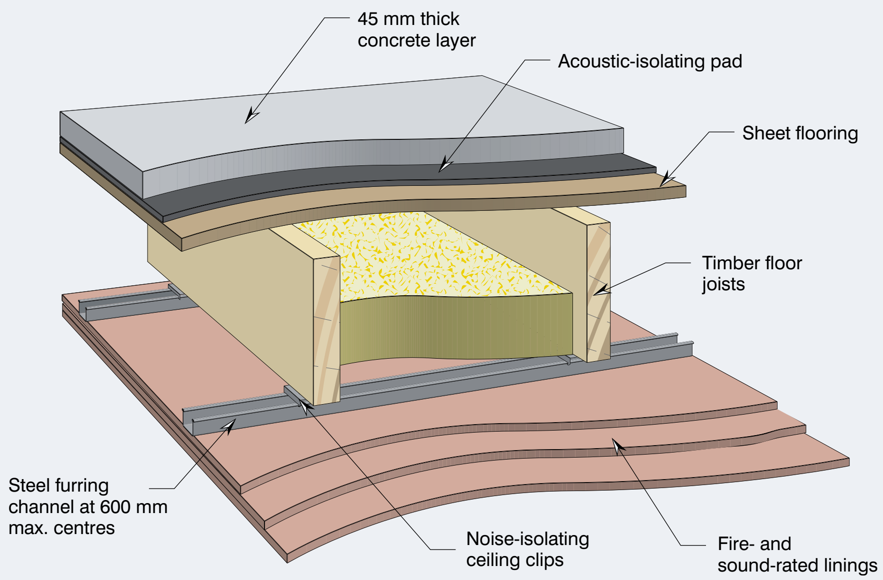


Figure 5.5: Adding mass to floor system through the use of concrete topping.

Extra sheet flooring. This method utilises standard sheet flooring on an isolating mat. This system does not perform as well as the higher mass products, sand or concrete (Figure 5.6).

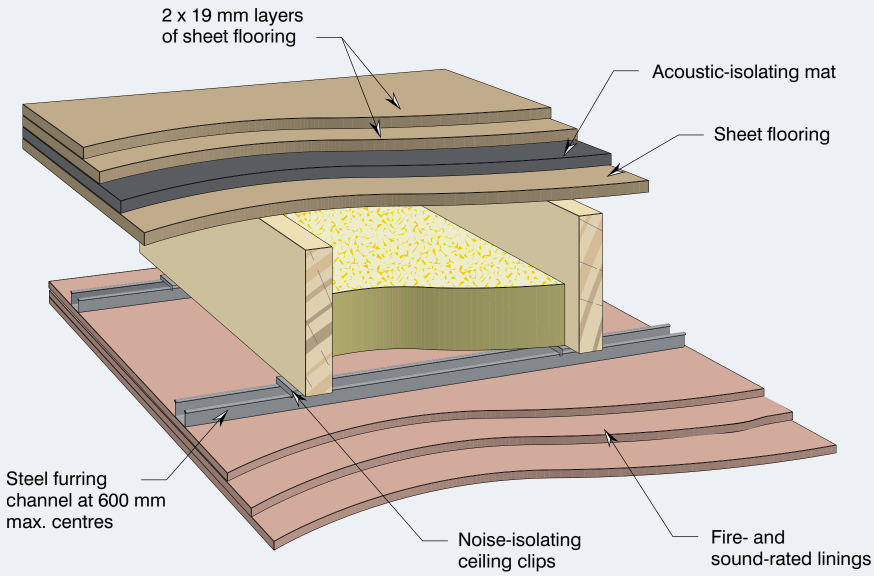


Figure 5.6: Adding mass to floor system through the use of additional floor sheets.

Separate floor and ceiling frame. By having two sets of joists (separate floor and ceiling joists) that are nested between but not touching each other, it is possible to isolate the two structures, thereby minimising the transference of impact sound through the structure. Care must be taken with this approach to prevent flanking noise running along the floor joists and into the walls below. This can be improved by sitting the ceiling joists onto strips of acoustic isolating mat (Figure 5.7).

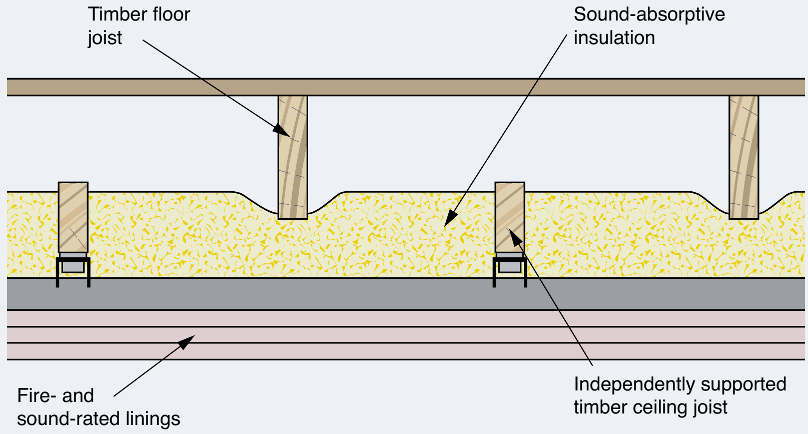


Figure 5.7: Separate ceiling and floor joist structures.

5.2 Establish Architectural Layout

The basic architectural layout of a building is determined by considering many variables; the relative importance of which will vary from project to project. Typically, these include:

- the project brief
- site conditions
- sustainable construction
- aesthetics
- economics
- planning, building and other regulations.

The design should then be refined with input from the various disciplines involved in the design team.

This process is demonstrated for a mid-rise timber office building with basement car parking and ground level retail as shown in Figure 5.8.

A typical office floor plan is shown in Figure 5.9.

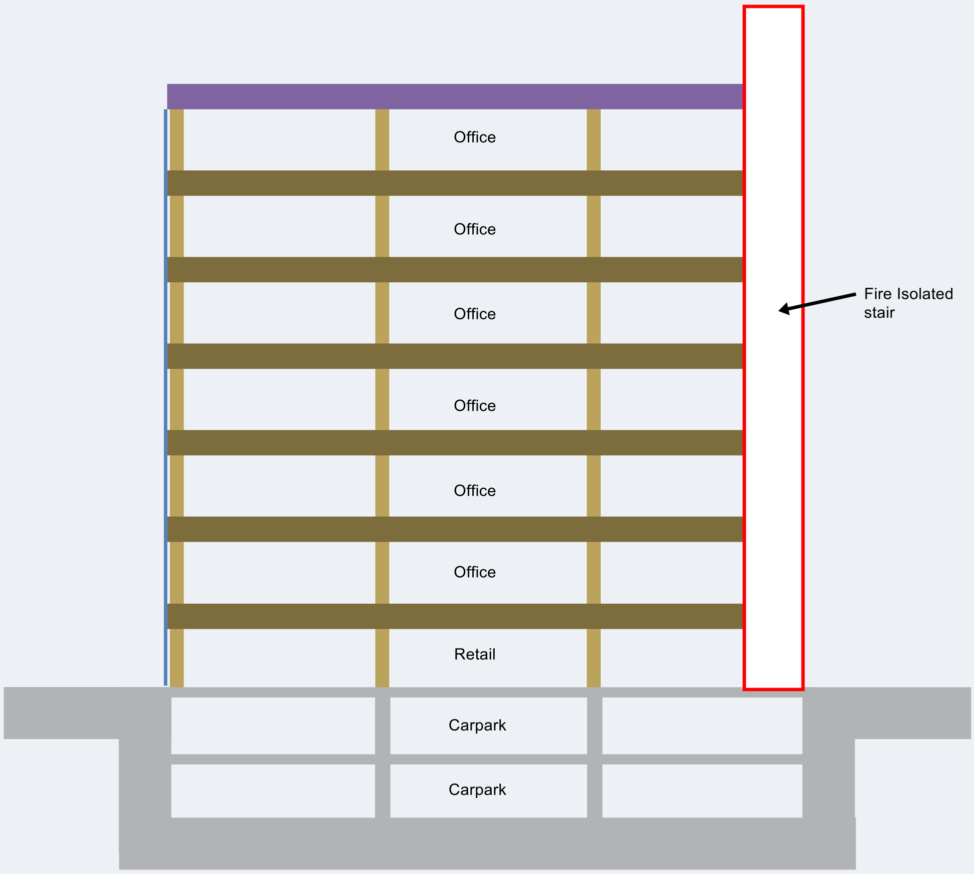


Figure 5.8: Typical schematic section through a mid-rise office building.

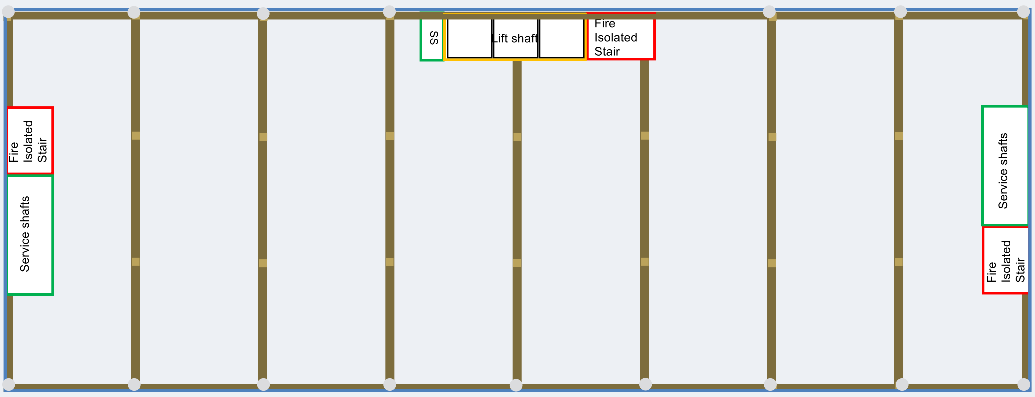


Figure 5.9: Plan of a typical office floor.

5.2.1 Optimising Building Layout - NCC DTS Provisions for Escape

As part of the strategy for evacuation of occupants during a fire emergency, the NCC prescribes DTS provisions for escape in Part D1.

The following is a brief overview of these requirements relevant to the optimisation of the building layout. Reference should be made to part D of the NCC for further details.

Since mid-rise buildings have an effective height less than 25 m for office buildings with a small footprint it is permitted to provide one exit only but in many cases more than one exit is required to satisfy other NCC provisions such as maximum distances of travel to fire exits. In this example three exits are required to satisfy the following NCC DTS requirements :

- no point on a floor is more than 20 m from an exit (entry to fire isolated stair for the example office floor) or a point of travel in different directions, in which case the maximum distance to one of those exits must not exceed 40 m.
- the exits are distributed as uniformly as practicable in positions where unobstructed access to at least two exits is available from all points of the floor
- the exits are not less than 9 m apart and not more than 60 m apart for office buildings
- paths of travel to exits do not converge within 6 m of each other.

The selected configuration shown in Figure 5.10 is consistent with the structural design enabling fire isolated exits to be incorporated into the structural cores and accommodating two independent occupancies on the same floor each with compliant access to two exits without the need to pass through the other occupancy simplifying door hardware requirements although access from the lift lobby to two exits is still required to be provided.

Figure 5.10 Comparison of Layout with Different Maximum Travel Distance to Exits

An open plan area and an area fitted out with individual offices is included in Figure 5.10 to demonstrate the estimation of travel distances which should take into account the location of walkways rather than the straight-line distance from a location on the floor to an exit.

In office buildings where only a single structural core is desired, with careful design, it is possible to locate a fire exit at each end of the structural core whilst complying with the NCC DTS provisions for the provision of escape although once the floor plate exceeds a certain size additional exits will be required as in the example case.

The distance of travel concession facilitates the design of more efficient floor layouts

5.3 Select Structural Form

For this example, the preferred structural material is timber for the Class 5 and 6 parts, which may have been selected for many reasons including:

- lightweight construction (useful if ground conditions are difficult)
- speed of construction
- sustainable construction
- prefabrication of elements.

This does not preclude the use of hybrid forms of construction.

The primary reasons for the selection of reinforced concrete construction for the basement were to address ground water penetration and also as part of the strategy to manage termite risk. In some instances, there may be advantages to extending the concrete construction to the ground floor. Typical examples including;

- where parts of the building ground floor level or close to or below the external ground level the extension of concrete construction may be appropriate to address moisture penetration and manage termite risk
- if part of the carpark was also extended to the ground floor it may be practicable to adopt the same form of construction for all levels that include carparking whilst the concrete trades are onsite.

There are several timber structural forms that can be adopted to suite the preferred architectural layouts. Typically for office and retail spaces more open forms of construction may be preferred in which case post and beam type construction may be preferred over lightweight timber-frame construction. In some cases, a mix of structural forms may be the most practical solution. For example, post and beam timber construction may be selected as the primary structural elements with lightweight timber-frame construction being adopted for pre-fabricated floor cassettes and lightweight timber framing for wall systems carrying lower loads or non-loadbearing wall systems.

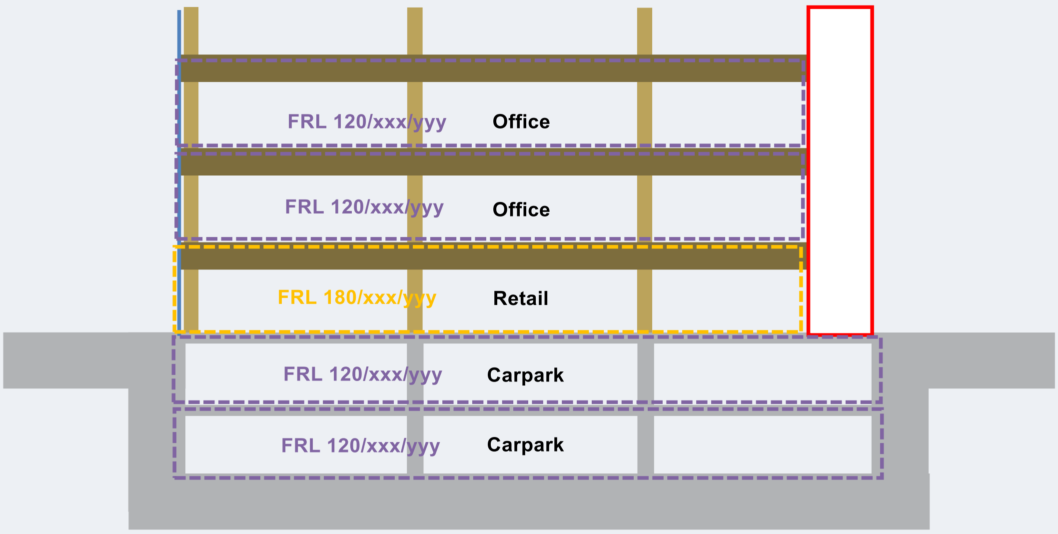
While subsequent sections consider both options, the proprietary nature of massive timber panels, columns and beams manufactured from engineered products such as Laminated Veneer Lumber (LVL) and Cross-Laminated Timber (CLT) limits the number of generic details that can be included in this Guide.

5.3.1 Determine Fire Resistance Levels Required for Structural Elements within Fire Compartments

The Structural Adequacy component of the FRLs required for the various fire compartments should be derived from Specification C1.1 of the NCC based on the required Type of Construction and Building Class for the relevant part of the structure. The distance from the allotment boundary and adjacent buildings also needs to be considered for external walls, external columns and other elements that could be exposed to fire from adjacent buildings. This generally defines the highest FRLs required within a fire compartment with relaxations or concessions being applied to some elements of construction particularly with respect to the integrity or insulation criteria.

For different classifications in different storeys in a Type A building (most mid-rise buildings), the floor between the adjoining parts must have an FRL not less than that prescribed by Specification C1.1 for the lower storey.

Figure 5.11 shows the general required structural adequacy levels for each of the floors of the example building which includes Class 5 (Office), 6 (Retail) and Class 7 (Carpark) parts. Integrity and Insulation criteria depend upon the role of the element of construction. These may require further adjustments to take into account the support of another part (NCC Spec C1.1 Clause 2.2) requirements and potential concessions such as a reduction in the FRL for the office floors (NCC Spec C1.1 Clause 3.3).



Note FRLs may be modified by NCC provisions such as support of another part. Check with the Building Surveyor / Certifier before detailing required systems

Figure 5.11: Example of multi-class building.

5.3.2 Select Basement and Ground Level Structural Forms

The NCC 2019 Deemed-to-Satisfy Provisions allow the use of fire-protected timber in all building Classes or parts of buildings with an effective height not greater than 25 m. However, concrete has been selected for construction below ground level to address ground water penetration and as part of the system providing protection from termites.

The Class 6 and 7 parts should be fire-separated in accordance with Clause C2.9 (or Clause C2.8 if different classes share the same floor) and comply with the Deemed-to-Satisfy solutions for the Class 6 and 7 parts.

The general Structural Adequacy FRL requirements for the carpark levels are 120 minutes and an FRL 120/120/120 fire separation is required by Specification C1.1 between the carpark level and retail level on the ground floor.

From Specification C1.1 the retail (Class 6) ground floor level requires a general Structural Adequacy FRL of 180 minutes. Figures 5.12 and 5.13 show typical options for fire separation between the carpark and retail level and between the retail level and fire isolated stair.

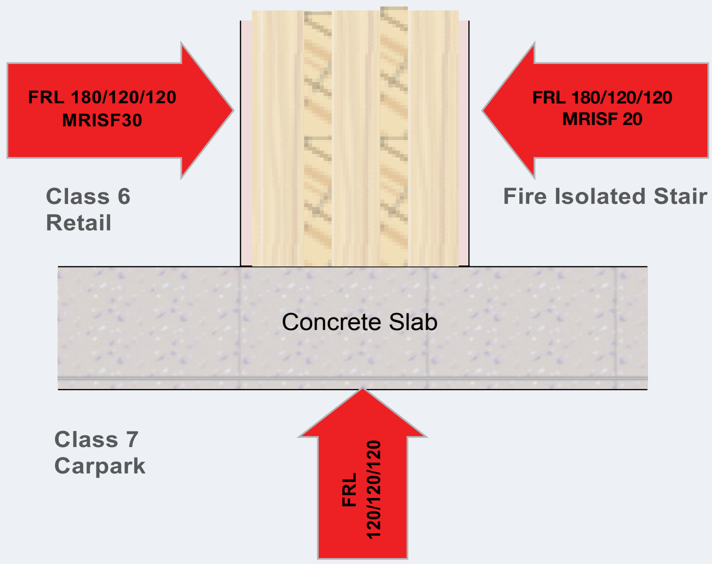


Figure 5.12: Typical FRL requirements for Class 6 and Class 7 parts (Massive timber construction)

Refer RIR 55945800.1B, available from the WoodSolutions website, for the assessment of timber frame wall systems for FRLs up to 180/180/180

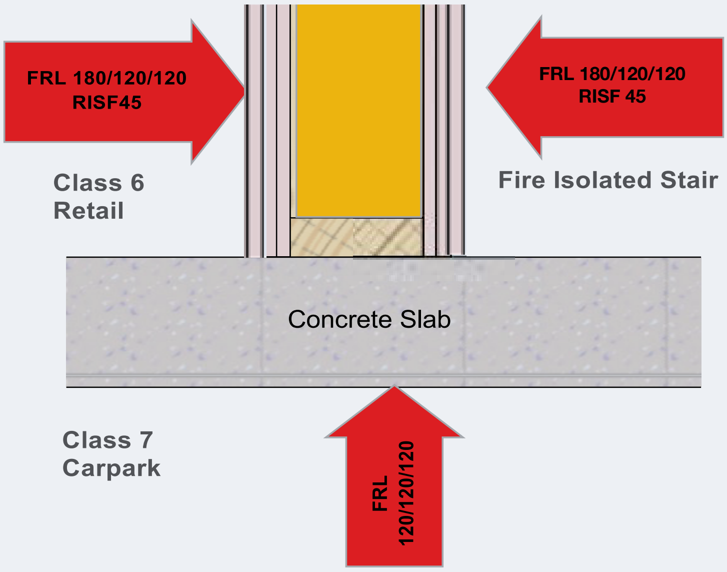


Figure 5.12: Typical FRL requirements for Class 6 and Class 7 parts (Lightweight timber-framed construction)

For the fire isolated stair shafts two options for timber construction have been shown.

For the massive timber option due to the large inherent fire resistance of the massive timber the fire protective coverings are provided to comply with the minimum MRISF requirements of 30 minutes with the outer face exposed to fire and if necessary to supplement the inherent fire resistance performance of the massive timber so that an FRL of at least 180/120/120 can be achieved.

For the timber frame option, substantially greater reliance is placed on the fire protective coverings which greatly exceed the minimum RISF requirements of 45 minutes and supplementary protection within the frame cavities may be required. Further details of a system that has successfully achieved an FRL of 180/180/180 and RISF of 120 minutes is provided in report RIR 55945800.1B and is available on the WoodSolutions website.

5.3.3 Select Fire Separation between Ground (Retail) and Level 1 (Office)

A floor having an FRL of 180/180/180 and an RISF of 45 minutes or an MRISF of 30 minutes is required by the Deemed-to-Satisfy provisions of the NCC for this application.

Due to the high FRL requirement, massive timber forms of construction are the most likely solution because serviceability considerations (deflection limits in service) tend to be the dominant criteria for determining the depth of structural members for floor systems rather than loadbearing capacity. Under these circumstances the load ratios tend to be low further increasing the inherent fire resistance of massive timber members which means that in some instances no, or minimal, increases to the fire protective coverings required to achieve an RISF of 45 minutes for suspended ceiling systems or 30 minutes for direct fix systems (without cavities) are required.

5.3.4 Select Office Level Structural Forms and Fire Resistance Construction

Refer WoodSolutions website, for the referenced floor system reports

All the upper levels in the example building are offices (Class 5) and fire-protected timber construction has been selected as the preferred option using a DTS solution. Whilst a minimum FRL for structural members with respect to the criteria for structural adequacy is 120 minutes, there is a floor loading concession included in Specification C1.1 Clause 3.3 of the NCC that permits a reduction in the required FRLs in Class 5 and 9b buildings for floors to 90/90/90 and roofs to 90/60/30 if the live load on the floor below does not exceed 3kPa.

This can be a significant consideration if lightweight timber-frame flooring systems or parts of the flooring systems use lightweight timber-frame elements since the additional protection may impact significantly on building costs. For example, fire tests of lightweight engineered timber-frame floor system have shown that typically two layers of 16mm fire protective grade plasterboard are required for a floor system to achieve an FRL of 90/90/90 (RIR 37600400) compared to three-layers of 16mm fire protective grade plasterboard if an FRL of 120/120/120 (FAS190034-RIR1.0) is required.

If the floor loading concession is adopted, compliance with the requirements for fire protection for a support of another part (Spec 1.1 Clause 2.2) states, amongst other things, that:

"a part of a building required to have an FRL depends upon direct vertical or lateral support from another part to maintain its FRL, that supporting part, must—

(i) have an FRL not less than that required by other provisions of this Specification; and

(ii) if located within the same fire compartment as the part it supports have an FRL in respect of structural adequacy the greater of that required—

(A) for the supporting part itself; and

(B) for the part it supports; and …"

Therefore, in the example case (with a live load not exceeding 3kPa), if the floor (or elements of a floor system) are likely to provide direct vertical or lateral support to columns / walls, an FRL in respect of structural adequacy of 120 minutes will be required. If post and beam construction is adopted for the structural frame, the primary beams providing the lateral and / or vertical support will be required to achieve an FRL of 120/-/- but secondary members not providing lateral support such as floor cassettes or secondary beams need only achieve FRLs of 90/90/90 or 90/-/- respectively.

In some cases, practical solutions can be developed using the same protection system for all members of a floor system since the larger cross-section of primary beams will have a substantially higher inherent fire resistance than lightweight engineered floor cassettes.

5.3.5 Select Lift and Fire Stair Shaft Construction

Lift and fire stair shafts in mid-rise timber buildings can be of timber, masonry or concrete construction. The choice will depend on the structural design of the building and numerous other factors.

If concrete or masonry shaft construction are adopted, it is important that the detailing can accommodate the possibility of differential movement between the timber structure and masonry/concrete shafts. Further information relating to masonry and concrete shaft construction lies outside the scope of this Guide.

Fire-protected timber shafts can be timber-framed construction or massive timber panel systems. Both options will be considered in subsequent sections.

An independent structural frame can be provided within the shaft as part of the lift installation, effectively isolating the lift system from the shaft walls and providing adequate acoustic separation.

If an independent steel frame is used within a fire-protected timber shaft, the possibility of differential movement between the timber and steel frame will need to be addressed. Typically, this can be addressed if the lift system can be readjusted for differences in floor levels.

5.3.6 Structural Design

Issues that should be considered in the structural design of mid-rise timber buildings include:

- The design must comply with the relevant NCC requirements including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1(a)(iii) (structural robustness). A first principles performance pathway can also be adopted that addresses both fire and structural performance. Further guidance is provided in WoodSolutions Technical Design Guide #39 Robustness in Structures.
- The lighter mass of timber to that of masonry/concrete construction – greater attention needs to be given to resistance against overturning.
- The greater effect from wind loads than expected on smaller structures. This is due to a greater height-to-width ratio, resulting in a need for attention to resistance to overturning.
- Potential for movement (and differential shrinkage in buildings of hybrid construction) in taller timber buildings. Movement can be minimised by:
  - using seasoned timber or engineered timber
  - constructing bearers and joists in the same plane
  - detailing to avoid differential shrinkage between dissimilar materials, e.g. steel to timber; timber to masonry or allowing articulation to absorb the differential movement
  - allowing for differential movement with respect to plumbing and other services.

A professional structural engineer with appropriate skills will be needed to ensure the above issues and structural performance in general are adequately addressed.

The following standards should be called on where appropriate:

- AS 1170.0 – Structural design actions – General Principles.
- AS 1170.1 – Structural design actions – permanent, imposed and other actions provides the basis for determination of appropriate dead, live design loads and loads combinations
- AS 1170.2 – Structural design actions – wind actions – which provides the basis for wind loads.
- AS 1170.4 – Structural design actions – Earthquake actions in Australia – which provides guidance and design procedures for earthquake forces.
- AS 1720.1 – Timber structures – Design methods.
- AS 1720.5 – Timber structures – Nailplated timber roof trusses

In addition:

- Select details that minimise the effects of shrinkage (especially since differential shrinkage may have an adverse impact on the function of fire-resisting wall and floor elements).
- Check walls and columns are capable of supporting multi-storey load paths from above. Enlist internal fire-resisting walls and columns if required.
- Check that any elements supporting loads (including bracing elements) are treated as fire-resisting construction and designed accordingly.

5.4 Establish Service Plant Areas, Service Runs, Risers and Shafts

5.4.1 Service Plant Areas

Service plant rooms are generally located away from public areas, either in basements or on roof tops, depending on the building design.

Clauses C2.12 'Separation of Equipment' and C2.13 'Electricity Supply System' generally require certain types of equipment to be fire separated from the rest of the building by construction having an FRL of 120/120/120 with doorways protected by self-closing fire doors with an FRL not less than -/120/30.

For applications similar to the example office building, the most practical solution may be to locate these service areas in the basement where FRLs of 120/120/120 are required.

5.4.2 Service Runs

In fire-protected mid-rise timber buildings, the timber elements are protected by fire-protective coverings and services tend to be concealed in a similar manner to conventional building designs using service risers, ducts and fitting cabling and pipes behind false wall and ceiling linings.

While the use of cavities within fire-protected timber construction to run cables and pipes can appear to be a simple solution, this choice presents several issues including:

- difficulty in maintaining the RISF or MRISF ratings of the elements at points of service penetrations
- risk of acoustic separation being compromised
- risk of fire protection systems not being correctly installed after modifications or additions to existing installations
- risk of disruption of concealed cavity barriers during modifications or additions.

A more reliable option is to plan the layout of required services and potential future services carefully utilising service risers, service shafts and ducts, and additional (false) linings to conceal services minimising penetrations through fire-protected timber elements as far as practicable. This approach is described in the following Sections.

5.4.3 Service Risers and Horizontal Distribution of Services

Adding/modifying services is simplified if services are protected at each floor level by framing out the opening and using a multi-service penetration system

Services such as electricity, water and telecommunications/data systems are normally distributed between floors through service risers/shafts that are commonly located close to the structural core (lift and stair shafts) as shown in Figure 5.14.

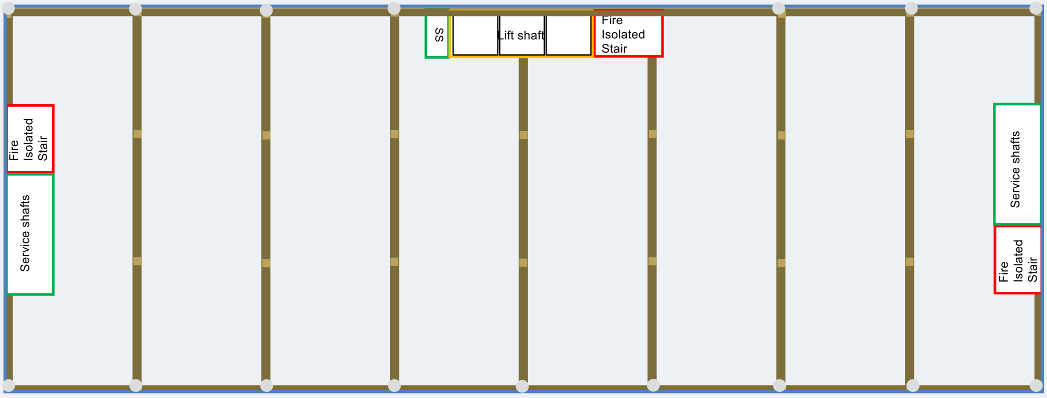


Figure 5.14: Typical position of service risers/shafts in the example office building.

Fire compartmentation can be maintained by protecting the service penetrations at each floor level or using fire-resisting construction for the risers (shafts), fitting fire doors or fire-rated access panels to the risers and fire protecting each service where it penetrates the riser wall.

Generally, the option of protecting the service penetrations at each floor level is the most practical solution. This can be achieved by forming an opening in the floor such that no timber is exposed, the FRL and RISF or MRISF is not compromised and services can be run through the opening. The opening can be protected by a multi-service penetration system such as a pillow, mineral fibre batt or other proprietary fire protection system that can be readily reinstated if additional services need to be run. A typical example is shown in Figure 5.15.

The face of openings cut in fire-protected timber members needs to be protected so that no timber is exposed. The required RISF or MRISF and the FRL of the fire-protected timber must also be maintained. Continuing fire-protective coverings around the opening is a typical solution

Using a framed opening also avoids the need to expose cavities and timber members if additional services need to be run substantially reducing the risk of cavity fires and premature ignition of timber members. Refer to Section 5.14 for further details on the selection of service penetration systems.

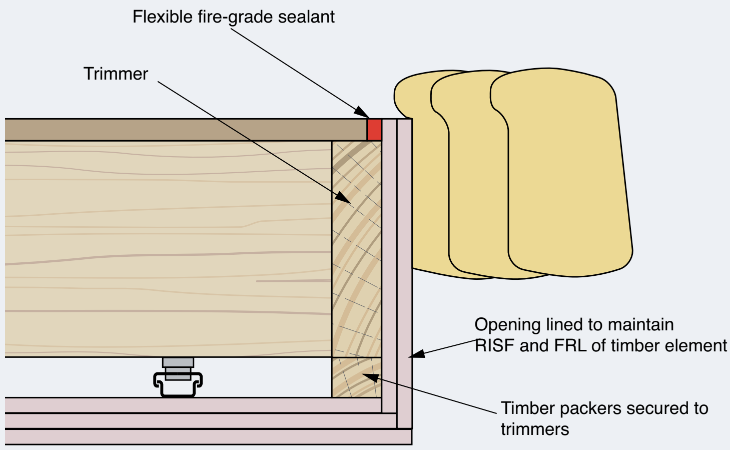


Figure 5.15: Typical riser penetration detail through a fire-protected timber floor.

For horizontal distribution of services, ducts may be created or more commonly the services can be run above a false ceiling fitted below a fire-protected timber floor/ceiling system or below a platform floor. For the office building example, the services have been distributed from the risers above a false ceiling as shown schematically in Figure 5.15 and Figure 5.16.

The use of false ceiling linings to conceal services can provide a number of advantages including a significant improvement in the reliability of fire-protection systems by:

- avoiding large numbers of individual service penetrations within fire-protected timber members for services such as power outlets, lighting, plumbing services including fire sprinklers
- concealing pipework and cable runs
- allowing reconfiguration of services without disrupting fire-protected timber elements
- reducing the risk of cavity fires during maintenance activities and the risk of fire spread to cavities if the fire protection of services is not reinstated after reconfiguration or repair to services
- enabling services to be grouped together and protected by a single multi-service penetration system fitted above the false ceiling (access panels can be provided to facilitate access for inspection and/or adding or modifying existing services).

The use of the false wall and ceiling linings can have additional benefits, such as reducing sound transmission and improving energy efficiency by reducing leakage/flanking paths and providing an additional layer of protection.

Services can be run through internal walls within a fire compartment without the need for protection provided the wall is not required to be of fire-resisting construction. Note: Most loadbearing members are required to achieve an FRL even if they are not part of a bounding wall.

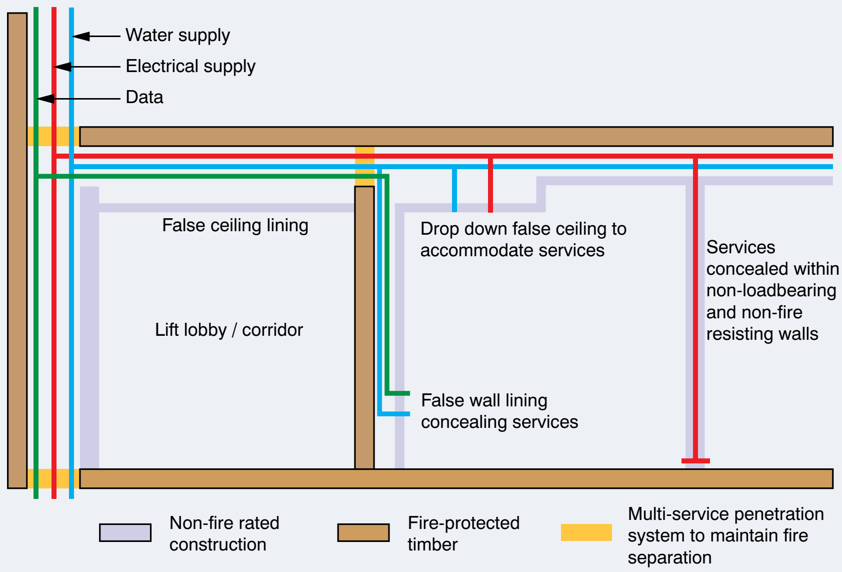


Figure 5.16: Typical distribution of services.

Walls that are not required to achieve an FRL can be a practical option

for the location of services such as power outlets.

5.4.4 Service shafts

Shafts are required to achieve a Fire Resistance Level and it is best to treat the shaft like an independent compartment (Figure 5.17). Care is needed to ensure the sound rating is achieved, as many wall systems are not adequate on their own.

There are advantages (space, economy and design options and reduced risk to critical structural elements) in ensuring that service shafts are non-loadbearing since the integrity and insulation criteria for non-loadbearing service shafts vary from -/90/90 for Class 5, 7a or 9b buildings to -120/120 for Class 6, 7b or 8 buildings. For loadbearing shafts the FRLs vary from 120/90/90 for Class 5, 7a or 9b buildings to 180/120/120 for Class 6 buildings and 240/120/120 for Class 7b and 8 buildings.

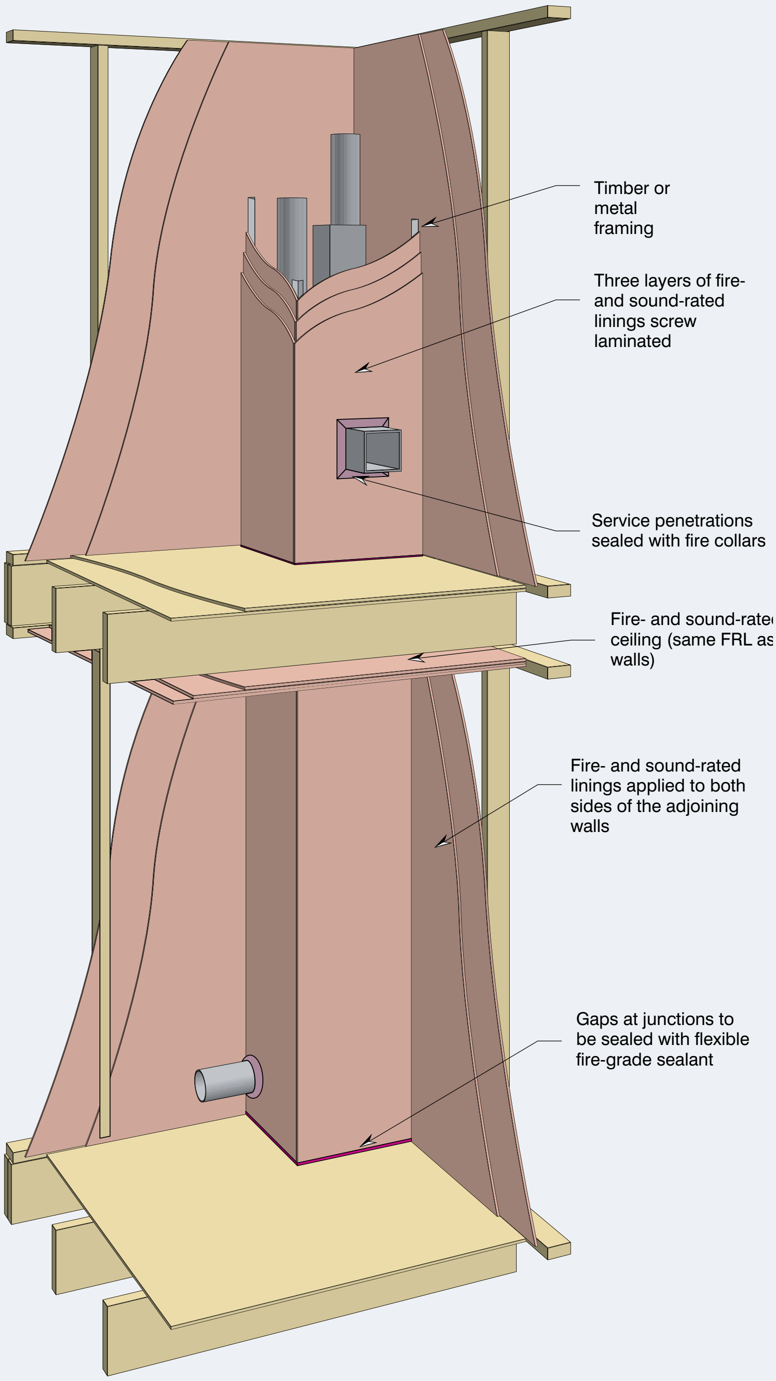


Figure 5.17: Fire-rated service riser/shaft.

Where timber framing is used to support the shaft linings it must be sheeted with fire/sound-grade linings on both sides of the shaft, including the part of the shaft that is the bounding wall of the SOU.

An alternative to using timber framing is to use laminated plasterboard or shaft wall systems. These systems are proprietary, developed by lining manufacturers, and reference to their details is required. Refer to Figures 5.17 and 5.18 for an illustration of a typical fire-grade plasterboard system.

The number of layers, type and thickness of plasterboard (or other fire-protective covering) and fixing methods selected depend on the required FRL and Evidence of Suitability that is available. The non-loadbearing criteria apply and therefore for the example office areas an FRL of –/90/90 is required and for the retail area an FRL of –/120/120.

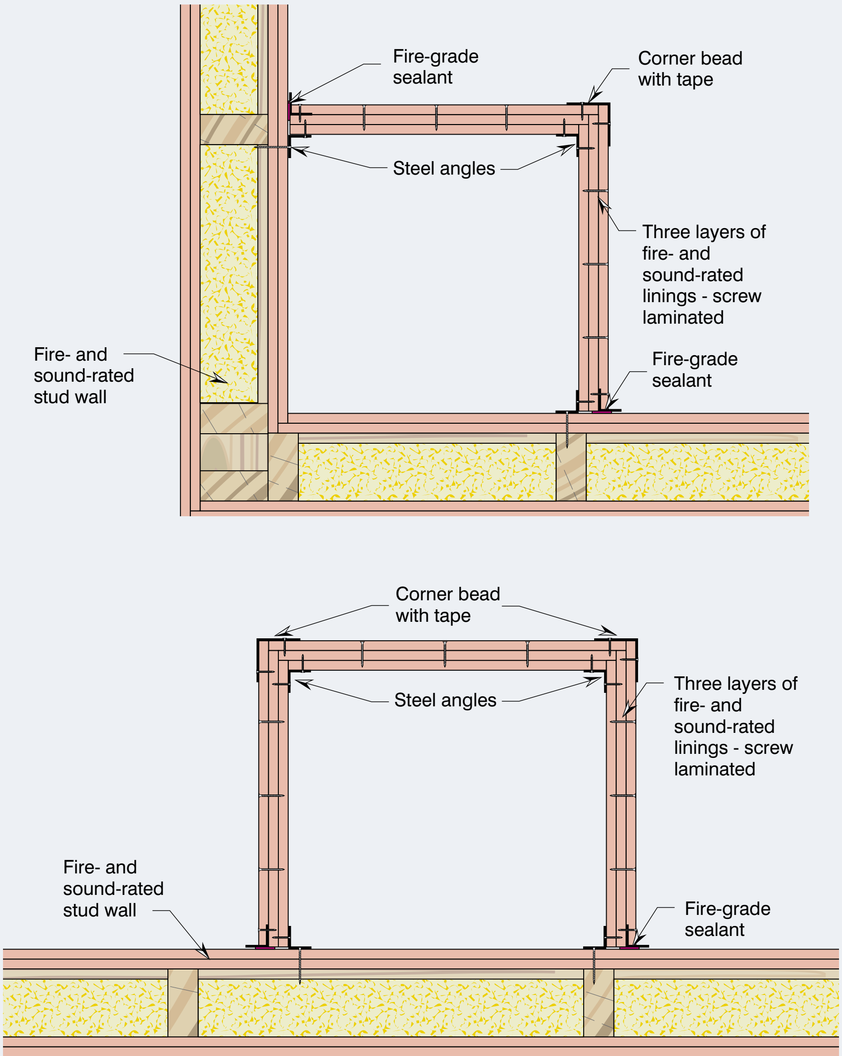


Figure 5.18: Laminated fire-grade plasterboard used to create shafts.

5.5 External Walls

External walls must be designed to satisfy a range of criteria including:

- fire performance
- structural performance (for safety and serviceability)
- weather resistance (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation (the control of transmission of sound from external sources is not required by the NCC but may be part of a design brief or planning control).

5.5.1 Fire Performance of Fire Protected Timber External Walls

The external face of the wall may form the fire-protective covering of a fire-protected timber element, brick veneer construction as shown in Figure 5.19.

If this option is used the specification will need to address the installation of cavity barriers to ensure correct placement and that moisture is not transported from the internal brickwork face to the timber frame through the cavity barrier.

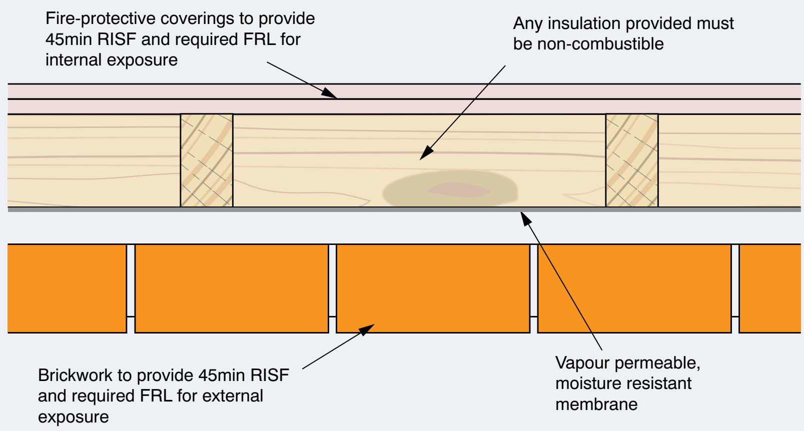


Figure 5.19: Fire-protected timber brick veneer external wall.

Alternatively, a cladding system may be fixed to a fire-protected timber element to prevent water penetration and serve other non-fire related functions. The cladding system could be a direct fix system or ventilated systems as shown schematically in Figures 5.20 and 5.21 for lightweight timber-frame and massive timber construction, respectively. These figures may not show all components that form part of proprietary systems.

Many massive timber panels are proprietary products. Fire (and other) properties depend on the adhesives used and manufacturing processes, which are currently not fully standardised. Evidence of Suitability for massive timber external wall systems will tend to be product specific in most instances and configurations will tend to vary to satisfy the relevant NCC and other design requirements.

Fixings for the cladding system must be detailed so that the performance of the fire-protective coverings is not compromised.

The NCC DTS provisions require the external walls to be of non-combustible construction or of fire protected timber construction. Therefore, any cladding systems applied to fire-protected timber external walls in mid-rise buildings must be non-combustible to comply with the NCC DTS provisions.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers. Further details of the evidence required are provided in Appendix C.

RIR 37401400 available from the WoodSolutions website determines that non-combustible external cladding systems can be fitted to fire-protected timber walls without compromising the FRL, RISF or MRISF.

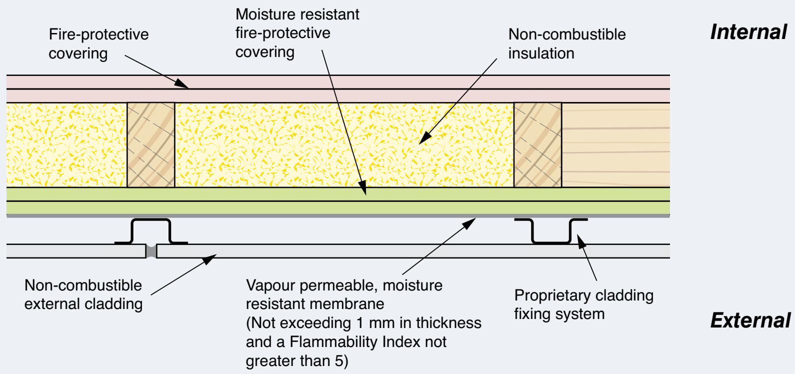


Figure 5.20: Fire-protected timber frame external walls with lightweight cladding.

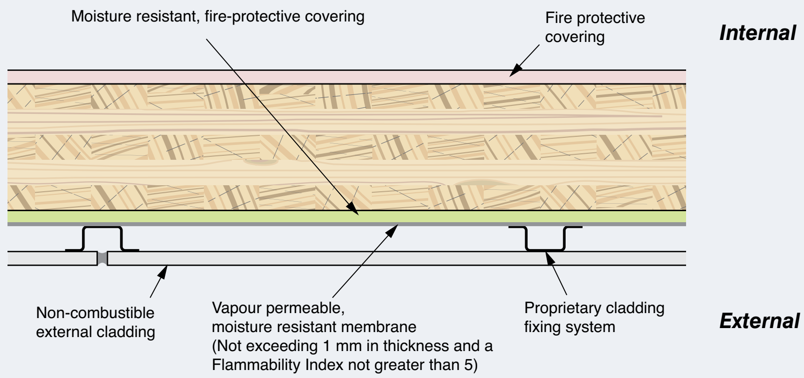


Figure 5.21: Fire-protected massive timber external wall with external lightweight cladding.

If combustible cladding systems are to be used, the Performance Solution pathway must be adopted to demonstrate compliance of the external wall system with the relevant NCC performance requirements. Verification method CV3, in conjunction with verification methods CV1 and CV2, and the classification standard AS 5113 define an appropriate method for demonstrating compliance in most States and Territories.

Table 5.1 summarises the required FRLs and RISF or MRISF based on the distance from the boundary for the example Class 5 Office Building of Type A construction - refer NCC Specification C1.1, Table 3 for Class 6, 7, 8 and 9b FRL requirements.

While there are significant reductions in the required FRLs for non-loadbearing elements as the distance from the fire source feature increases, the design of the external walls will not vary significantly because the required RISF or the MRISF, in combination with the minimum thickness requirement of 75 mm for massive timber, will become the dominant design factors.

If the subject building is of massive timber construction and is not more than 1 metre from a fire source feature, the required MRISF is increased to 45 minutes externally to minimise the risk of fire spread from adjacent structures.

Table 5.1: Fire-resistance requirements for external walls for a Class 5 building of Type A construction.

Distance from fire source feature	**FRL – Structural Adequacy /Integrity/ Insulation – minutes**		**General Timber**	**Massive Timber**
	Loadbearing	Non-Loadbearing	RISF (minutes)	MRISF (minutes)
≤1.0 m	120/120/120	-/120/120	45	45 external 30 internal
<1.5 m	120/120/120	-/120/120	45	30
≥1.5 and <3 m	120/90/90	-/90/90	45	30
≥3 m	120/60/30	-/-/-	45	30
External Columns	120/-/-	-/-/-	45	30

Table 5.1 indicates that an FRL of -/-/- is applicable to an external wall system that is non-loadbearing and located more than 3m from a fire source feature. In these applications a common solution for office buildings is to use a glazed curtain walling system secured to floor plates Any openings between the floor plate and the curtain walling systems need to be protected to maintain the same FRL as that required for the floor system. These perimeter seals are sometimes also referred to as cavity barriers, but the performance levels and Deemed-to-Satisfy solutions provided in Specification C1.13 for cavity barriers within timber frame construction should not be used since amongst other things higher FRLs are required at the perimeter of a floor plate. For the example office building the opening between the edge of the floor plate and curtain walling system would need to be protected by a system capable of achieving an FRL of -90/90 or -120/120 depending on weather the floor loading concession is adopted.

5.5.2 External Noise

Currently, there are no NCC requirements to provide external noise attenuation for buildings. However, Government authorities have regulatory or legislative powers to require control of noise entering buildings and market forces may generate requirements.

The *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise* provides examples of lightweight external wall systems that can be used as guidance.

5.5.3 Weatherproofing

There are currently no Deemed-to-Satisfy Provisions in the NCC in relation to the weatherproofing of external walls and therefore suppliers of waterproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). A weatherproofing Verification Method (FV1.1) is described in the NCC to enable compliance with FP1.4 via a tested prototype. It is important that installed waterproofing membranes/systems for timber construction are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate through to the structural timber building elements (moisture resistant).

5.6 Fire-protected Timber Floors

Floor systems must be designed to satisfy a range of criteria including:

- structural performance (for safety and serviceability)
- fire performance
- acoustic separation
- durability.

Common structural elements used for timber floors include:

- solid timber beams
- LVL beams
- I-section beams with OSB or plywood webs
- parallel chord steel web trusses
- parallel chord timber web truss
- I-section with Steel Web
- massive timber panel systems (e.g. CLT or LVL).

These structural members can be used with a range of flooring systems, internal insulation systems and soffit/ceiling lining systems in keeping with the building finishes and to achieve the required fire and acoustic performance.

5.6.1 Fire Performance of Flooring Systems Protected by Typical Ceiling Systems

Typical floor systems that may satisfy the fire related NCC Deemed-to-Satisfy fire requirements for fire-protected timber in the example Class 5 building are shown in Figures 5.22 to 5.27 for FRLs of 90/90/90 and 120/120/120. The required FRL should be determined having regard for the Floor Loading Concession (NCC Spec C1.1 Clause 3.3) when considering Class 5 and 9b buildings.

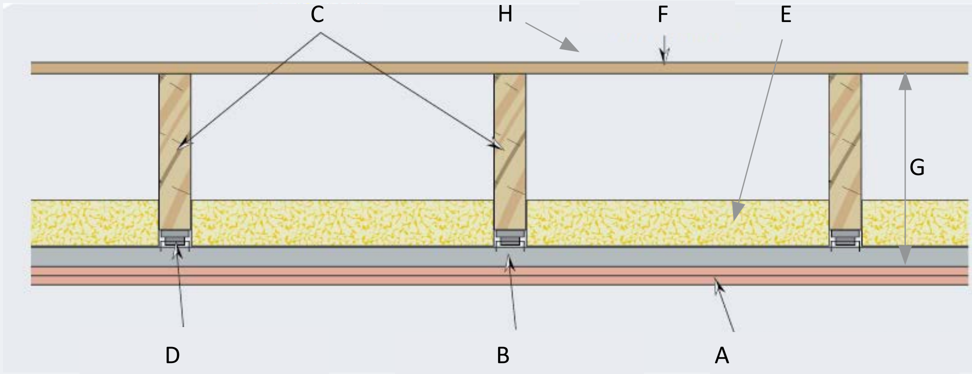
These systems incorporate a ceiling system comprising two or three layers of 16 mm thick fire-protective grade plasterboard secured to steel furring channels supported from the structural element. Since the ceiling provides the largest contribution to the FRL of the floor/ceiling systems, and the performance will be largely independent of the structural members prior to structural failure, the results can be applied to a large range of combinations of structural element, cavity insulation and flooring systems to which additional finishes may be applied, provided compliance with other all NCC requirements is not compromised.

For timber-framed floor systems the required thickness of the fire-protective coverings tends to be dominated by the FRL criteria rather than the RISF criteria of 45 minutes.

For massive timber panel floor systems and beams, where thicknesses of 150 mm or more may be required to achieve adequate structural performance, the required thickness of fire-protective coverings will tend to be dominated by the MRISF or RISF criteria or acoustic considerations rather than FRL criteria because of the high inherent fire resistance of the massive timber panels.

In many situations, fire-protective coverings based on a single layer of 16 mm fire-protective grade plasterboard may be suitable for massive timber panel floor systems. However, the fire performance of these systems depends on many factors, including adhesives, number of layers and thicknesses of lamella, manufacturing process, timber species and grade, and applied loads. These factors are not fully standardised and vary between manufacturers therefore Evidence of Suitability in the form of fire-resistance test reports from Accredited Testing Laboratories should be sought from the suppliers of the specific CLT system to confirm the FRL of fire-protected timber members.

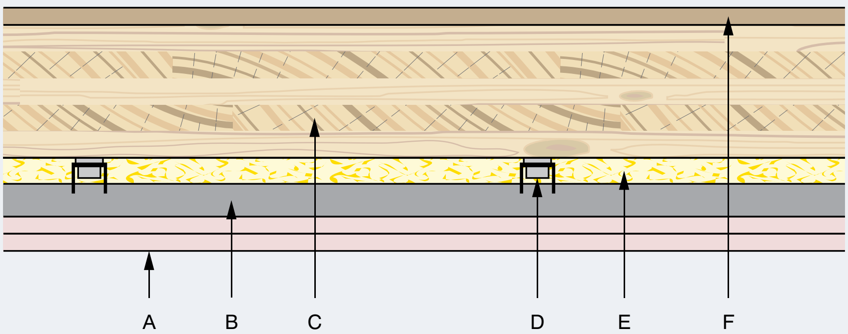
RIR 37401400 can be found on the WoodSolutions website that assesses the FRL and RISF of the floor system shown in Figures 5.24 and 5.25



Construction
A Linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier and/or report RIR 37401400)
RISF 60 mins (verify with supplier and/or report RIR 37401400) or RISF 45 mins (NCC Spec C1.13a DTS)

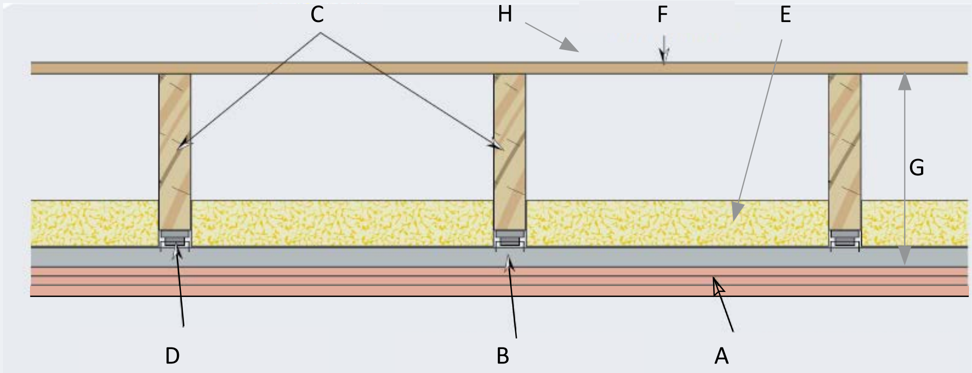
Figure 5.22: Typical timber-framed floor.



Construction
A Ceiling linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Massive Timber Panel (min 75 mm thick)
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Cavity minimum 40 mm (must be filled with non combustible insulation if massive timber 'concession' applied otherwise may be unfilled)
F Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (***verify with supplier***). Note higher FRLs may be achieved depending upon massive timber element design
MRISF/RISF 60 mins (verify with supplier) or MRISF/RISF> 45 mins (NCC Spec C1.13a DTS)

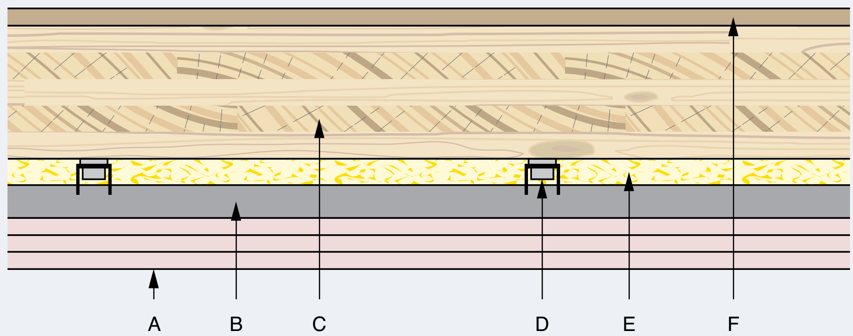
Figure 5.23: Typical massive timber panel floor.



Construction
A Linings 3 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 120/120/1200 (verify with supplier and/or WF report FAS190034-RIR 1.0)
RISF 120 mins (verify with supplier and/or WF report FAS190034-RIR 1.0) or RISF 45 mins (NCC Spec C1.13a DTS)

Figure 5.24: Typical timber-framed floor.



Construction
A Ceiling linings 3 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Massive Timber Panel (min 75 mm thick)
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Cavity minimum 40 mm (must be filled with non combustible insulation if massive timber 'concession' applied otherwise may be unfilled)
F Floor finishes such as carpet, acoustic treatments may be added

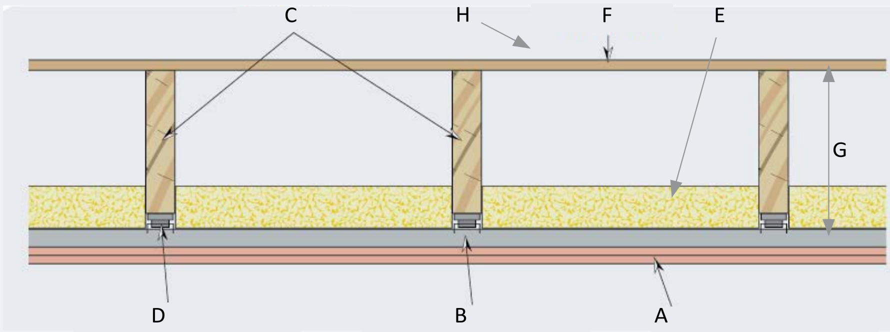
Performance
FRL 120/120/120 (verify with supplierand/or WF report FAS190034-RIR 1.0) Note higher FRLs may be achieved depending upon massive timber element design
MRISF/RISF 120 mins (verify with supplier and/or WF report FAS190034-RIR 1.0) or MRISF/RISF> 45 mins (NCC Spec C1.13a DTS)

Figure 5.25: Typical massive timber panel floor.

5.6.2 Sound

The sound performance of a floor/ceiling system depends on a number of elements including: the density of the floor covering (tile, timber, carpet), isolation from the structure (acoustic underlay), ceiling insulation (density), ceiling installation (acoustic mounts) and layers and thicknesses of ceiling plasterboard. The objective is to minimise both airborne (R_w+C_{tr}) and impact sound ($L_{n,w}$) transmission through the floor/ceiling system and the performance of the floor/ceiling system should be verified with the plasterboard supplier for DTS Solutions.

There are a range of flooring products (e.g. timber overlay, carpet) that can be used and achieve the minimum NCC acoustic requirements. The use of a hard flooring surface will influence the impact performance ($L_{n,w}$) of the floor/ceiling system. As acoustic performance is not an NCC mandatory requirement for Class 5, 6, 7, 8, or 9b buildings, the following is provided for guidance.



Construction
A Linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)

E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm thick
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – thickness 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier)[xxv]
RISF 60 mins (verify with supplier)
or RISF 45 mins (NCC Spec C1.13a DTS)
Acoustics (Verify with supplier):
Above system incorporates: 10 mm overlay solid strip flooring, 4.5 mm acoustic underlay on top of flooring products (F), R2.5 ceiling insulation (E), Acoustic (resilient) mounts (D).
R_w+C_{tr} ≥ 50
$L_{n,w}$ ≤ 62 (bare floor)
$L_{n,w}$ ≤ 50 (carpet and underlay)

Figure 5.26: Sound performance of typical timber-framed floor/ceiling system.

The acoustic performance of various tested CLT floor/ceiling system configurations can be found in the *WoodSolutions Technical Design Guide #44 CLT Acoustic Performance.*

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers.

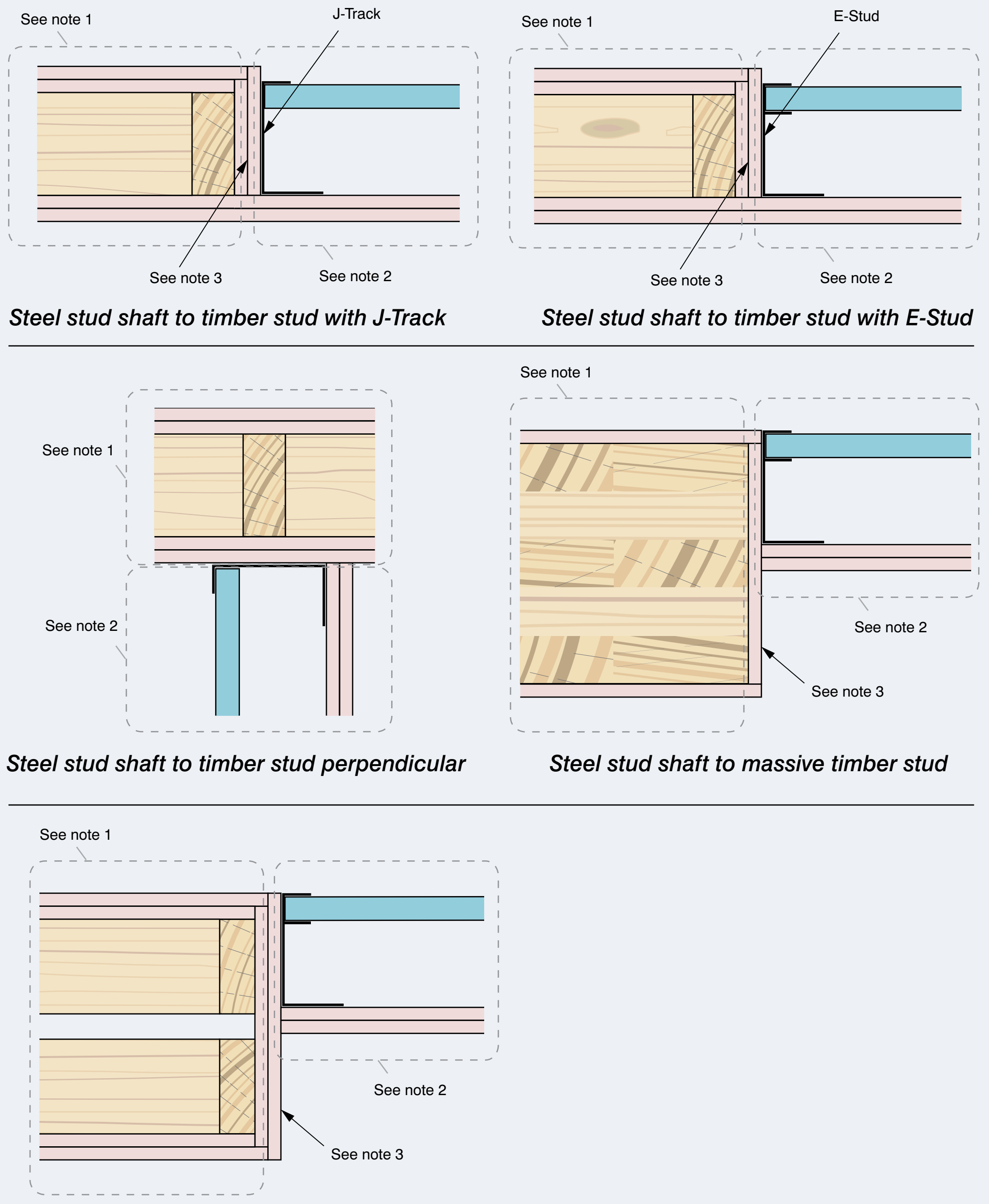
5.8 Service Shafts

While service shafts can be constructed from fire-protected timber walls, in many instances there are substantial advantages in using either steel stud shaft wall or laminated shaft wall construction particularly if the shafts are in locations where sound transmission is not a significant consideration.

The advantages include:

- ease of construction
- smaller footprint (more usable space)
- simplification of treatment of service penetrations
- greater selection of proprietary fire protection systems for service penetrations that already have Evidence of Suitability to demonstrate the FRLs of the systems.

Report RIR 37401400 available from the WoodSolutions website assesses the impact of the interface details in Figure 5.27 on the FRL, RISF and MRISF of the systems



Steel stud shaft to timber stud with J-Track

Steel stud shaft to timber stud with E-Stud

Steel stud shaft to timber stud perpendicular

Steel stud shaft to massive timber stud

Steel stud shaft to massive timber panel

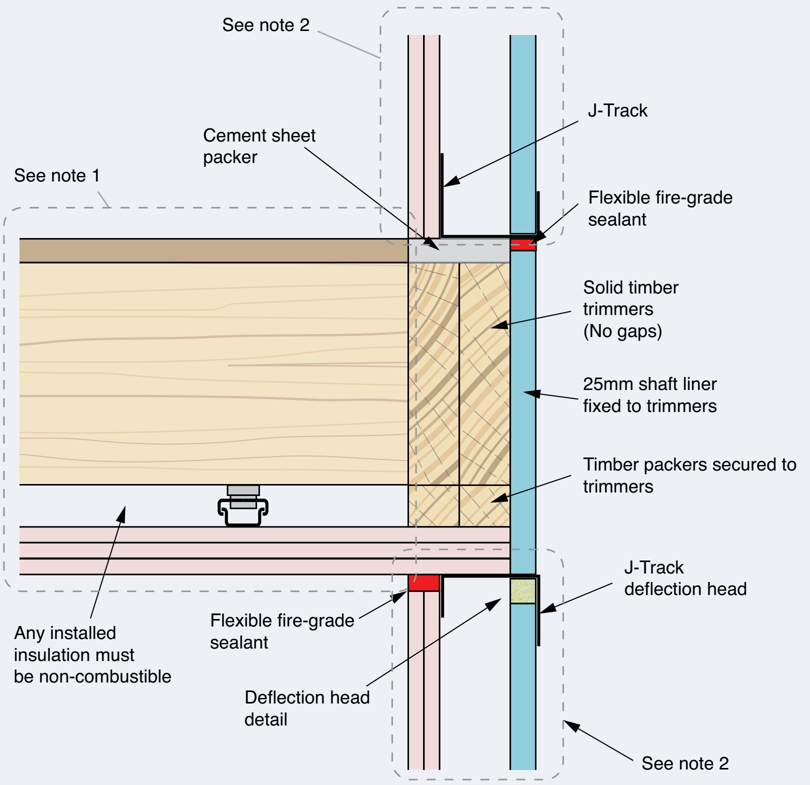
Figure 5.27: Interfaces between fire-protected timber and steel stud shafts (continued next page)

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

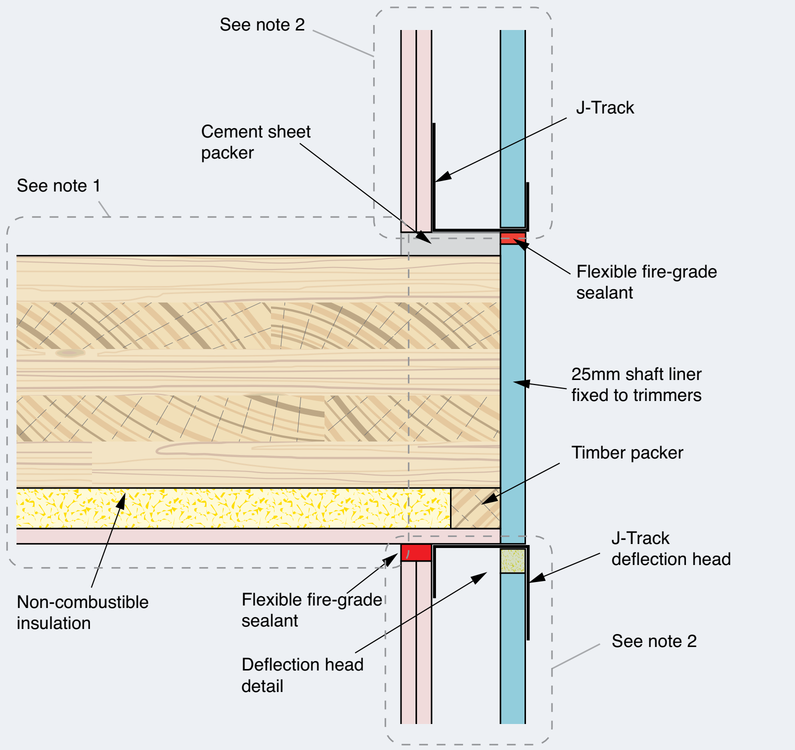
Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws.

EWFA RIR 37401400 available from the WoodSolutions website assesses the impact of the interface details in Figure 5.28 on the FRL RISF and MRISF of the systems



Shaft continuity at timber framed floor



Shaft continuity at massive timber panel floor

Figure 5.28: Interfaces between fire-protected timber and steel stud shafts (continued from previous page)

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

It is important that the fire performance is not compromised at the interfaces between the shaft and fire-protected timber walls and floors. Figures 5.27 and 5.28 shows typical interface details for steel framed shaft construction and Figure 5.29 shows typical interface details for laminated shaft construction. These interface details have been assessed by an Accredited Testing Laboratory (EWFA report reference RIR 37401400) and found not to compromise the performance of the wall or shaft systems.

RIR 37401400 assesses the impact of the interface details on the FRL of the systems

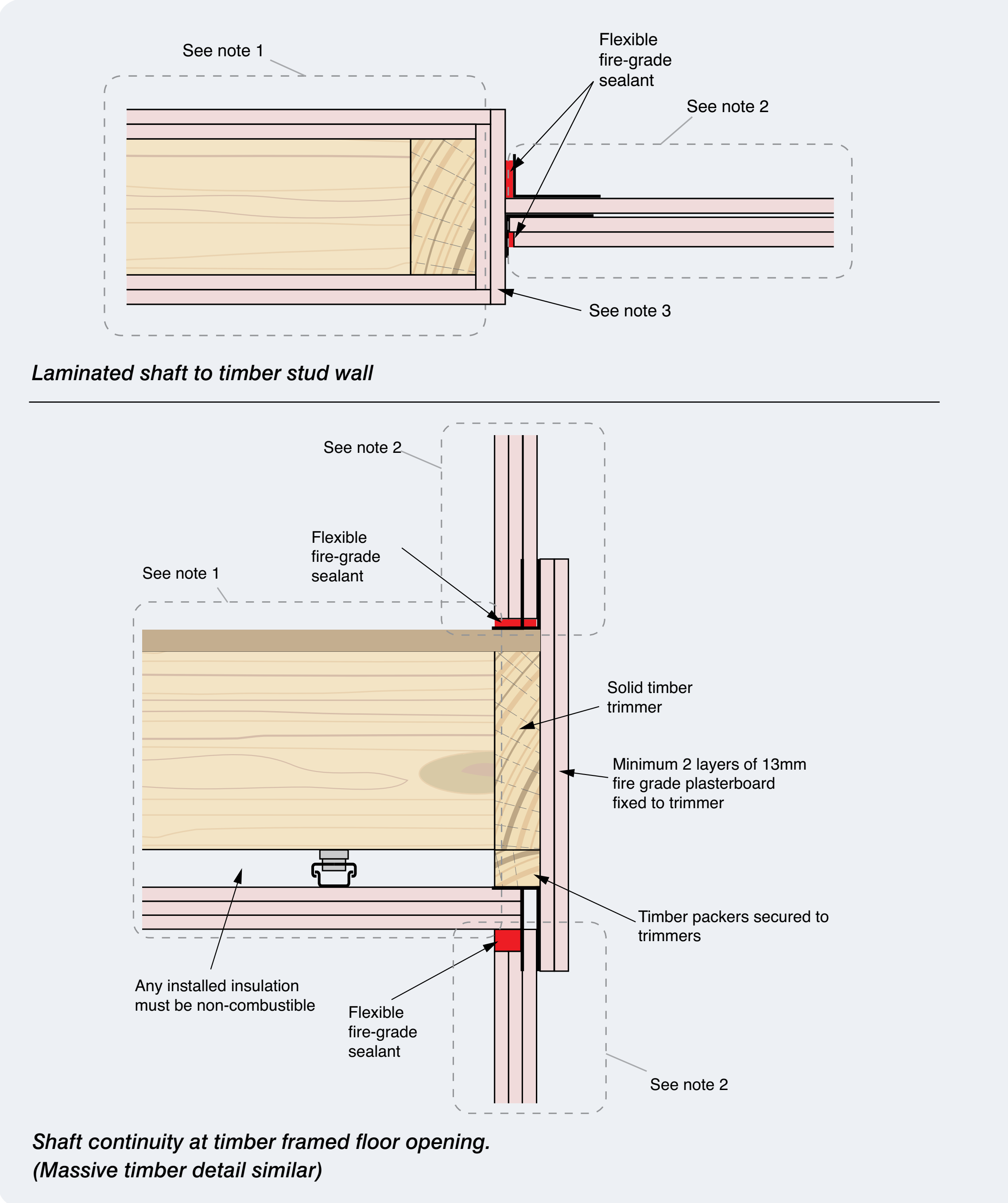


Figure 5.29: Interfaces between fire-protected timber and laminated board shafts.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws.

5.9 Fire Doors in Fire-protected Timber Walls

Fire door assemblies are required to comply with AS 1905.1 as appropriate in addition to achieving the required FRL. Generally, fire doors are required to be tested when mounted in a wall of representative construction. Evidence of Suitability should therefore be provided from the supplier that relates to the performance of their fire doors when mounted in representative timber elements of construction.

In addition, the fire doors must not compromise the RISF or MRISF performance of the wall. The frame fixing details shown in Figure 5.30 have been assessed by an Accredited Testing Laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber-frame systems and 30 minutes for the massive timber panel systems (Refer report reference RIR 37401400). Other details may be adopted if appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements for fire doors and fire-protected timber elements is provided.

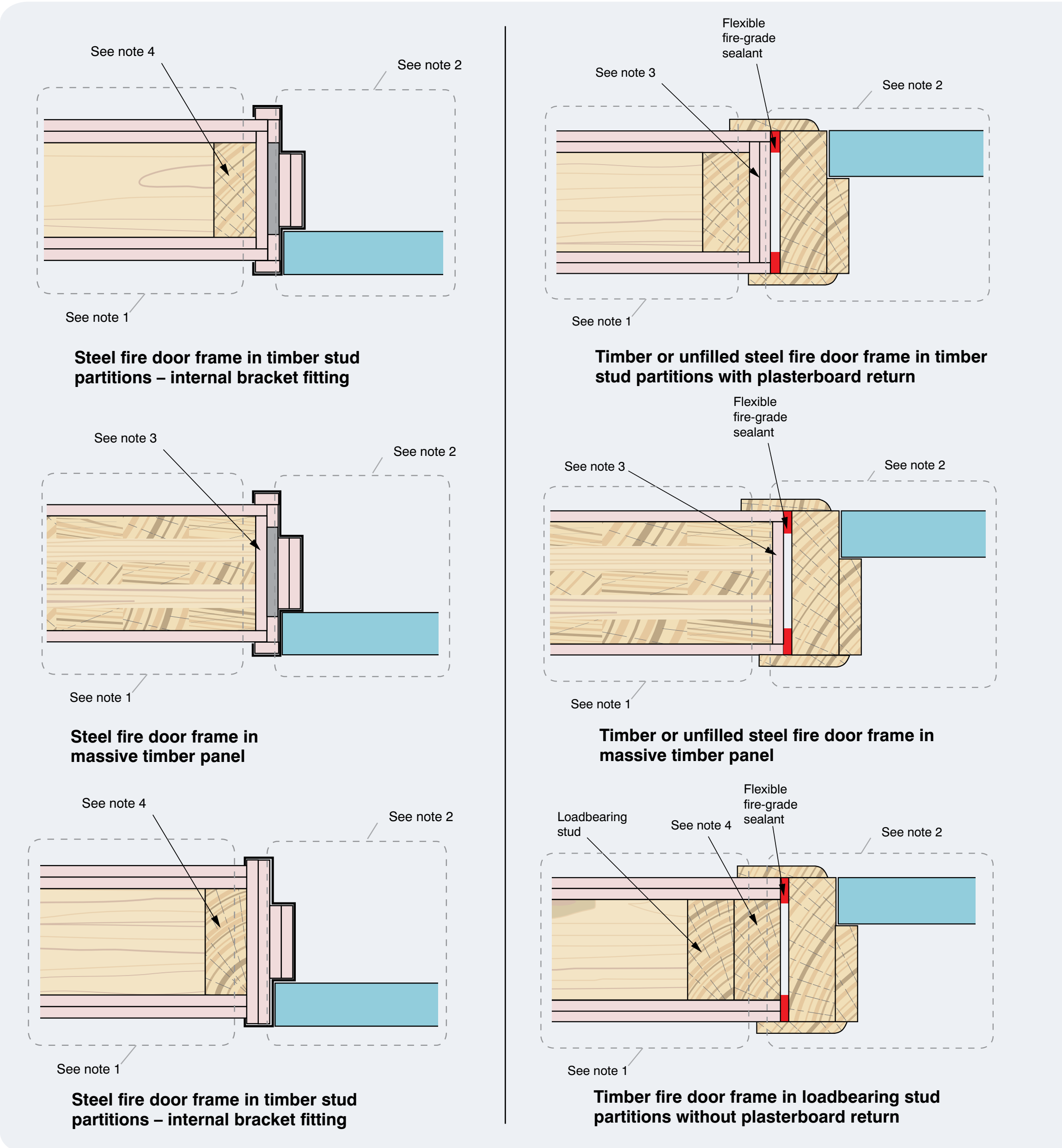


Figure 5.30: Fire door interface details.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Fire Door Assembly with the required FRL determined in accordance with AS 1530.4 and AS 1905.1 as appropriate.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Minimum of 45 mm thick non-loadbearing solid timber cavity barrier framing the cavity opening around the door.

5.10 Construction for Fire-Isolated Stair Shafts

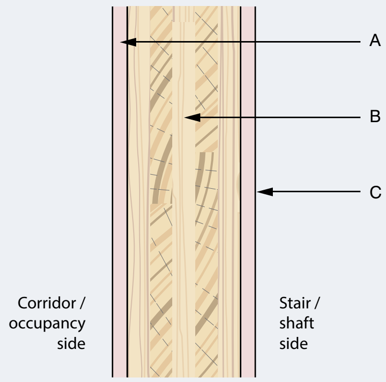
Fire-isolated stair shafts can be constructed from fire-protected timber, concrete masonry and other non-combustible non-loadbearing materials or a hybrid construction may be adopted. The selection will depend on the structural design of the building, construction programming, and other factors.

Where concrete or masonry shafts are used the design will need to account for differential movement between the shaft and timber structure.

It should be noted that whilst the integrity and insulation FRL components are 120 minutes for the commercial buildings considered in this Guide the Structural Adequacy component varies with the Class of building from 120 minutes (e.g. office buildings) to 240 minutes.

A further concession is provided for massive timber panels in that the fire-protective covering for the internal face of the shaft is permitted to achieve a MRISF of 20 minutes compared to the 30 minutes required for the outer face as shown in Figure 5.31 and Figure 5.32. The fire protective coverings around the face of openings for doors and access panels should be the greater of that required for the external facing or to achieve the required FRL.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers



Construction
A Linings min 1 x 16 mm fire-protective grade plasterboard (corridor/occupancy side)
B Massive timber min 75 mm thick ***(note greater thickness may be required to achieve required FRL unless additional fire protective coverings applied)***
C Linings min 1 x 13 mm fire-protective grade plasterboard (internal shaft side)

Performance
FRL 120/120/120(office areas) 180/120/120 (retail areas) obtain Evidence of Suitability from supplier
MRISF 30 mins (1 x 16 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS
MRISF 20 mins (1 x 13 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS

Figure 5.31: Typical stair and lift shaft construction for single skin massive timber panel construction.

Although a minimum panel thickness of 75 mm is permitted, in most instances substantially greater thicknesses will be required as part of the structural design and/or to achieve the required FRLs. The FRL should be checked to ensure the load levels during the test were comparable to the loads that will be applied under fire conditions.

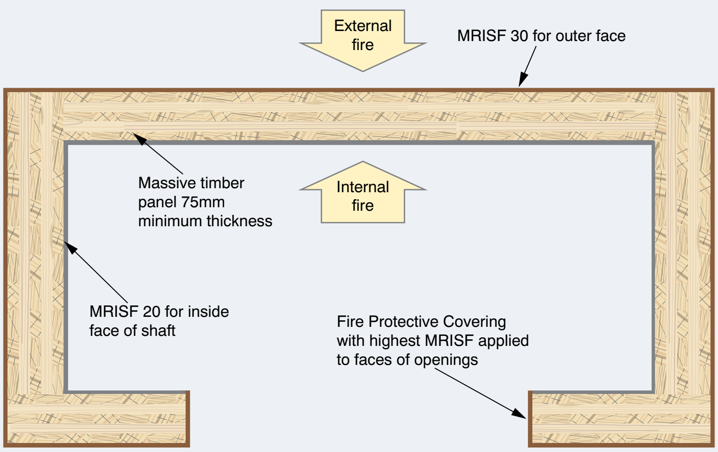


Figure 5.32: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

5.11 Construction for Stairways within Fire-isolated Stairs

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway provided the timber used:

- has a finished thickness of not less than 44 mm with an average timber density of not less than 800 kg/m^3 (at 12% moisture content).
- the building is protected throughout by a sprinkler system complying with Specification E1.5 (other than a FPAA101D or FPAA101H system) that is extended to provide coverage within the fire-isolated enclosure
- the underside of flights of stairs directly above landings providing access to ground level or car parking levels being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres (Figure 5.33).

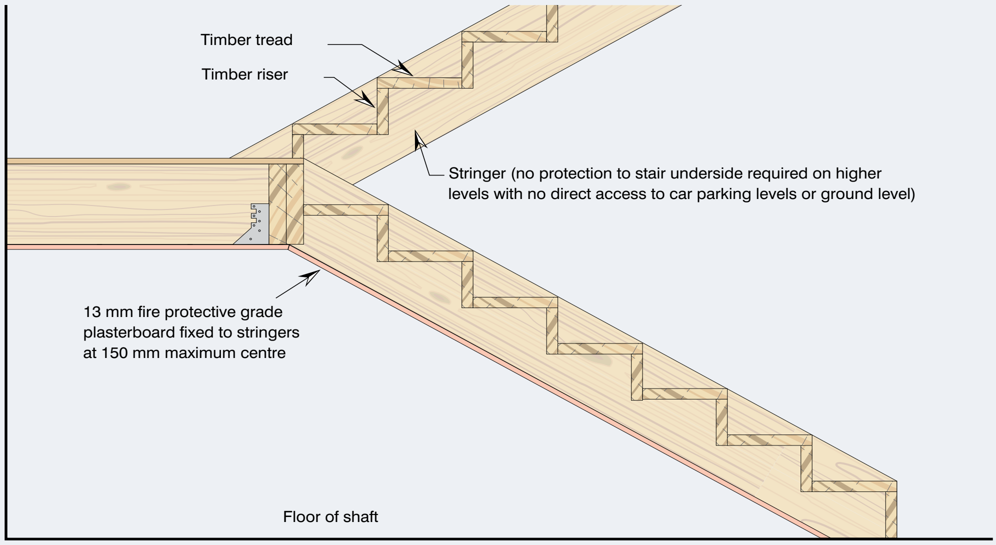


Figure 5.33: Stairway fire protection.

Refer to Section 5.15 for further information about sprinkler installations.

Impact sound from stair usage may vibrate the stair shaft walls, creating a pathway for sound transmission. A practical way to prevent this is by isolating the support for the stair structure by using stringers to support the stairs (top and bottom) rather than the wall adjoining areas requiring sound isolation (Figure 5.34). In some instances, newel posts to support the stringers may be necessary.

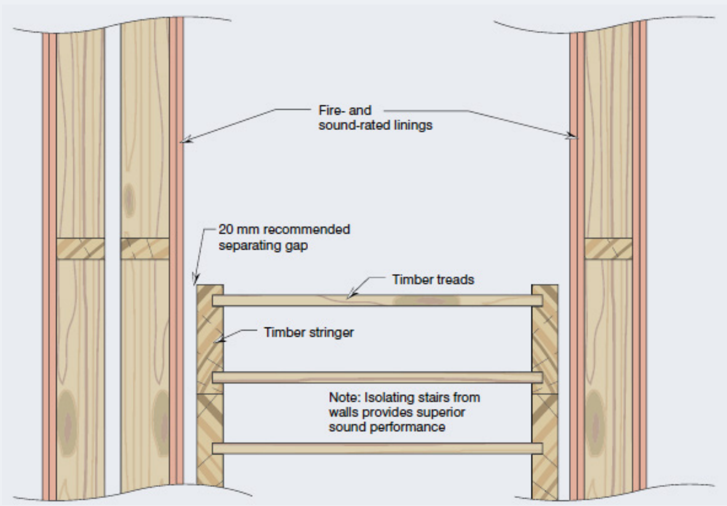


Figure 5.34: Sound isolation of stairway.

If a non-combustible stair (e.g. steel) is installed within the fire-protective timber stair shaft, the sprinkler system does not require extending to provide coverage in a fire-isolated stair.

5.12 Construction for Lift shafts

Lift shafts can be constructed in a similar manner to stair shafts as described in Section 5.10.

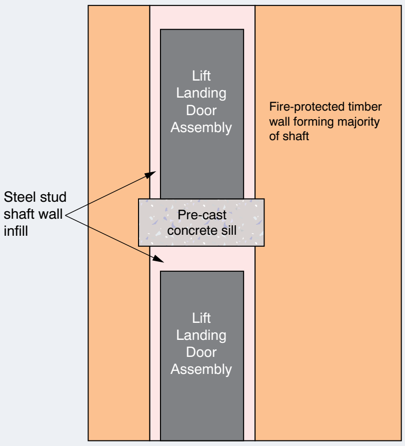
Care is needed to ensure that the lift shaft is compatible with the selected lift system. Compatibility issues should be resolved early in the design process and early liaison with the lift supplier is strongly recommended.

In the short term, most lift landing door assemblies will have been fire tested in masonry/concrete or steel stud shaft wall systems. The following details provide an interface between fire-protected timber and a pre-cast concrete sill and steel shaft wall systems. This can enable lift doors to be installed within sections of the wall of steel stud /plasterboard shaft wall and concrete construction to which existing lift landing door fire-resistance test results can be applied.

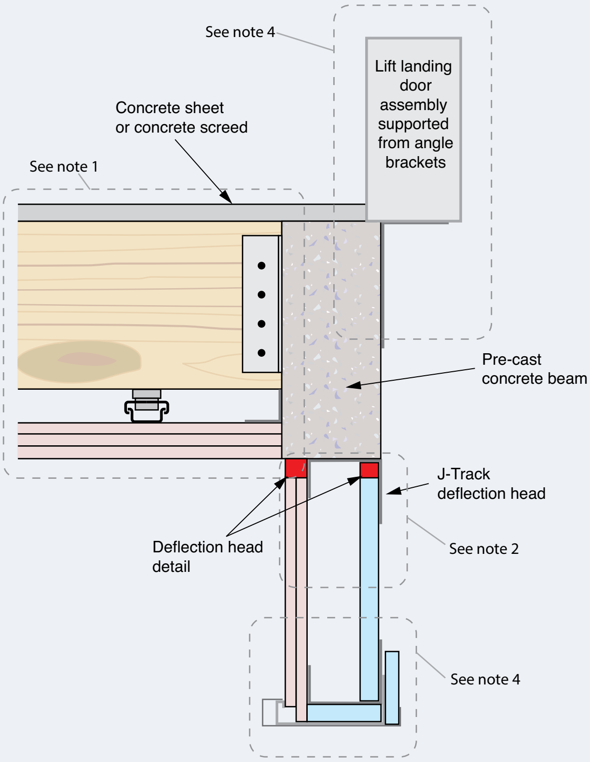
In the longer term, a larger range of lift landing doors is expected to be fire tested in fire-protected timber construction, providing simpler installation details.

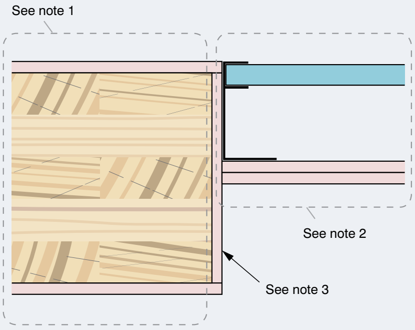
The interface details in Figure 5.35 have been assessed by an Accredited Testing Laboratory (refer report RIR 37401400). The applicability of the Evidence of Suitability to an application should be checked with the authority having jurisdiction.

Impact sound from lift use may vibrate the lift shaft walls, creating a pathway for sound transmission. While this can be addressed to some extent using double stud wall assemblies or twin-skin massive timber panel construction utilising two layers of 13 mm plasterboard, there are other options, such as the construction of a framework within the lift shaft that supports the lift assembly independently of the shaft walls.



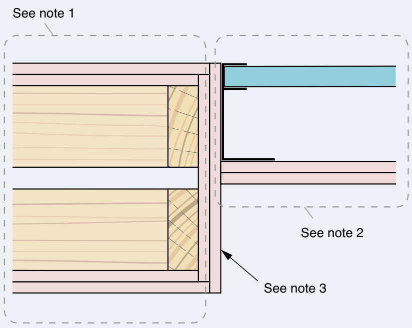
Elevation of lift doors in shaft





Side interface between shaft-wall and massive timber panel

Head and sill detail for interfaces with shaft wall and concrete sill (timber-frame)



Side interface between shaft-wall and double timber stud shaft

Figure 5.35: Typical details for shaft wall conversion for lift-landing door installation.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Lift landing door assembly having the required FRL installed in accordance with the lift door and shaft wall supplier instructions and evidence of compliance confirming the FRL.

5.13 Cavity Barriers and Junction Details

Although it is expected that massive timber in conjunction with post and beam construction will be adopted in many applications there will be applications more suited to timber-frame construction and therefore typical cavity barrier details and junction details have been provided in this Section.

5.13.1 Typical Junction Details at Intersection of Fire-Protected Timber Walls and Floors

Cavity barriers are required at the junctions between fire-protected timber floor assemblies and fire-protected timber walls in framed construction. In many instances the Deemed-to-Satisfy solutions permitting the use of solid timber and/or mineral fibre enable integration of cavity barriers with typical wall and floor junction details.

The key design parameters are to achieve, as a minimum, the required seal thickness in the direction of potential fire spread through the cavity and ensure the seals are continuous.

Typical details for double stud walls and external walls are shown in Figures 5.36-5.39. These details are based on a 'ring beam' design concept which can be useful in the management of the risk of disproportionate collapse. This form of construction is also compatible with the prefabrication of floor cassettes. Prefabrication can provide several advantages including:

- acceleration of the construction program
- improved quality control
- improved safety.

Although the mineral fibre cavity barrier is only required to be 45mm or 60 mm thick (depending on the required FRL of the elements) in the potential direction of fire spread, where practical, installation of cavity barriers the full floor depth provides a more robust solution since any joins in the ring beam/ blocks will also be backed by the mineral fibre.

For single stud internal walls, the detail is simplified because the top and bottom plates of the wall frame close off the cavities within the wall as shown in Figure 5.39.

Massive timber panel designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Figures 5.36 to 5.39 include typical examples of joint seals to allow for movement and maintain acoustic and fire separations. The joint sealing details may vary depending upon the installation order of wall and ceiling fire-protective grade coverings amongst other things. Reference should be made to the plasterboard and / or sealant suppliers for Evidence of Suitability.

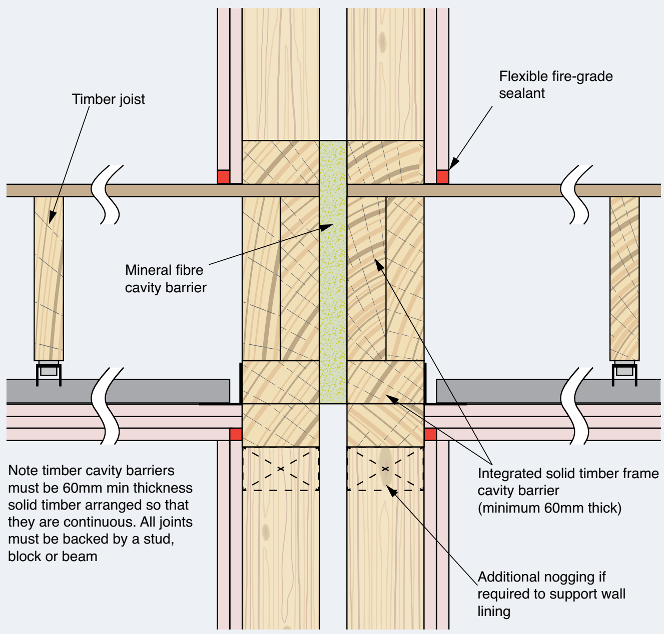


Figure 5.36: FRL 120/120/120 Fire-protected timber frame wall/floor junction with integral cavity barriers – beams parallel to wall

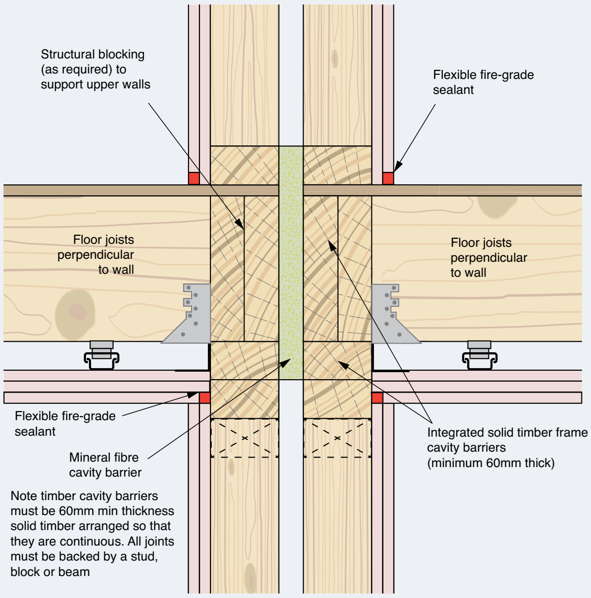
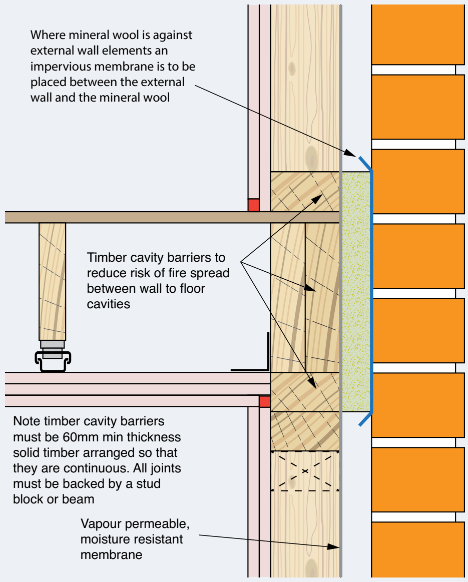


Figure 5.37: FRL 120/120/120 Fire-protected timber frame wall /floor junction with integral cavity barriers – beams perpendicular to wall



Performance can vary dependant on stud frame depth, installed insulation, cavity insulation and masonry veneer material and thickness. Evidence of suitability must be obtained to demonstrate compliance with the required FRL.

Figure 5.38: FRL 120/120/120 Fire-protected timber frame wall /floor junction with integral cavity barriers – beams parallel to wall

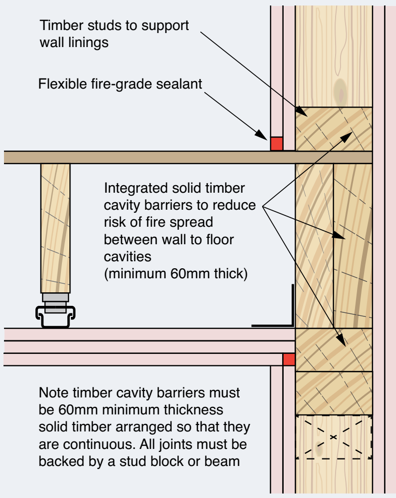


Figure 5.39: FRL 120/120/120 Fire-protected single stud timber frame wall /floor junction with integral cavity barriers – beams parallel to wall

5.13.2 Vertical Cavity Barriers

Vertical cavity barriers are required at the intersection of walls and at 10 metres maximum horizontal centres. Typical details for double stud walls and external walls are shown in Figures 5.40-5.42. Single stud details adopt a similar approach.

For double stud walls separate cavity barriers can be provided for each skin as shown in Figure 5.41 but in most instances a more practical solution is to fit a wider section spanning the full width of the intersecting wall, as shown in Figures 5.40 and 5.42.

Massive timber panel designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Where external cladding or veneer systems form part of the fire-protective coverings (e.g. brick veneer) at cavity barrier positions an impervious membrane must be placed between the mineral fibre and cladding or veneer surface to control moisture transfer from the cladding or veneer.

An alternative approach for external walls that may avoid the risk of bridging at cavity barrier positions is to apply the fire-protective coverings to the outer face of the timber elements as well as the inner face and then fit a non-combustible external cladding system that satisfies the NCC DTS requirements. Typical examples are shown in Figures 5.19 to 5.21.

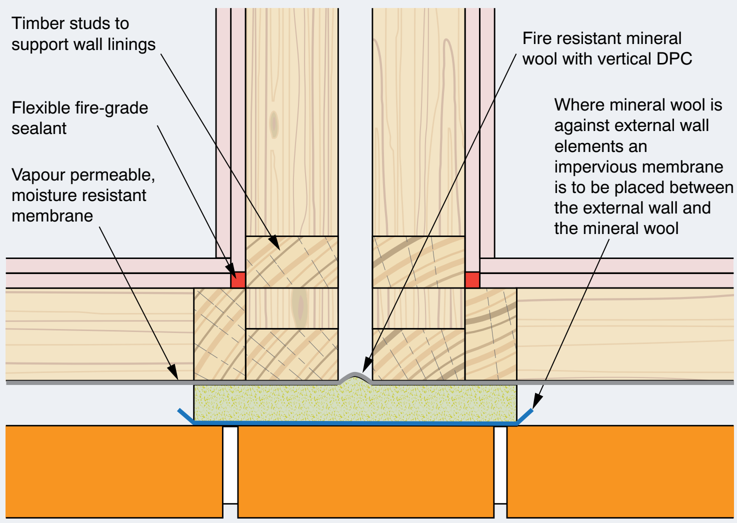


Figure 5.40: Double stud fire-protected timber internal wall intersecting a brick veneer wall.

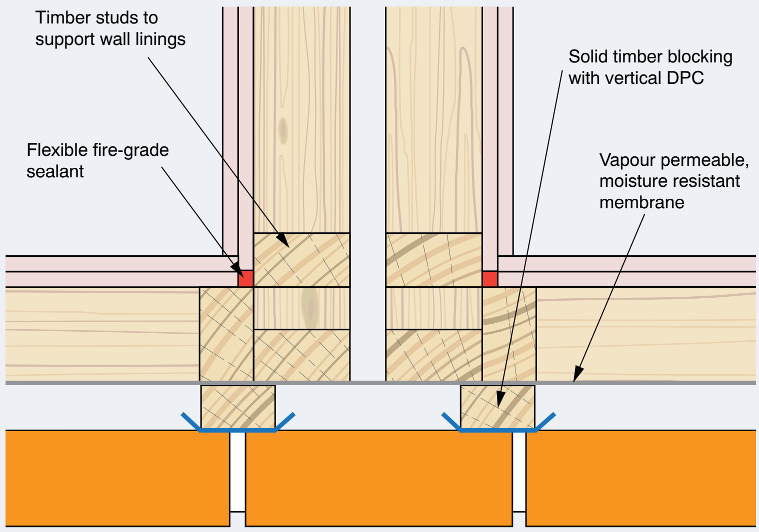


Figure 5.41: Double stud fire-protected timber internal wall intersecting a brick veneer wall with split cavity barrier system.

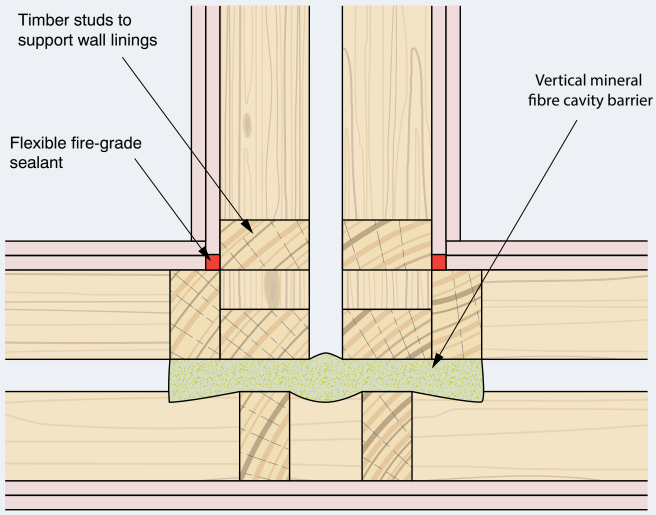


Figure 5.42: Double stud fire-protected timber internal wall intersection.

Provided the timber studs are a minimum of 45 mm thick, intermediate cavity barriers (at maximum 10 metres centres) can be fitted at a stud position as shown in Figure 5.43 if FRLs for the elements no greater than 90/90/90 are required. In elements with higher FRLs the timber thickness must be increased to a minimum of 60mm which can be achieved by nailing 2 pieces of 35mm studs (70mm total thickness) together.

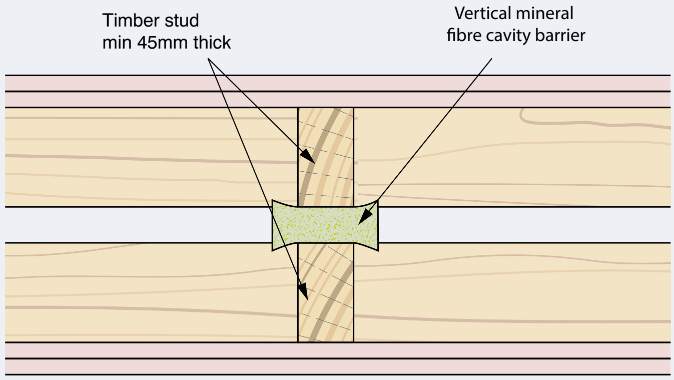


Figure 5.43: Intermediate vertical cavity barrier in double stud wall.

5.13.3 Unprotected Openings in External Walls

Cavity barriers are required around the perimeter of openings, such as unprotected windows in external walls, to prevent premature entry into the fire-protected timber cavities at these positions. A typical example is shown in Figure 5.44 for an external wall required to achieve an FRL of up to 90/90/90.

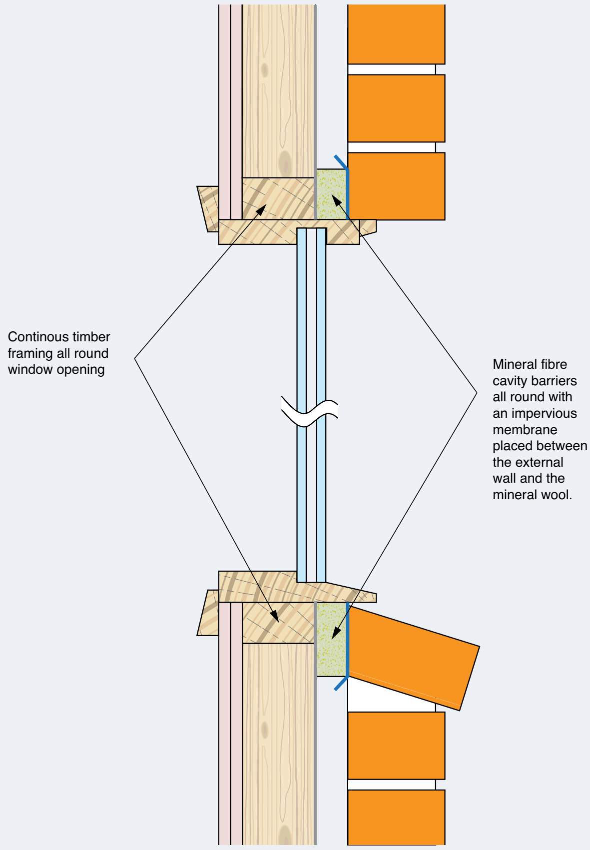


Figure 5.44: Cavity barrier around window in external wall. (minimum thickness of timber and mineral fibre must be in accordance with NCC Spec C1.13 Table 1)

5.13.4 Intersection of Non-Fire-Resisting Walls with Fire-Protected Timber Elements

Fire-protective coverings of fire-protected timber elements should not be interrupted at the point of intersection with non-fire-resisting walls to ensure the FRLs and RISF or MRISF are not compromised. Typical examples are shown in Figures 5.45 and 5.46.

Where the non-fire-resisting element is fixed to the fire-protected element additional framing may be required to avoid the risk of failure of the non-fire-resisting element compromising that of the fire-protected element. A typical detail for additional framing is shown in Figure 5.46.

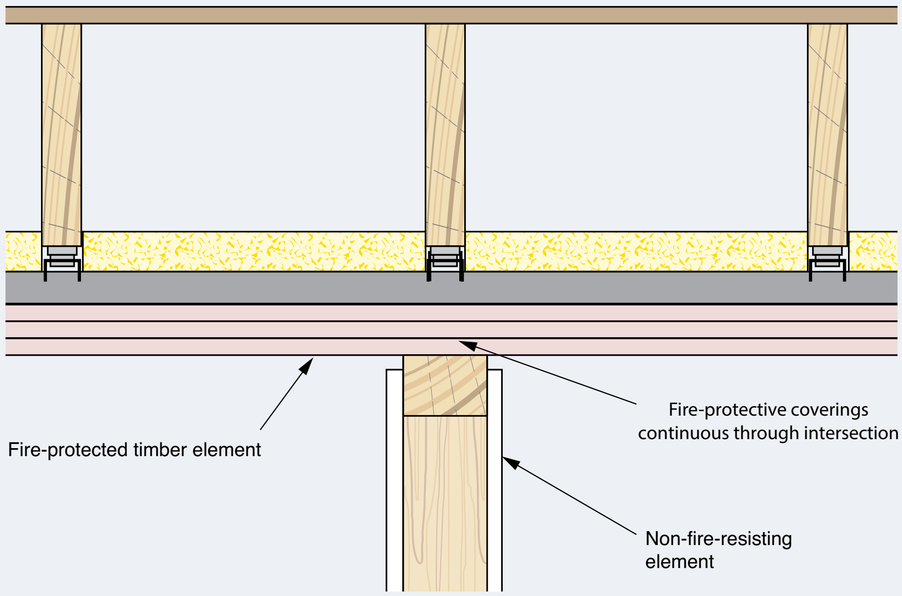


Figure 5.47: Junction of non-fire-resistant wall and fire-protected timber floor.

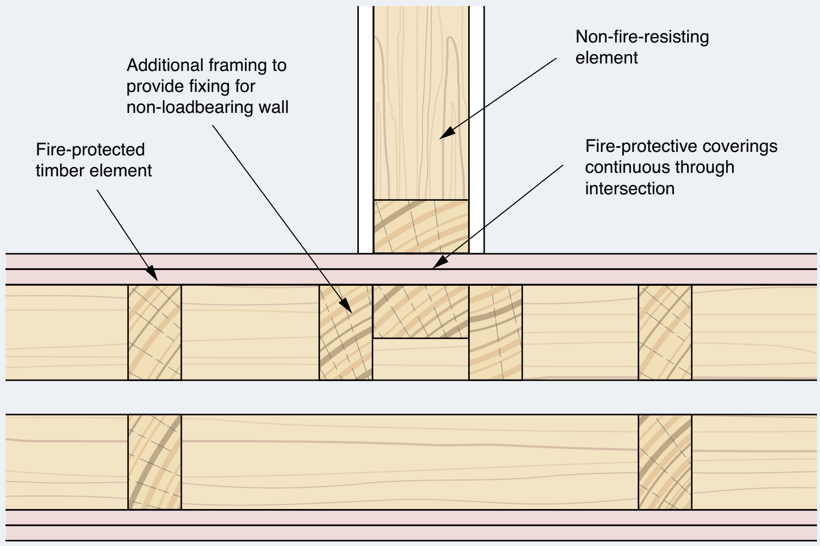


Figure 5.46: Junction of non-fire-resistant wall with fire-protected timber wall including additional framing detail.

With massive timber, the fixing point is less likely to require additional stiffening.

5.13.5 Roof Space Cavity Barriers or Fire-protected Timber Wall Extension

Special attention needs to be given to the design of roof spaces to address the risk of uncontrolled fire spread if timber frame construction is adopted although for many applications massive timber construction may be preferred. There are generally two approaches that can be adopted for timber frame construction.

Option 1: Extend fire resistant timber walls to roof level

The bounding construction around SOUs is continued to roof level (Figure 5.47). This has the advantage that the ceilings to the top floor do not need to be fire-resisting because the wall extension can provide the necessary fire and sound separation.

It is critical that the seal against the underside of the roof can achieve the required FRL, RISF or MRISF and that the fire separation is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect extension of the SOU boundary walls.

If this option is adopted a horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 5.47.

Good practice principles for service penetrations:

1 If practicable avoid service penetrations through fire-protected timber elements.

2 If fire-protected timber elements have to be penetrated by services, group the services and run them through lined openings protected by multi-penetration systems.

3 Ensure the FRLs and the RISF or MRISF levels are maintained at service penetrations.

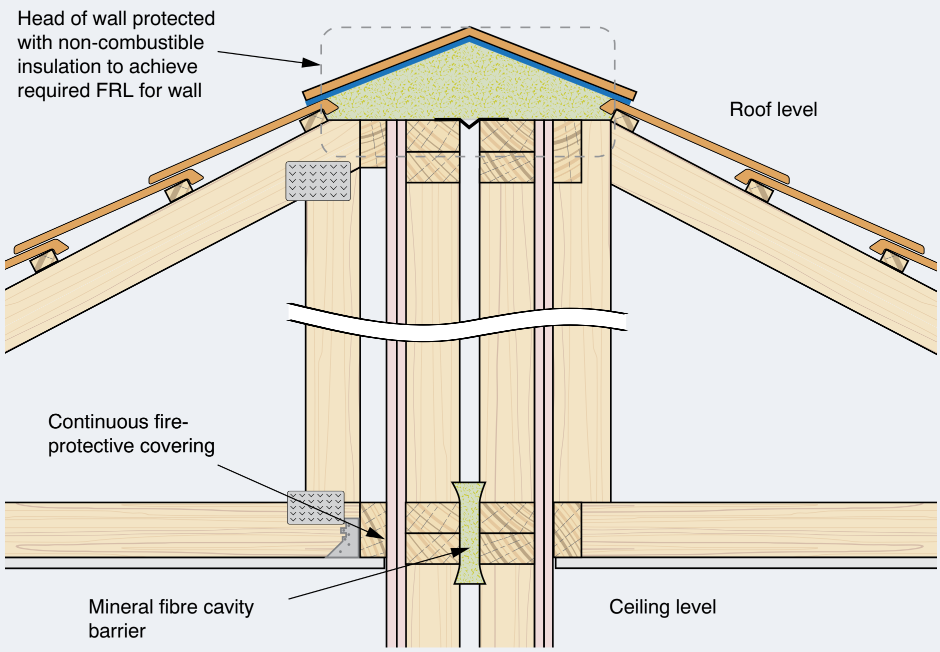


Figure 5.47: Roof space option 1 extending SOU bounding fire-protected timber walls to roof level.

Option 2: Provide fire-protected timber ceiling and cavity barriers within roof space

If Option 2 is applied to the Class 5 example building, assuming timber-framed construction, the ceiling / roof system would require an FRL of 90/60/30 and a RISF of 45 minutes assuming the floor loading concession is applied. The roof spaces would need to be divided by cavity barriers above each wall required to be fire resisting. Where the roof void is relatively deep it may be impractical to apply the Deemed-to-Satisfy solutions of solid timber or mineral fibre and a plasterboard partition achieving the required FRL of 45 minutes for a cavity barrier may provide a more practical solution as shown in Figure 5.48.

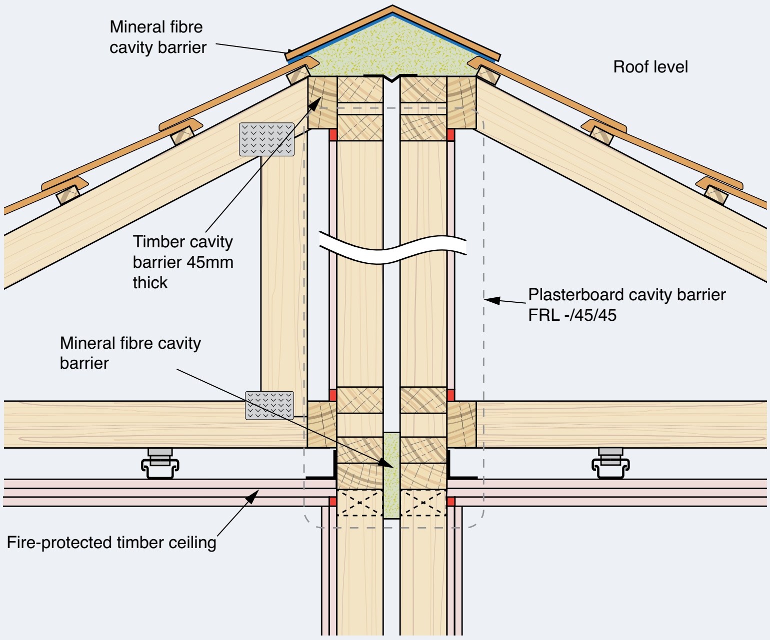


Figure 5.48: Roof space Option 2 fire-protected timber ceiling and cavity barriers within roof space.

If this option is selected it is critical that the seal against the underside of the roof is capable of achieving the required performance and the fire separation provided by the cavity barrier is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect at the extension of the fire resistant wall

A horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 5.48

Depending on the roof design the roof cavity height can vary from nominally 150 mm to several metres and careful consideration should be given to detailing and checking installations to ensure the design objectives are achieved.

5.14 Service Penetration Treatments

Careful detailing of services and service penetration systems during the design stages and subsequent correct installation during construction can simplify construction details and streamline the construction process as described in early chapters of this Guide.

The general design approach can be expressed as three fundamental principles

- Select services, service locations and service runs to avoid, as far as practical, the need for service penetrations through fire-protected timber elements (e.g. the use of false walls and ceilings can substantially reduce the number of penetrations that require protecting).
- If service penetrations cannot be avoided, where practical they should be grouped and penetrate lined openings or non-combustible shaft walls, which minimises the risk of exposing the cavity during maintenance operations. This approach also simplifies the installation of new services.
- If service penetrations are required to pass through fire-protected timber elements, ensure the FRL and RISF or MRISF as appropriate at service penetration positions.

The following Sections provide typical generic examples. Over time, it is expected that proprietary systems will become available simplifying the installation process. Refer to Section 5.8 Service Shafts for typical interface details between non-combustible shaft construction and fire-protected timber.

5.14.1 Multi-penetration Systems with Lined Openings

Typical multi-penetration systems with lined openings are shown in Figure 5.49.

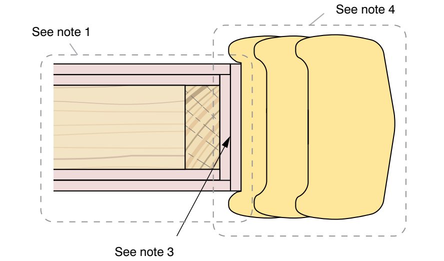
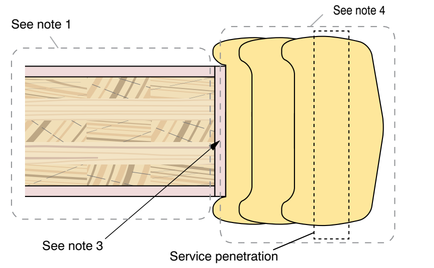
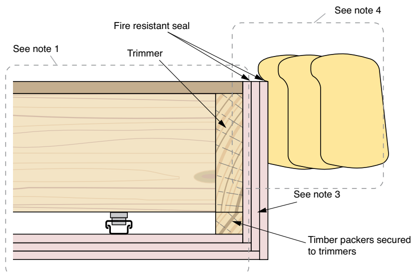
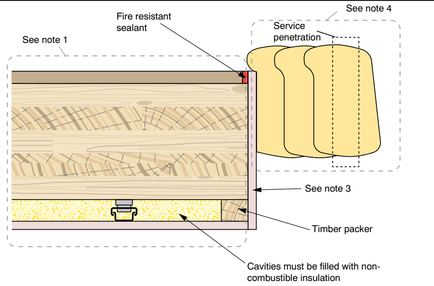
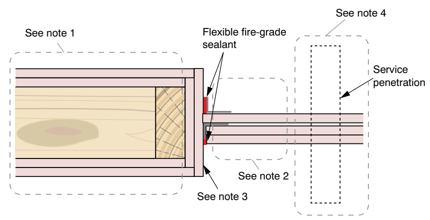
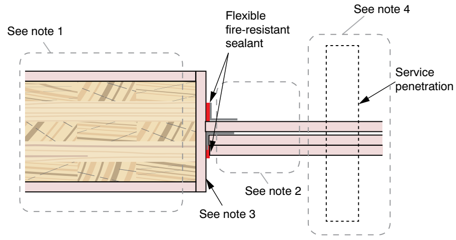
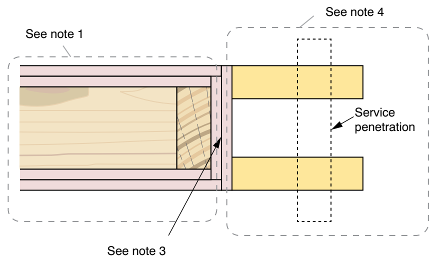
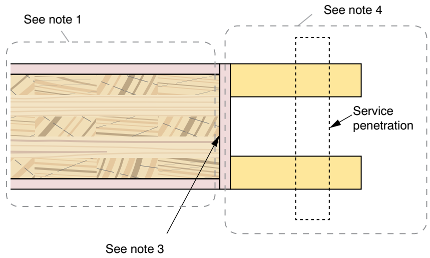
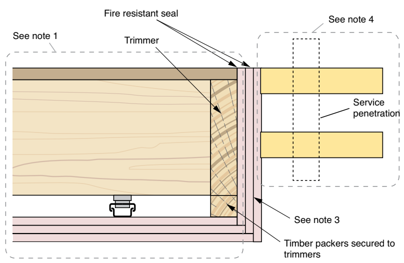
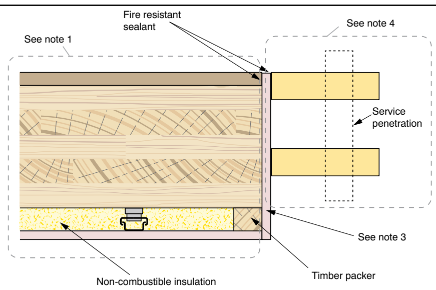
System	Timber Frame	Massive Timber Panel
Pillow in wall opening		
Pillow in floor opening		
Shaft-wall infill in wall opening		
Mineral fibre batt wall opening		
Mineral fibre batt floor opening		

Figure 5.49: Typical multi-penetration systems with lined openings.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of interface details shown in Figure 5.49

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of interface details shown in Figure 5.50

Interface details shown in Figure 5.49 have been assessed by a registered test laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber stud systems and 30 minutes for the massive timber panel systems (Refer Report RIR 37401400). Other details may be adopted provided appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements is provided.

5.14.2 Fire Damper and Duct Penetrations

The lined opening approach can also be applied to duct and damper penetrations (Figure 5.50).

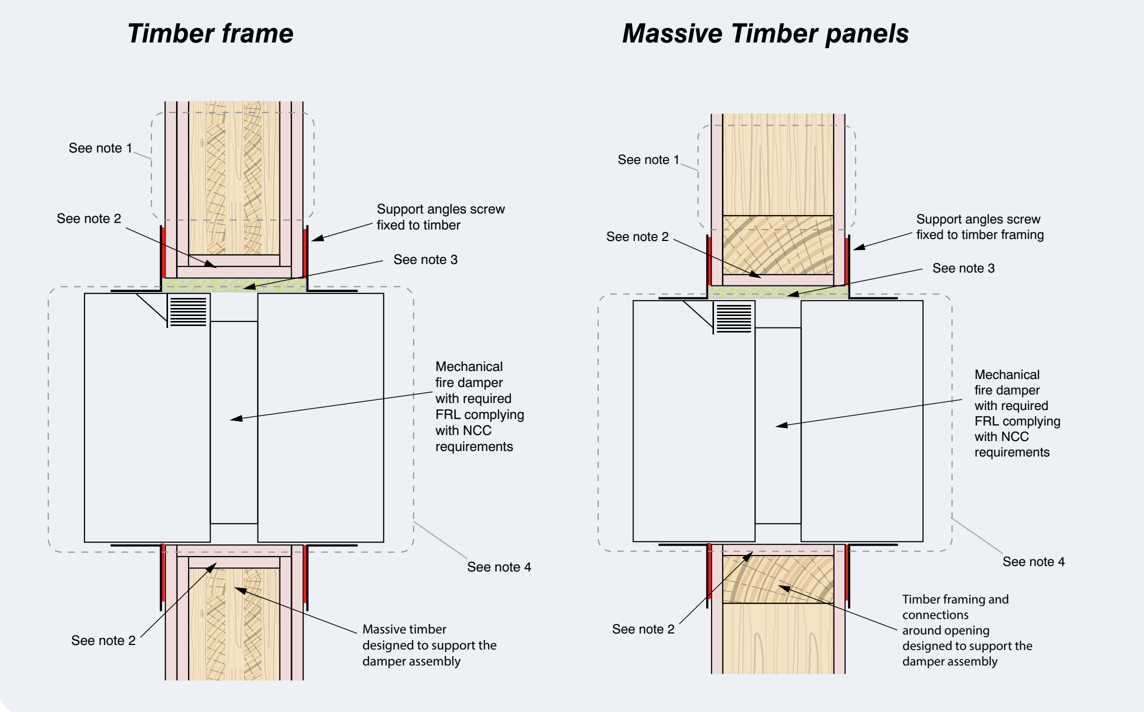


Figure 5.50: Typical details for fire damper and duct penetrations.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 3: Non-combustible mineral fibre packing may be used for fire damper penetration seal or proprietary fire damper penetration seals that achieve the required FRL with evidence of performance in the form of a report from an Accredited Testing Laboratory to be in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 4: Mechanical fire damper having the required FRL when tested in accordance with AS 1530.4 and complying with AS 1682 Parts 1 and 2 as appropriate.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of wall lining detail shown in Figure 5.51

5.14.3 GPO Outlets and Switches

Where practical, the need to protect GPO outlets, switches and similar penetrations should be avoided by mounting them within internal (non-fire-resisting walls) or false (decorative) linings fitted in front of fire-protected timber elements as shown in Figure 5.51.

Methods of attaching non-fire-resisting decorative linings that will not compromise the FRL, RISF or MRISF performance of wall and floor systems, such as shown in Figure 5.51, have been assessed in a report from a Accredited Testing Laboratory (refer RIR 37401400).

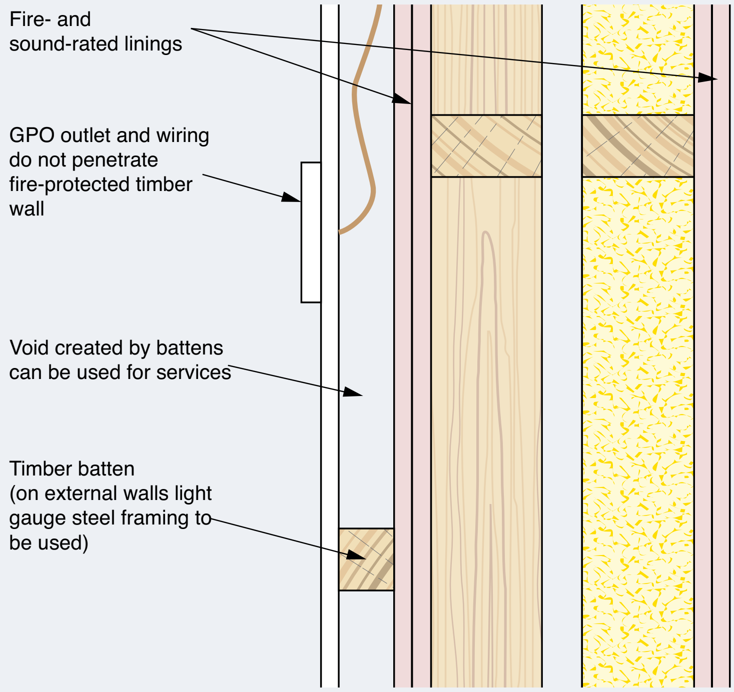


Figure 5.51: False wall system.

If it is impractical to apply an additional lining, a proprietary GPO protection system may be adopted, if it has Evidence of Suitability, demonstrating that the required FRL and RISF or MRISF for the element will not been compromised.

Alternatively, the generic systems shown in Figures 5.52 and 5.53 may be adopted.

New products (e.g. skirting service ducts) also enable services to be run without penetrating fire-protective grade linings.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of the GPO interface details shown in Figures 5.52 and 5.53

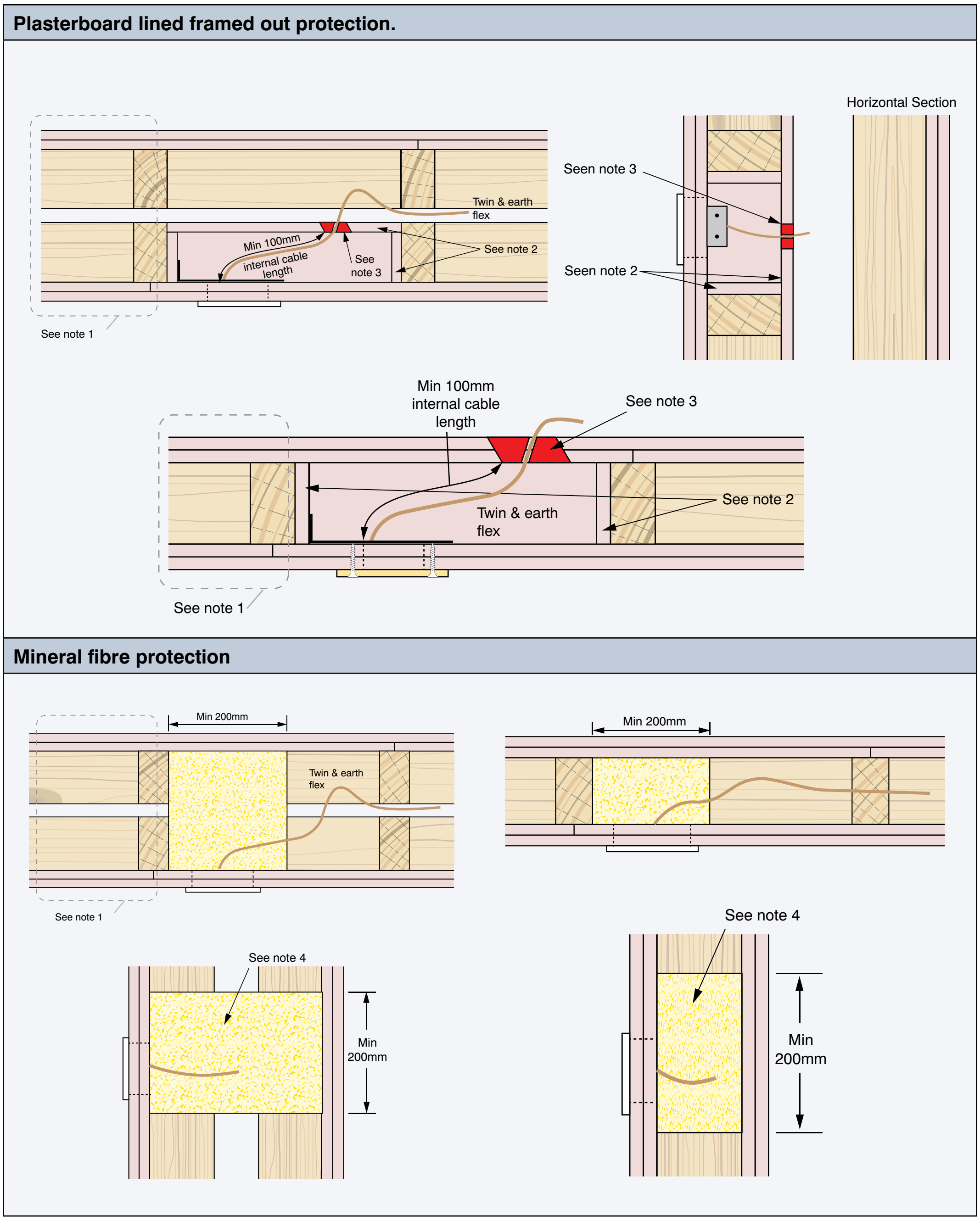


Figure 5.52: Generic GPO protection systems in timber-framed construction.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic' The mastic should have evidence of performance in the form of a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve an FRL of –/60/–.

Note 4: Cavity filled full depth with mineral fibre of minimum density 60 kg/m³ for at least 100 mm to the sides and above and below the centreline of the GPO.

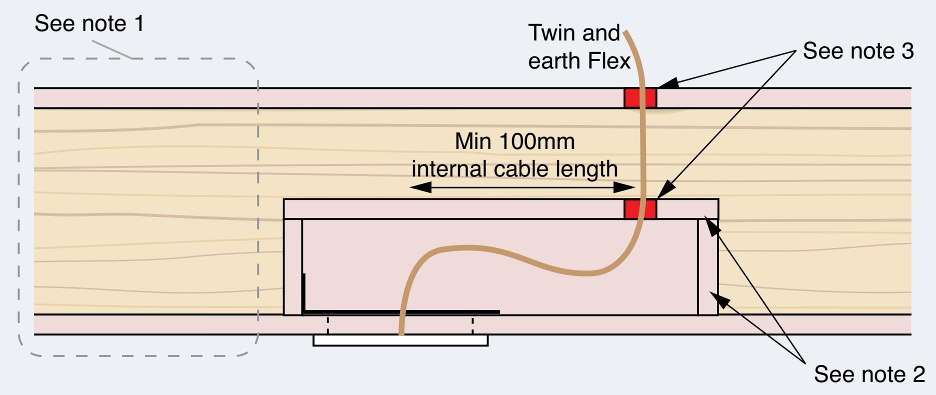


Figure 5.53: Generic GPO protection systems in massive timber construction.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Aperture lined with a minimum of one layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic'. The mastic should have a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve an FRL of –/60/–.

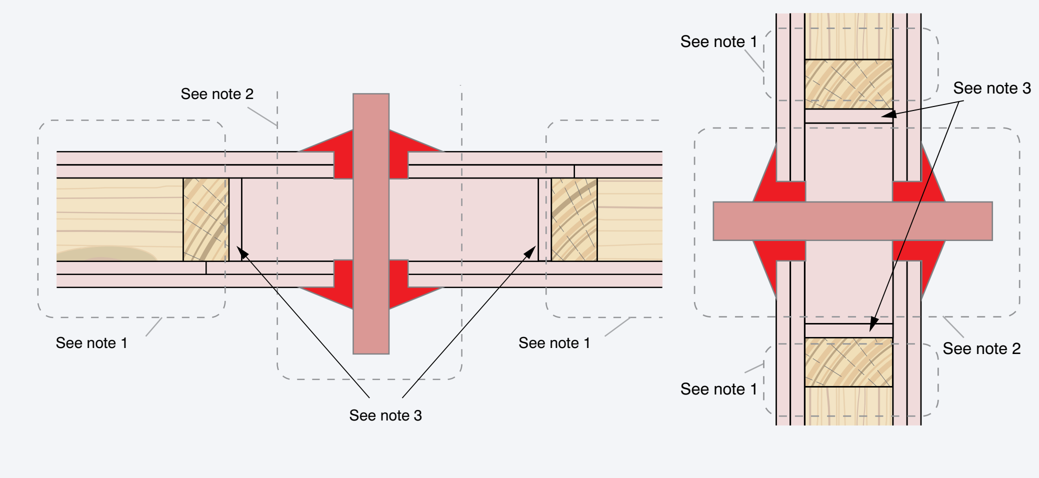
5.14.4 Single Cable and Metal Pipe Penetrations

Where single cable and pipe penetrations through fire-protected timber members cannot be avoided, existing proprietary protection systems that have achieved the required FRL in plasterboard systems can be used in conjunction with internal plasterboard linings or mineral fibre insulation packing as shown in Figure 5.54 to satisfy a RISF of 45 minutes or MRISF of 30 minutes as appropriate.

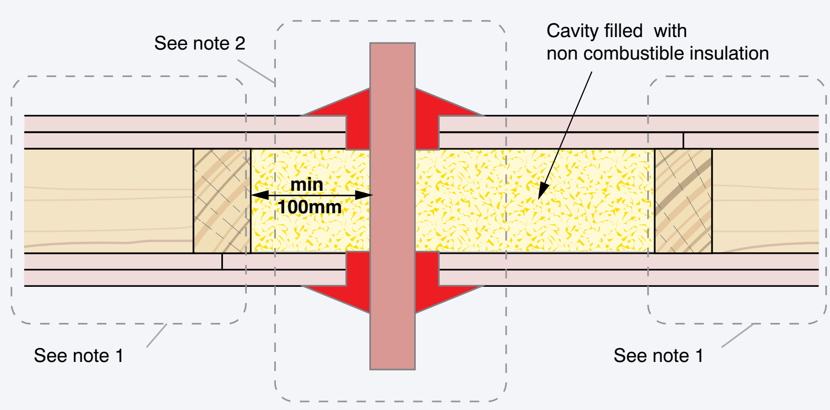
Fire tested proprietary systems may provide more practical options, subject to adequate Evidence of Suitability being available.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of the systems shown in Figure 5.54

Plasterboard lined timber frame. RISF 45 minutes



Mineral fibre fill timber frame. RISF 45 minutes.



Mineral fibre fill massive timber. MRISF 30 minutes.

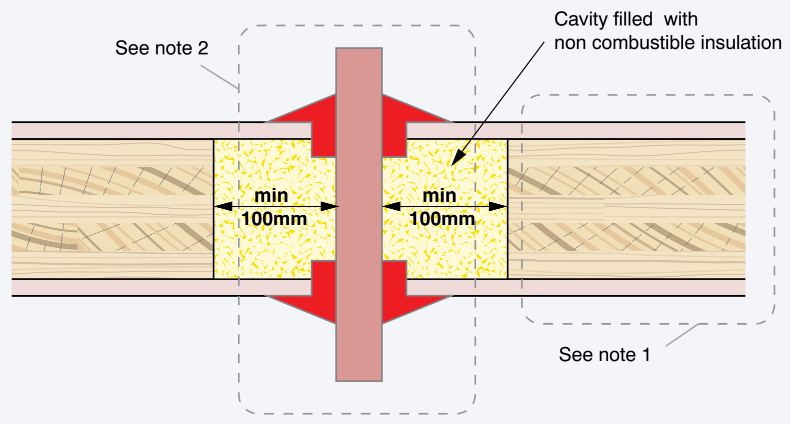


Figure 5.54: Pipe and cable penetrations through fire-protected timber.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Refer Report FAS 190034, available from the WoodSolutions website, for assessment of ceiling lining detail shown in Figure 5.55.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of back blocking system shown in Figure 5.56

The preferred option for lighting cables, sprinkler pipe penetrations and the like is to run them through the cavity above a false ceiling. A typical false ceiling detail is shown in Figure 5.55. Larger cavities can be provided above false ceilings by using suspended ceiling fixings to accommodate down lights and larger services.

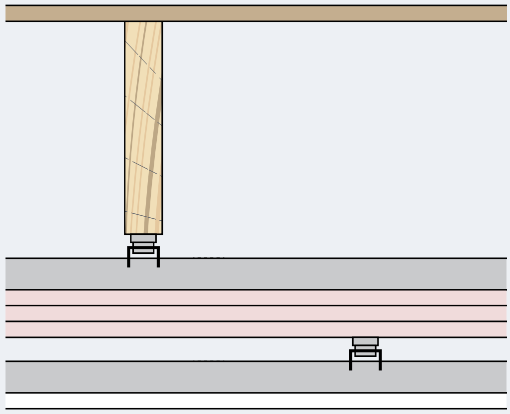


Figure 5.55: False ceiling detail for minimising service penetrations through ceiling systems.

If it is impractical to provide a false ceiling a solution for lighting cable penetrations through fire-protected timber ceilings is to use cover blocks as shown in Figure 5.56. Proprietary systems may be available to protect down-light penetrations and sprinkler pipe penetrations but access for the long-term service and maintenance of these systems and options for reconfiguration would be very limited.

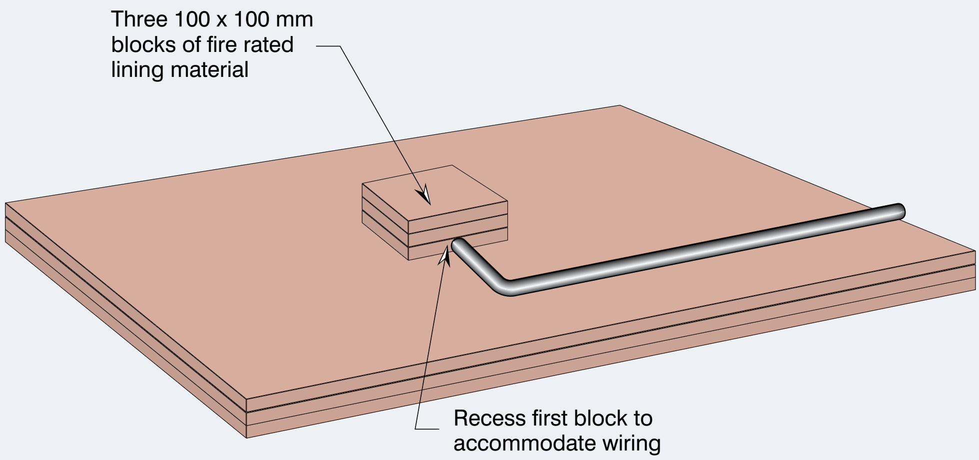


Figure 5.56: Recess block protection system for lighting cables penetrating fire-protected timber floors.

5.14.5 Rebated Ceiling Details for Housing Services

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of rebated ceiling system shown in Figure 5.57

Another alternative for ceiling systems is to create a rebate to house services without penetrating a fire-protected element such as a fire-protected timber floor/ceiling system as shown in Figure 5.57. This detail has been assessed by an Accredited Testing Laboratory as achieving an FRL of 120/120/120 and a RISF greater than 45 minutes. Care should be taken not to attach the rebate framing members to the floor structure to avoid short-circuiting the sound separation.

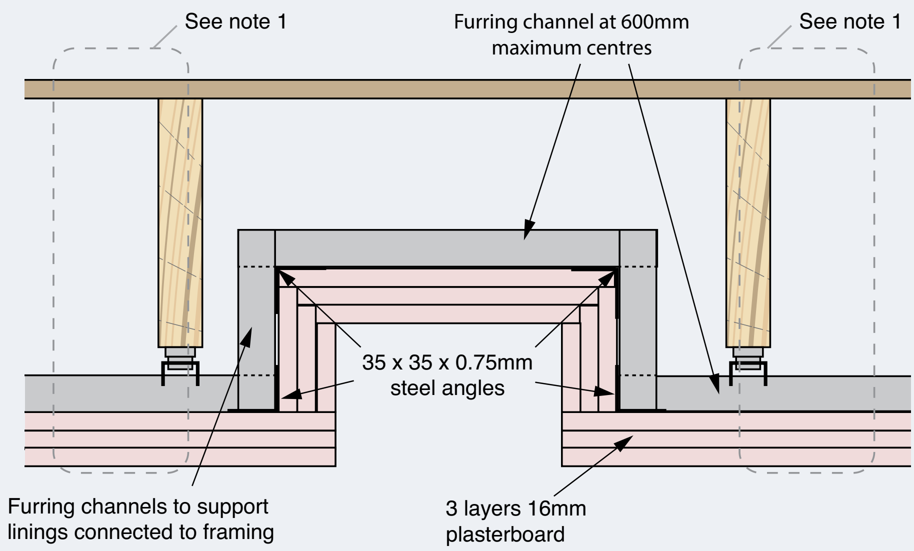


Figure 5.57: Rebated ceiling system.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

The rebate may be fitted with a grill, a section of false ceiling or may be sized to mount individual items of equipment.

5.14.6 Plastic Pipe Penetrations

Where it is impractical to adopt false wall and ceiling linings or utilise non-combustible shaft construction or lined opening multi-penetration systems, the following details, shown in Figures 5.58, 5.59 and 5.60, have been developed to maintain a RISF of 45 minutes or a MRISF of 30 minutes. The systems must have achieved an FRL of at least -/90/90 in plasterboard partitions when used to protect individual plastic pipe penetrations. The following notes apply:

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of Suitability to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Refer Report RIR 37401400, available from the WoodSolutions website, for assessment of interface details to maintain the RISF and MRISF performance of elements penetrated by plastic pipes as shown in Figures 5.58 to 5.60

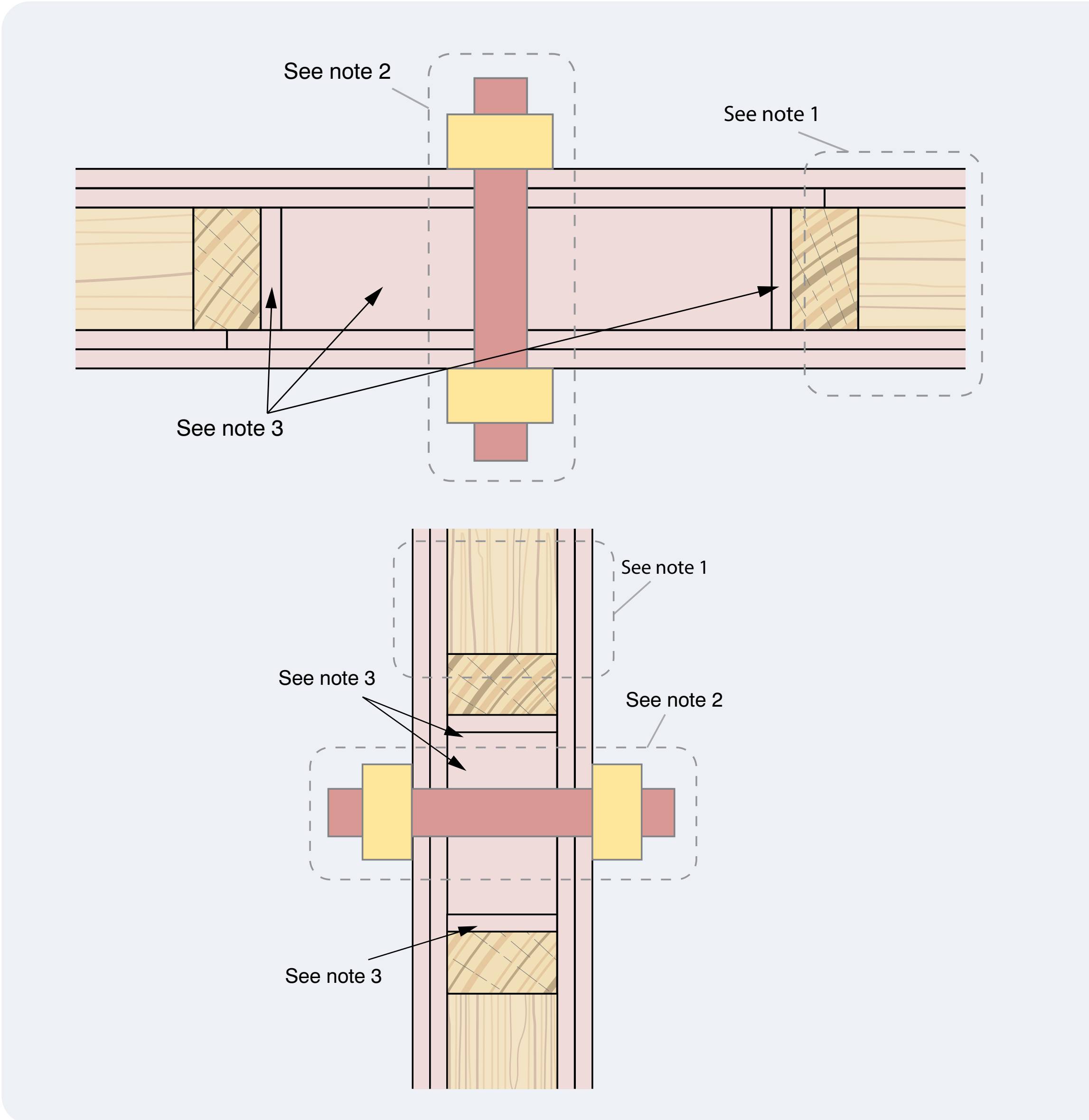


Figure 5.58: Plastic pipe penetration through fire-protected timber-framed walls with internal linings.

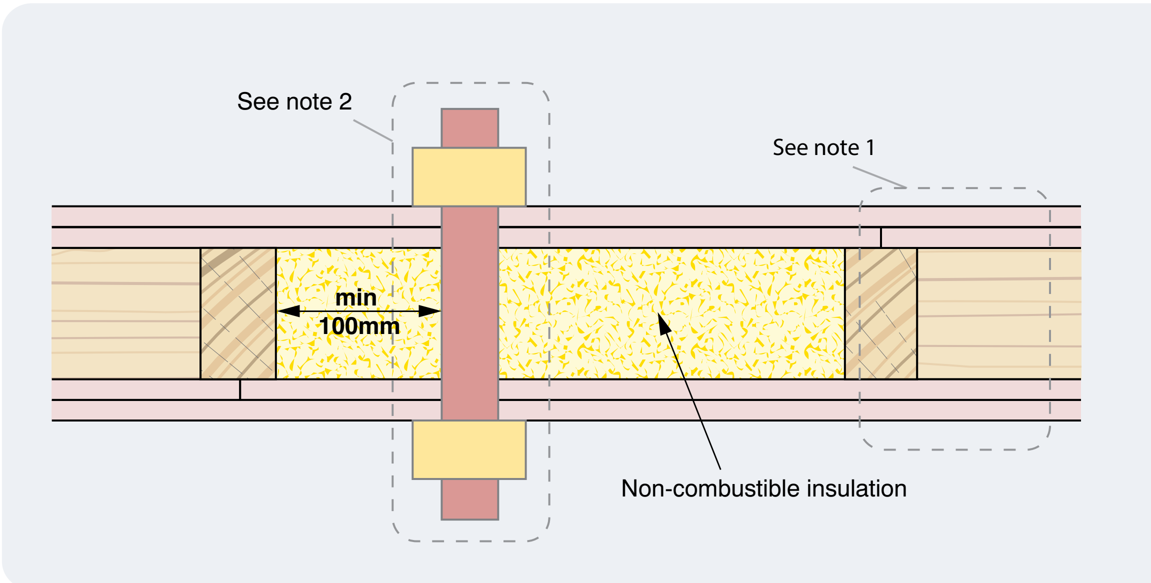


Figure 5.59: Plastic pipe penetration through fire-protected timber-framed walls with non-combustible mineral fibre insulation.

Evidence of Suitability required from supplier to confirm required RISF and MRISF performance of penetrated elements is maintained in addition to the FRL for the system shown in Figure 5.61

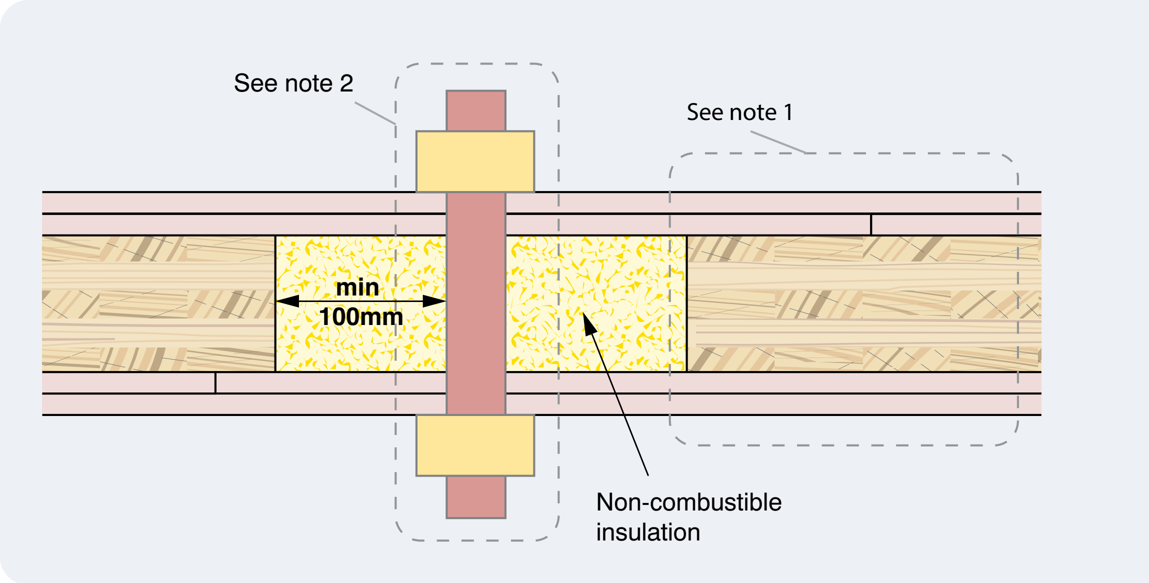


Figure 5.60: Plastic pipe penetration through fire-protected massive timber walls with non-combustible mineral fibre insulation.

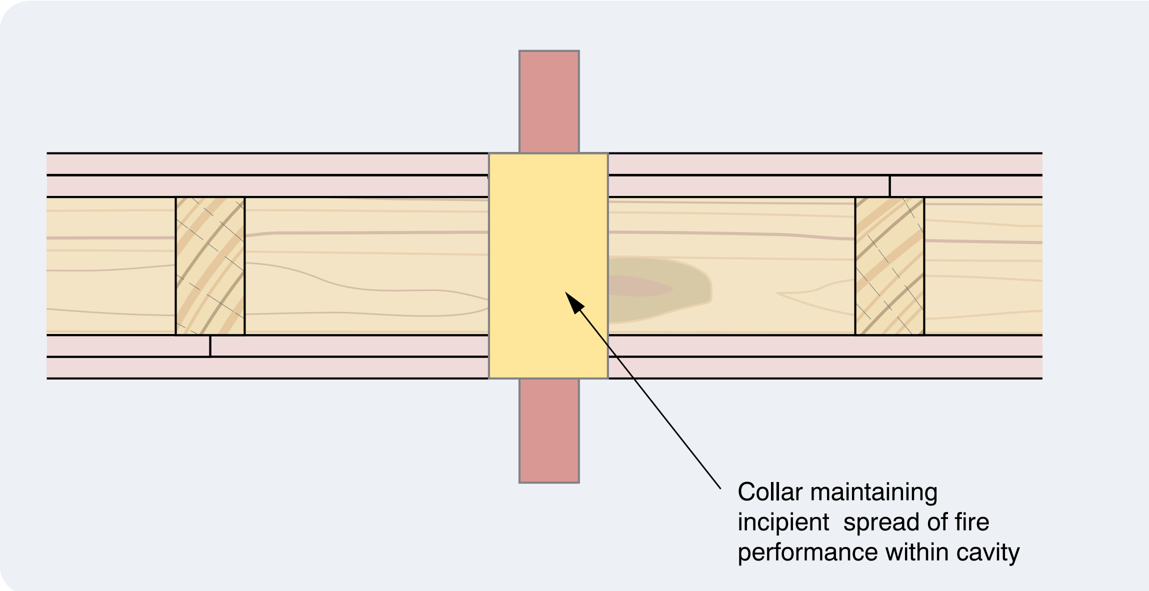


Figure 5.61: Option for a proprietary system with integral insulation protecting a plastic pipe penetration.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of interface details to maintain the RISF and MRISF performance of elements pentetrated by access panels as shown in Figures 5.62 to 5.64

5.14.7 Access Panels

Access panels may be used to protect openings providing access to a floor/ceiling cavity as shown in Figure 5.62 or to shafts through fire-protected timber walls as shown in Figures 5.63 and 5.64.

Providing access panels will tend to compromise the sound separation and therefore they should normally be located in areas that are not sound 'sensitive'.

The following notes apply to the typical details shown in Figure 5.64 through Figure 5.66.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire-protective coverings that are applied to the fire-protected timber element face.

Note 3: Proprietary access panel system with the required FRL. For access panels providing access to ceiling cavities an RISF rating of 45 minutes or a MRISF rating of 30 minutes as appropriate is also required to be satisfied.

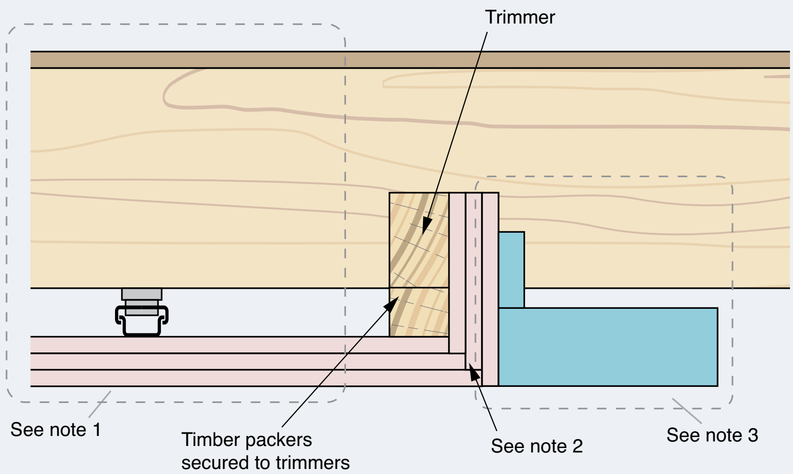


Figure 5.62: Access panel in a fire-protected floor/ceiling system.

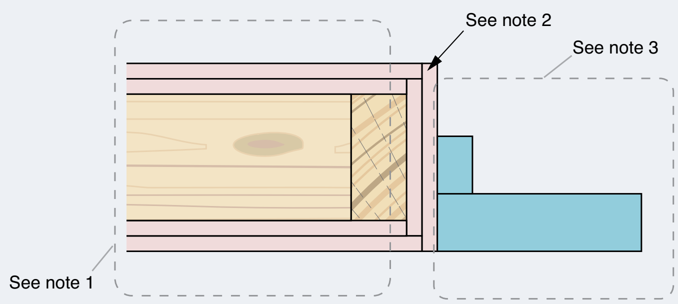


Figure 5.63: Access panel in a fire-protected timber-framed wall.

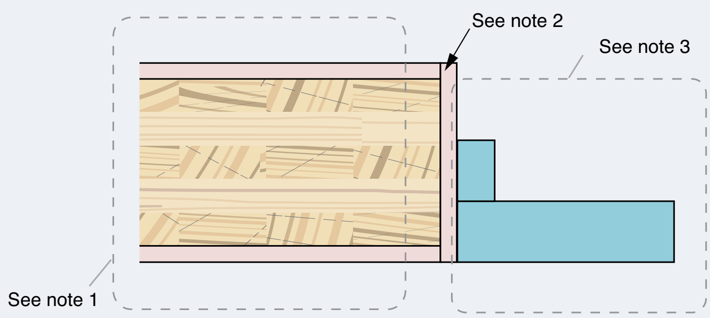


Figure 5.64: Access panel in a fire-protected massive timber wall.

5.15 Automatic Fire Sprinkler Systems

The provision of an automatic fire sprinkler system is accordance with NCC Specification E1.5 (other than a FPAA101D or FPAA101H system) is a mandatory requirement for mid-rise timber buildings if the DTS Solution pathway is adopted.

The automatic fire sprinkler system is a critical component of the fire safety design and must be designed and installed by organisations and/or individuals with appropriate competency. Detailed information about the design of automatic fire sprinkler systems is outside the scope of this Guide, however in common with all services there is a need for the design to be integrated with the architectural, structural and passive fire protection systems. The following sub-sections highlight some key considerations, but it is not an extensive summary.

It is important that the design documentation clearly specifies the requirements for the sprinkler systems such as locations of pipe runs, types of materials and components to be used, treatment of penetrations, types of sprinkler head and positions.

5.15.1 Piping Materials and Connections

Materials for piping and connection details for fire sprinkler systems should be carefully selected to:

- comply with the NCC Specification E1.5 requirements (other than a FPAA101D or FPAA101H system)
- suit the environment
- minimise the time the system is unavailable after maintenance/repair
- minimise hot works on site such as cutting and welding metal pipes
- facilitate the reinstatement of the performance of fire-protected timber at the points of penetration by sprinkler pipes.

While plastic pipes (e.g. CPVC) can largely negate the need for hot works, if alterations are made to plastic pipes, the sprinkler system could be unavailable while the adhesive sets. This is an important consideration for buildings that undergo regular refits such as offices and retail premises. The reinstatement of the performance of fire-protected timber when penetrated by plastic pipes can be more complex than metal pipe penetrations

Metal pipes may be more appropriate for some applications, but they should be pre-prepared so that, as far as practical, all on-site connections can be made without hot works. Fittings can be selected that can be adjusted on site, such as flexible sprinkler fittings minimising the need for hot works.

Once the materials and components have been selected the pipe runs should be clearly defined to minimise the number of penetrations through fire-protected timber and that if they cannot be avoided, they occur where the performance of the fire-protected timber can be readily reinstated.

5.15.2 Sprinkler Head Selection

Although not mandatory in AS 2118.1, fast response heads should be used where practicable and appropriate since they respond faster, reducing the risk of occupants and increasing the likelihood that the sprinkler system will suppress the fire.

Sprinkler head options include concealed and semi-recessed. Concealed heads are a common choice in public areas because they can reduce the risk of vandalism and accidental impact. However, the following issues should be considered during the selection process and appropriate mitigation measures adopted:

- larger cut outs in ceilings are required which can be addressed by use of a non-rated false ceiling. The false ceiling depth should be designed to allow for the fitting of the concealed heads and related pipework.
- the response time will tend to be slower - this should be checked with the manufacturer.
- overpainting and use of sealants to retain covers can compromise the performance of a head - this should be addressed through regular inspections.

5.15.3 Monitored Isolation Valves

The reliability of an automatic fire sprinkler system can be enhanced by specifying monitored isolation valves incorporating a check valve and flow switch at each level that is permanently connected to a fire alarm monitoring service provider by a direct data link.

This approach allows the water supply to the sprinkler system on individual floors to be isolated for maintenance or reconfiguration of the system without the need to isolate the whole building. Since the valves are monitored, the risk of the water supply not being reinstated is also significantly reduced.

This arrangement is compatible with the progressive commissioning of automatic fire sprinkler systems during construction, allowing protection of the lower levels while work progresses on the upper levels and individual floors to be easily isolated for adjustments to systems. This may be adopted as part of the fire safety strategy to address fire safety during construction.

5.15.4 Fire-isolated Stairs and Passageways with Timber Stairways

The NCC allows the use of timber stairways in fire-isolated stairs and passageways subject to the automatic fire sprinkler system coverage being extended to cover the fire-isolated stair in addition to other precautions (refer Section 4.8.2).

In the absence of other specifications, sprinkler heads should be provided in the following locations:

- at the top of the shaft
- under the landings at each floor level
- under intermediate landings
- providing coverage to other positions where there is a significant risk of accumulation of combustible materials.

5.16 Other NCC Requirements

This is a guide to the use of fire-protected timber for mid-rise timber buildings as a DTS solution in the NCC. It does not address all NCC requirements that apply to mid-rise buildings nor does it address all NCC fire-related requirements (e.g. fire hazard properties of linings).

Advice should be sought from appropriately qualified practitioners and relevant regulatory authorities regarding compliance with the NCC for specific projects.

6

Class 4 Parts of Buildings

Refer NCC 2019 for further details on NCC fire safety requirements for Class 4 parts of Buildings

A Class 4 part of a building is a dwelling in a Class 5, 6, 7, 8 or 9 building (e.g. a caretaker's flat). A building can only contain one Class 4 dwelling. If two or more dwellings are within a building of another Class, the dwellings must be classified as a Class 2 part of the building.

The Class 4 part has much in common with Class 2 and 3 buildings and the fire separation of the dwelling (SOU) and other fire safety measures within the dwelling are closely aligned with Class 2 and 3 Residential Buildings. This chapter should therefore be read in conjunction with;

WoodSolutions Design Guide #37R Mid-rise Timber Buildings Multi-Residential Class 2 and 3

The remaining fire safety related content of the NCC for Class 4 buildings deals with ensuring egress provisions and general fire safety measures are compatible with both the residential and commercial usage.

The following section highlights some of the more significant matters for consideration if a commercial building contains a Class 4 Part. Reference should be made to the NCC to check the requirements for provisions not listed below.

6.1 Type of Construction Required

If a Class 4 part of a building is on the top storey, the classification applying to the top storey (and all other storeys) must be the classification applicable to the next highest storey if the Class 4 part occupies the whole of the top storey, or the classification applicable to the adjacent part if the Class 4 part occupies part of the top storey. (NCC Clause C1.3)

Refer WoodSolutions Design Guide #37R for information on SOU Bounding Construction and detailing of service penetrations and other openings in bounding Construction

6.2 Fire Separation

A Class 4 part of a building requires the same FRL for building elements and the same construction separating the Class 4 part from the remainder of the building as a Class 2 part in the same Type of Construction (Refer WoodSolutions Design Guide #37R for further information).

This requirement includes doors serving the SOU, service penetrations and other openings.

6.3 Travel distance from SOU door

As required for a Class 2 or 3 building, the entrance doorway to any Class 4 part of a building must be not more than 6 m from an exit or a point from which travel in different directions to 2 exits is available.

6.4 Smoke Hazard Management

Class 4 part of a building must be provided with an automatic smoke detection and alarm system complying with NCC Spec E2.2a.

A fire protected timber mid-rise building is required to be provided by an automatic fire sprinkler system (other than a FPAA101D or FPAA101H system) complying with Specification E1.5 throughout and therefore a required fire isolated stairway serving the Class 4 part may also serve one or more storeys of Class 5, 6, 7 (other than an open-deck carpark), 8 or 9b parts.

.

A

Appendix A – Determination of Compliance of Fire-protected Timber

Refer NCC A5.4 and Schedule 5 for FRL

Refer NCC A5.6 for RISF

Refer NCC A5.2 for non- combustibility

There are three components to the performance of fire-protected timber that need to be satisfied:

- the protected element must achieve the required Fire Resistance Level (FRL)
- the protected element must achieve the required Resistance to the Incipient Spread of Fire (RISF or MRISF as appropriate), and
- fire-protective coverings must be non-combustible.

A1 Non-Combustible Fire-Protective Covering

Unless the NCC deems a material or element of construction to be non-combustible, non-combustible means:

- Applied to a material – not deemed combustible as determined by AS 1530.1 – Combustibility Tests for Materials.
- Applied to construction or part of a building – constructed wholly of materials that are not deemed combustible.

If the fire-protective covering is a composite or multi-layer system, each layer must be non-combustible. It is not acceptable to undertake a single combustibility test on the composite or just the facing materials and claim the fire-protective covering is non-combustible.

Typical examples of multi-layer systems are shown in Figure A1.

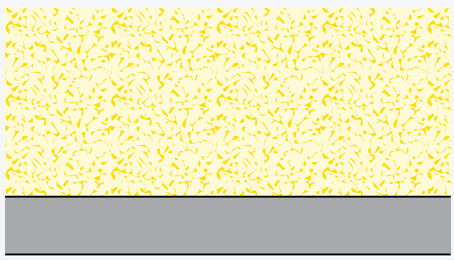
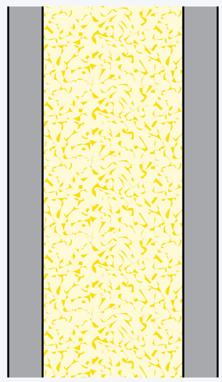
	
Multi-layer system – each layer must be non-combustible	Composite panels – each layer of the composite must be non-combustible
Commonly fire-resistant board supporting non-combustible lightweight insulation used in ceilings protecting floors/beams	Commonly non-combustible lightweight insulating core between non-combustible durable facings used for external claddings

Figure A1: Example of multi-layered fire-protective coverings (all layers).

Clause C1.9(e) of the NCC allows (deems) the following materials, though combustible or containing combustible fibres, to be used wherever a non-combustible material is required:

- plasterboard
- perforated gypsum lath with a normal paper finish
- fibrous-plaster sheet
- fibre-reinforced cement sheeting
- pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm thickness and where the Spread-of-Flame Index of the product is not greater than 0
- sarking-type materials that do not exceed 1 mm in thickness and have a Flammability Index not greater than 5
- bonded laminated materials where:
  - each laminate is non-combustible
  - each adhesive layer does not exceed 1 mm in thickness
  - the total thickness of the adhesive layers does not exceed 2 mm
  - the Spread-of-Flame Index and the Smoke-Developed Index of the laminated material as a whole does not exceed 0 and 3 respectively.

All materials forming the fire-protective covering are either permitted to be used in accordance with NCC Clause C1.9(e) or determined to be non-combustible by testing to AS1530.1.

A2 Fire Resistance Level

A fire-protected timber element must achieve the required FRL specified in the NCC for the particular application. The fire resistance of a fire-protected timber element has to be determined in accordance with Schedule 5.2(b) and (c) of the NCC.

Generally, Schedule 5.2(b) requires a prototype to be submitted to the Standard Fire Test (AS1530.4), or an equivalent or more severe test, and the FRL achieved by the prototype, without the assistance of an active fire suppression system, is confirmed in a report from an Accredited Testing Laboratory which:

- describes the method and conditions of the test and the form of construction of the tested prototype in full
- certifies that the application of restraint to the prototype complied with the Standard Fire Test;

or differs in only a minor degree from a prototype tested under Schedule 5.2(b) and the FRL attributed to the building element is confirmed in a report from an Accredited Testing Laboratory which:

- certifies that the building element is capable of achieving the FRL despite the minor departures from the tested prototype; and
- describes the materials, construction and conditions of restraint which are necessary to achieve the FRL.

The option to use AS 1720.4 char-based calculation methods to determine the fire resistance is not permitted for fire-protected timber. This is because concerns were expressed with respect to the suitability of the AS 1720.4 approach for certain types of adhesives and connections forming parts of engineered timber products. The proprietary nature of massive timber panel products and lack of standardisation of adhesives and other critical materials used in their construction meant that there was insufficient data available at the time to demonstrate the suitability or otherwise of AS 1720.4.

A3 Resistance to the Incipient Spread of Fire

A3.1 Determine Applicable Resistance to the Incipient Spread of Fire Requirements

The Resistance to the Incipient Spread of Fire (RISF) in relation to a fire-protective covering means the ability of the covering to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

The general requirement for fire-protected timber is an RISF of 45 minutes.

The NCC permits a relaxation to the RISF requirements for fire-protected timber providing both the following additional criteria are satisfied.

- the minimum timber panel thickness is not less than 75 mm
- there are no cavities between the surface of the timber and the fire protective covering or between timber members.

The 75 mm dimension relates to the inherent fire resistance achieved when using a timber panel member. If the relaxation conditions are satisfied, the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria are applicable. Typical examples of massive timber installations satisfying these conditions are shown in Figure 4.3 in the body of this Guide.

Figure A2 shows the process for determining the applicable Resistance to the Incipient Spread of Fire requirements. The general requirement for fire-protected timber is a RISF of 45 minutes.

The relaxed requirements for massive timber construction without voids and cavities is a MRISF that applies a higher interface temperature limit and the time periods for which the temperature limit applies varies according to the application in accordance with Table A1.

Table A1: Modified Resistance to the Incipient Spread of Fire required performance for applications where criteria are relaxed (massive timber construction without voids and cavities).

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)
Inside a fire-isolated stairway or lift shaft	20 min
External walls within 1 metre of an allotment boundary or 2 metres of a building on the same allotment	45 min
All other applications	30 min

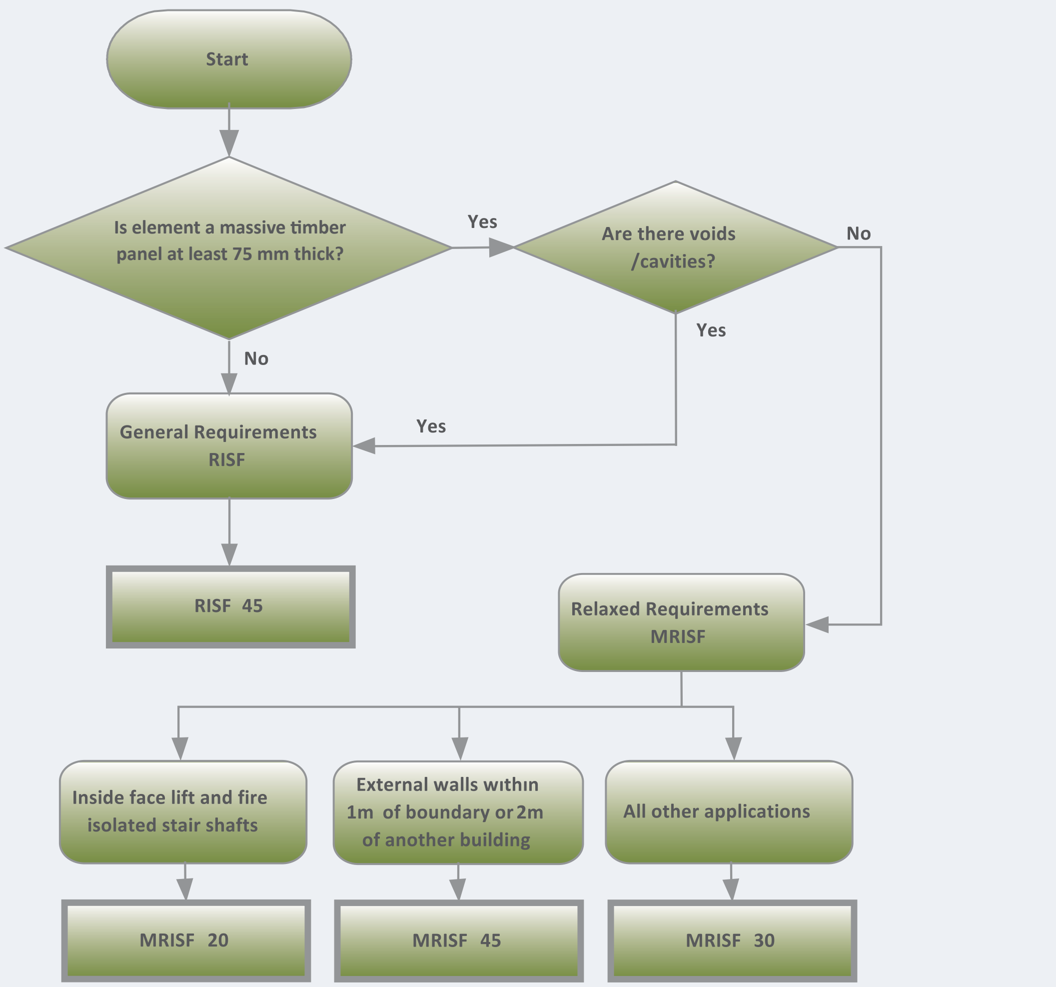


Figure A2: Determination of Resistance to the Incipient Spread of Fire acceptance requirements.

A3.2 Compliance Paths for Resistance to the Incipient Spread of Fire

Three paths are permitted to demonstrate compliance with the RISF requirements;

- simultaneous determination during a full-scale fire resistance test
- smaller-scale fire resistance test (at least 1 metre x 1 metre specimen)
- selection of Deemed-to-Satisfy fire-resisting grade plasterboard coverings.

Simultaneous determination during a full-scale fire resistance test

When a fire resistance test is undertaken to determine the FRL of an element, additional instrumentation can be included in the test to also determine the RISF or MRISF performance, providing a cost-effective approach for new protection systems.

Smaller-scale fire resistance test

There are a large number of systems that have been tested previously to determine their FRLs but in most cases insufficient data will have been recorded to determine the RISF or MRISF performance. Under these circumstances, the use of a smaller specimen (not less than 1 metre x 1 metre) is permitted to obtain supplementary data to determine the RISF or MRISF of the system in a cost effective manner. The fire-protective covering should be fitted in the same manner as that used for the original test that determined the FRL of the system.

Deemed-to-Satisfy Fire-Protective Grade Plasterboard coverings

Specification C1.13 deems fire-protective grade plasterboard facings, if fixed in accordance with the requirements to achieve the required FRL of the element, to also satisfy the requirements for Resistance to the Incipient Spread of Fire (RISF) or Modified Resistance to the Incipient Spread of Fire (MRISF). Table A2 shows the minimum requirements for plasteboard coverings.

Table A2: Fire-protective grade plasterboard coverings Deemed-to-Satisfy RISF requirements.

Requirements	Application	Performance	Minimum Deemed-to-Satisfy fire-protective grade plasterboard
General Requirements	All applications	RISF 45min	2 layers x 13 mm thick
Relaxed requirements for timber panels not less than 75 mm thick without cavities voids or cavities voids filled with non-combustible material	Inside a fire-isolated stairway or lift shaft	MRISF 20 min	1 layer x 13 mm thick
	External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	MRISF 45 min	2 layers x 13 mm thick
	All other applications	MRIFS 30 min	1 layer x 16 mm thick

A3.3 Resistance to the Incipient Spread of Fire (RISF) Test Procedures

The test procedure for determining the Resistance to the Incipient Spread of Fire (RISF) of horizontal elements during a full-scale fire resistance test is provided in Section 4 of AS 1530.4 . Specification C1.13a of the NCC requires the relevant procedures from AS 1530.4 Section 4 to be applied to other elements.

AS 1530.4 requires walls to be full size or not less than 3 m high x 3 m wide and floor/ceiling systems to be full size or not less than 4 m long x 3 m wide. Floor systems are exposed to furnace heating conditions (Figure A3) from the underside and fire-resisting walls are exposed from one side. Asymmetrical walls generally require two tests to evaluate the response to exposure to fire from either side unless the side exposed to fire is specified.

Smaller-scale specimens (not less than 1 m x 1 m) can be used to retrospectively determine the RISF performance of a floor or wall system that has previously achieved the required FRL in a fire resistance test satisfying the minimum size requirements specified in AS 1530.4.

For universal application of results the minimum cavity depth should be fire tested.

To determine the RISF, five thermocouples with insulating pads as prescribed in AS 1530.4 are fixed to the inner face of the fire-protective covering system. They are placed at approximately the centre and the centre of each quarter section as shown in Figure A4.

When testing corrugated specimens, increase the number of thermocouples to six to provide an equal number of thermocouples at the maximum and minimum specimen thickness.

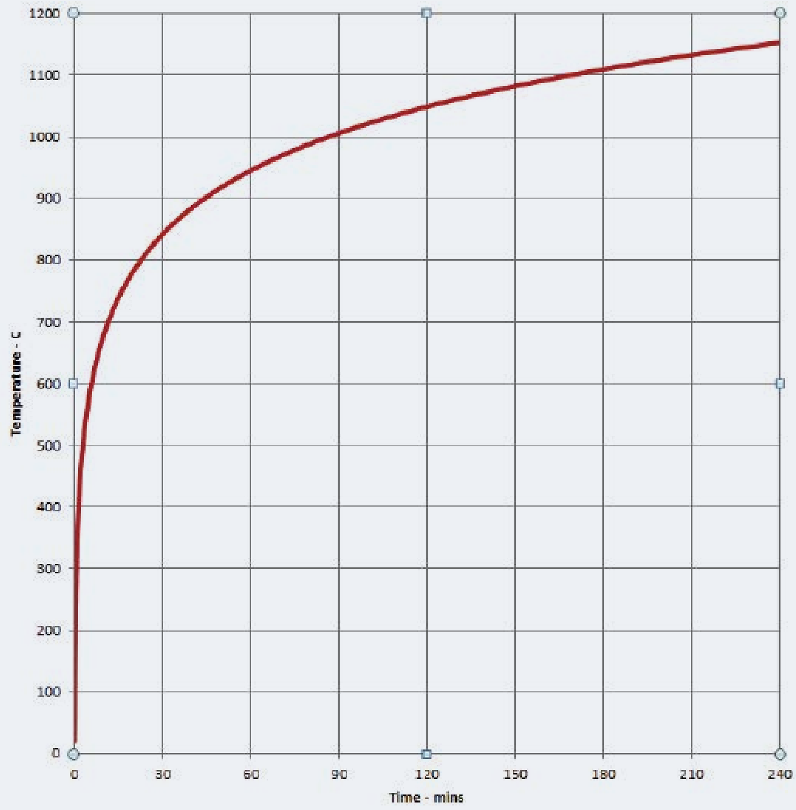


Figure A3: Standard fire resistance test heating regime.

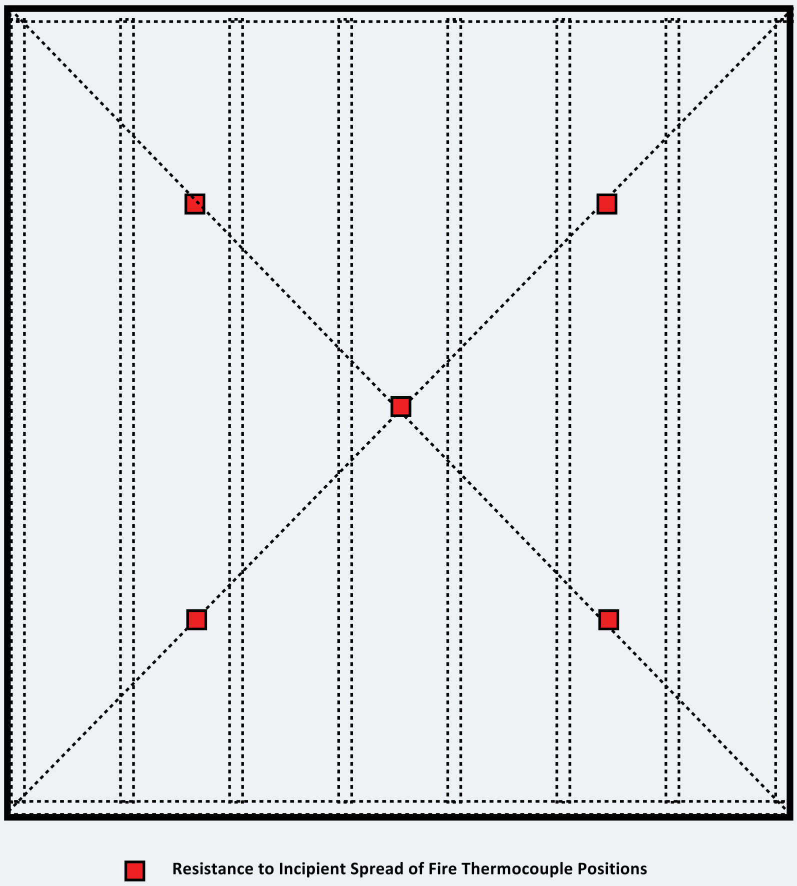


Figure A4: Elevation of a wall showing RISF thermocouple positions.

Sections through typical specimen configurations are shown in Figure A5 to illustrate the correct surfaces to apply thermocouples to determine the RISF. For fire-protected timber, the temperature has to be maintained below the prescribed temperature on the surface of the fire-protective covering facing the void and at the interface with timber elements within the wall or floor. If a wall or ceiling system is protected by a board system, for example, the temperatures are measured on the board surface within the cavity even if non-combustible insulation is applied between the timber studs or beams. However, if the non-combustible insulation forms a continuous layer between the timber elements and the board the thermocouples (t/c) should be applied to the surface of the insulation as shown in Figure A5.

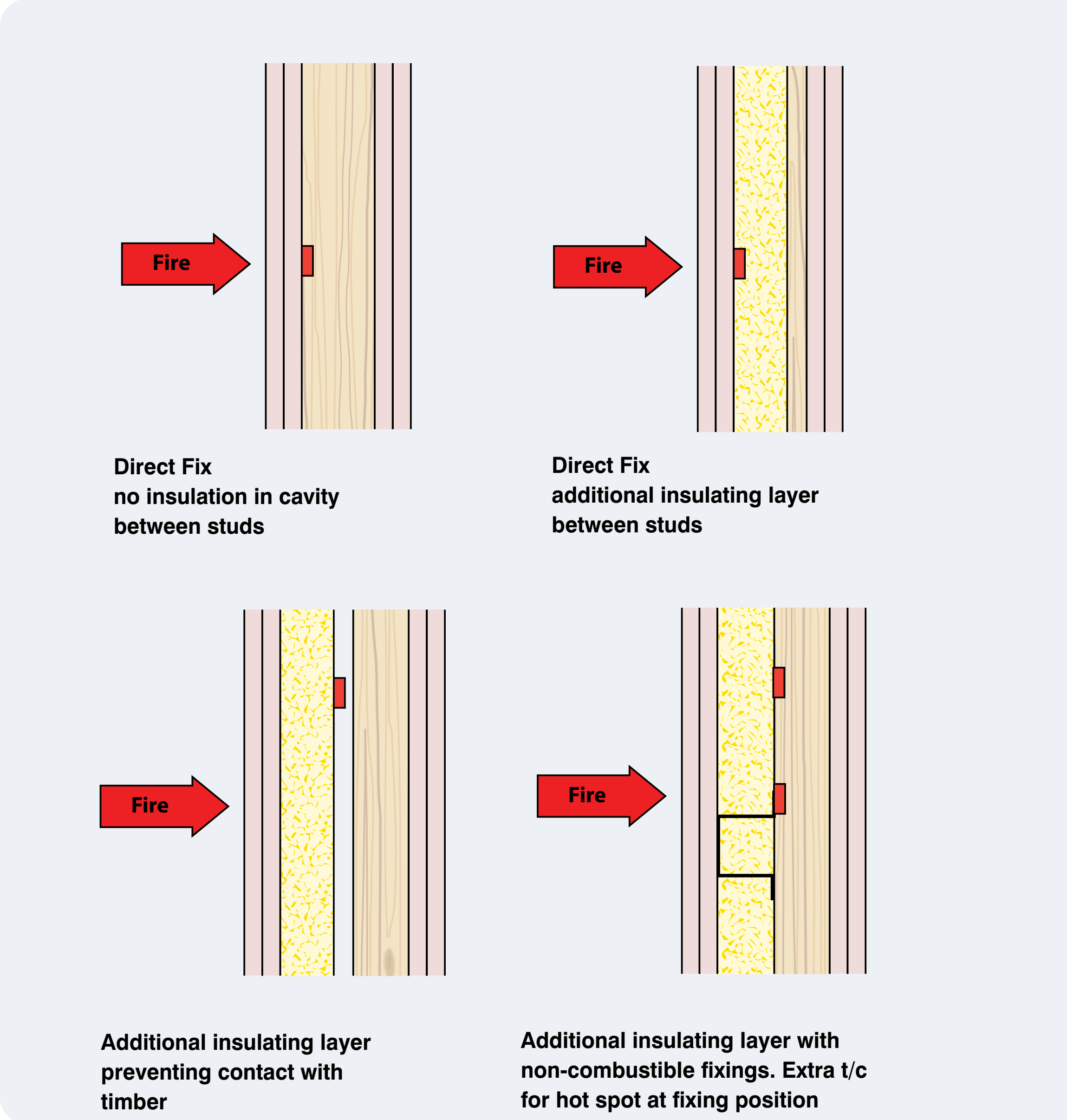
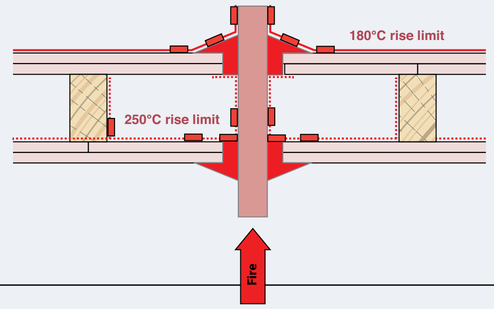


Figure A5: Resistance to the incipient spread of fire thermocouple positions for typical specimen configurations.

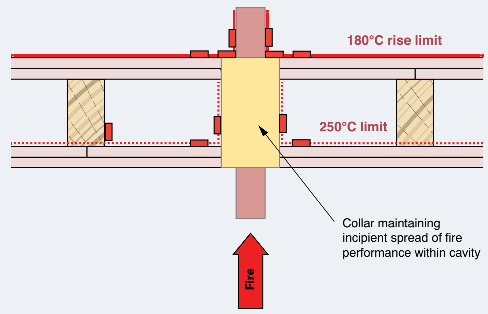
Failure in relation to the RISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 250^{o}C.

Smaller scale specimens 1 m x 1 m can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations are shown in Figure A6. Additional thermocouples are shown to allow the simultaneous determination of the FRL of the service penetration system.

Cable/metal pipe penetration protected with fire-resistant mastic.

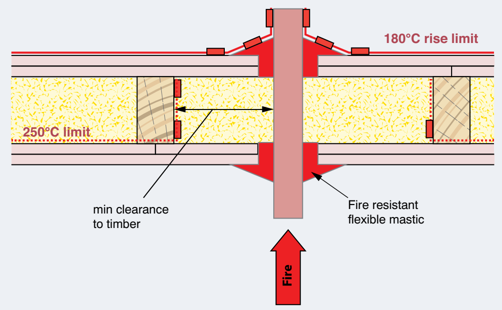


Plastic pipe protected by insulating collar system.



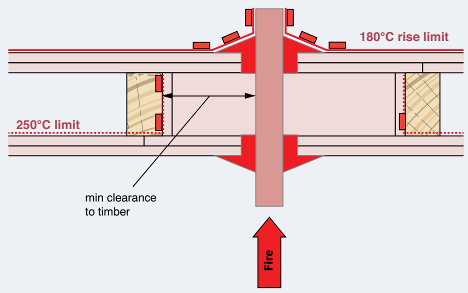
Cable/metal pipe penetration protected with fire resistant mastic and non-combustible cavity infill.

The critical interface for RISF for the service penetration system is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system



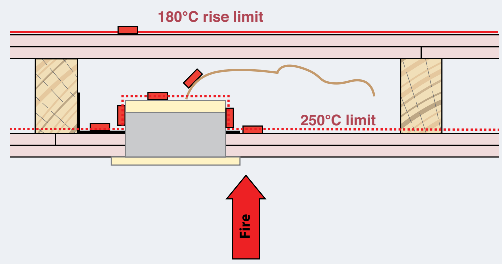
Cable/metal pipe penetration protected with fire-resistant mastic and cavity lined with non-combustible board.

The critical interface for RISF for the service penetration is the surface of the lining board where it is in contact with timber elements.



Proprietary GPO outlet protection system.

Note: Thermocouples applied to cable surface connected to the GPO, on fixing bracket and adjacent element.



GPO outlet with non-combustible cavity infill protection.

The critical interface for RISF is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system

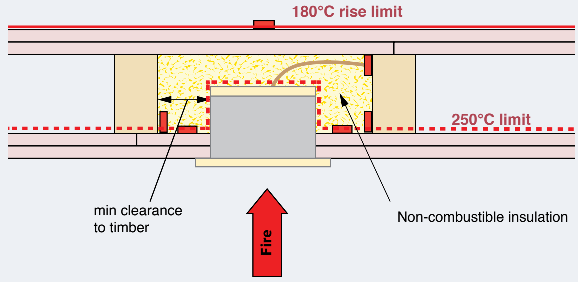


Figure A6: Typical thermocouple positions for determining the RISF of service penetrations.

The thermocouples positions must satisfy the following requirements:

- At not less than two points about 25 mm from the edge of the hole made for the passage of the service.
- Attached to adjacent structural members and those elements that support the penetrating service.
- At points on the surface of the penetrating service or its fire stopping encasement, as follows:
  - at least two thermocouples about 25 mm from the plane of the general surface of the covering and non-combustible insulation
  - where the seal or protection around the service is tapered or stepped, two additional thermocouples beyond the step or the end of any taper if it is expected that the temperatures will be higher at these points.
- Where practicable, at two points on the seal or protection around the service.
- One in the centre of the surface of the penetration nominally parallel to the plane of the fire protective covering if it terminates within the cavity (e.g GPO outlets or down lights).

Failure in relation to the RISF is deemed to occur for the service penetration when the maximum temperature of the thermocouples described above exceeds 250^{0}C.

A3.4 Modified Resistance to the Incipient Spread of fire (MRISF) Test Procedures

The MRISF is applicable to massive timber panels having a thickness not less than 75 mm if there are no voids/cavities through which fire and smoke can spread. The MRISF, amongst other things, relaxes the failure temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures are described in Section 3 of Specification C1.13a of the NCC and are summarised below:

- Tests must be carried out in accordance with AS 1530.4, or an equivalent or more severe test, on the timber element with the proposed non-combustible fire protective coverings fixed in a representative manner.
- Smaller scale specimens (not less than 1m x 1m) can be used to retrospectively determine the MRISF performance of a system that has previously achieved the required fire resistance level in a fire resistance satisfying the minimum size requirements specified in AS 1530.4. If a fire protection system incorporates joints, the test specimens must incorporate representative joints.

To determine the MRISF interface, temperatures must be measured over the following features by a minimum of two thermocouples complying with Appendix C1 and Section 2 of AS 1530.4 as appropriate:

- at joint positions in the protection systems
- at least 200 mm from any joint
- at any other locations where, in the opinion of the Accredited Testing Laboratory, the interface temperature may be higher than the above positions.

Where the fire protective covering is not in contact with the timber (e.g. multi-layer system), the surface of the fire-protective covering is deemed to be the interface.

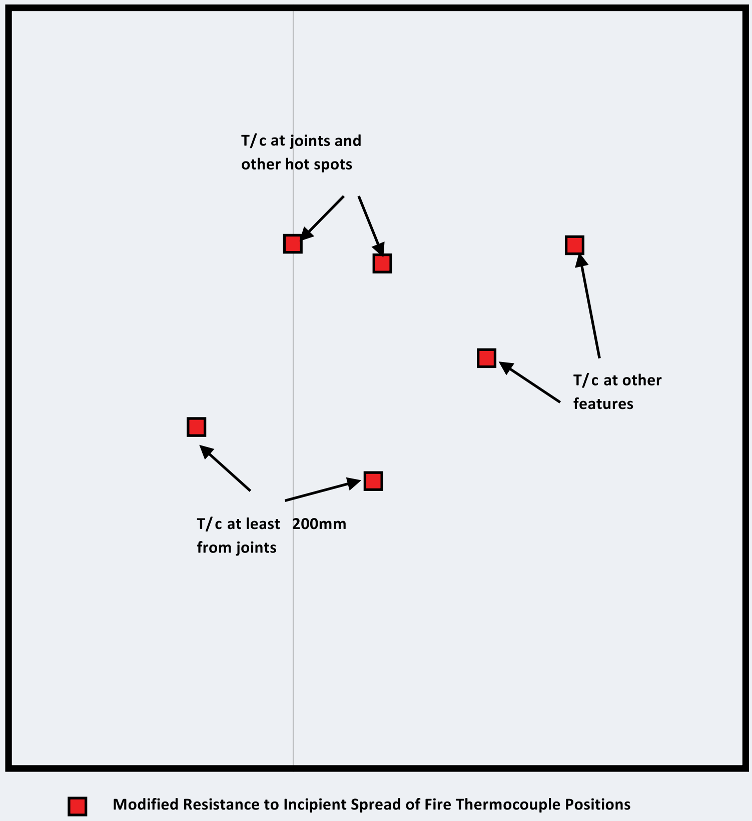


Figure A7: Elevation of a wall showing modified RISF thermocouple positions.

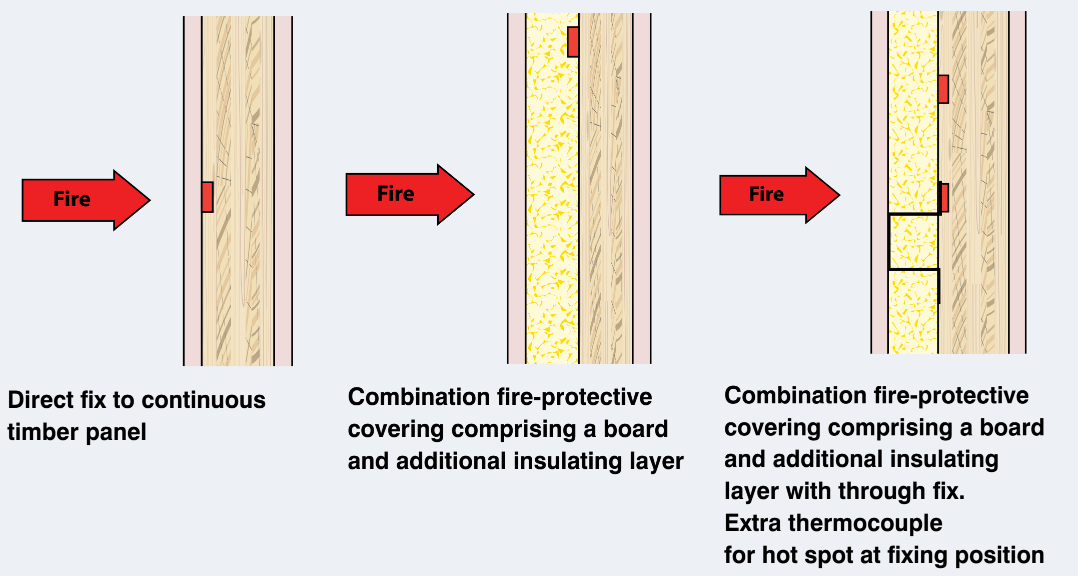


Figure A8: Modified RISF thermocouple positions for typical specimen configurations.

Failure in relation to the MRISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 300°C.

Smaller scale specimens 1 metre x 1 metre can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations to determine both the MRISF and FRLs are shown in Figure A9.

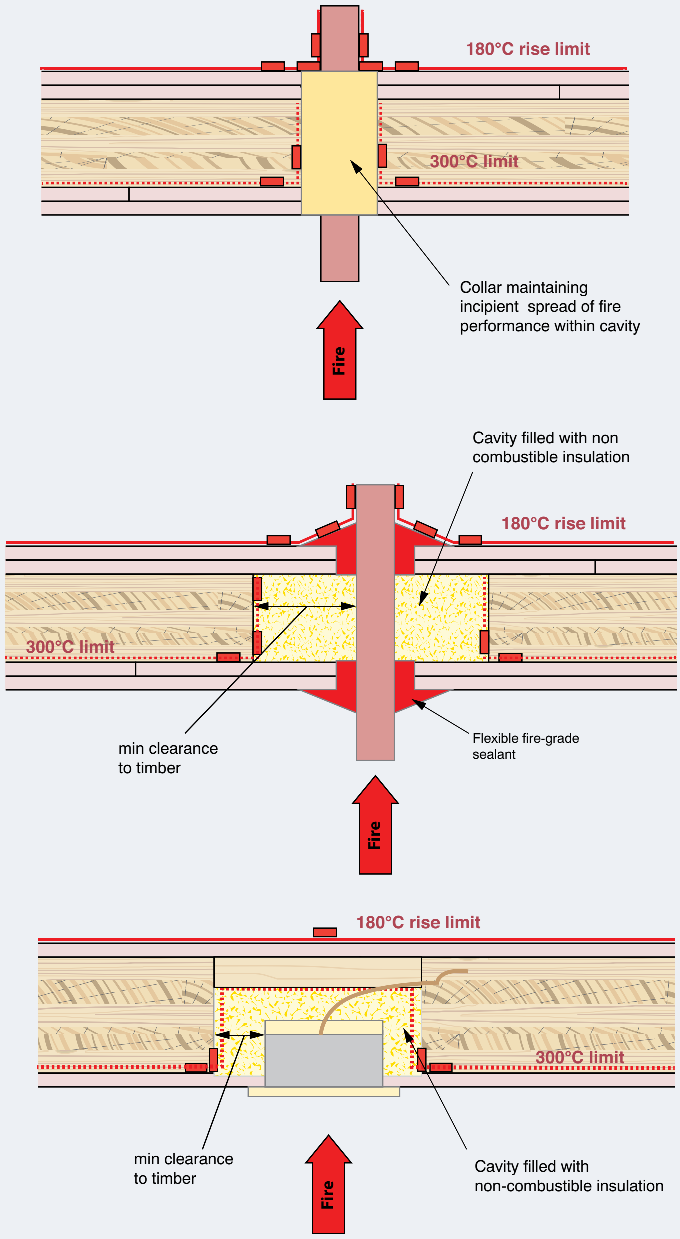


Figure A9: Typical thermocouple positions for determining the MRISF of service penetrations.

B

Appendix B – Determination of the Performance of Cavity Barriers in Fire-Protected Timber Construction

Specification C1.13 of the NCC sets out the requirements for cavity barriers in fire-protected timber construction.

The following compliance options are provided for cavity barriers:

- the cavity barrier system must achieve the FRLs specified in Table B1 when mounted in timber elements having the same or a lower density than the timber members in the proposed application or
- comprise timber of minimum thickness as specified in Table B1 or
- comprise polythene-sleeved mineral wool or non-sleeved mineral wool slabs or strips placed under compression and of minimum thickness as specified in Table B1 or
- another option is that, for cavity barriers around doors and windows, steel frames are also Deemed-to-Satisfy the requirements for cavity barriers provided that the steel frames should be tightly fitted to rigid construction and mechanically fixed. It should, however, be noted that if the windows or doors are of fire-resistant construction, the windows or door system needs to be capable of achieving the required fire resistance when mounted in the wall system, notwithstanding the requirements for cavity barriers.

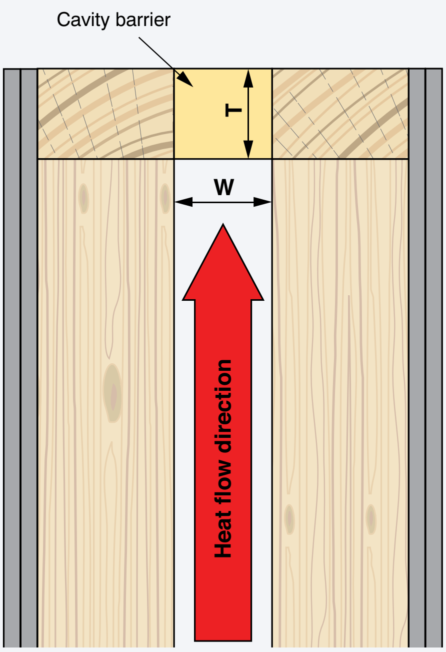
Table B1: Cavity barrier requirements for fire-protected timber.

Cavity Barrier Compliance Options	**FRL required for element cavity barrier is fitted to (minutes)**	
	–/90/90 or less	greater than –/90/90
Cavity Barrier Required FRL – minutes	–/45/45	–/60/60
Timber required minimum thickness	45 mm	60 mm
Mineral wool required minimum thickness	45 mm	60 mm

The minimum thicknesses of protection are required to be measured in the direction of heat flow. The role of a cavity barrier is normally to prevent a fire spreading from the cavity on one side of the cavity barrier to the other. The top plate of a double stud partition (Detail A of Figure B1) is a typical example of this where the direction of heat flow for the cavity barrier would be from the underside to the upper face of the barrier.

The other role for cavity barriers is to reduce the risk of fire spread to cavities occurring around openings for doors and windows within a fire-resisting wall. This configuration is shown as Detail B in Figure B1. For this scenario, the heat flow is from the occupied area of the building through the framing to the cavity. In the Figure, the thickness dimension is identified as 'T'.

Detail A
Cavity barrier at the head of a double stud partition



Detail B
Cavity barrier around door opening

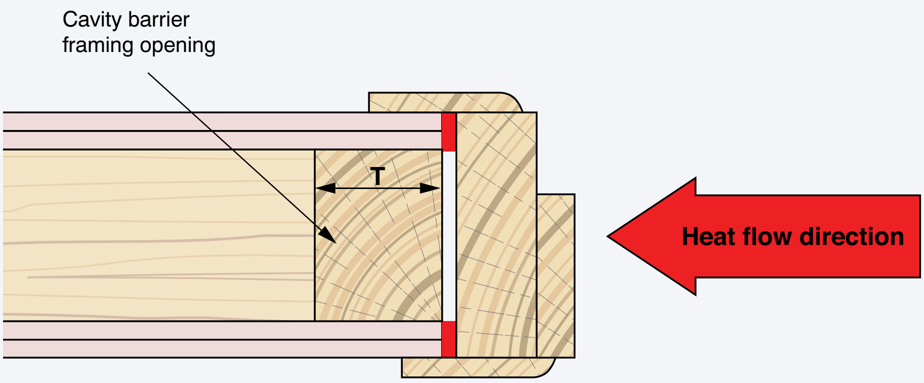


Figure B1: Heat flow direction for cavity barriers.

Proprietary cavity barrier systems may provide more practical options than the Deemed-to-Satisfy solutions for some applications. To encourage the development and use of these systems a compliance path has been provided through the specification of FRLs. For smaller cavity barriers, the performance should be determined by testing the cavity barrier as a control joint system in accordance with Section 10 of AS 1530 using timber members as the separating element. Specification C1.13 permits the results from such a test to be used for applications where the fire-protected timber is constructed from timber with a nominal density at least equal to the tested timber.

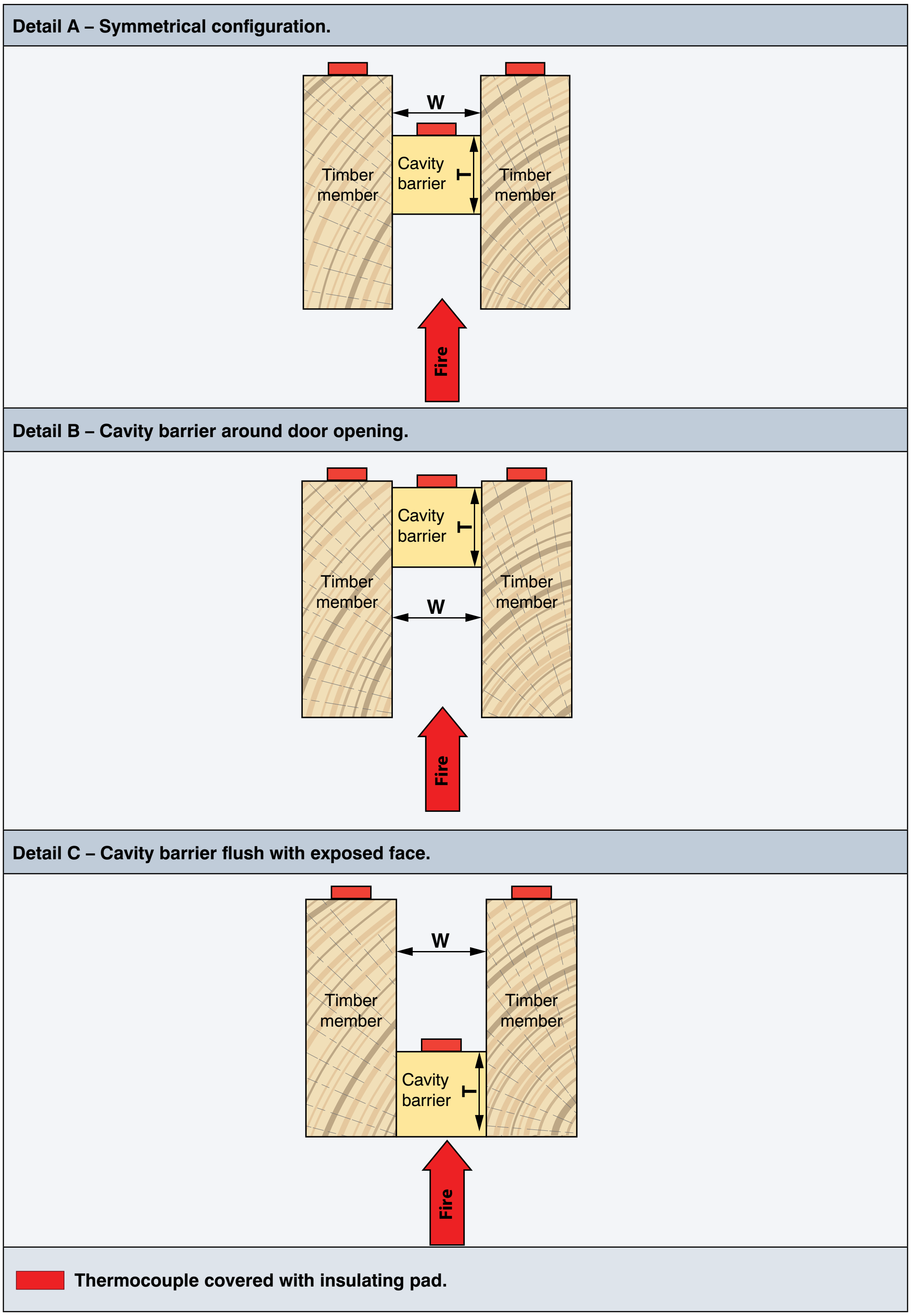


Figure B2: Typical cavity barrier test configurations.

Typical test configurations are shown in Figure B2. The selection of the test configuration(s) depends on how the cavity barrier will be mounted. If it is symmetrical (e.g. fitted at the mid-depth of a timber member), Detail A is appropriate. If the cavity barrier system is not symmetrical both details B and C should be tested unless the most onerous configuration can be determined by the test laboratory or the cavity barrier use is restricted to one configuration. A report from an Accredited Testing Laboratory should state the field of application for the cavity barrier based on the test results.

Cavity barriers can be of combustible construction and therefore a timber framed partition with exposed timber members could be used subject to the wall achieving the required FRL.

In some instances, it may be more practicable to continue the fire-resisting walls up to roof level in lieu of providing a fire-protected timber roof system with cavity barriers. This option is shown in Figure B3.

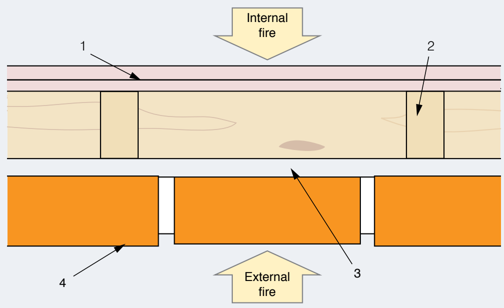
C

Appendix C – Example Data Sheets for an External Wall System

The following data sheet provides an example of the Evidence of Suitability required by the NCC. A brick veneer external wall system has been used because, in addition to fire and sound requirements, thermal resistance, weatherproofing and structural tests apply.

System External Wall 1

External Brick Veneer Timber framed wall system



1 Fire protective grade plasterboard, 2 x 13 mm thick

2.Timber framing in accordance with Evidence of Suitability

3 Cavity. – Cavity insulation may be required to achieve sound ratings and R –value (insulation must be non-combustible)

4 Outer brick veneer 90 mm thick

Typical Performance

Fire-protected timber	FRL90/90/90: RISF45: NC
Sound transmission and insulation	R_w 50: R_w+C_{tr} 50
Thermal resistance	R Value 3.3 m²K/W
Damp and weatherproofing	NCC performance requirement FP1.4
Structural tests	NCC specification C1.8 Clause 3.4

Evidence of Suitability

Fire-protected timber:	
Internal Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 – (e.g.Exova Warringtonfire report 22567A-01) RISF – 45 (NCC Spec C1.13a DTS)
External Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 or design in accordance with AS 3700 RISF – 45 (AS 3700 design for insulation or test or assessment report from an Accredited Testing Laboratory)
Non-combustibility	Plasterboard NCC C1.9(e)(i) DTS Fire-protected timber NCC C1.13 Concession Cavity Insulation AS 1530.1 test report Brickwork –traditional building material

Obtain Evidence of Suitability from product supplier before specifying or installing any product or system

Sound Transmission and Insulation No NCC requirement for external walls in NCC 2019 but commonly specified for inner city locations. Report from a laboratory or acoustics engineer stating performance achieved.

Thermal Resistance R-Value Report complying with NCC Clause A5.2

Weatherproofing Statement of compliance with relevant requirements of AS 3700 and report confirming applicability of AS 3700 – complying with NCC Clause A5.2.

Structural tests for lightweight construction Report complying with NCC Clause A5.2 expressing results of tests in accordance with NCC specification C1.8.

Notes

Selection of systems that are fit for the purpose and the provision of Evidence of Suitability to the satisfaction of the relevant authority is the responsibility of the designers and product suppliers. Forest and Wood Products Australia Limited (FWPA) and the authors of this Guide make no warranties or assurances with respect to the fitness for purpose of the systems described in this Guide.

Ensure installation is in accordance with Evidence of Suitability, manufacturer's instructions and design drawings.

Primary Distributors

Various plasterboard distributors

D

Appendix D: Glossary

National Construction Code (NCC)
National Construction Code Volume One: Building Code of Australia 2019.

Cavity barrier
A barrier placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements, that complies with Specification C1.13 of the NCC, to limit the spread of fire, smoke and hot gases to other parts of the building.

Discontinuous construction
A wall system typically having a minimum of 20 mm cavity between two separate wall frames (leaves) with no mechanical linkage between the frames except at the periphery intended to reduce sound transmission.

Exit
Includes any of the following if they provide egress to a road or open space:

- an internal or external stairway
- a ramp complying with Section D of the NCC
- a doorway opening to a road or open space
- a fire-isolating passageway
- horizontal exit.

Fire-protected Timber
Fire-resisting timber building elements that comply with Specification C1.13a of the NCC.

Fire-protective grade plasterboard
Plasterboard with glass fibre and mineral additives used to improve strength and control shrinkage under fire conditions. Typically a lightweight loadbearing timber framed wall protected by one layer of 16 mm fire-protective grade plasterboard applied to each face would be expected to achieve an FRL of at least 60/60/60 and if protected by two layers of 13 mm fire-protective grade plasterboard on each face an FRL of at least 90/90/90.

Fire-isolated stair or ramp
A stair or ramp construction of non-combustible materials and within a fire-resisting shaft or enclosure.

Fire-isolated passageway
A corridor or hallway of fire-resisting construction that provides egress to a fire-isolated stairway or ramp or to open space.

Fire Resistance Level (FRL)
The time in minutes, determined in accordance with Clause A5.4 (of the BCA) for the following, in order:

- structural adequacy
- integrity
- insulation

Fire-resisting
As applied to a building element means, having the FRL appropriate for that element

Fire-resisting sealant
Fire-grade material used to fill gaps at joints and intersections in fire-protective linings and around service penetrations to maintain Fire Resistance Levels and Resistance to Incipient Spread of Fire performance of elements of construction. Note: The material should also be flexible to allow for movement and where necessary waterproof.

Fire-source feature

- The far boundary of a road adjoining the allotment; or
- a side or rear boundary of the allotment; or
- an external wall or another building on the allotment which is not of Class 10.

Habitable room
A room for normal domestic activities, e.g. bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room and sunroom. Excludes bathroom, laundry, water closet, pantry, walk-in wardrobe, corridor, hallway, lobby, clothes-drying room, and other spaces of a specialised nature occupied neither frequently nor for extended periods.

Internal walls
Walls within, between or bounding separating walls but excluding walls that make up the exterior fabric of the building. Note: Fire walls or common walls between separate buildings or classifications are NOT internal walls.

Lightweight construction
Construction that incorporates or comprises sheet or board material, plaster, render, sprayed application, or other material similarly susceptible to damage by impact, pressure or abrasion.

Massive Timber 'Concession'
A relaxation allowing the Resistance to Incipient Spread of Fire requirements for fire-protected timber to be modified if both the following conditions are satisfied:

- the timber is at least 75 mm thick
- any cavity between the surface of the timber and the fire-protective covering is filled with non-combustible materials.

Massive Timber Panels
Large engineered wood panels of minimum thickness of 75 mm thick. Typical examples include Cross-laminated Timber (CLT), Laminated Veneer Lumber (LVL) and Glulam panels.

Modified Resistance to the Incipient Spread of Fire (MRISF)
The MRISF, amongst other things, relaxes the RISF limiting temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures for MRISF are described in Section 3 of Specification C1.13a of the NCC.

Multi-service penetration system
A service penetration system used to protect a group of services penetrating a single opening in a fire-resisting element such that the FRL, RISF or MRISF of the element is not reduced. Note: Fire protective coverings or other means may be required to be fitted around the opening to ensure that the RISF or MRISF are not reduced.

Performance Requirements
The requirements in the NCC that describe the level of performance expected from the building, building element or material.

Resistance to the Incipient Spread of Fire (RISF)
The ability of a fire-protective covering to insulate voids and the interfaces with timber elements to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

Sole-Occupancy Unit (SOU)
A room or other part of a building for occupation by one or joint owner, lessee, tenant, or other occupier to the exclusion of any other owner, lessee, tenant, or other occupier and includes:

- a dwelling (e.g. apartment)
- a room or suite of rooms in a Class 3 building which includes sleeping facilities
- a room or suite of associated rooms in a Class 5, 6, 7, 8 or 9 building
- a room or suite of associated rooms in a Class 9c building, which includes sleeping facilities and any area for the exclusive use of a resident

References

WoodSolutions Technical Design Guides

#1 Timber-framed Construction for Townhouse Buildings Class 1a

#2 Timber-framed Construction for Multi-residential Buildings Class 2 and 3

#3 Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b

#4 Building with Timber in Bushfire-prone Areas

#5 Timber service life design – Design guide for durability

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)

#17 Alternative Solution Fire Compliance, Timber Structures

#18 Alternative Solution Compliance Facades Forest and Wood Products Australia Ltd 2015

#20 Fire Precautions during Construction of Large Buildings

#37R Mid-rise Timber Buildings Multi-Residential Class 2 and 3

#39 Robustness in Structures

#38 Fire Safety Engineering Design of Mid-Rise Buildings

#44 CLT Acoustic Performance

Other:

R-values for Timber-framed Building Elements

Australian Standards

AS 2118.1 Automatic fire sprinkler systems – General requirements

AS 2118.4 Automatic fire sprinkler systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height

AS 2118.6 Automatic fire sprinkler systems – Combined sprinkler and hydrant systems in multi-storey buildings

AS 1170 series – Structural design actions

AS 1720.1 Timber structures – Design methods

AS 5113 Amd 1 Classification of external walls of buildings based on reaction to fire performance

AS 1905.1 Components for the protection of openings in fire-resistant walls – Fire-resistant doorsets

AS 1530.1 Methods for fire tests on building materials, components and structures - Combustibility test for materials

AS 1530.4 Methods for fire tests on building materials, components and structures - Fire-resistance tests for elements of construction

AS 4072.1 Components for the protection of openings in fire-resistant separating elements – Service penetrations and control joints

AS 2444 Portable fire extinguishers and fire blankets – Selection and location

AS 1682.1 Fire, smoke and air dampers – Specification

AS 1682.2 Fire, smoke and air dampers – Installation

Other References

National Construction Code Volume One: Building Code of Australia 2019 – Australian Building Codes Board, Canberra ACT – © Commonwealth of Australia and the States and Territories 2019

International Fire Engineering Guidelines (2005) – Australian Government, Sate and Territories of Australia

Safe Design of Structures – Safe Work Australia

Exova Warringtonfire Australia Pty Ltd (EWFA) Regulatory Information Reports (RIR) issued to Forest & Wood Products Australia:

RIR 22567A-04 – The fire resistance performance of timber-framed walls lined with plasterboard if tested in accordance with AS1530.4-2005

RIR 37600400 – The fire resistance level (FRL) of timber-framed floor/ceiling systems incorporating timber and metal web floor trusses or various engineered joists when tested in accordance with AS 1530.1-2014

RIR 37401400 – The fire resistance level (FRL), Resistance to the Incipient Spread of Fire (RISF) and Modified Resistance to the Incipient Spread of Fire (MRISF) performance of various timber-framed and massive timber panel systems

RIR 55945800.1B – The fire resistance performance of timber framed walls lined with 3 layers of 16mm fire protective grade plasterboard if tested in accordance with AS 1530.4-2014 for 180 minutes

WF report FAS190034-RIR 1.0 - Timber-framed floor / ceiling systems incorporating various timber and metal web floor trusses or engineered joists with an FRL of 120 / 120/120.

37H

Mid-rise Timber Buildings
Healthcare
Class 9a and 9c

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

WoodSolutions design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-43-0

Researchers

Paul England
EFT Consulting
PO Box 1288, Upwey, VIC 3158
paul.england@eftconsult.net

Boris Iskra
TPC Solutions (Aust) Pty Ltd
Level 6, 10-16 Queen Street

Published: July 2021

Table of Contents

1

Introduction

This document provides detailed guidance in relation to the design of mid-rise fire protected timber healthcare buildings but will also be a useful resource if other forms of construction are adopted. For example, the protection of service penetration and maintenance of penetration seals through the life of a health-care building has always been a major challenge. Practical designs have been developed that can be adapted to all forms of construction to simplify installations and subsequent maintenance of fire protection systems. As for all forms of construction it is critical to develop detailed documentation and ensure evidence of suitability is available during the design stage to avoid unnecessary complications during construction and commissioning

1.1 Scope

The National Construction Code (NCC) Volume One, Building Code of Australia 2019 Amendment No. 1 2020 allows the use of fire-protected timber construction using the Deemed-to-Satisfy (DTS) pathway for all buildings up to 25 metres in effective height ('mid-rise construction', see Figure 1.1).

The DTS pathway provisions cover both traditional 'lightweight timber framing' and 'massive timber' products, such as Cross-laminated Timber (CLT) and Laminated Veneer Lumber (LVL), in conjunction with the use of appropriate non-combustible fire-protective coverings – termed 'fire-protected timber' in the NCC – and appropriate compliant automatic sprinkler systems, among other things.

Low-rise timber healthcare buildings
are buildings of:

- Type B or C construction (1 or 2 storey)

Mid-rise timber healthcare buildings
have an effective height
of not more than 25 metres.}
Typically, they are 3-8 storeys high
(the maximum number of storeys depends
on the floor-to-floor height)

High-rise timber healthcare buildings
have an effective height
greater than 25 m.

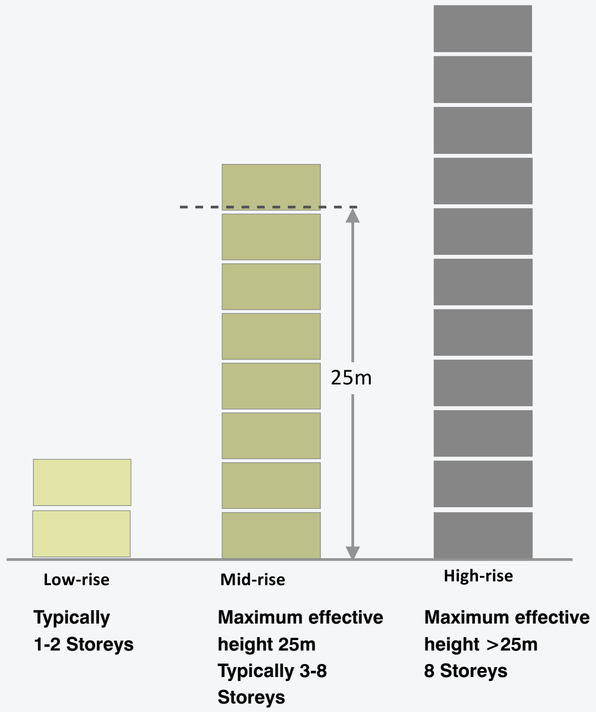


Figure 1.1: Low, mid and high-rise Class 9 healthcare buildings.

This Guide applies to mid-rise NCC Class 9a and Class 9c healthcare buildings or parts of buildings that are to be constructed using fire-protected timber in accordance with the DTS pathway.

The NCC defines a healthcare building as a building whose occupants or patients are undergoing medical treatment and generally need physical assistance to evacuate the building during an emergency and include:

- a public or private hospital
- a nursing home or similar facility for sick or disabled persons needing full-time care
- a clinic, day surgery or procedure unit where the effects of the predominant treatment administered involve patients becoming non-ambulatory and requiring supervised medical care on the premises for some time after the treatment.

Healthcare buildings can be classified under the NCC as Class 9a, Class 9c or Class 3 (refer Note 1 and Note 3).

Class 9a – A building that is a healthcare building including any parts of the building set aside as laboratories and may include a healthcare building used as a residential care building (e.g. a hospital).

Class 9c – A building that is a residential care building.

A residential care building is defined as a place of residence where 10% or more of persons who reside there need physical assistance in conducting their daily activities and to evacuate the building during an emergency, including any aged care building or residential aged care building, but does not include a hospital.

Note 1: Some residential care buildings providing long-term or transient accommodation for the aged, children, or people with disabilities may be classified as a Class 3 building; particularly if full-time care is not provided. Reference should be made to WoodSolutions Technical Design Guide #37R for further information if the building has been classified as Class 3.

Note 2: Buildings or parts of buildings used as consulting suites by medical practitioners within an office setting, may be considered as Class 5 buildings under some circumstances. Reference should be made to WoodSolutions Technical Design Guide #37C for further information if the building has been classified as Class 5.

Note 3: The determination of the appropriate Class for a healthcare building can be complex and open to interpretation. It is critical that the classification is defined by the appropriate authority at the start of the project as it may have a significant impact on the design. Healthcare buildings can be subject to change and allowance for future flexibility can be important including the choice of a more onerous classification for various parts of the building.

1.2 Design of Healthcare Buildings

Healthcare buildings vary substantially with respect to size and complexity. For example:

- Hospitals can vary from small single-storey rural hospitals with small numbers of beds and limited or no emergency facilities through to multi-storey city hospitals with large numbers of beds providing a broad range of health services and typically include restaurants, shops, carparks as well as facilities such as laboratories to support healthcare services. Some ward areas may require extensive security provisions, further complicating the design.
- Residential care facilities can vary from small single-storey facilities to multi-storey facilities housing large numbers of residents requiring various levels of support and healthcare.

It is critical that the building designs are developed to satisfy the specific needs for each healthcare facility and that all members of the design team have a detailed understanding of the operational needs of the facility and the various drivers and constraints that apply to the specific project.

Various National and State guidance documents have been produced relating to the design of healthcare buildings to clarify key drivers and constraints and in some cases specify additional administrative processes which should be adopted as appropriate.

It is also good practice to involve key stakeholders and the design team in a Design Brief process early in the project to ensure all drivers and constraints have been identified, and highlight essential criteria before the design is prepared. This is important as there may be additional criteria that may not be fully addressed by compliance with the minimum NCC requirements. Figure 1.2 shows a typical process. The proposed design will need to be assessed for compliance with the NCC and other project drivers and constraints. The primary focus of this Guide is NCC compliance via the Deemed-to-Satisfy pathway with the fire safety provisions.

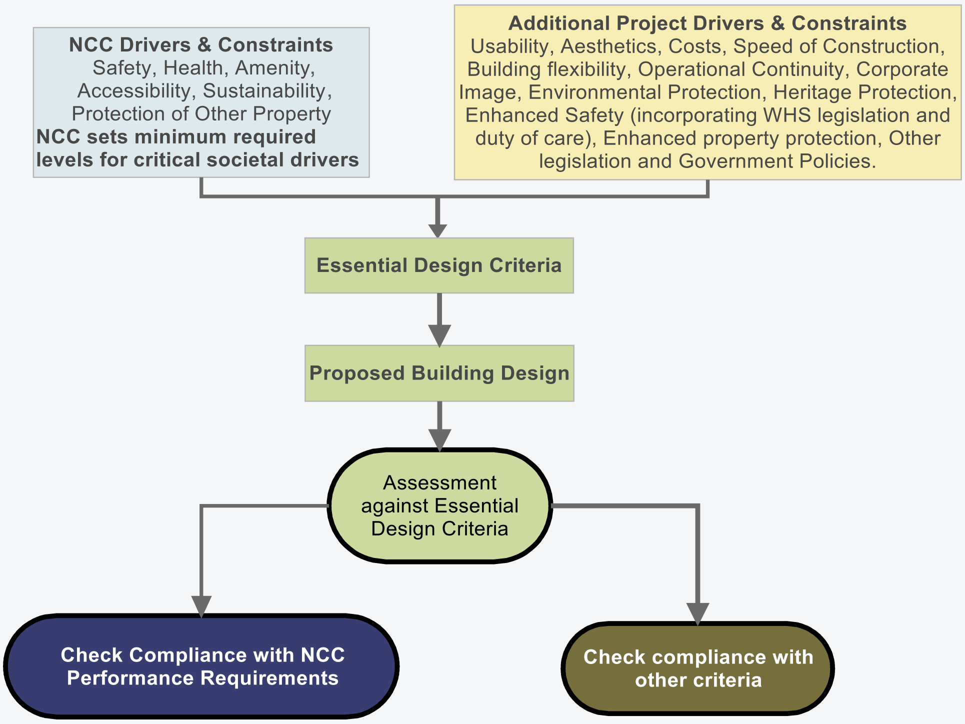


Figure 1.2: Identification of essential design criteria.

Details of compliance checks for other project design criteria such as those described below are not discussed in detail.

1.2.1 Robust Structural Design

The structural design of mid-rise timber buildings must comply with the relevant NCC requirements, including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1(a)(iii).

Further guidance is provided in WoodSolutions Technical Design Guide #39 Robustness in Structures.

1.2.2 Fire Precautions During Construction

Mid-rise timber buildings, when complete, provide a high level of safety because of the combination of automatic fire sprinklers and fire-protected timber, among other things.

While the use of timber significantly reduces a number of risks during construction, the fire risk can be increased as a result of the increased volumes of unprotected timber. WoodSolutions Technical Design Guide #20 Fire Precautions during Construction of Large Buildings helps building professionals, and organisations with responsibilities for fire safety on a construction site, reduce the risk of fire.

There are additional matters that must be addressed to satisfy work, and health and safety requirements in addition to compliance with minimum NCC DTS provisions for fire precautions during construction – refer NCC Clause E1.9 and Australian State and Territory guidelines.

1.2.3 Other Design Criteria

Designers need to take account of a broad range of matters to ensure that a building is fit-for-purpose and complies with all requirements of the NCC and other legislation. These include:

- structural design (for safety and serviceability)
- weatherproofing
- safe access and egress
- light and ventilation (including condensation control)
- energy efficiency
- durability (including termite management)
- design in bushfire-prone and flood-prone areas.

Some sources of information on these matters are referenced in the Appendices of this Guide.

1.3 Checking Compliance with the National Construction Code (NCC)

To comply with the NCC, it must be demonstrated that the NCC's Governing Requirements and Performance Requirements have been satisfied.

The Governing Requirements are documented in Section A of the NCC and provide rules and instructions for using and complying with the NCC including:

- interpreting the NCC
- complying with the NCC
- application of the NCC in States and Territories
- applying documents referenced in the NCC
- documenting the suitability of the design, construction and/or use of materials to comply with the NCC
- classifying buildings by their characteristics and intended use.
- The performance requirements can be satisfied by means of a Performance Solution or a Deemed-to-Satisfy Solution or a combination (see Figure 1.3).

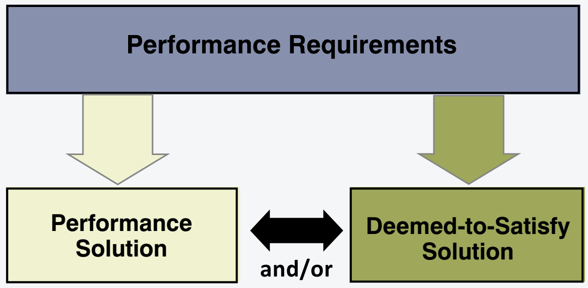


Figure 1.3: Pathways for demonstrating compliance with NCC performance requirements.

Figure 1.3 is a derivative of Figure 1 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au****/ Australian Building Codes Board © 2020.*

The focus of this Guide is the Deemed-to-Satisfy Pathway but some of the content may have relevance to Performance Solutions. For example whilst many Class 9a and 9c buildings may be designed generally in compliance with a Deemed-to-Satisfy Solution there may be some variations to these provisions to provide the necessary functionality. Reference should be made to other WoodSolutions publications and Design Guides, and Australian Building Codes Board publications for further advice in relation to developing Performance Solutions.

1.4 Checking Interpretations of Regulations and Standards

This Guide focuses on NCC requirements. From time-to-time, State and Territory-based NCC amendments or other State legislation and department policies may vary requirements. Users of this Guide should make themselves aware of these differences and develop a full understanding of the resulting implications. It is prudent to check any interpretations of the regulations and required evidence of suitability with the relevant regulatory authorities during the early stages of the design process.

1.5 Guide Structure

An overview of Timber Construction Options is provided as a general introduction to typical forms of timber construction.

A 6-step design process for sound and fire-resisting construction has been followed to broadly follow the development and assessment of a design against the NCC DTS provisions.

Step 1 High-level NCC design Issues (schematic design)

Step 2 Determine NCC design requirements for sound, thermal resistance, weatherproofing and structural tests

Step 3 Improve and upgrade sound performance

Step 4 Determine NCC fire-protected timber design requirements (design development)

Step 5 Integrate architectural, structural and building service designs (detailed design)

Step 6 Further design assistance (Appendices)

2

Timber Construction Options for Mid-rise Buildings

2.1 Overview of Structural Timber Construction Methods

A number of timber system options are available for the construction of mid-rise timber buildings, such as those shown in Figure 2.1. Note: Under the NCC DTS provisions, only fire-protected timber building systems are permitted where an element is required to be of non-combustible construction or of masonry or concrete construction.

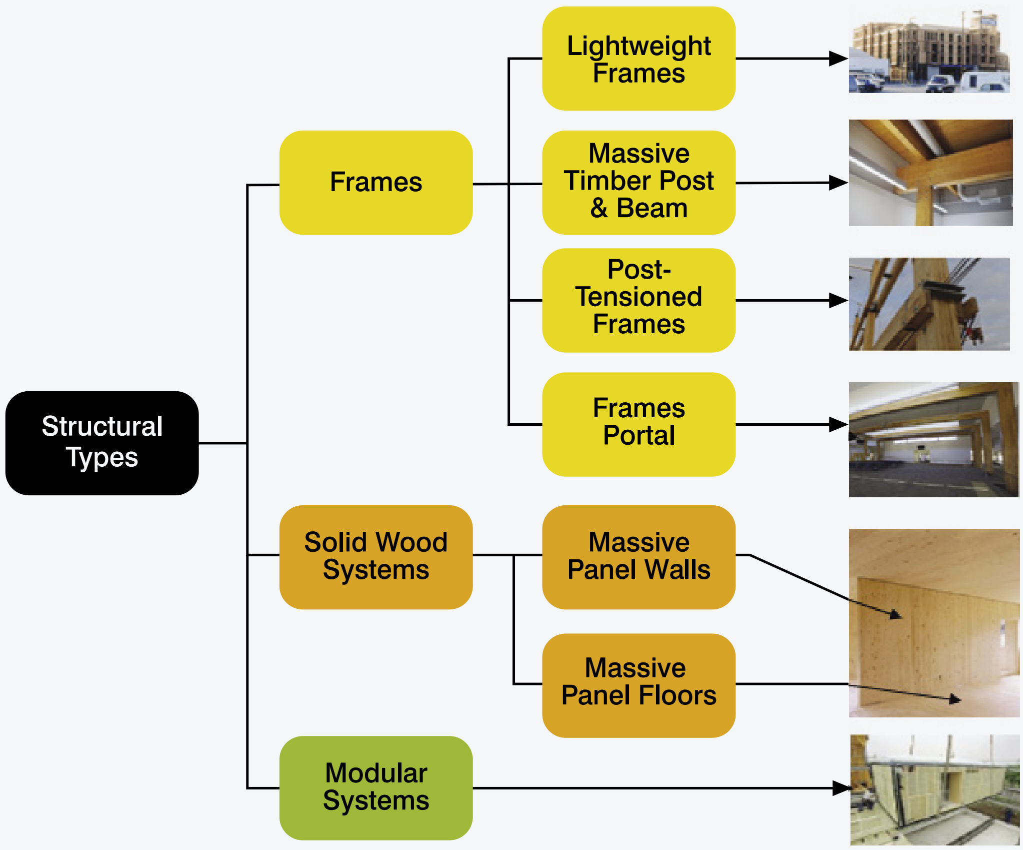


Figure 2.1: Summary of structural timber construction options

No exposed structural timber is currently permitted under Deemed-to-Satisfy Provisions where an element must be of non-combustible construction.

Loadbearing fire-protected timber-framed construction is most suited to applications where individual rooms have closely spaced walls that, in effect, form a 'honeycombed' structure; with many individual load paths. Lightweight timber-framed systems (up to around six storeys) combined with fire protective coverings are an efficient form of construction. Alternatively, a solid massive timber system, or a mixture of massive and lightweight timber construction, might be considered. Fire-protected timber-framed construction is also useful for non-loadbearing elements where smoke or fire-resistant separations are required.

For larger open-plan spaces, a post and beam approach for the overall structure may be more appropriate. Typically, the main columns and beams might be constructed using glued-laminated timber (Glulam) or laminated veneer lumber (LVL) with floors being either lightweight prefabricated cassettes or solid massive timber panel floor plates. Refer Figure 2.2 for a comparison of timber frame and post and beam construction.

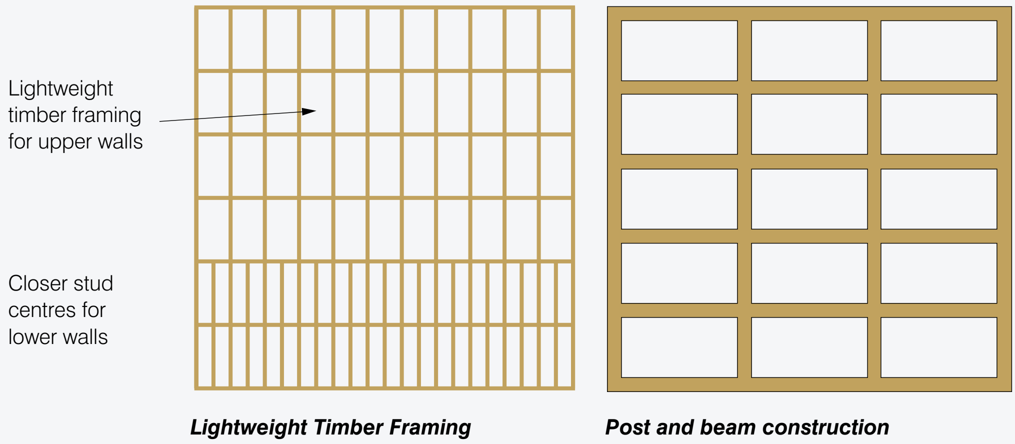


Figure 2.2: Schematic comparison of timber frame and post and beam construction.

There are significant efficiency, speed and cost benefits in using structural timber systems compared to alternative material such as reinforced concrete. These include:

- Reduced on-site construction infrastructure (preliminary costs) such as fixed cranes, site accommodation, storage areas, scaffolding and edge protection, hoists, etc.
- Direct savings from faster methods of construction compared to traditional steel and concrete structures due to:
  - increased scope for off-site prefabrication and panelisation
  - lighter and more easily manoeuvred and installed materials.
- Reduced foundation requirements due to a lighter above-ground structure.
- Significantly reduced on-site costs and Work Health and Safety (WHS) issues, particularly with a shift to more prefabricated solutions.
- Increased ability to commence follow-on trades earlier in the construction process, reducing the overall construction time.
- Increased accessibility of the construction site and significantly lower impacts of noise and site activities on local neighbourhoods (less truck movements), which is a major benefit for suburban multi-residential developments.

Detailed information on the specific construction cost benefits of timber systems in different classes of buildings can be found in various WoodSolutions Technical Design Guides, including *#28 Rethinking Aged Care Construction – Consider Timber – A material cost comparison of typical aged care accommodation.*

2.2 Fire-protected Timber Options for Mid-rise Buildings

Whichever timber construction option is selected, the prescriptive Deemed-to-Satisfy (DTS) solutions for mid-rise timber buildings require timber members to be fire-protected where an element is required to be of non-combustible construction, or of concrete or masonry construction.

The ‘general timber’ requirements that apply for fire-protected timber are:

- the building element must be protected to achieve the required FRL
- a non-combustible fire-protective covering must be applied to the timber that achieves a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4
- cavity barriers must be provided in accordance with relevant NCC requirements.

The NCC permits a ‘relaxation’ to the general requirements in the case of fire-protected massive timber panels if the following additional criteria are satisfied:

- the timber panel is at least 75 mm thick
- any cavities between the surface of the timber and the fire-protective covering and between timber elements within the fire protective covering are filled with non-combustible materials.

If both these conditions are satisfied, it is still necessary for the fire-protected timber member to achieve the required FRL and have a non-combustible fire-protective covering. However, the thickness of the fire-protective coverings, based on the covering's RISF performance and required FRL, can be modified depending on the application (e.g. internal SOU wall, external wall).

The basis for allowing specific provisions for massive timber panels is that timber with a large cross-section can achieve high fire-resistance due to the formation of a char layer that protects the timber core and allows it to continue to support an imposed load or maintain a fire-separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for longer than lighter forms of construction. By not permitting any concealed spaces between the massive timber members, or between the timber and fire-protective coverings, the risk of fire spread is reduced.

The different timber construction systems generally use one or more of a range of different sawn or engineered timber products, including:

- sawn timber – softwood (MGP) and hardwood (F- and A-grade)
- engineered timber – particleboard, plywood, Oriented Strand Board (OSB), Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL), I-beams, fabricated floor and roof trusses, Glued Laminated Timber (Glulam) and Cross-laminated Timber (CLT).

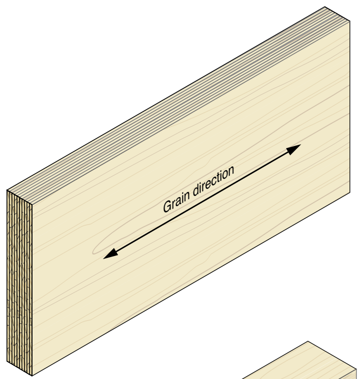
2.3 Timber Frame Construction

Lightweight timber-frame construction systems use commonly available structural timber framing products assembled into lightweight systems such as wall frames, floor and roof trusses, and prefabricated cassette floor modules.

Sawn Timber Products

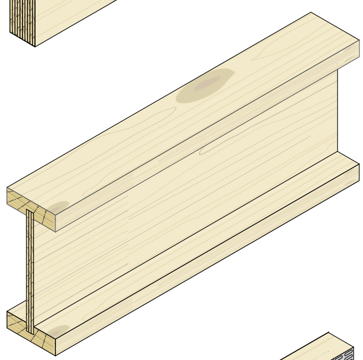
Sawn timber products include seasoned structural softwood (MGP10, 12 & 15) or seasoned structural hardwood (typically F17 or F27). Typical thicknesses are 35 and 45 mm and depths include: 70, 90, 120, 140, 190, 240* and 290* mm. (* available on order)

Laminated Veneer Lumber (LVL)



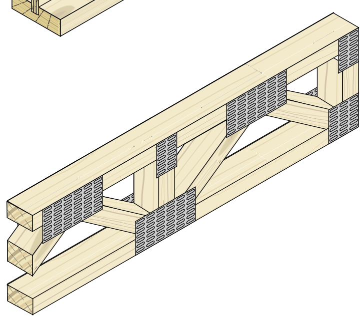
Lightweight framing elements are available in LVL, a widely used softwood engineered wood product, available in all the standard framing sizes. LVL is manufactured by bonding together rotary peeled or thin-sliced wood veneers under heat and pressure. As LVL is typically used in a beam or stud application, the grains of the veneers are all oriented in the same direction. LVL is typically manufactured in slabs 1200 mm wide, known as billets, which are then cut into the commonly available framing member depths required. LVL is typically manufactured in lengths up to 12 metres in 0.3 metres increments.

I-Beams



I-Beams are lightweight, high-strength, long-span structural timber beams. They typically comprise top and bottom flanges of LVL or solid timber – which make the distinct shape. The flanges are separated by a vertical web, usually manufactured from structural plywood, Oriented Strand Board (OSB) or light gauge steel. Typical depths are: 200, 240, 300, 360 and 400 mm in lengths up to 15 metres.

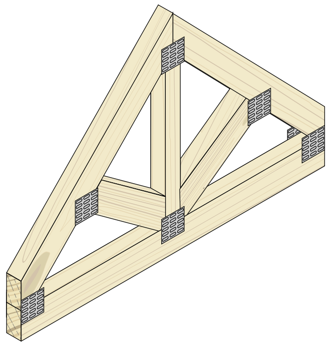
Parallel Chord Trusses



Parallel Chord Trusses are similar to I-beams in that they have top and bottom chords (flanges) of LVL or solid timber but instead of solid webs, web struts are used.

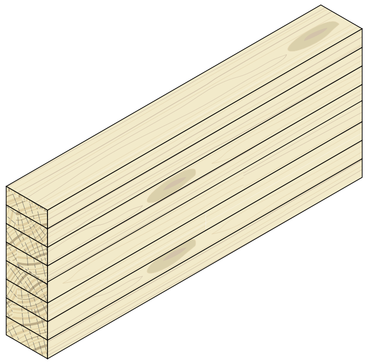
The struts may be either timber or light gauge steel and are secured to the chords typically with nailplates. The struts may be diagonal (more common for steel struts) or a mix of vertical and diagonal (more common with timber struts).

Roof Trusses



Timber roof trusses provide an engineered roof frame system designed to carry the roof or roof and ceiling, usually without the support of internal walls. The characteristics of a roof depend on the purpose of the building, the available roofing materials and the wider concepts of architectural design and practice. Light truss roofs are formed from sawn or LVL timber elements connected with nailplates or other mechanical fixings designed and supplied by frame and truss manufacturers.

Glued-laminated Timber (Glulam)



Glulam consists of a number of strength-graded, kiln-dried laminations face bonded and finger-jointed together with adhesives. Elements can be manufactured to practically any length, size or shape: beams are often manufactured with a built-in camber to accommodate dead load deflection or curved for aesthetic appeal.

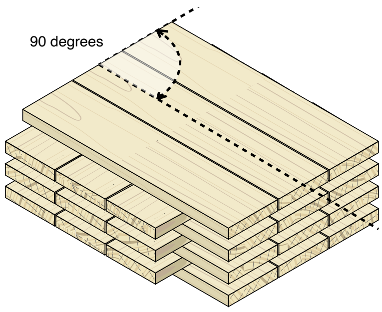
A range of GL Grades are produced in Australia or imported depending on the timber species used in manufacture: GL10 (Cypress), GL13 (Radiata Pine, Oregon), GL17 (Slash Pine, Merbau), GL18 (Tas Oak, Vic Ash), GL21 (Spotted Gum) – the GL descriptor refers to the element's Modulus of Elasticity (E), i.e. GL10 describes a Glulam member that has an E-value of 10GPa.

A wide range of depths are available in increments from 90 mm to more than 1,000 mm; and thicknesses from 40 mm to 135 mm (65 mm and 85 mm are commonly used). Lengths up to 18 metres are available in 0.3 metre increments from traditional suppliers and up to 27 metres in length from specialist manufacturers.

2.4 Solid Massive Wood Panel Construction Systems

Solid wood panel construction systems utilise massive timber engineered wood panels such as Laminated Veneer Lumber, Cross-laminated Timber (CLT), or Glue/nail-laminated Mass Timber panels, in minimum panel thicknesses of 75 mm when used in accordance with the NCC DTS provisions. Solid wood panels can be used to form complete floors, walls and roofs and construction methods have more in common with precast concrete panels than timber framing; except that timber panels are much lighter, more easily worked and easier to erect.

Cross-laminated Timber (CLT)



Cross-laminated Timber (CLT) utilises individual planks of timber 12-45 mm thick and 40-300 mm wide face-glued together (and edge-glued in some instances), each layer at 90° to its neighbouring lamella – effectively 'jumbo plywood'. CLT panels are typically 57-320 mm thick and made up of 3, 5, 7 or 8 layers depending on application. Panels are available in 2.2 to 2.95 metres wide and up to 11.9 metres long (dictated mainly by transport restrictions).

More detailed information on CLT can be found at the WoodSolutions website or in *WoodSolutions Technical Design Guide #16 Massive Timber Construction Systems: Cross-laminated Timber (CLT),* which introduces the use of CLT in construction and provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

2.5 Modular Systems

A major benefit in using timber structural systems in mid-rise construction is the ability to prefabricate off-site and manufacture frames or cassettes, panelised elements or full volumetric modules to minimise the on-site construction requirements and costs.

Prefabricated Cassette Floor Systems

Prefabricated cassette floor systems use a range of timber structural products, typically for flooring (particleboard, plywood or OSB panels) and for floor joists and bearers (sawn timber, LVL, OSB beams, floor trusses or I-beams). Cassettes tend to be about 3 metres wide and up to about 12 metres long (due to travel restrictions). Cassette floor systems are highly effective in mid-rise construction as they are extremely fast to install and far safer for on-site workers, dramatically reducing 'fall-from-height' risks for workers.

With mid-rise construction, where effective acoustic separation is required, a Timber/Concrete Composite Cassette Floor might be considered; the concrete screed adding mass for acoustic and vibration control as well as acting compositely with the timber components for improved structural performance. For more detailed information on this refer to *WoodSolutions Technical Design Guide #30, Timber Concrete Composite Floor Design.*

Panelised Elements

Fully panelised elements involve the total off-site manufacture of all components, including structural members, insulation, sarking, plumbing and electrical fittings, window and/or door installation and internal lining installation (and external cladding if appropriate).

Modular Wood Construction Systems

Modular wood construction systems (volumetric modules) use either light-frame systems (mainly) or solid wood panel systems. The main principle is that the entire volumetric box consisting of walls, floor and ceiling, as well as inner lining and all services are assembled in a factory and transported to the construction site for erection. To assemble the modules on top of each other, a male-female connector arrangement is often used. The size of the modules is generally limited by transportation restrictions with maximum dimensions of about 4.2 metres wide, up to 13 metres long and 3.1 metres high. The overall building height is generally the same as for light-frame systems, around 6–7 storeys. Modular systems are used in a range of applications, including aged care facilities.

2.6 Hybrid Construction

Mid-rise timber buildings may also use a 'mix' of materials – hybrid construction – to achieve cost-effective, practical and robust solutions.

There are two main categories of Hybrid construction. The first applies to buildings that have a mix of timber and other structural elements.

For example, a common configuration is to use concrete construction for below-ground structures, such as car parks and basements, to reduce the risks to timber elements associated with groundwater and for ground floor construction to provide a physical separation from the ground as part of a termite management strategy (where required). Another common configuration is the use of concrete cores housing lifts and fire exits providing lateral and some vertical support for the structure supplemented by timber floors, beams and walls

The second form hybrid construction may use timber and other structural materials within the same element of construction, for example:

- concrete toppings to timber floors with shear connections to use composite action
- floors constructed with a mix of steel beams and timber beams to manage a mix of spans as shown in Figure 2.3 (LVL timber beams are increasingly used to replace steel beams for this application).

Figure 2.3: Mix of steel and timber beams in a floor.

3

Step 1 – High-Level NCC Design Issues (Schematic Design)

The National Construction Code (NCC) provides a regulatory framework for determining the minimum design and construction requirements for buildings in Australia. This step covers high-level design issues relating to fire-resisting and sound-insulating construction for the DTS pathway.

3.1 Application of this Guide and Additional Sources of Information

WoodSolutions Technical Design Guide 37H applies to mid-rise NCC Class 9a and Class 9c healthcare buildings.

For low-rise buildings, refer to:

- *WoodSolutions Technical Design Guide #03 Timber Framed Construction for Commercial Buildings Classes 5, 6, 9a & 9b, and*
- *WoodSolutions Technical Design Guide #42 Building Code of Australia Deemed to Satisfy Solutions for Timber Aged Care Buildings (Class 9c)*

Other mid-rise timber resources include:

- *WoodSolutions Technical Design Guide #37R Mid-rise Timber Buildings – Multi-residential Class 2 and 3*
- *WoodSolutions Technical Design Guide #37C Mid-rise Timber Buildings -Commercial and Education Class 5, 6, 7, 8 and 9b (including Class 4 parts).*

For high-rise timber buildings (greater than an effective height of 25 m), or for other buildings that vary from an NCC DTS provisions, the performance pathway should be followed. Further general information on the application of this pathway is provided in the NCC and related publications.

Useful additional resources if a Performance Solution Pathway is being followed are:

- *WoodSolutions Technical Design Guide #38*, which provides the technical basis behind the changes to the NCC to provide a DTS pathway for mid-rise timber buildings.
- Fire Safety of Hospitals – a guide for designers (Bennetts, I., et al., *Fire Safety of Hospitals – a guide for designers. 2018: Melbourne)*

3.2 Determine the Type of Construction Required

Note: For mid-rise Class 9a and 9c buildings or parts of buildings, Type A construction will be required in most situations.

The Class of Building in conjunction with the building height, expressed in terms of the rise in storeys, and the maximum size of fire compartments, are used to determine the Type of Construction required. Maximum fire compartment sizes (floor area, volume) as defined in the NCC Clause C2.2 are not usually the primary driver of the Type of Construction in Class 9a and 9c buildings.

The NCC defines three types of construction:

Type C construction is generally applicable to buildings with a rise in storeys of 1 or 2 depending on the class of building. It is the least fire-resisting form of construction and places few fire-related restrictions on the use of structural timber members.

Type B construction is generally applicable to buildings with a rise in storeys of 2 or 3 depending on the class of building. Type B construction requires greater use of fire-resisting construction and applies greater controls on the combustibility of construction materials than Type C construction. Fire-protected timber options provided in the NCC DTS provisions, or using the performance pathway, can be applied to address these constraints.

Type A construction generally applies to buildings with a rise in storeys of 3 or more and so applies to most mid-rise structures. Type A construction is the most fire-resisting and the prescriptive solutions within the NCC can impose severe limitations on the use of timber through the prescription of masonry and concrete construction, and non-combustibility for elements required to achieve a prescribed Fire Resistance Level (FRL). The fire-protected timber options provided in the NCC DTS provisions may be applied or the performance pathway may be taken to address these limitations..

An overview of the process for determining the Type of Construction is described below.

3.2.1 Determine the Building Classification

The NCC contains mandatory performance requirements that apply to 10 primary classes of building that are determined by the building's purpose. This classification is generally undertaken by the appropriate authority (e.g. building surveyor or building certifier). The classes directly applicable to this Guide from NCC Clause A6.9 and the relevant definitions providing further explanation from Schedule 3 of the NCC are summarised in the accompanying box.

A6.9 Class 9 buildings

A Class 9 building is a building of a public nature that includes one or more of the following sub-classifications:

1. Class 9a – a *healthcare building* including any parts of the building set aside as laboratories and includes a *healthcare building* used as a *residential care building*.
2. Class 9c — a residential care building.

Healthcare building means a building whose occupants or patients undergoing medical treatment generally need physical assistance to evacuate the building during an emergency and includes—

a) a public or private hospital; or

b) a nursing home or similar facility for sick or disabled persons needing full-time care; or

c) a clinic, day surgery or procedure unit where the effects of the predominant treatment administered involve patients becoming non-ambulatory and requiring supervised medical care on the premises for some time after the treatment.

Residential care building means a Class 3, 9a or 9c building which is a place of residence where 10% or more of persons who reside there need physical assistance in conducting their daily activities and to evacuate the building during an emergency (including any aged care building or residential aged care building) but does not include a hospital.

Residential aged care building means a Class 3 or 9a building whose residents, due to their incapacity associated with the ageing process, are provided with physical assistance in conducting their daily activities and to evacuate the building during an emergency.

Aged care building means a Class 9c building for residential accommodation of aged persons who, due to varying degrees of incapacity associated with the ageing process, are provided with personal care services and 24 hour staff assistance to evacuate the building during an emergency.

Note: Refer NCC A6 for details of other Classes of Building

Where areas of hospitals are likely to be assigned different uses during the life of a building, the most onerous classification may be selected to provide flexibility for future applications. These matters should be discussed with the client / operators during the design phase.

The demarcations between Class 3, Class 9a and Class 9c buildings can overlap, reflecting the continuum of occupant characteristics; especially where a facility allows for ageing 'in place' where these characteristics can vary over time.

In addition, large hospitals may have parts with different classifications, for example restaurants, shops or carparks. These may be classified separately or treated as an ancillary use, depending on the proportion of floor area occupied, among other things. Where it is unclear which classification should apply, the appropriate authorities have the discretion to decide.

Note: The DTS provisions for fire-resisting and sound-insulating construction vary significantly between the Building Classes and therefore the applicable Building Class(es) should be determined at the start of the Schematic Design Stage and be confirmed by the appropriate authority.

3.2.2 Determine the Rise in Storeys

The rise in storeys is the sum of the greatest number of storeys at any part of the external walls of the building and any storeys within the roof space:

- above the finished ground next to that part; or
- if part of the external wall is on the boundary of the allotment, above the natural ground level at the relevant part of the boundary.

Note: NCC Clause C1.5(b) allows Class 9c buildings to have a rise in storeys of 2 if it is protected throughout with a sprinkler system (other than a FPAA101D or FPAA101H system) complying with Specification E1.5 and complies with the maximum compartment size specified in Table C2.2 for Type C construction. Refer to WoodSolutions Technical Design Guide #42 (Class 9c) if this option is adopted.

3.2.3 Determine Type of Construction from Rise in Storeys and Building Class

Table 3.1 Shows the Type of Construction required by Clause C1.1 of NCC Volume One.

Table 3.1: Types of construction required by NCC Volume One.

Rise in storeys	Multi-residential (includes some Class 3 residential care buildings)		Office	Retail	Car Park/ Storage	Factory/ Laboratory	Hospitals/ Residential Care Facilities/ Assembly
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
4 or more	**A**	**A**	**A**	**A**	**A**	**A**	**A**
3	**A***	**A***	**B**	**B**	**B**	**B**	**A**
2	**B(C)***	**B(C)***	**C**	**C**	**C**	**C**	**B(C)****
1	**C**	**C**	**C**	**C**	**C**	**C**	**C**

Table 3.1 is a derivative of Table C1.1 of the National Construction Code 2019 Amendment 1 provided by **https://www.abcb.gov.au/** *Australian Building Codes Board © 2020.*

** Refer low-rise concessions (e.g. Specification C1.1 Clause 3.10 and Clause 4.3) for Type B construction or Clause C1.5 for applications where Type C applies.*

***For Class 9a and 9c healthcare buildings, Type B construction is required for buildings having a rise in storeys of 2 unless Type C construction is permitted for Class 9c buildings having a rise in storeys in accordance with Clause C1.5.*

Type A construction is required for all Class 9a and Class 9c buildings having a rise in storeys of 3 or more and this Guide applies to mid-rise buildings of Type A construction with an effective height not greater than 25 m.

3.3 Determine NCC Compliance Pathway

3.3.1 NCC Compliance Pathway

To comply with the NCC the relevant Performance Requirements must be satisfied; as demonstrated by means of the Assessment Methods specified in the NCC. There are two pathways (or a combination of the two) that can be followed.

- For a Deemed-to-Satisfy Solution, it is necessary to provide Evidence of Suitability to show that the Deemed-to-Satisfy Provisions have been met.
- For a Performance Solution, specific building solutions are developed for a building that may vary from the Deemed-to-Satisfy Provisions.

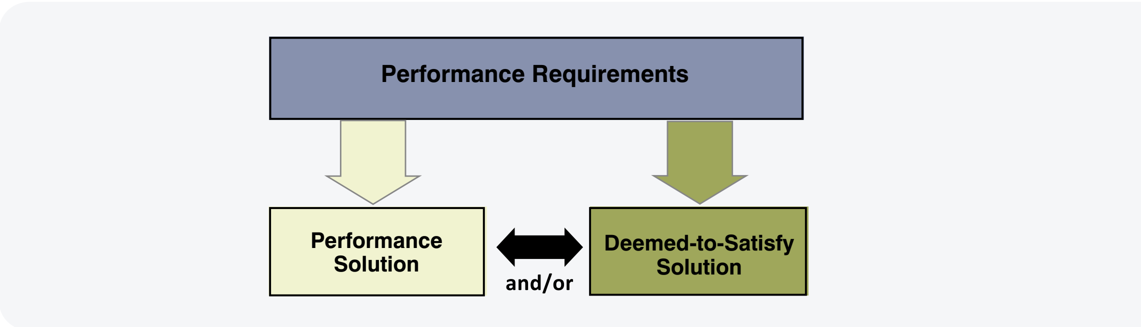


Figure 3.1: Pathways for demonstrating compliance with NCC performance requirements.*

Figure 3.1 is a derivative of Figure 1 of the National Construction Code 2019 Amendment 1 provided by* **https://www.abcb.gov.au/ *Australian Building Codes Board © 2020.*

The construction systems and details in this Guide are based on the Deemed-to-Satisfy (DTS) Solution pathway for mid-rise timber buildings that was first introduced into the 2016 edition of the NCC for Class 2, 3 and 5 buildings and extended to all classes of buildings in the 2019 edition. This does not prevent designers using Performance Solutions, but it should be ensured that any variations from the DTS Solution do not adversely affect the fire safety strategy for the building.

More information on the design principles and supporting data for fire-protected buildings is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-Rise Buildings.*

3.3.2 NCC Compliance Options for Timber Buildings

In the context of this Guide, timber buildings are defined as buildings where timber is the predominant material in the structure. There are still opportunities to use timber for some structural and non-structural applications in buildings using other materials for the primary structure.

Table 3.2 summarises options for complying with the NCC Performance Requirements for Class 2 to 9 buildings with further details provided below. DTS Solutions are available for the building configurations shaded in green (light, mid and dark green) with this Guide being directly applicable to Class 9a and 9c buildings of 3 or more storeys and an effective height not greater than 25 m, which are shaded in mid green and highlighted with a red box.

At the time of writing, Performance Solutions are required to be developed for the areas shaded in blue.

Table 3.2: Design options for timber buildings.

Rise in storeys or effective height	**Multi-residential (includes some Class 3 residential care buildings)**		**Office**	**Retail**	**Car Park/ Storage**	**Factory/ Laboratory**	**Hospitals/ Residential Care Facilities/ Assembly**
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
Effective height greater than 25m	High	High	High	High	High	High	High
8 (*Effective height of not more than 25 m)*	Mid	Mid	Mid	Mid	Mid	Mid	Mid
7	Mid	Mid	Mid	Mid	Mid	Mid	Mid
6	Mid	Mid	Mid	Mid	Mid	Mid	Mid
5	Mid	Mid	Mid	Mid	Mid	Mid	Mid
4	Mid[1]	Mid[1]	Mid	Mid	Mid	Mid	Mid
3	Low[1]	Low[1]	Mid	Mid	Mid	Mid	Mid
2	Low[1]	Low[1]	Low	Low	Low	Low	Low
1	Low	Low	Low	Low	Low	Low	Low

Note 1: Refer to Technical Design Guide #02 to check if low-rise timber concessions apply.

Low	*Low-rise DTS options – Guide #42 for Class 9c, #02 for multi-residential and #03 for commercial buildings.*
Mid	*Mid-rise fire-protected timber DTS – Guide #37R for Class 2 and 3 buildings*
Mid	*Mid-rise fire-protected timber DTS – Guide #37C for Class 5, 6, 7, 8 and 9b commercial and educational buildings and Guide #37H for Class 9 healthcare buildings (this Guide).*
High	*High-rise Performance Solution*

Low-rise timber buildings

There are relatively few fire-related restrictions on the use of structural timber members in Buildings of Type C construction, irrespective of the Class of Building, under the DTS Solution pathway. While fire-protected timber options could be adopted for these buildings, there may be more practical options and reference should be made to the relevant guides identified in the key to Table 3.2.

The NCC DTS pathway also includes concessions that facilitate the use of timber-frame construction for Class 2 and 3 buildings up to a rise in storeys of 3 and, in limited cases, up to 4 storeys. Guidance in relation to construction of these low-rise options is provided in the relevant guides identified in the key to Table 3.2.

Note: The NCC defines effective height as "the vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units)". If there is any doubt as to whether a building's effective height does not exceed 25 metres, it is recommended that the effective height is checked with the relevant authorities.

Low-rise buildings would normally be designed following the Deemed-to-Satisfy Solution pathway with Performance Solutions being used to address minor variations and/or unusual design circumstances as required.

Mid-rise timber buildings

Mid-rise buildings have an effective height of not more than 25 metres. The use of timber structural members under the NCC prescriptive pathway is restricted for mid-rise buildings unless the option to use fire-protected timber in conjunction with automatic fire sprinklers is adopted. This Guide applies to fire-protected timber Class 9a and Class 9c healthcare buildings with a rise of storeys of 3 or more up to a maximum effective height of 25 m fitted with an automatic fire sprinkler system throughout.

Some healthcare buildings (e.g. large hospitals) may be of mixed classes and parts of a building may fall under different classifications. Refer to the key to Table 3.2 for relevant design guides.

While the focus of this Guide is mid-rise healthcare buildings, which can be designed following the DTS pathway, Performance Solutions can be adopted to address minor variations and/or innovative design circumstances. Guidance on the technical derivation of the mid-rise fire-protected timber solution is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-rise Timber Buildings*, which may assist with the development of a Performance Solution.

High-rise buildings

All high-rise timber buildings (effective height greater than 25 m) need to follow the Performance Solution pathway.

3.4 Overview of Fire-protected Timber DTS Solutions for Mid-rise Buildings

3.4.1 Overview of NCC Prescriptive Requirements specific to fire-protected timber

The NCC 2019 includes Deemed-to-Satisfy Provisions that allow the construction of mid-rise timber buildings. The main features specific to the mid-rise fire-protected timber building DTS Solutions are:

- the building has an effective height of not more than 25 metres
- the building has a sprinkler system, other than a FPAA101D or FPAA101H system, complying with Specification E1.5 of the NCC throughout
- fire-protected timber complying with Specification C1.13a of the NCC can be used for loadbearing timber elements, non-loadbearing timber walls required to achieve an FRL and for elements of construction required to be non-combustible
- cavity barriers are provided in accordance with Specification C1.13 of the NCC
- any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible.

These fire safety precautions aim to provide a robust building solution on the following basis:

Automatic fire sprinkler suppression system: Objective is to suppress a fire before the structure is threatened and greatly reduce the risk to people and property.

Fire-protected timber (NCC prescribes FRLs AND non-combustible fire protective coverings): The objective of these measures is to prevent or delay ignition of the timber structural members so that the response to an enclosure fire will be similar to non-combustible elements, masonry or concrete during the fire growth period and prior to fire brigade intervention.

Cavity barriers: The objective is to prevent uncontrolled spread of fire through cavities in the low probability events of either failure of the protective covering or fire starting within the cavity.

Non-combustible insulation: The objective is to minimise the risk of fire spread through cavities by removing a potential source of fuel, i.e. combustible insulating materials.

Note: These above specific fire-protected timber requirements are used in conjunction with other DTS provisions of the NCC that apply to healthcare buildings.

3.4.2 Evidence of Suitability

The NCC requires every part of a building to be constructed in an appropriate manner to achieve the performance requirements, using materials and construction that are fit for the purpose for which they are intended, including safe access for maintenance.

The NCC Volume One specifies requirements for Evidence of Suitability in Clause A5.2 but there are additional specific requirements that apply to certain aspects of fire safety under NCC prescriptive requirements:

- NCC Clause A5.4 Fire-Resistance of Building Elements
- NCC Clause A5.5 Fire Hazard Properties
- NCC Clause A5.6 Resistance to the Incipient Spread of Fire.

In most instances, for the materials and systems considered in this Guide, the Evidence of Suitability for the fire resistance or Resistance to the Incipient Spread of Fire (RISF) of an element of construction will be in a report from an Accredited Testing Laboratory.

If a Performance Solution is proposed, compliance should be demonstrated using the procedures prescribed in Clauses A5.4 and A5.6 of the NCC as appropriate for fire-protected timber elements.

3.5 Determine Schematic Building Layout

3.5.1 Mixed Class Buildings

The NCC DTS Solution, using fire-protected timber in conjunction with an automatic fire sprinkler system, can be applied to mixed class buildings; provided the different classes are adequately fire separated (refer Step 4) and the entire building is protected by an automatic fire sprinkler system complying with NCC Volume One Specification E1.5, other than a FPAA101D or FPAA101H system.

This provides added flexibility for the design of new buildings and facilitates the refurbishment or expansion of existing buildings which is relatively common for major hospitals.

3.5.2 Check Fire and Smoke Compartment Sizes

General floor area and volume limitations

The NCC Clause C2.2 does not nominate a maximum fire compartment size for Class 2, 3 and 4 buildings or parts of buildings but the NCC does require the bounding construction of Sole Occupancy Units (SOUs) to be of fire-resisting construction.

For Class 5 to 9 buildings maximum compartment sizes are prescribed in Clause C2.2 of the NCC and are summarised in Table 3.3 for the mid-rise buildings considered in this Guide.

Table 3.3: NCC Maximum Fire Compartment Sizes for mid-rise Commercial and H ealthcare Buildings.*

Building Class	Type A Construction		Type B Construction	
	Floor Area	Volume	Floor Area	Volume
5	**8,000 m²**	**48,000 m³**	**5,500 m²**	**33,000 m³**
9b				
9c See sub-section below for 'Additional fire and smoke compartmentation' requirements				
6	**5,000 m²**	**30,000 m³**	**3,500 m²**	**21,000 m³**
7				
8				
9a See sub-section below for 'Additional fire and smoke compartmentation' requirements				

Table 3.3 is a derivative of Table C2.2 of the National Construction Code 2019 Amendment 1 provided by* ***https://www.abcb.gov.au*/ *Australian Building Codes Board © 2020.*

Note: There are concessions permitted by the NCC for large isolated buildings that allow relaxations to the above limits under certain circumstances (refer NCC Clause C2.3).

Individual floors are normally substantially less than the maximum floor areas specified in Table 3 above and under these circumstances each floor will generally comprise at least one separate fire compartment unless relaxations are permitted (e.g. non-required, non-fire isolated connections as permitted in Clause D1.2 of the NCC).

Additional fire and smoke compartmentation for Class 9a patient care areas

NCC Clause C2.5(a) requires additional fire and smoke compartmentation of patient care areas in Class 9a buildings. These are summarised in Table 3.4.

Table 3.4: Additional fire and smoke compartmentation for Class 9a patient care areas.

<table>
<tr><th rowspan="2">Part</th><th rowspan="2">Smoke Compartment (smoke-proof walls, NCC Spec C2.5)</th><th colspan="2">Fire Compartmentation Max areas</th></tr>
<tr><th>Sub Compartment FRL 60/60/60</th><th>Fire Compartment FRL 120/120/120</th></tr>
<tr><td>Treatment area</td><td>1,000 m²</td><td>-</td><td rowspan="3">2,000 m²</td></tr>
<tr><td>Ward areas</td><td>500 m²</td><td>1,000 m²</td></tr>
<tr><td>Hazardous ancillary areas within patient care areas</td><td>-</td><td>Fire separation from remainder of patient care area</td></tr>
</table>

Hazardous ancillary areas that contain equipment or materials that are a high potential fire hazard are required to be separated from the rest of the patient care area by walls having an FRL of not less than 60/60/60 and include:

- a kitchen and related food preparation areas with floor area of more than 30 m^2
- a room containing a hyperbaric facility (pressure chamber)
- a room used for the storage of medical records having a floor area of more than 10 m^2
- a laundry, where items of equipment are of the type that are potential fire sources.

Additional fire and smoke compartmentation for Class 9c Buildings

A fire compartment must be separated from the remainder of the building by fire walls and floors with an FRL of not less than 60/60/60. (Refer to Table 3.3 for maximum fire compartment sizes). Lift and stair shaft walls are required to achieve an FRL of at least 120/120/120 or –/120/120 if non-loadbearing.

NCC Clause C2.5(b) requires Class 9c buildings to be subdivided into smoke compartments having a maximum floor area of 500 m^2.

Hazardous ancillary areas that contain equipment or materials that are a high potential fire hazard are also required to be separated from the rest of the patient care area by smoke-proof walls and include:

- a kitchen and related food preparation areas with floor area of more than 30 m^2
- a room containing a hyperbaric facility (pressure chamber)
- a room used for the storage of medical records having a floor area of more than 10 m^2
- a laundry, where items of equipment are of the type that are potential fire sources.

Note: Note the NSW variations to the NCC modify Clause C2.5 to require additional smoke/fire separation around SOUs in Class 9c buildings.

3.5.3 Determine Schematic Fire Safety Design Strategy

The preliminary specification of a fire safety strategy for a building is important as it may significantly affect the building layout. This is applicable irrespective of the compliance pathway chosen (Performance Solution or DTS Solution) since, even within the DTS pathway, options have to be selected that affect the building layout, detailed design and on the use of the building through its life cycle.

The schematic fire safety design strategy should, at the preliminary stage, provide as a minimum:

- A summary of the fire safety objectives.
- Building uses that the design needs to address.
- Occupant characteristics that the design addresses.
- Approach to demonstrating compliance with the NCC (Performance Solution, DTS Solution or a combination).
- Where a DTS Solution is specified, it is still necessary to provide details of the DTS options selected as detailed below.

- Schematic drawings and brief descriptions as appropriate indicating:
  - design requirements for automatic fire sprinkler systems
  - design requirements for detection and alarm systems
  - general layout showing fire/smoke resistant compartmentation and structural elements
  - active smoke control measures if provided
  - means of egress during a fire emergency including travel distances to exits, discharge of exits, door operations, etc
  - evacuation strategy and associated emergency warning and intercom system (EWIS)
  - means to alert the fire brigade and equipment to facilitate fire brigade intervention
  - any other fire protection measures.
- An implementation plan relating to the scheduling and inspection of all fire safety measures and systems, stating who is responsible for ensuring compliance and measures that will be in place to facilitate compliance (e.g. inspection schedule).
- Protection measures required during construction.
- Fire safety management measures after completion to ensure ongoing effectiveness of the fire safety strategy through the life of the building.

This preliminary strategy should be regularly reviewed and updated with further details added as the design develops.

3.5.4 Determine Building Services and Preliminary Layout

The preliminary selection of building services and service locations should be considered when determining the general building layout and provision allowed for safe maintenance, modification, and addition of services without compromising fire safety and sound separation.

Typical matters for consideration include:

- location of plant rooms (noting that some plant rooms may contain transformers, emergency generators and associated fuel tanks that may present additional hazards that need to be addressed)
- selection of heating, ventilation and air conditioning (HVAC) systems (centralised v localised systems)
- locate service shafts to minimise nuisance noise and facilitate fire compartmentation
- minimise service penetrations through fire barriers
- group service penetrations together and select treatments that minimise the risk of fire spread to cavities
- avoid the need for hot works (e.g. welding, grinding) on services at the position of penetrations through fire-resistant elements
- provide for maintenance and additional services (e.g. allow for the provision of sufficient inspection hatches to allow adequate viewing of potential penetrations through smoke or fire walls)
- select services and materials that minimise the need for hot works (e.g. welding, grinding, soldering).
- distribution systems for medical gasses and bulk storage locations.

3.6 Consideration of Other Drivers and Constraints

3.6.1 Safe Design

Safe design principles should be applied throughout the design process and consider the entire building life cycle. This section gives some general guidance in relation to mid-rise timber buildings.

Building life cycle

It is important to consider the impacts of design decisions on all phases of the building's life cycle.

For example, the NCC Deemed-to-Satisfy Provisions may require a fire safety feature to be incorporated into a building. During the design process it is necessary to determine:

- how the provision can be installed/constructed safely to achieve its required performance (e.g. avoiding hot works during installation)
- how the feature will be commissioned and its performance verified, (e.g. provision of safe access for inspection)
- that the feature will not present a hazard during occupation of a building, (e.g. where there is a significant risk that occupants may self-harm detail fire protection measures top avoid hanging points.)
- how the feature can be maintained and repaired safely
- measures to be taken to ensure the feature does not present a hazard during renovation/modification or demolition and to ensure that the performance of the feature is not compromised during the renovation/modification process.

Many of these matters lie outside the scope of the NCC but they are addressed through State and Territory Building Acts and regulations and Work Health and Safety (WHS) legislation.

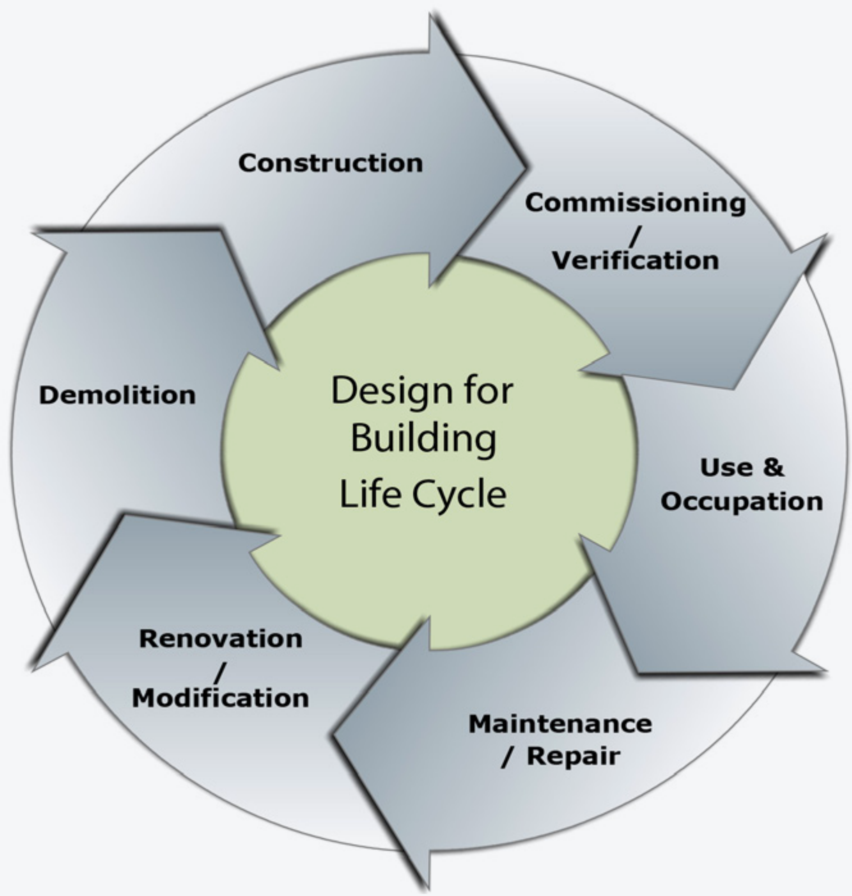


Figure 3.7: Typical Building Life Cycle.

Responsibilities for safe design

This Guide focuses on NCC 2019 requirements relating to Deemed-to-Satisfy Solutions for mid-rise timber buildings.

The NCC does not regulate matters such as the roles and responsibilities of building practitioners and maintenance of fire safety measures that fall under the jurisdiction of the States and Territories. State and Territory building legislation is not consistent in relation to these matters, with significant variations with respect to:

- registration of practitioners
- mandatory requirements for inspections during construction
- requirements for maintenance of fire safety measures.

WHS legislation requires safe design principles to be applied. A Code of Practice – Safe Design of Structures published by Safe Work Australia provides guidance to persons who design structures that will be used, or could reasonably be expected to be used, as a workplace. It is prudent to apply these requirements to all buildings as they are generally a workplace for people doing building work, maintenance, inspections and the like.

The Code defines safe design as: "The integration of control measures early in the design process to eliminate or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed."

It indicates that safe design begins at the start of the design process when making decisions about:

- the design and its intended purpose
- materials to be used
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal, (e.g. avoidance of working-at-height issues through the provision of safe access to elevated equipment)
- what legislation, codes of practice and standards need to be considered and complied with.

The Code also provides clear guidance on who has health and safety duties in relation to the design of structures and lists the following practitioners:

- architects, building designers, engineers, building surveyors, interior designers, landscape architects, town planners and all other design practitioners contributing to, or having overall responsibility for, any part of the design
- building service designers, engineering firms or others designing services that are part of the structure such as ventilation, electrical systems and permanent fire extinguisher installations
- contractors carrying out design work as part of their contribution to a project (for example, an engineering contractor providing design, procurement and construction management services)
- temporary works engineers, including those designing formwork, falsework, scaffolding and sheet piling
- persons who specify how structural alteration, demolition or dismantling work is to be carried out.

In addition, WHS legislation places the primary responsibility for safety during the construction phase on the builder.

The design team, in conjunction with owners/operators and the builder, have a responsibility to document designs, specify and implement procedures that will minimise risks to health and safety throughout the life of the structure being designed.

Applying safe design principles

A key element of safe design is consultation with stakeholders to identify risks, practical mitigation measures and to assign responsibilities to individuals/organisations for ensuring the mitigation measures are satisfactorily implemented.

This approach should be undertaken whichever NCC compliance pathway is adopted and applies to all forms of construction.

Some matters specific to fire safety are summarised below:

- The NCC and associated referenced documents represent nationally recognised standards for fire safety for new building works.
- The NCC's limited treatment of fire precautions during construction focuses on manual fire-fighting, egress provisions and fire brigade facilities. Additional precautions are required to address WHS requirements such as fire prevention and security. Refer to Section 1.4.4 and WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings, for further information.
- Minimising service penetrations through fire-resisting construction.
- Grouping of service penetrations through fire-resisting walls with safe access for installation, inspection and maintenance.
- Detailed design of fire safety measures to optimise reliability and facilitate safe installation, maintenance and inspection where practicable. Special attention should be given to protection of service penetrations and cavity barriers.

Importance of Ongoing Documentation

Throughout the design and construction phase it is important to undertake the following tasks culminating at the end of the commissioning stage with the development of a Fire Safety Manual.

- Documentation of procedures and allocation of responsibilities for determining Evidence of Suitability for fire safety measures.
- Documentation of procedures and allocation of responsibilities for the verification and commissioning of all fire safety installations.
- Provision of specifications and drawings of all fire safety measures within the building, Evidence of Suitability, commissioning results and requirements for maintenance and inspection to the owner as part of the fire safety manual. (Note: Some State and Territory legislation contains minimum requirements for inspection of fire safety measures).
- The fire safety manual should also provide information on how to avoid compromising fire safety through the life of a building (e.g. preventing disconnection of smoke detectors or damage to fire-resisting construction or preventing unauthorised / unitentional isolation of automatic fire sprinkler systems).

Fire precautions during construction

Fires may occur on building construction sites due to the nature of the works. Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Mid-rise timber buildings complying with the NCC 2019 Deemed-to-Satisfy Provisions offer a safe and economical building solution. The addition of the fire-protective coverings plays an important role in providing this fire safety and, due to the construction sequencing, there may be a period where the timber is not fully protected and/or automatic fire sprinkler protection is not fully operational. During this period timber buildings are at their highest risk from construction fires.

The builder and design team need to consider fire precautions during construction. The scope of the NCC is limited to specifying minimum requirements for fire hydrants, hose reels and extinguishers and egress provisions (NCC Clause E1.9).

Addressing WHS requires a broad holistic approach that considers the building layout and site layout throughout the construction process to minimise the fire risk at a time when the building could be at its most vulnerable. Typical matters that should be considered include:

- progressive installation of services
- progressive installation of fire-protective grade covering of timber members and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works
- security provisions (to address arson)
- safe access for maintenance of equipment and minimising the down time of fire safety equipment during maintenance
- detailing service penetration and construction interfaces to minimise the risk of cavity fires during installation.actual construction phase.throughout the life of the structure being designed.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

3.6.2 Additional User Requirements

Additional user requirements may include the following items. Many State and Territory Health Authorities provide guidance on these matters and may have standard specifications/room layouts.

- usability
- aesthetics
- costs
- speed of construction
- building flexibility
- operational continuity
- corporate image
- environmental protection
- heritage protection
- enhanced safety (incorporating WHS legislation and duty of care)
- enhanced property protection
- other legislation and government policies.

It is critical these matters are addressed at the schematic stage by the design team to reduce the risk of costly reworks of a design.

4

Step 2 – Determine NCC design requirements for sound, thermal resistance, weatherproofing and structural tests

Timber building systems can be designed to achieve the regulatory requirements of the National Construction Code Volume One (NCC). From a performance perspective, the NCC sound provisions tend to govern the choice of timber building systems more than the fire provisions, due to the lightweight nature of the timber systems.

In relation to Class 9a (hospitals) and Class 9c (aged care), the NCC only applies sound trasmission provision to Class 9c buildings.. Keep in mind that the NCC Provisions are minimum requirements and may not meet the expectations of the building occupants. There is also a parallel need to address sound induced by poor spatial design of a building, flanking noise issues and, where appropriate, upgraded sound performance requirements to meet end user needs.

4.1 Utilising the Deemed-to-Satisfy Provisions for Sound

Part F5 of the NCC is concerned with safeguarding 'occupants from illness or loss of amenity as a result of undue sound being transmitted' and primarily addresses Class 2, 3 and 9c buildings. Therefore, there are no mandatory sound transmission NCC Performance Requirements for Class 9a buildings.

However, in large open plan spaces in hospitals, consideration should be given to the way occupants will work and interact in the spaces and the effects of generated sound within the built environment. In hospital facilities, this would include workstations, private meeting rooms, waiting rooms through to kitchen areas and should consider levels to enable the ability to hold 'comfortable' conversations without affecting adjacent occupied spaces.

A holistic acoustic design would consider:

- sound insulation between spaces (e.g. wards and lift lobbies)
- internal room spaces
- noise from building services/plant
- vibration
- external noise.

The NCC Performance Requirements for Class 9c buildings focus on limiting the transmission of both airborne and impact-generated sound via floor and wall building elements bounding sole-occupancy units (SOUs) where separating:

- adjoining SOUs
- SOUs from a common space (e.g. kitchen)
- SOUs from another building classification within the building.

The sound performance of these floor and wall building elements must consider the impact of any pipe penetrations or other service elements (e.g. air-conditioning) as well as door openings on bounding construction. (Note: The provisions include the sound isolation of pumps but issues pertaining to this are not dealt with in this Guide). When interpreting these requirements, it is important to understand the difference between airborne and impact sound (Figure 4.1).

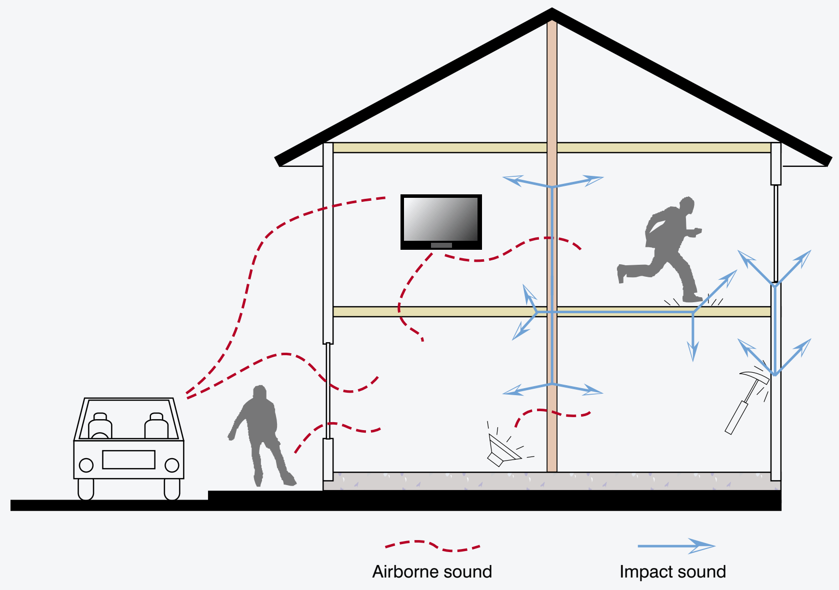


Figure 4.1: Examples of impact and airborne sound.

It is also important to understand how each type of sound is measured in order to select appropriately sound-insulated wall, floor and ceiling elements. The nomenclature used in the Deemed-to-Satisfy Provisions using results from laboratory requirements, is explained in Figures 4.2 and 4.3.
Note: Alternative methods of sound measurement also exist.

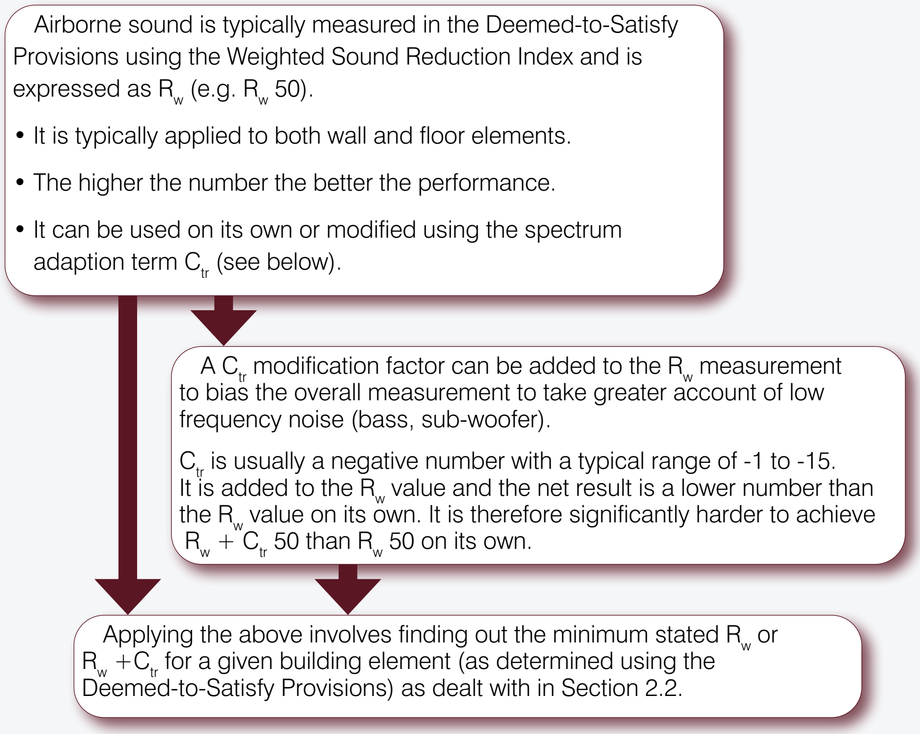


Figure 4.2: Airborne sound.

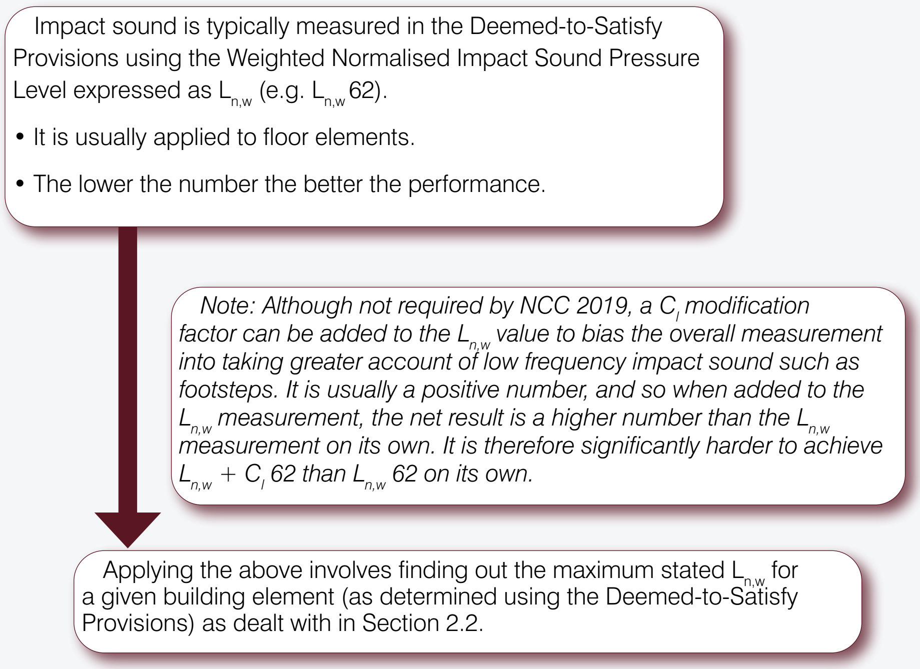


Figure 4.3: Impact sound.

There are no NCC requirements for sound ratings of external walls, but in some parts of Australia state planning regulations or local government requirements may apply. For information on the sound performance of common timber-framed external walls, refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise;* and for Cross Laminated Timber internal walls, floors and for service isolation refer to *WoodSolutions Technical Design Guide #44 CLT Acoustic Performance.*

4.2 Determining Sound Insulation Requirements for Individual Building Elements

Of importance to construction is the minimum airborne and impact sound insulation requirements for individual building elements, e.g. wall and floor elements. Table 4.1 provides a simple means for finding out such information and is necessary for selecting appropriate timber-framed construction, systems.

Table 4.1: Deemed to Satisfy Requirements for sound insulation of wall and floor elements in Class 9c buildings.

Situation			**Floor Rating**
First Space		**Adjoining Space**	
SOLE OCCUPANCY UNIT – all spaces	Separates	**Sole Occupancy Unit – all spaces**	$D_{nTw} + C_{tr}$ (airborne) ≥ 45 & $L_{nT,w}$ (impact) ≤ 62
Public corridor or lobby or the like	Separates	**Sole Occupancy Unit – all spaces**	$D_{nTw} + C_{tr}$ (airborne) ≥ 45 & $L_{nT,w}$ (impact) ≤ 62
Stair and lift shaft	Separates	**Sole Occupancy Unit – all spaces**	$D_{nTw} + C_{tr}$ (airborne) ≥ 45 & $_{LnT,w}$ (impact) ≤ 62
Plant Rooms	Separates	**Sole Occupancy Unit – all spaces**	$D_{nTw} + C_{tr}$ (airborne) ≥ 45 & $L_{nT,w}$ (impact) ≤ 62
Different NCC Building Classification	Separates	**Sole Occupancy Unit – all spaces**	$D_{nTw} + C_{tr}$ (airborne) ≥ 45 & $L_{nT,w}$ (impact) ≤ 62

Where a wall required to have sound insulation has a floor above, the wall must continue to the underside of the floor above, or the ceiling must provide the equivalent sound insulation required for the wall. (Professional advice should be sought to upgrade the ceiling to the required wall sound insulation.)

4.3 Services

If a duct, soil, waste or water supply pipe serves or passes through more than one SOU, the duct or pipe must be separated from any SOU by construction with an Rw + Ctr (airborne) not less than:

- 40 if it is adjacent to a bedroom or living room (other than a kitchen); or
- 25 if it is adjacent to a kitchen or bathroom.

It is also required where a duct or pipe is located within a wall or ceiling cavity.

If a storm water pipe passes through an SOU, it must comply with both of these provisions.

4.4 The Next Step

Having used the previous information to obtain an understanding of the NCC's minimum sound-insulation requirements, the next step is to either:

- go to Step 3 to find out about possible options for consideration for improving and/or upgrading sound performance; or
- go to Step 5 to select timber building systems that will comply with minimum NCC sound requirements.

4.5 Other Design Considerations

There are other design considerations that need to be taken into account in meeting NCC requirements. The following are not covered in detail in this Guide but are listed as requiring consideration.

4.5 Other Design Considerations

There are other design considerations that need to be taken into account in meeting NCC requirements. The following are not covered in detail in this Guide but are listed as requiring consideration.

4.5.1 Thermal Resistance (R-value)

NCC 2019 Volume One, Section J, provides the energy-efficiency requirements that a building, including its services, must achieve in order to address the annual greenhouse gas emissions of buildings.

The energy efficiency provisions can be met via the NCC 2019 Verification (modelling) Methods, including the Additional Requirements (e.g. floor edge insulation, building sealing) contained in Specification JVa or complying with the Deemed-to-Satisfy Provisions. These provisions will vary based on a range of factors including: the building's location (climate zone), direction of heat flow, level of external wall/window ratio and shading, size and performance of external glazing/windows and form of construction of the external building fabric.

The thermal resistance of timber building elements depends on the level of installed insulation (i.e. thickness), number and thickness of sheet lining layers (plasterboard, flooring), element construction (e.g. incorporating furring channels) and overall thickness of the building element.

Guidance on timber wall and floor/ceilings can be found in the *WoodSolutions' R-values for Timber-framed Building Elements* publication.

4.5.2 Damp and Weatherproofing

The requirements for the damp and weatherproofing of buildings are provided in NCC Volume One Part F1. The intent is to protect the building from external (rain) and internal water (e.g. laundry overflow) and the accumulation of internal moisture in a building causing unhealthy conditions for occupants and potential damage to building elements.

Key areas of consideration include:

- External walls. There are currently no Deemed-to-Satisfy Provisions in the NCC for weatherproofing and therefore suppliers of weatherproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). It is important that installed weatherproofing membranes/ systems are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate (i.e. water barrier) through to the structural timber building elements.
- Internal wet areas (e.g. bathroom) must be waterproofed in accordance with the NCC requirements (F1.7) and have adequate overflow systems (e.g. floor waste) in place to deal with the possibility of wastewater overflow.
- Roof coverings. For the purposes of this Guide, and as required by the NCC, roof coverings must be of non-combustible materials (e.g. concrete, metal, terracotta) and be fixed in accordance with, and comply with, the relevant Standard as specified in the NCC Clause F1.5.

Note: *The drawings in this Guide have either omitted damp, weatherproofing and waterproofing details or provided indicative details only. Specific details may vary with climatic conditions and in many instances the only compliance pathway is a Performance Solution that may yield solutions that vary from project to project. Take care to ensure that the Performance Solutions for damp, weatherproofing and waterproofing do not conflict with NCC fire safety requirements.*

4.5.3 Structural Tests

NCC Specification C1.8 describes structural tests for fire-resisting, lightweight wall construction that bounds lift, stair and service shafts, fire-isolated passageways and ramps as well as external and internal walls. The test methods and criteria for compliance are stated in relation to materials, damage, deflection (under static pressure and impact) and surface indentation.

Lightweight wall systems do not require testing if designed and constructed in accordance with the relevant design and loading standards specified in the NCC Part B1 Structural Provisions.

5

Step 3 – Improve and Upgrade Sound Performance

The NCC 2019 Volume One Amendment 1 does not nominate sound transmission requirements for Class 9a (hospitals) buildings but it does provide minimum Deemed-to-Satisfy requirements for Class 9c buildings.

Sound performance between floor levels and between adjacent rooms has a positive benefit for building occupants and can often be improved by simple attention to the form and spatial arrangement of the building design. Attention to flanking noise is another important way to improve sound performance. End users may wish to enhance the sound performance of their buildings and, as a result, this Step in the Guide focuses on ways to improve and upgrade sound performance.

5.1 Attention to Building Design to Reduce Sound Transmission

Aspects of the form and spatial design of a building that can be adapted to improve sound performance are dealt with under the following headings.

5.1.1 Floor Layout

Check that the floor layout is beneficial rather than detrimental to sound transmission. Service rooms, including photocopying/collation rooms and kitchens, create extra sound compared the typical work environment. Adequate sound insulation between spaces will assist in enhancing the acoustic environment for occupants.

5.1.2 Windows

Windows normally have lower sound insulation than the walls around them and can be used to improve the building occupant's comfort level from external noise. To improve the acoustic performance of window, consider one or more of the following:

- use thicker glass or double glazing
- use fixed glazing in lieu of opening windows (this may also require sound-insulated ventilation)
- locate windows so that they do not face noisy areas
- reduce the area of windows in the façade.

5.1.3 Doors

As with windows, doors tend to be a weak link in sound-rated wall systems. Where sound control is desired, solid core doors should be used and be treated with soft acoustic gaskets at interfaces with door jambs. Threshold closers at the bottom of the door or air seals will also help reduce sound transmission. In most cases, achieving the required sound rating will involve the use of gaskets and seals. Sliding doors should be avoided where optimum sound-control is desired.

5.1.4 Services

The location and detailing of services are two of the most important considerations in controlling sound transmission in residential buildings.

Generally, services and service penetrations should not be located on sound-insulated walls between SOUs but rather on internal walls or dedicated sound resisting service shafts. In all instances, service pipes should be located away from noise-sensitive parts of the dwelling, such as bedrooms.

5.1.5 External Walls

There are no NCC requirements for sound ratings of external walls, but in some parts of Australia there may be state planning regulations or local government requirements for external wall sound rating. For information on the sound performance of common timber-framed external walls, refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

5.2 Addressing Flanking Noise

The ability to insulate against sound moving from one location in building to the next depends not only on insulating individual wall and floor elements, but also on stopping noise from jumping or transferring from one building element to the next or, worse still, moving through the building in an uncontrolled way. As a result, the effectiveness of sound-insulated construction is concurrently dependent on addressing flanking noise. Flanking noise refers to sound passing around rather than through wall/ floor elements, causing sound to unexpectedly manifest itself in unwanted places.

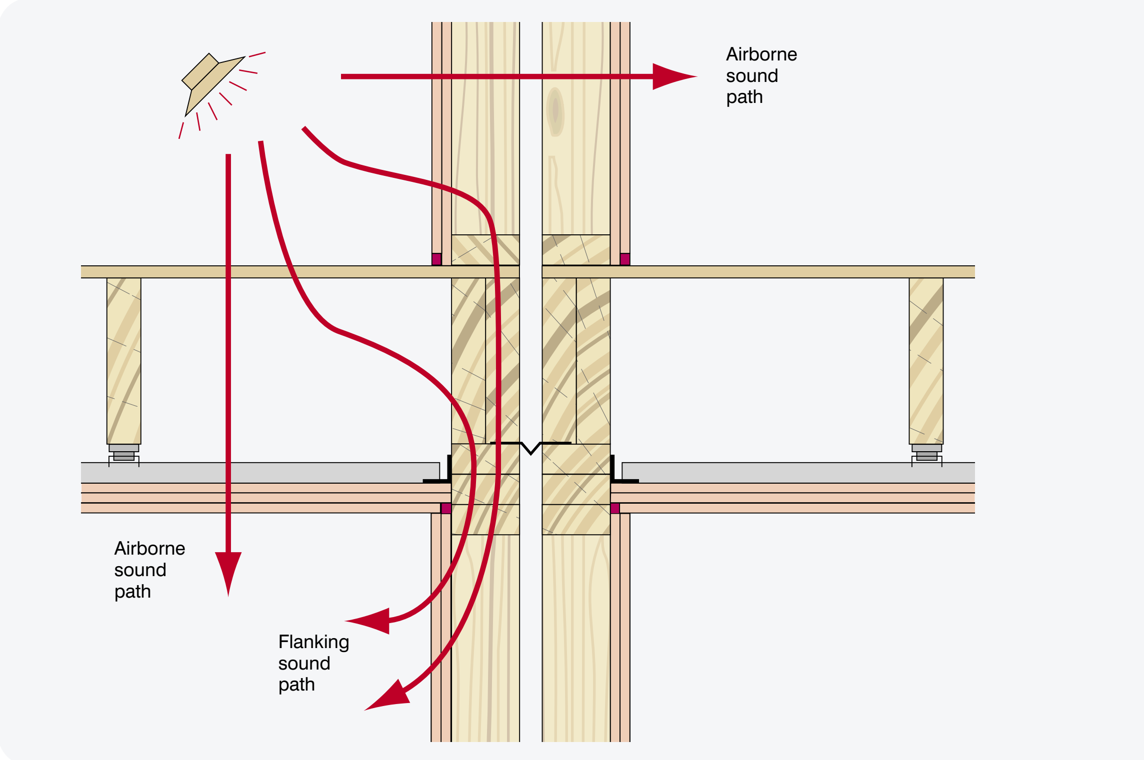


Figure 5.1: Flanking and airborne noise pathways – elevation view.

There are no minimum requirements addressing flanking noise in the NCC's Deemed-to-Satisfy Provisions, though there is an onus on designers and builders to address flanking noise in order to ensure that laboratory-tested wall and floor elements perform to their full potential in the field.

This Guide's approach is to consider reducing flanking noise paths wherever possible. The content is the result of careful thought, taking into account issues such as the limits on what could be achieved in reducing flanking because of their effect on fire and structural integrity. Even though direct reference to reducing flanking noise has not been made, many of the details incorporate elements within them.

An example of reducing flanking noise can be seen in the standard detail for floor joist and flooring over bounding walls where the joist and flooring are not continuous (Figure 5.1).

.

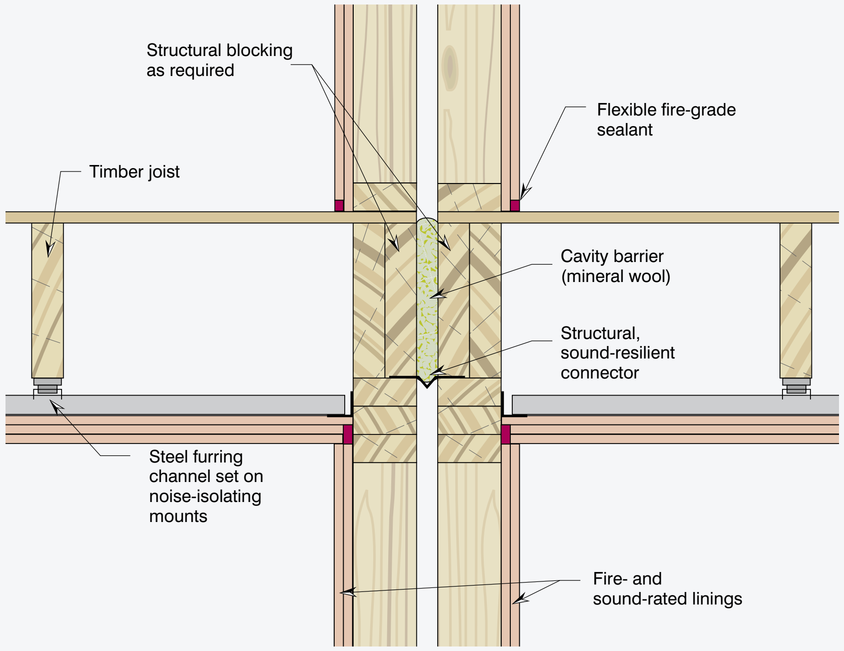


Figure 5.2: Discontinuous floor joist and floor sheeting – elevation view.

There are two main approaches used for addressing flanking noise in timber-framed buildings:

- Limit the ability of the noise to migrate from one element to another, e.g. dampening and isolation at junctions between elements (Figure 5.2).
- Limit the noise getting into wall/floor element, e.g. carpet, floating floors (Figure 5.3).

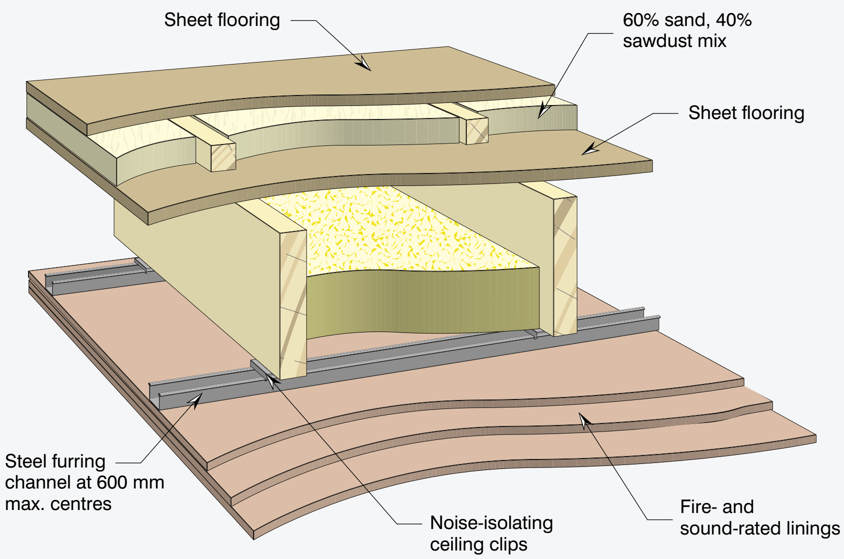


Figure 5.3: Acoustic isolating pad to reduce flanking noise.

In addition to these, timber-framed construction details orientated to improving flanking sound are provided in Section 5.3 and include:

- discontinuous elements at walls, floors and ceilings
- cavities within sound rated elements blocked or travel path increased to reduce noise
- introduced isolating elements, e.g resilient mats or brackets
- platform flooring discontinuous over double stud walls.

5.3 Strategies for Upgrading Sound Performance in Construction

Building occupants often desire higher sound performance than the NCC's minimum requirements. This is especially the case for impact sound and the related issue of vibration from footsteps, water movement through pipes, water hammer and sources such as washing machines, air conditioning units and dishwashers. Other scenarios not dealt with in the NCC include acoustic requirements for home entertainment areas, noise transfer within a dwelling and noise from outside the building (e.g. busy roads, trains, aircraft noise). Options for upgrading typical construction are provided below. Using a combination of options is more likely to give the best performance.

Isolating one side of a bounding construction from the other (e.g. using double stud cavity wall construction). This is also known as decoupling and can be useful in reducing both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.

Avoiding rigid connections between the opposing sides of isolated (decoupled) elements. This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at floor or ceiling level.

Using absorptive materials to fill wall and floor cavities (non-combustible glass fibre or mineral wool) can reduce airborne sound transmission.

Sealing sound leaks at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

For information for the upgrade of external walls refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

5.3.1 Walls

Extra mass on the walls – the addition of mass is a simple yet effective way to improve sound performance in timber construction. In its simplest form, it involves adding extra layers of material such as plasterboard to the outer layer of the sound-rated wall system.

Use a 90 mm rather than 70 mm wall studs – The deeper the wall, the better its sound performance. This is particularly the case where trying to improve C_{tr} scores (being the modification factor for low frequency bass noise applied to R_w scores). The simplest way to do this is to use 90 mm deep studs instead of 70 mm deep studs in a double stud wall system.

Upgrade batts in the wall/floor – There are many types and grades of non-combustible insulation batts in the marketplace. Sound insulation specific batts are best and high-density materials tend to outperform low-density materials. Always refer to the supplier's documented recommendations; some systems require insulation or linings to affect different frequencies and therefore may have differing advice.

5.3.2 Floors

Extra mass on the ceilings – adding mass is a simple yet important way to improve sound performance in timber-framed construction. At its simplest manifestation, this involves adding extra layers of material such as plasterboard to the sound-rated ceiling system.

Extra mass on floors – the addition of mass on floors is an effective way to address impact noise (e.g. footsteps). The additional mass can be in the form of additional layers of sheet flooring.

5.4 The Next Step

The strategies and methods shown in this Step of the Guide may involve specialist proprietary systems that go beyond the scope of this publication. As a result, the next step is:

- Go to proprietary system suppliers and ask for advice on how to integrate their systems with those discussed in this Guide. As part of this, care must be taken to ensure that the fire performance of systems in this Guide are not compromised in any way;
- Go to Step 4 to find out about fire-resisting construction requirements so that these requirements can be considered in tandem with sound requirements before selecting the appropriate timber construction system in Step 5.
- Go to Step 5 to select timber construction that will comply with minimum NCC fire requirements.

6

Step 6 – Determine NCC fire-protected timber design requirements (design development)

Designing fire-resisting construction involves a process of understanding how the NCC's Performance Requirements translate into the more objective and measurable Deemed-to-Satisfy Solutions for mid-rise timber buildings, prior to finalising the building layout and selecting timber construction systems that meet these requirements.

6.1 Utilising the Deemed-to-Satisfy Solutions for Fire Design

Section C of the NCC Volume One is concerned with safeguarding people if a building fire occurs. Specific attention is given to facilitating the evacuation of occupants and activities of emergency services personnel by restricting fire spread within a building and providing structural adequacy for the required length of time, avoiding the spread of fire between buildings, and protecting other property from damage as a result of fire.

The NCC details Deemed-to-Satisfy (DTS) Solutions that satisfy the Performance Requirements under:

Part C1 – Fire-resistance and stability

Part C2 – Compartmentation and separation

Part C3 – Protection of openings

These Parts deal with a wide range of issues but it is primarily the fire-resistance of building elements and provisions that relate specifically to mid-rise timber buildings that are dealt with in this Guide. To this end, only the more relevant clauses from Parts C1, C2 and C3 are discussed in more detail below together with the following provisions that apply specifically to mid-rise timber buildings.

- Protection of the building with an automatic fire sprinkler system complying with Specification E1.5 of the NCC (other than a FPAA101D or FPAA101H system)
- Fire-protected timber complying with Specification C1.13a of the NCC used for loadbearing internal walls, loadbearing fire walls and for elements of construction required to be non-combustible
- Any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible
- Cavity-barriers provided in accordance with Specification C1.13 of the NCC.

The NCC Deemed-to-Satisfy Provisions that facilitate the construction of mid-rise timber buildings are shown in Figure 6.1.

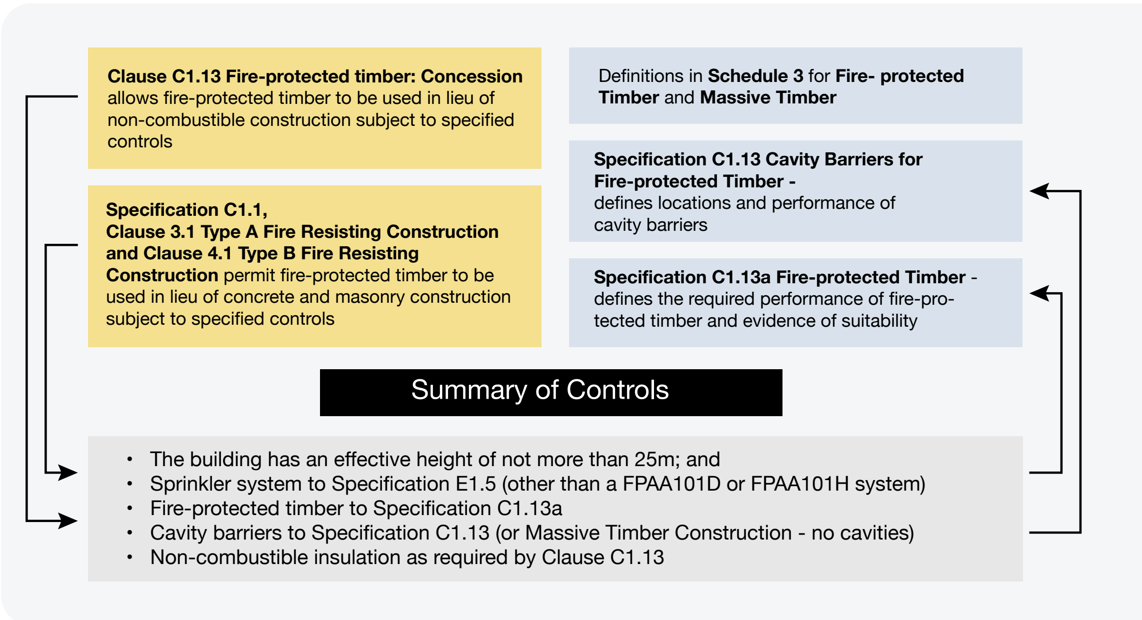


Figure 6.1: Mid-rise timber buildings overview of NCC DTS provisions.

6.2 Automatic Fire Sprinklers

A key fire safety feature for mid-rise timber buildings is the requirement to provide automatic fire sprinkler systems in accordance with NCC Specification E1.5 throughout the building (other than a FPAA101D or FPAA101H system), including any parts of the building that are not of timber construction. This requirement, in conjunction with other fire safety measures, is expected to significantly reduce the risk from fires in mid-rise timber buildings below that in other forms of construction complying with the minimum NCC DTS requirements that do not incorporate sprinkler systems.

6.2.1 Sprinkler Design Standards permitted by NCC Specification E1.5

Specification E1.5 allows sprinkler systems to be designed in accordance with:

- AS 2118.1: Automatic Fire Sprinkler Systems – General Requirements
- AS 2118.4: Automatic Fire Sprinkler Systems – Sprinkler protection for accommodation buildings not exceeding 4 storeys in height
- AS 2118.6: Combined sprinkler and hydrant systems in multi-storey buildings.

AS 2118.4 is limited to accommodation (residential) buildings not exceeding 4 storeys high. Therefore, most mid-rise timber building sprinkler systems will be designed to comply with AS 2118.1 or AS 2118.6.

Note: FPAA101D and FPAA101H systems are not currently permitted to be used in conjunction with fire-protected timber systems in accordance with the NCC DTS provisions.

6.2.2 Designing Fire Sprinkler Systems to Improve their Effectiveness

There are opportunities during the design process to incorporate features that can enhance the effectiveness of an automatic fire sprinkler system and simplify ongoing maintenance. A few examples are described below.

Fast response residential heads

Both AS 2118.1 and 2118.6 allow the use of appropriately listed fast response heads. These heads have a more rapid response than standard heads and are therefore more likely to either suppress a fire or limit its size, thus reducing the risk to occupants within the building. Therefore, where appropriate, fast response heads should be specified.

Monitored valves

The reliability of automatic fire sprinkler systems can be enhanced by providing monitored components, such as main stop valves and subsidiary stop valves. Monitored stop valves on each floor, for example, enables sprinkler protection to be maintained throughout the remainder of the building while work is undertaken on part of the sprinkler system. If the valve is left closed when the work is completed, the building owner/operator can be alerted to ensure the error is quickly corrected. This minimises the time periods and extent of areas where sprinkler protection is unavailable. The progressive installation of monitored valves during construction can be used as part of the strategy to address fires during construction by facilitating the progressive commissioning of the sprinkler system.

False ceilings

If sprinkler pipes are run above a ceiling system that is required to have Resistance to the Incipient Spread of Fire (RISF), the ceiling will need to be penetrated to accommodate sprinkler heads; potentially compromising the fire performance of the ceiling if the sprinkler system fails to operate successfully. This can be avoided by providing a false ceiling and running the pipes below the RISF ceiling and the penetrations for the sprinkler heads need only penetrate the non-fire-resisting false ceiling.

This detail also provides flexibility for lighting systems, air conditioning and other services.

Selection of materials and pipe connections

The use of materials and pipework connections that minimise the need for hot works on site and reduce the time the sprinkler system is not in operation during maintenance should be considered. The use of non ferrous sprinkler heads, piping and connections may be needed in MRI and similar environments where there are strong magnetic fields.

Note: Major modern hospitals can include a broad range of functional areas. It is therefore important to identify the appropriate hazard classifications that may apply to the sprinkler system at an early stage in the design process since it will impact on water supply requirements and potential requirements for on-site water storage and pumps.

Note: NCC Clause C1.9(e) permits some materials, including plasterboard and fibre-reinforced cement sheeting, to be used wherever a non-combustible material is required by the NCC. This dispensation does not apply for applications where non-combustible materials are specified under other legislation, e.g. specification of materials for enclosure of medical gas storage facilities.

Protection of voids/concealed spaces

Concealed spaces within fire-protected timber elements greater than 200 mm deep generally require protection in accordance with AS 2118.1 and AS 2118.6. Where these voids include elements such as beams, the void depth is measured from the soffit of the beam.

6.2.3 Hazard Classes of Occupancies for Sprinkler System Design

Sprinkler systems are classified based on hazard classes of occupancy, which depend on the expected rate of heat release rate together with the fuel loading and burning characteristics of materials in a fire compartment.

The following major classifications apply (including sub-classes).

- Light Hazard Occupancies
- Ordinary Hazard Occupancies
- High Hazard Occupancies.

The Hazard Class can have an impact on the required water supply and as consequence the building layout and costs. The Sprinkler Hazard Class(es) for a building or part of a building should be identified early in the project to determine if the existing water supply is adequate or pumps and water tanks are required to supplement the existing supply.

6.3 Fire-Protected Timber Requirements

The NCC defines fire-protected timber as fire-resisting timber building elements that comply with Specification C1.13a of the NCC.

6.3.1 Fire-Protected Timber – General Requirements

Specification C1.13a applies the following General Requirements to fire-protected timber:

- the building element must be protected to achieve the required FRL
- a non-combustible fire-protective covering must be applied to the timber; which must achieve a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

To adequately specify or check Evidence of Suitability of a fire-protected timber element, three items of information are required:

- Fire-resistance Level (FRL) – determined from AS 1530.4 test or an equivalent or more severe test
- Resistance to the Incipient Spread of Fire (RISF) – determined from AS 1530.4 test or an equivalent or more severe test
- results from a non-combustibility test in accordance with AS 1530.1 – for materials not deemed non-combustible by the NCC.

FRL is the grading period in minutes for the following three criteria expressed in the order listed below separated by forward slashes (/).

- structural adequacy – ability of a loadbearing element to support an applied load
- integrity – ability of an element of construction to resist the passage of flames and hot gases from one space to another
- insulation – ability of the surface of an element of construction, on the non-fire side of the element, to maintain a temperature below the specified limits.

For example, if an FRL of 120/60/30 is specified, the element would need to satisfy the structural adequacy criteria for 120 minutes, the integrity criteria for 60 minutes and the insulation criteria for 30 minutes. A dash means that there is no requirement for that criterion, i.e. an FRL of 90/–/– means that only the criterion of structural adequacy applies for 90 minutes.

The RISF in relation to a fire-protective covering means the covering's ability to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits.

A material is classified as non-combustible if flaming is not observed and specified temperature rise limits are not exceeded when a sample of material is exposed to the heating conditions specified in AS 1530.1; or it is 'deemed' non-combustible in accordance with NCC Clause C1.9(e).

To facilitate a consistent approach to specifying the performance of fire-protected timber the following format is recommended: **Fire-Protected Timber – FRL120/120/120:RISF45:NC.**

This means that the element must satisfy the structural adequacy, integrity and insulation requirements for 120 minutes; the RISF criteria for 45 minutes and the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

While individual test/assessment reports from NATA Accredited Testing Laboratories can be used as Evidence of Suitability, it may be more practical for Accredited Testing Laboratories to provide consolidated reports stating the performance in the above format. Further information relating to the test procedures to determine the FRL and RISF are provided in Appendix A.

Cavities are permitted within fire-protected timber elements that, without adequate measures in place, can allow fire spread through concealed spaces. The risk of fire spread from enclosure fires to the cavities is substantially reduced by the requirement for an RISF45 applied to the fire-protective covering, among other things. There is a small residual risk of fire spread to the cavity from an enclosure fire or a fire started within a cavity due to hot works, for example. The risk of fire spread via concealed spaces is further reduced by the provisions for cavity barriers and requirements for wall/ceiling cavity insulation, if present, to be non-combustible.

- Specification C1.13a deems two-layers of 13 mm fire-protective grade plasterboard fixed in accordance with manufacturer's system requirements to achieve equivalent performance to an RISF45:NC fire-protective covering.
- The timber-framed wall system in Figure 6.2 with two-layers of 13 mm fire-protective grade plasterboard either side of a cavity between studs could be classified as fire-protected timber FRL90/90/90:RISF45:NC if Evidence of Suitability (as required by the NCC) is provided for the loadbearing wall system to verify that it achieves an FRL of 90/90/90 under similar or more severe load conditions.
- This evidence would normally be an AS 1530.4 fire test report from an Accredited Testing Laboratory. The RISF45 for two-layers of 13 mm thick fire-protective grade plasterboard does not require further verification since it is Deemed-to-Satisfy the 45-minute requirement and plasterboard is also deemed non-combustible by Clause C1.9(e) of the NCC.
- The primary objective for the inclusion of the non-combustibility requirement for the fire-protective covering is so that the reaction of the fire-protected timber to external and enclosure fires is comparable to elements of construction that are non-combustible: such as reinforced concrete or steel protected with non-combustible materials.

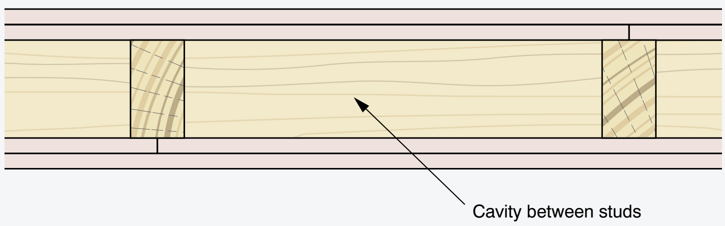


Figure 6.2: Horizontal section through typical FRL120/120/120: RISF45:NC timber stud wall showing allowable cavity.

The primary objective for the specification of RISF45 is to reduce the risk of the timber structural elements being ignited prior to burn-out of the contents or fire brigade intervention, in the unlikely event of the automatic fire sprinkler system failing. To achieve this, it is necessary that the RISF performance is not compromised by the presence of building service penetrations and openings for doors and windows. Refer to the relevant sections in this Step for further details on how the RISF performance can be maintained through appropriate penetration fire stopping systems, cavity barriers and lining of openings.

6.3.2 Relaxations for Massive Timber Panels

The NCC permits the General Requirements for fire-protected timber to be 'relaxed' if both the following additional criteria are satisfied:

- the minimum thickness of timber panels is not less than 75 mm
- there are no cavities between the surface of the timber and the fire-protective covering system or between timber members.

This 75 mm dimension relates to the minimum dimension of the dressed/finished timber member. In most instances, massive timber elements will have minimum dimensions much greater than 75 mm to meet the structural adequacy and integrity criteria of AS 1530.4. Typical examples of massive timber installations satisfying the conditions are shown in Figure 6.3. The rationale for allowing the 'concession' for massive timber is that it is reasonable to reduce the performance of the fire-protective covering, subject to maintaining the required FRL, because the consequences of ignition of timber structural members are significantly reduced:

- Timber with a large cross-section can achieve high fire-resistance levels due to its relatively high inherent fire resistance allowing it to continue to support an imposed load or maintain a fire separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for a greater period than lightweight construction.
- By not permitting any concealed spaces between the timber and fire-protective coverings or between timber members, the risk of fire spread through concealed cavities is mitigated.
- If the massive timber conditions are met, the following requirements can be adopted for fire-protected timber in lieu of the General Requirements:
  - The building elements must be protected to achieve the required FRL and have a non-combustible fire-protective covering applied to the timber which achieves the Modified Resistance to the Incipient Spread of Fire (MRISF) of not less than the values stated in Table 6.1 when tested in accordance with AS 1530.4.
  - The Modified Resistance to the Incipient Spread of Fire (MRISF) is determined in accordance with Clause 3 of NCC Specification C1.13a. Further information relating to the test procedures to determine the Fire Resistance and the MRISF are provided in Appendix A.

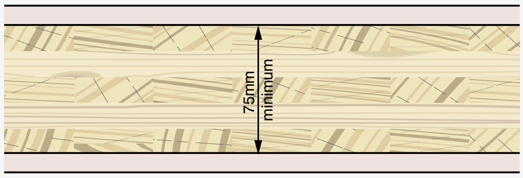
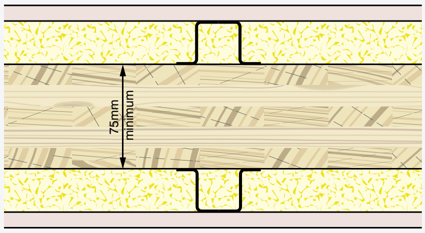
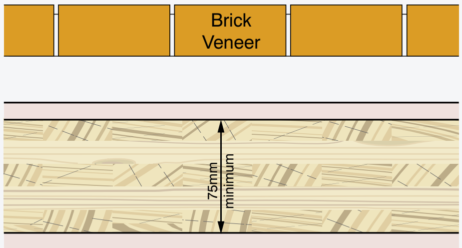
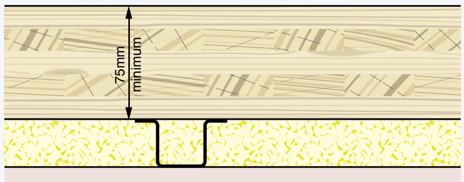
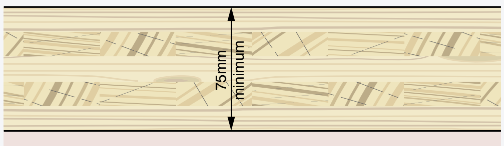
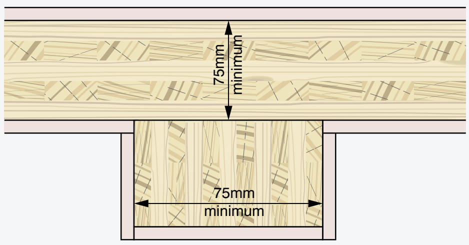
Description	Schematic section
Massive Timber Wall Panels	
Fire-protective covering direct fix to massive timber panel.	
Multi-layer fire-protective covering system direct fix to massive timber panel.	
External brick veneer wall – Note massive timber is faced on both sides with fire-protective coverings.	
Massive Timber Floor Panels	
Multi-layer fire-protective covering system direct fixed to the underside of the massive timber panel.	
Fire-protective covering direct fix to massive timber panel.	
Profiled ceiling fire-protective covering system follows the floor profile maintaining contact with massive timber element.	

Figure 6.3: Typical massive timber panel details for which the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria may be applied.

To facilitate a consistent approach to specifying the performance of fire-protected massive timber the following format is recommended:

Fire-Protected Timber – FRL90/90/90: MRISF30: NC

This means that the element must satisfy:

- the structural adequacy, integrity and insulation requirements for 90 minutes
- the Modified Resistance to the Incipient Spread of Fire criteria for 30 minutes
- the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

Table 6.1: Minimum fire-protective covering requirements – Massive timber.

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)	Minimum Deemed-to-Satisfy Fire-protective Grade Plasterboard
Inside a fire-isolated stairway or lift shaft	20 min	1 layer x 13 mm thick
External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	45 min	2 layers x 13 mm thick
All other applications	30 min	1 layer x 16 mm thick

Table 6.1 is a derivative of Specification C1.13a Table 1 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

Table 6.1 also includes Deemed-to-Satisfy fire-protective grade plasterboard minimum requirements if fixed in accordance with the manufacturer's system requirements in order to achieve the required FRL of the element for massive timber.

For example, if a non-loadbearing wall system is required to achieve an FRL of –/60/60, an appropriate specification for a massive timber element would be:

Fire-Protected Timber – FRL–/60/60: MRISF30: NC

If there is appropriate Evidence of Suitability to show a massive timber element can achieve an FRL of –/60/60 when protected by 16 mm fire-protective grade plasterboard, then no further evidence is required since the 16 mm thick plasterboard is Deemed-to-Satisfy the MRISF30 requirement and the plasterboard is also deemed to be non-combustible.

6.3.3 Fire-protected Timber Smoke-proof Walls (application of Specification C2.5)

Smoke-proof walls are used for sub-compartmentation in both Class 9a (e.g. hospitals) and Class 9c (e.g. aged care buildings). When implementing the fire-protected timber options, if the smoke walls are of timber-framed or massive timber construction, it is necessary for the level of encapsulation to be consistent with the minimum required for fire-resisting elements of construction. To achieve this, the required performance of fire-protective coverings and protection of openings and service penetrations needs to be increased.

To simplify the implementation, the requirements for Class 9a and 9c buildings have been consolidated and increased requirements for fire-protective coverings and protection of openings and service penetrations have been included in the following specification.

Fire-protected smoke-proof walls:

- Must be of fire-protected timber construction and extend to:
  - the floor above; or
  - a non-combustible roof covering; or
  - a ceiling having a resistance to the incipient spread of fire to the space above itself of not less than 60 minutes for a Class 9a building or 45 minutes for a Class 9c building subject to cavity barriers being provided directly above the smoke wall position within the ceiling cavity. The cavity barriers shall comply with NCC specification C1.13.
- Not incorporate any glazed areas unless the glass is safety glass as defined in AS 1288.
- Only have doorways that are fitted with smoke doors complying with Specification C3.4 of the NCC.

Note: While the NCC uses the term smoke proof and states that smoke doors must be constructed so that smoke will not pass from one side of the doorway to the other, smoke-proof walls constructed in accordance with NCC Specification C2.5 and smoke doors in accordance with the DTS provisions within Specification C3.4 may allow appreciable volumes of smoke to spread to adjacent smoke compartments particularly around the perimeter of smoke doors.

To manage the risk of smoke spread around the perimeters of doors, consideration should be given to the selection of appropriate and compatible door and seal combinations which can be evaluated by testing to AS1530.7 and requiring installation and construction in accordance with AS 6905.

- Have all openings around penetrations and the junctions of the smoke-proof wall and the remainder of the building stopped to prevent the free passage of smoke and maintain the Resistance to the Incipient Spread of Fire performance of fire-protected elements of construction and the required fire-resistance performance of elements of construction.
- Incorporate smoke dampers where air-handling ducts penetrate the wall (or cavity barrier above a wall) unless the duct forms part of a smoke hazard management system required to continue air movement through the duct during a fire.

Typical examples are shown in Figure 6.4.

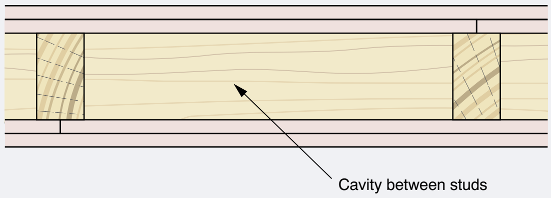
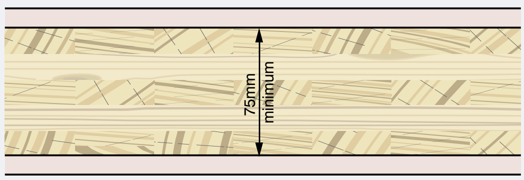
Description	Schematic section
Timber framed smoke-proof wall lined with two layers of 13 mm fire-protective grade plasterboard on each face (Evidence of Suitability to demonstrate the ability of fixings to retain plasterboard in place for ≥45 minutes in a standard fire-resistance test)	
Massive timber smoke-proof wall lined with one layer of 16 mm fire-protective grade plasterboard each side (Evidence of Suitability to demonstrate the ability of fixing to retain plasterboard in place for ≥30 minutes in a standard fire-resistance test)	

Figure 6.4: Typical examples of fire-protected timber smoke-proof walls.

The following requirements apply to the protection of doorways in smoke-proof walls.

A door required by C2.5 or Specification 2.5 of the NCC to be smoke-proof or have an FRL, other than one that serves a fire compartment provided with a zone pressurisation system in accordance with AS 1668.1, must provide a smoke reservoir by not extending within 400 mm of the underside of:

- a roof covering; or
- the floor above; or
- an imperforate false ceiling that will prevent the free passage of smoke.

Doors in smoke-proof walls have to comply with Specification C3.4, which states that smoke doors must be constructed so that smoke will not pass from one side of the doorway to the other and, if they are glazed, there is minimal danger of a person being injured by accidentally walking into them. The clause provides the following Deemed-to-Satisfy construction requirements for smoke doors with one or two leaves:

- The leaves are side-hung to swing:
  - in the direction of egress; or
  - in both directions.
- The leaves are solid-core and at least 35 mm thick, or are capable of resisting smoke at 200°C for 30 minutes.
- The leaves are fitted with smoke seals.
  - The leaves are normally in the closed position; or
  - (A) The leaves are closed automatically with the automatic closing operation initiated by smoke detectors, installed in accordance with the relevant provisions of AS 1670.1, located on each side of the doorway not more than 1.5 m horizontal distance from the doorway; and
    (B) in the event of power failure to the door, the leaves fail-safe in the closed position.
- The leaves return to the fully closed position after each manual opening.
- Any glazing incorporated in the door complies with AS 1288.
- If a glazed panel is capable of being mistaken for an unobstructed exit, the presence of the glass must be identified by an opaque mid-height band, mid-rail, crash-bar or other opaque construction.

6.4 Selection of NCC DTS Compliance Pathways for Mid-rise Class 9c Buildings

Class 9c buildings of 3 or more storeys are required to be of Type A construction. Using this classification, the Fire Resistance Levels, smoke-resistance and fire-protected timber requirements applicable to various elements of construction can be derived.

Before proceeding further, it is important to decide the most appropriate pathway to follow to demonstrate compliance with the NCC DTS requirements. For mid-rise Class 9c buildings, there are two main pathways that can be followed to demonstrate compliance if timber construction is to be adopted:

- Pathway 1 Fire-protected timber option
- Pathway 2 Hybrid timber option

6.4.1 NCC Compliance Pathway 1: Fire-protected timber option (this Guide)

Note: The fire-protected timber requirements must be applied to all timber structural elements, external walls that include structural timber elements and non-loadbearing timber walls that are required by the NCC to provide fire and/or smoke separation functions.

Under Pathway 1, fire-protected timber can be used for external walls and the construction of lift and fire stair shafts, among other things, but other timber elements of construction such as internal fire-resisting walls, floors and smoke-proof walls must also satisfy the fire-protected timber requirements. This provides consistent levels of encapsulation for timber elements intended to be fire or smoke resistant throughout the building.

As a consequence, the levels of encapsulation for timber smoke-proof walls, floors and fire-resisting internal walls may be greater than those under Pathway 2.

6.4.2 NCC Compliance Pathway 2: Hybrid option without fire protected timber

Note: There is a NSW variation in the NCC that requires additional protection to some of the timber elements that are permitted to be used under Pathway 2

Under Pathway 2, timber construction is permitted to be used for internal walls (excluding lift and fire stair shafts), floors and smoke walls. The required levels of protection for the timber elements are prescribed within the NCC DTS provisions.

However, the use of timber construction for external walls, and lift and fire stair shafts, is not permitted under Pathway 2. Non-combustible construction must be used for external wall construction and masonry or concrete construction is required for stair and lift shafts. Hence a hybrid approach needs to be adopted.

Where these constraints are acceptable, the hybrid timber option may be preferred. For more information on this pathway reference should be made to *WoodSolutions Technical Design Guide #42*.

This Guide addresses Pathway 1.

6.5 Requirements for Fire-resistant Elements and Smoke-proof Walls in Class 9c Buildings

A typical section through a mid-rise Class 9c Aged Care Building is shown in Figure 6.5. It comprises a below-ground reinforced concrete carpark above which are timber aged care general and resident use areas.

For the timber part of the building, typically timber frame construction may be adopted because individual rooms have closely spaced walls that in effect form a 'honeycombed' structure with many individual load paths and, as such, the use of lightweight timber-framed systems combined with fire protective coverings is an efficient form of construction. Alternatively, a solid massive timber system, or a mixture of massive and lightweight timber construction may be considered.

Post and beam construction for the main structural frame may be considered, allowing greater design flexibility.

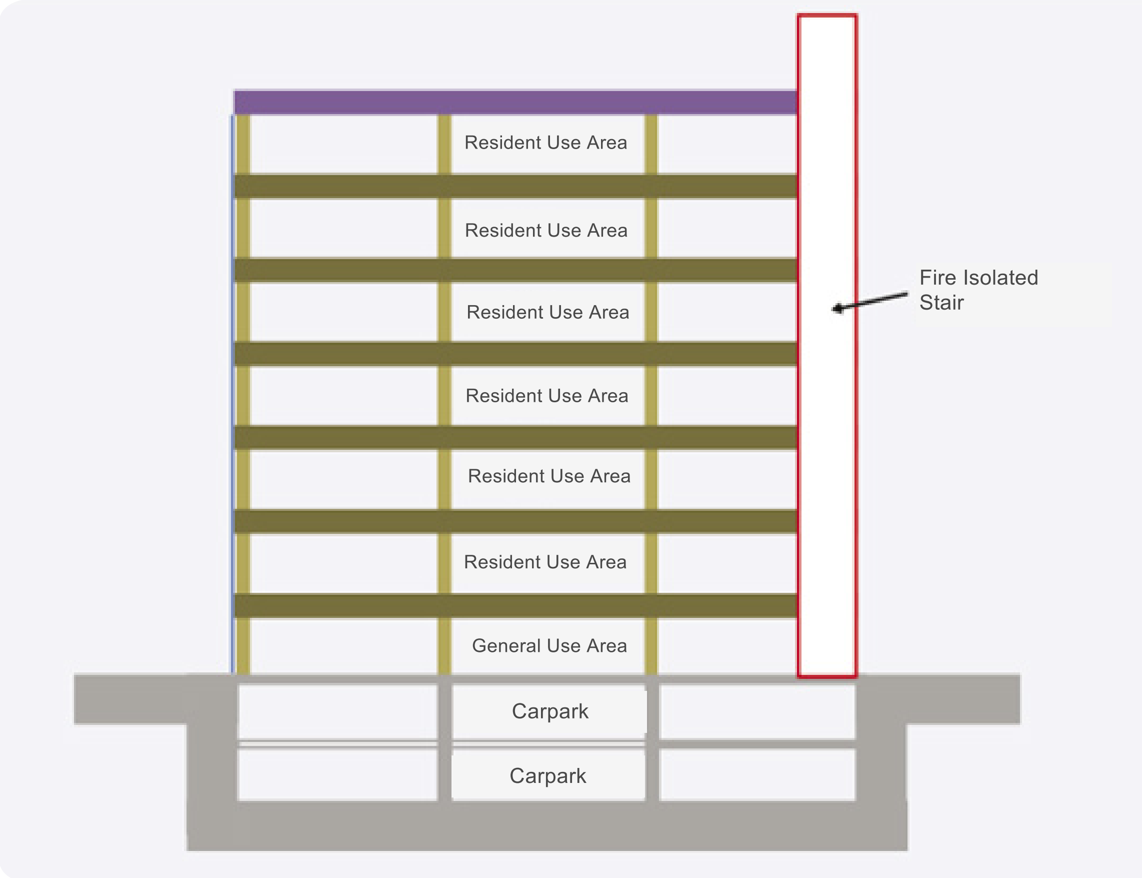


Figure 6.5: Typical Schematic Section through mid-rise Aged Care Building.

A typical floor plan of a residential floor is shown in Figure 6.6.

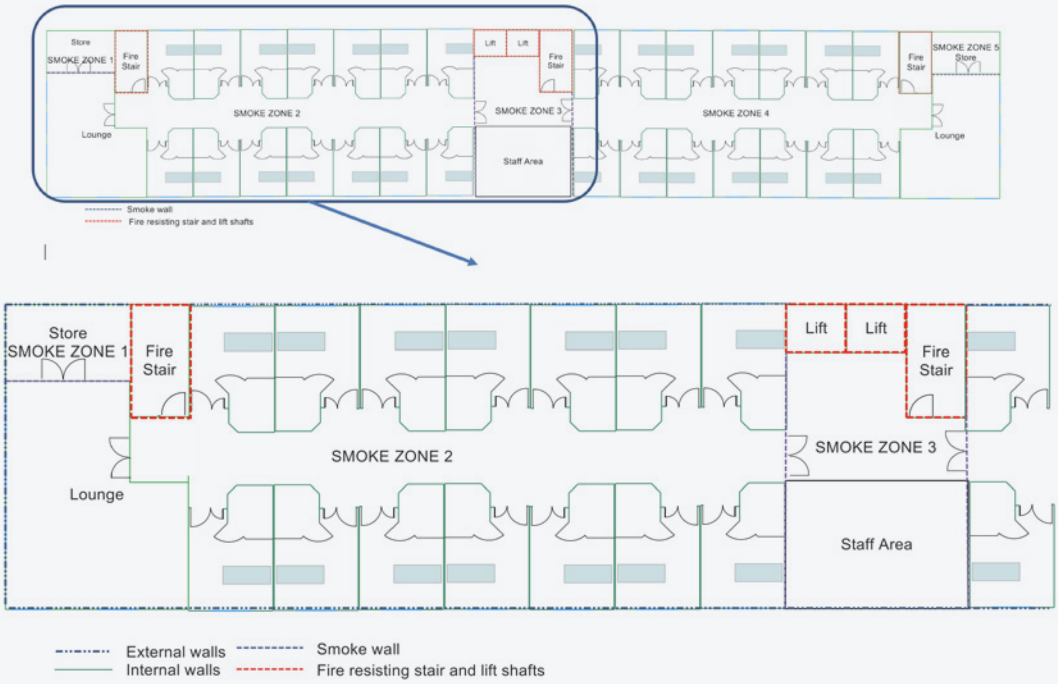


Figure 6.6: Typical Floor of a Class 9c Building.

Note: The requirements for fire-protected timber apply to all elements required to have an FRL or be non-combustible and to smoke-proof walls in addition to the prescribed FRLs and forms of construction.

An overview of the general FRL requirements for a Class 9c building and other classes that may form part of a Class 9c building are summarised in Table 6.2. Refer to the sections following the table for more detailed explanations and details of required smoke compartmentation.

Table 6.2: Overview of FRLs applicable to a Class 9c aged care building.

Description	FRL – Structural Adequacy /Integrity/Insulation (minutes)			
	Class 9c	Class 5, 7a or 9b	Class 6	Class 7b or 8
External wall – maximum[1]	120/120/120	120/120/120	180/180/180	240/240/240
Fire stair shaft	120/120/120	120/120/120	180/120/120	240/120/120
Service Shaft[2]	60/60/– or –/60/–	120/90/90	180/120/120	240/120/120
Internal loadbearing walls	60/–/–	120/–/–	180/–/–	240/–/–
Lift Shaft walls	120/120/120	120/120/120	180/120/120	240/120/120
Common Walls and Fire Walls	60/60/60	120/120/120	180/180/180	240/240/240
Door to fire Stair	–/60/30	–/60/30	–/60/30	–/60/30
Fire Door to service shaft	–/60/30	–/60/30	–/60/30	–/60/30
Lift door	–/60/–	–/60/–	–/60/–	–/60/–
Fire doors to services risers[3]	–/60/30	–/60/30	–/60/30	–/60/30
Floors	60/60/60	120/120/120	180/180/180	240/240/240
Roofs	120/ 60/ 30	120/ 60/ 30	180/ 60/ 30	240/ 90/ 60

Table 6.2 is a derivative of Specification C1.1 Table 3 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

Note 1 External wall FRLs vary with distance to the fire source feature and whether the element is loadbearing.

Note 2 The NCC is open to interpretation in relation to treatment of service shafts. Check interpretation with the appropriate authority before implementing.

Note 3 Not required if service penetrations are protected between floors.

6.5.1 Fire and Smoke Compartmentation Mid-Rise Class 9c Buildings

Fire separation between floors

Generally, each floor forms a fire compartment. Levels within the Class 9c part of the building are required under NCC Clause C2.5(b) to be fire separated by floors with an FRL of not less than 60/60/60. The slab separating the Class 7a and 9c parts has to comply with NCC Specification C1.1 Table 3 for the lower storey in accordance with NCC Clause 2.9 which requires an FRL of 120/120/120.

Sub-compartments

NCC Clause C2.5(b) requires Class 9c buildings to be subdivided into smoke compartments having a maximum floor area of 500 m^2 and hazardous ancillary areas that contain equipment or materials that are a high potential fire hazard are also required to be separated from the rest of the patient care area by smoke-proof walls. These areas include but are not limited to;

- a kitchen and related food preparation areas with floor area of more than 30 m^2.
- a room containing a hyperbaric facility (pressure chamber).
- a room used for the storage of medical records having a floor area of more than 10 m^2.
- a laundry, where items of equipment are of the type that are potential fire sources.

In the example shown in Figure 6.6, the stores adjacent to the lounge area at the end of each wing contain high levels of combustibles and have a floor area greater than 10 m^2 and have therefore been treated as a hazardous ancillary area. As such, the store will be smoke separated from the remainder of the wing with smoke-proof walls. This requires one smoke wall since the store boundaries include external walls on two sides and a fire stair on the third side, subject to all walls being adequately sealed to prevent smoke spread.

Each of the wings was marginally over 500 m^2 in floor area but by enclosing the store with a smoke wall the area was reduced to less than 500 m^2. It is then practical to integrate the smoke-proof walls with the boundary separating the staff area from the resident wings creating five smoke zones per floor as shown in Figure 6.6 (smoke-proof walls shown as purple dashed lines).

This arrangement is considered a good solution since a lift lobby is created with a large volume facilitating a phased horizontal evacuation in an emergency and the location of the smoke walls ties in with the functional use of the spaces.

Note: When defining smoke compartments, consider the evacuation strategy for the building and potential fire locations to optimise life safety; in addition to consideration of the functional use of the building and access for maintenance and inspection of smoke-proof walls and associated service penetrations.

Shafts

Lift and stair shaft walls are required to achieve an FRL of at least 120/120/120, or –/120/120 if non-loadbearing, and may be constructed from masonry, concrete or fire-protected timber.

There is no specific statement regarding the treatment of walls forming ventilating, pipe, garbage shafts that are not used for the discharge of hot products of combustion however these shafts are grouped under internal walls in Table 3 of Specification C1.1 of the NCC. If the shafts are loadbearing, an FRL of 60/–/– would apply if a literal interpretation is made of Clause C2.5(b)(iii) of the NCC.

Clause C2.5(b)(vi) infers the intent of the NCC by requiring openings in fire walls, other than doorways and windows, to be protected by construction having an FRL not less than –/60/–.

A reasonable interpretation of the intent of C2.5 with respect to internal walls forming ventilating, pipe, garbage and the like shafts not used for the discharge of hot products of combustion would be:

- non-loadbearing service shaft walls require an FRL of at least –/60/–
- loadbearing service shaft walls require an FRL of at least 60/60/–
- service penetrations and other openings in the internal shaft walls are required to be protected by systems that will achieve an FRL not less than FRL of –/60/–
- service shafts should be of non-combustible construction or fire-protected timber construction
- if the option of fire-protected timber shafts is adopted, service penetrations must be protected by systems that will maintain the Resistance to the Incipient Spread of Fire (RISF) or modified resistance to the incipient spread of fire (MRISF) required for fire-protected timber elements.

This interpretation should be confirmed with the appropriate authority prior to documenting the required FRLs for service shafts and treatment of service penetrations.

Note: Where external walls are required to be of non-combustible or fire-protected timber construction, internal and external linings including rainscreens form part of the external wall and must be non-combustible.

Note: For non-sprinkler protected buildings the NCC DTS provisions require vertical separation of openings, but the NCC waives the requirements if automatic fire sprinklers are provided in accordance with Specification E1.5. The removal of requirements for spandrel panels or horizontal projections simplifies construction and provides greater design flexibility.

Note: Even though non-loadbearing external walls do not require an FRL if more than 3 metres from a fire-source feature, if timber external walls are used, the fire-protective coverings must be applied since the external wall would otherwise be required to be non-combustible. This is to address the risk from external fires on balconies or external areas adjacent to the building and the risk of vertical fire spread through openings if a fully developed fire occurs.

6.5.2 Fire Resistance of External Walls of Mid-Rise Class 9c Buildings

In addition to maintaining loadbearing capacity when subjected to fires within a building, the external walls also need to address the risk of fire spread via the building facade under the following scenarios:

- Fire spread from adjacent buildings (or the fire source feature as defined in the NCC) to the subject building. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by means of specification of minimum separation distances, fire-resisting construction, and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Fire spread from the subject building to the fire source feature as defined in the NCC or adjacent buildings. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by specifying minimum separation distances, fire-resisting construction, and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.
- Fire spread from an external fire source adjacent to the façade other than adjacent fire sources such as balcony fires. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction for loadbearing elements and the requirement for external walls to be non-combustible or of fire-protected timber construction. In addition, sprinkler protection of external balconies is required for some applications
- Vertical fire spread between openings from a fully developed fire within the subject building. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by the requirement for external walls to be non-combustible or of fire-protected timber construction, and by providing automatic fire sprinklers.

The FRLs required for external walls are nominated in NCC Specification C1.1 and depend on the Class of Building, Type of Construction and proximity to the boundary (fire source feature) or other buildings. The focus of this section is Class 9c buildings of Type A construction and the required fire-resistance levels are summarised in Table 6.3 Other classes have been included since they may apply to other parts of the building (e.g. Figure 6.5 where Class 7a applies to the carpark levels).

Table 6.3: FRLs for external walls of mid-rise Class 5 to 9 buildings of Type A construction.

Distance from fire source feature	Loadbearing (Y/N)	FRL – Structural Adequacy /Integrity/Insulation (minutes)		
		Class 5, 7a or 9a, b and c	Class 6	Class 7b or 8
<1.5 m	Y	120/120/120	180/180/180	240/240/240
	N	–/120/120	–/180/180	–/240/240
≥1.5 and <3 m	Y	120/90/90	180/180/120	240/240/180
	N	–/90/90	–/180/120	–/240/180
≥3 m	Y	120/60/30	180/120/90	240/180/90
	N	–/–/–	–/–/–	–/–/–
External columns	Y	120/–/–	180/–/–	240/–/–

Table 6.3 is a derivative of Specification C1.1 Table 3 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

6.5.3 Structural Fire Resistance of Mid-Rise Class 9c Buildings

The requirements for structural fire resistance for Class 9c buildings are unusual in that Clause C2.5(b) provides a dispensation for floors and internal loadbearing walls reducing the period the structural adequacy has to be maintained to 60 minutes when subjected to a standard fire-resistance test for floors and internal loadbearing walls but retains the requirement for external loadbearing walls to maintain structural adequacy for a minimum of 120 minutes and for fire stair shafts and lift shafts to achieve FRLs of at least 120/120/120. To achieve this configuration the requirements of Specification C1.1, including the support for another part criteria, are waived for the elements requiring a FRL of 60/–/– and 60/60/60.

Note: The NCC Guide to Volume One indicates that the lower FRL allowed by Clause C2.5(b) (iii) recognises the effectiveness of the required sprinkler systems in Class 9c buildings. If a Performance Solution is being developed that varies some aspects of the DTS Solution, the analysis should have regard for the impact of the dispensation permitting lower Fire Resistance Levels.

6.6 Requirements for Fire-resistant Elements and Smoke-proof Walls in Class 9a Buildings

An indicative schematic section through a mid-rise Class 9a hospital is shown in Figure 6.7. It comprises a below-ground reinforced concrete carpark above which is a ground level of fire-protected timber construction comprising retail outlets and cafes, the emergency department and the main reception. Fire-protected timber construction is continued throughout the upper levels, which provide various functional areas including inpatient wards.

For the timber part of the building, timber frame construction may be preferred where individual rooms have closely spaced walls that in effect form a 'honeycombed' structure with many individual load paths. Alternatively, a solid massive timber system, or a mixture of massive and lightweight timber construction may be considered as well as using post and beam construction to provide larger open areas and greater design flexibility.

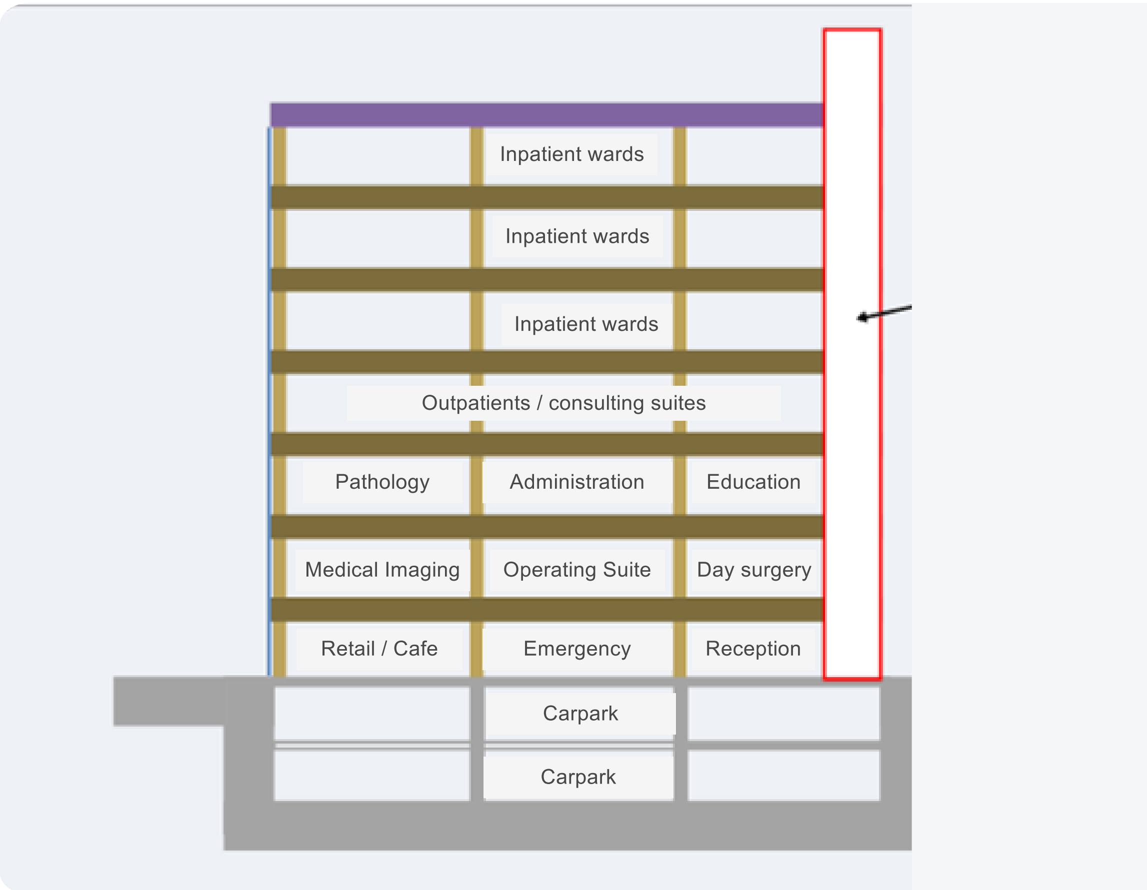


Figure 6.7: Typical Schematic Section through mid-rise hospital building.

A typical plan of a floor with inpatient wards is shown in Figure 6.8 to provide an example of the application of the fire and smoke compartmentation required for hospital buildings.

An overview of the general FRL requirements for a Class 9a building and other classes that may form part of a Class 9a building are summarised in Table 6.4.

Note: Since the required FRLs can vary significantly if the Class that applies to part of a hospital building is changed, it is important to clarify the classifications that the appropriate authority will be applying at an early stage in the project.

Note: The requirements for fire-protected timber apply to all elements required to have an FRL or be non-combustible and to smoke-proof walls in addition to the prescribed FRLs and forms of construction.

Table 6.4: Overview of FRLs applicable to a Class 9a hospital building.

Description	FRL – Structural Adequacy /Integrity/Insulation (minutes)			
	Class 9a	Class 5, 7a or 9b	Class 6	Class 7b or 8
External wall – maximum[1]	120/120/120	120/120/120	180/180/180	240/240/240
Fire stair shaft	120/120/120	120/120/120	180/120/120	240/120/120
Service Shaft[2]	120/90/90	120/90/90	180/120/120	240/120/120
Internal loadbearing walls	120/–/–	120/–/–	180/–/–	240/–/–
Lift Shaft walls	120/120/120	120/120/120	180/120/120	240/120/120
Common Walls and Fire Walls	120/120/120	120/120/120	180/180/180	240/240/240
Door to fire Stair	–/60/30	–/60/30	–/60/30	–/60/30
Fire Door to service shaft	–/60/30	–/60/30	–/60/30	–/60/30
Lift door	–/60/–	–/60/–	–/60/–	–/60/–
Fire doors to services risers[3]	–/60/30	–/60/30	–/60/30	–/60/30
Floors	120/120/120	120/120/120	180/180/180	240/240/240
Roofs	120/ 60/ 30	120/ 60/ 30	180/ 60/ 30	240/ 90/ 60

Table 6.4 is a derivative of Specification C1.1 Table 3 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

Note 1 External wall FRLs vary with distance to the fire source feature and whether the element is loadbearing.

Note 2 The NCC is open to interpretation in relation to treatment of service shafts. Check interpretation with the appropriate authority before implementing.

Note 3 Not required if service penetrations are protected between floors.

Refer to the following sections for more detailed explanations and for details of smoke compartmentation and relevant sections of this Guide for additional criteria such as the resistance to the incipient spread of fire (RISF) requirements for fire-protected timber construction.

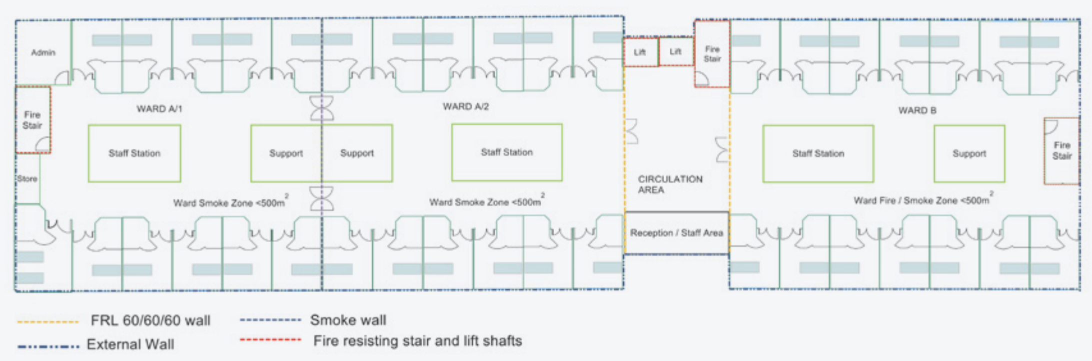


Figure 6.8: Typical ward level of a Class 9a building.

Refer to the following sections for more detailed explanations and for details of smoke compartmentation and relevant sections of this Guide for additional criteria such as the Resistance to the Incipient Spread of Fire (RISF) requirements for fire-protected timber construction.

6.6.1 Fire and Smoke Compartmentation Mid-Rise Class 9a Buildings

Fire separation between floors

Generally, each floor forms a fire compartment and the floor is required to achieve an FRL of 120/120/120 within Class 9a parts. (Refer Table 6.4 for examples of FRLs that apply to parts of a hospital building where more than one class may apply).

If a building has multiple classifications, determining the FRLs can become more complex and it is critical at an early stage that the classifications of the various parts of the building are determined and confirmed by the appropriate authority.

For the example shown in Figure 6.7, if the retail areas are small and considered ancillary to the hospital functions it may be determined that the entire building can be considered as a Class 9a building except for the carpark levels which would be treated as a Class 7a part. The slab separating the Class 7a and 9a parts has to comply with Specification C1.1 Table 3 for the lower storey in accordance with NCC Clause 2.9, which also requires an FRL of 120/120/120.

However, if the retail area is large it may be determined that it should be treated as a separate part of the building and Class 6 would be applied. This would require, among other things, the floor and fire wall separating the Class 9a and Class 6 parts to have an FRL of 180/180/180 and elements supporting the floor above the Class 6 part to have an FRL for structural adequacy of 180 minutes. The structural adequacy applicable to shafts also has to be increased from 120 minutes to 180 minutes.

This example illustrates the impact that the change of classification of part of a building can have on the building as a whole and the need for these matters to be resolved in the early stages of a project.

Compartments and sub-compartments

The maximum floor area of a fire compartment in a Class 9a building must not exceed 5,000 m2 (NCC Clause C2.2) and patient care areas must be divided into fire compartments not exceeding 2,000 m^2 (NCC Clause C2.5).

The example in Figure 6.8 has a patient care area/level less than 2,000 m^2 and therefore subdivision by fire walls in patient care areas is not required if each floor is fire separated from the floor above.

NCC Clause C2.5(a) requires additional fire and smoke compartmentation of Patient Care Areas in Class 9a buildings which are summarised in Table 6.5.

Table 6.5: Additional fire and smoke compartmentation for Class 9a patient care areas.

Part	Smoke Compartment (smoke-proof walls, NCC Spec C2.5)	Fire Compartmentation Max areas	
		Sub Compartment FRL 60/60/60	Fire Compartment FRL 120/120/120
Treatment Area	1,000m^2	–	2,000m^2
Ward Areas	500m^2	1,000m^2	
Hazardous Ancillary Areas within Patient Care Areas[1]	–	Fire separation from remainder of Patient Care Area	

Note 1 Hazardous Ancillary areas that contain equipment or materials that are a high potential fire hazard are therefore required to be separated from the rest of the patient care area by walls having an FRL of not less than 60/60/60 and include but are not limited to,

- a kitchen and related food preparation areas with floor area of more than 30 m^2
- a room containing a hyperbaric facility (pressure chamber)
- a room used for the storage of medical records having a floor area of more than 10 m^2
- a laundry, where items of equipment are of the type that are potential fire sources.

The application of these requirements for additional fire and smoke compartmentation within patient care areas are shown in Figure 6.8.

In this example, the area of the store in Ward A/1 is below 10 m^2 and is not anticipated to represent a hazardous area and therefore does not require fire separation (FRL 60/60/60) from the remainder of the ward. No other potentially hazardous ancillary areas were identified.

Ward A is a double ward with an area greater than 500 m^2 but less than 1,000 m^2. Smoke compartments each with an area less than 500 m^2 need to be provided. In this case the obvious location of the smoke wall is at the interface between the two standard-sized ward areas (A/1 and A/2). In addition, a fire sub-compartment is required to separate Ward A from the general circulation zone. The required FRL is 60/60/60.

Ward B has a floor area less than 500 m^2 and therefore the smoke compartmentation from the remainder of the building also needs to provide fire sub-compartmentation with a required FRL of –/60/60.

Note: Where external walls are required to be of non-combustible or of fire protected timber construction, internal and external linings including rainscreens form part of the external wall and must be non-combustible.

Note: For non-sprinkler protected buildings the NCC DTS provisions require vertical separation of openings, but the NCC waives the requirements if automatic fire sprinklers are provided in accordance with Specification E1.5. The removal of requirements for spandrel panels or horizontal projections simplifies construction and provides greater design flexibility.

Note: Even though non-loadbearing external walls do not require an FRL if more than 3 metres from a fire-source feature, if structural timber external walls are used, the fire-protective coverings must be applied. The external wall would otherwise be required to be non-combustible. This is to address the risk from external fires on balconies or external areas adjacent to the building and the risk of vertical fire spread through openings if a fully developed fire occurs.

The arrangement shown in Figure 6.8 is consistent with a typical progressive horizontal evacuation strategy. If a fire occurs in a ward area the occupants can be evacuated to the next smoke or fire sub-compartment initially and then, if necessary, progressively from the building. The large circulation area facilitates this phased evacuation process by providing adequate space for the patients and staff.

6.6.2 Fire Resistance of External Walls of Mid-Rise Class 9a Buildings

In addition to maintaining loadbearing capacity when subjected to fires within a building, the external walls also need to address the risk of fire spread via the building façade under the following scenarios:

- *Fire spread from adjacent buildings (or the fire source feature as defined in the NCC) to the subject building.* Under the DTS Solution pathway for mid-rise timber buildings this is addressed by means of specification of minimum separation distances, fire-resisting construction, and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- *Fire spread from the subject building to the fire source feature as defined in the NCC or adjacent buildings.* Under the DTS Solution pathway for mid-rise timber buildings this is addressed by specifying minimum separation distances, fire-resisting construction, and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.
- *Fire spread from an external fire source adjacent to the façade other than adjacent fire sources including balcony fires.* Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction for loadbearing elements and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- *Vertical fire spread between openings from a fully developed fire within the subject building.* Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers (or horizontal projections or spandrel panels if sprinkler systems are not provided).

The FRLs required for external walls are nominated in NCC Specification C1.1 and depend on the building use (Class of Building), Type of Construction and proximity to the boundary (fire source feature) or other buildings. The focus of this Section is Class 9a buildings of Type A construction and the required fire-resistance levels are summarised in Table 6.6. Other classes have been include since they apply to other parts of the building, as in the example shown in Figure 6.7 where Class 7a applies to the carpark levels and depending upon the determination of the appropriate authority Class 6 may apply to the retail parts of the ground floor.

Table 6.6: FRLs for external walls of mid-rise Class 5 to 9 buildings of Type A construction.

Distance from fire source feature	Loadbearing (Y/N)	FRL – Structural Adequacy /Integrity/Insulation (minutes)		
		Class 5, 7a or 9a, b and c	Class 6	Class 7b or 8
<1.5 m	Y	120/120/120	180/180/180	240/240/240
	N	–/120/120	–/180/180	–/240/240
≥1.5 and <3 m	Y	120/90/90	180/180/120	240/240/180
	N	–/90/90	–/180/120	–/240/180
≥3 m	Y	120/60/30	180/120/90	240/180/90
	N	–/–/–	–/–/–	–/–/–
External columns	Y	120/–/–	180/–/–	240/–/–

Table 6.6 is a derivative of Specification C1.1 Table 3 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

6.6.3 Structural Fire Resistance of Mid-Rise Class 9a Buildings

The requirements for structural fire resistance for Class 9a buildings are mainly defined in Specification C1.1 and Clause C2.5(a) and are summarised in Table 6.4 and Table 6.5. However, where multiple classifications can apply to a building the requirements in NCC Specification C1.1 Clause 2.2 relating to support for another part can significantly impact on the required FRLs as demonstrated in the example included in sub-section 6.6.1 'Fire and Smoke Compartmentation Mid-Rise Class 9a Buildings'.

6.7 Cavity Insulation

Combustible cavity insulation can facilitate ignition of cavity fires and the rapid spread of fire through cavities. Therefore, if cavity insulation is provided within fire-protected timber elements, it is required to be non-combustible.

Typical solutions include mineral fibre or glass wool insulation with very low organic binder contents. It is important to check that Evidence of Suitability in the form of a current AS 1530.1 report from a NATA Accredited Testing Laboratory is available for the specific products selected.

6.8 Cavity Barriers

The purpose of this section is to describe where cavity barriers are required and what performance is to be achieved.

Although, the provision of cavity barriers may appear onerous and complex, once the principles are understood, a simple systematic approach can be adopted to identify the required locations and the cavity barriers can be readily incorporated within standard construction.

Practical details in relation to each of the locations where cavity barriers are required are illustrated in Step 5

Cavity barriers are barriers placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements.

They are required to be provided by the following NCC clauses as part of a Deemed-to-Satisfy Solution:

- Clause C1.13
- Clause 3.1d(iii) of Specification C1.1
- Clause 4.1e(iii) of Specification C1.1.

Specification C1.13 prescribes the requirements for cavity barriers that are intended to restrict the spread of fire, smoke and hot gases to other parts of the building through cavities in conjunction with other measures such as the use of non-combustible cavity insulation.

Since the use of fire-protected timber requires the presence of a sprinkler system, cavity barriers are provided as a further measure of limiting fire and smoke spread within the building in the unlikely event of sprinkler failure, ignition of timber framing within the wall or fire ignition within the wall cavity.

The risk of fire spread via cavities and voids in designs that use massive timber can be addressed by prohibiting designs that incorporate cavities and voids. Hence the level of protection to the timber element can be reduced under certain circumstances.

6.8.1 Determining the Positions of Cavity Barriers/Compartment Boundaries

Note: Typical cavity barrier details and interface details are shown in Step 5.

Note: All interfaces between fire-resisting elements or smoke-proof elements, and walls/ceilings not providing a fire- or smoke-separating function, must be detailed to ensure the performance of the fire-resisting or smoke-proof element is not compromised.

Cavity barriers are required at the following positions where fire protected timber is used in any of the listed elements of construction unless massive timber construction has been adopted:

- junctions between fire-resisting floor/ceiling assemblies and fire-resisting walls
- junctions between fire-resisting floor/ceiling assemblies and fire-resisting or non-combustible external walls
- junctions between fire-resisting walls and fire-resisting or non-combustible external walls
- around the perimeter of door and window openings in fire-resisting construction.

Smoke-proof walls should also be provided with cavity barriers with the cavity barrier positions determined as they would be for fire-resisting construction.

Additional cavity barriers shall be provided if the following distances between cavity barriers are exceeded:

- Horizontal cavity barriers – 5 m centres.
- Vertical cavity barriers – 10 m centres.

Typical positions of cavity barriers are shown for representative Class 9a and Class 9c buildings or parts of buildings of timber-frame or Hybrid construction in Figure 6.9 to Figure 6.11.

Figure 6.9 shows a typical schematic section through a mid-rise Class 9a building which incorporates fire-protected massive timber columns supporting a fire-protected timber-frame floor assembly with fire-protected timber-framed construction for fire and smoke compartment walls, shaft walls and external walls.

Class 9a construction was selected for Figure 6.9 to demonstrate the broad range of smoke and sub-fire compartments in ward areas, and fire compartments separating hazardous areas and parts of the building having a classification other than Class 9a. The compartmentation of Class 9c tends to be simpler/more consistent.

Cavity barriers are required at the interface of the floor/ceiling assembly (see Figure 6.9):

- external walls
- shaft walls
- fire compartment walls
- FRL 60/60/60 sub-fire compartment walls
- smoke-proof walls.

In addition, cavity barriers are required around the perimeter of external windows (shown in Figure 6.9) and doors fitted into fire-resisting and smoke-proof walls (omitted from Figure 6.9).

Floor to floor heights are less than 5 m and therefore no supplementary cavity barriers are required to provide horizontal subdivision of vertical walls.

The intersection between the massive columns and fire-resisting floor/ceiling is highlighted to indicate that detailing is required to ensure that the fire resistance of the ceiling or column is not compromised where they intersect. Although not shown in Figure 6.9, careful detailing is also required at the junction of non-fire-resisting walls and the ceiling assembly to ensure the fire-resistance performance of the ceiling is not compromised.

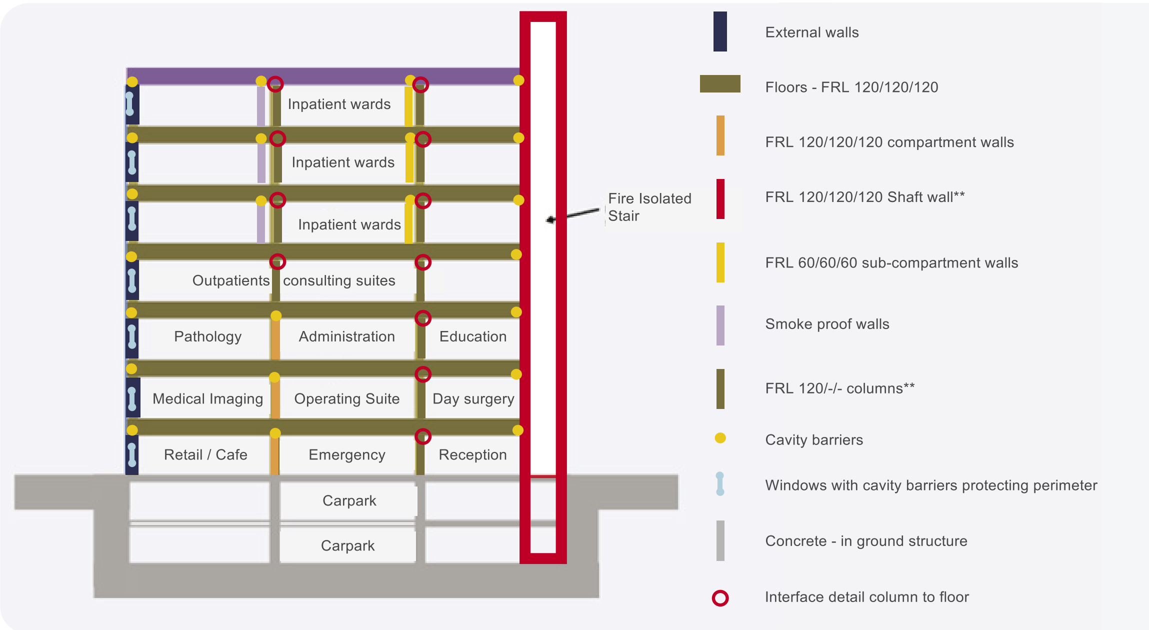


Figure 6.9: Vertical section through a Class 9a hospital building.

Figure 6.10 shows a typical section through a wing of a floor containing ward areas. The ward is divided into smoke compartments less than 500 m² by a smoke-proof wall and separated from the main circulation area by an FRL 60/60/60 wall.

Note: When defining smoke compartmentation, there can be significant advantages if the locations of the smoke-proof walls are consistent with functional areas and evacuation strategies.

A smoke-proof wall separates the two ward areas and a smoke-proof/fire-resisting wall separates the wards from a central circulation area that is large enough to hold the occupants from a whole wing and is therefore consistent with the functional areas and a phased evacuation facilitating an initial horizontal evacuation from the area most at risk.

The stair locations have been selected to avoid dead-end details, maximising the opportunity for occupants to move away from the seat of a fire. The area of the store is small and is not classified as a hazardous area and does not require fire and smoke separation.

Individual room walls are non-loadbearing and are not required to be fire resisting or smoke proof.

It is helpful to adopt the following approach to identify the required positions for cavity barriers.

a) Locate cavity barriers at the following positions:

   - junctions between fire-resisting floor/ceiling assemblies and fire-resisting/smoke-proof walls
   - junctions between fire-resisting floor/ceiling assemblies and fire-resisting or non-combustible external walls
   - junctions between fire-resisting walls and fire-resisting or non-combustible external walls
   - around the perimeter of door and window openings in fire-resisting or smoke-proof construction.

b) Identify junctions between non-fire-resisting and fire-resisting construction that require special treatment.

c) Check horizontal barriers are located within walls at maximum of 5 m centres (typically not required if floor-to-floor height is less than 5 m since cavity barriers are provided at junction between floor and walls).

d) Check vertical cavity barriers are located within walls at maximum of 10 m centres along the walls. If this is not achieved, provide additional cavity barriers. It is often most practical to locate these additional barriers coincident with junctions between fire-resisting and non-fire-resisting construction.

The same principles for determining cavity barrier positions apply to Class 9c residential aged care buildings as shown in Figure 6.11 but with a greater emphasis on smoke-proof construction. Some specific features of the layout include a large store that requires smoke separation from the remainder of the resident area, which conveniently reduces the remainder of the resident area in the wing to less than 500 m^2. Therefore, it is only necessary to smoke separate the resident area from the general staff area and circulation space. The example assumes the structure is predominantly of post and beam construction. If lightweight timber-frame construction is applied, generally the individual room walls will become loadbearing, as shown in the pulled out detail, and additional cavity barriers are required.

For the Class 9c building, the selection of the stair at the end of the wing was not straightforward because the layout dictated that either a dead-end (complying with the DTS provisions of the NCC) was accepted or occupants would have to travel through the lounge room to reach the exit stairs, potentially having to navigate various obstacles. Other details of the compartmentation are similar to the Class 9a example and are consistent with the expected evacuation strategy.

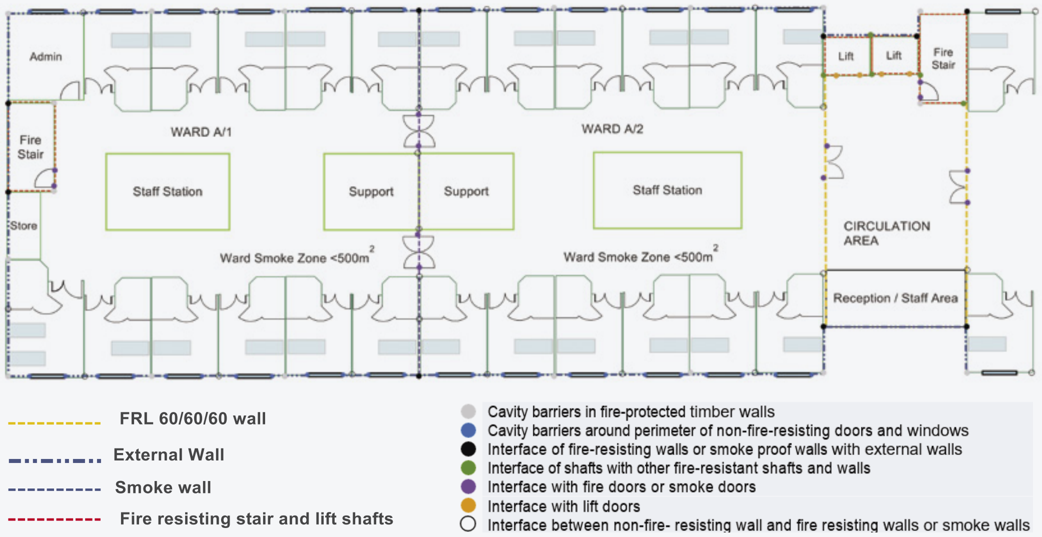


Figure 6.10: Floor plan section of a Ward area of a Class 9a building showing typical cavity barrier locations.

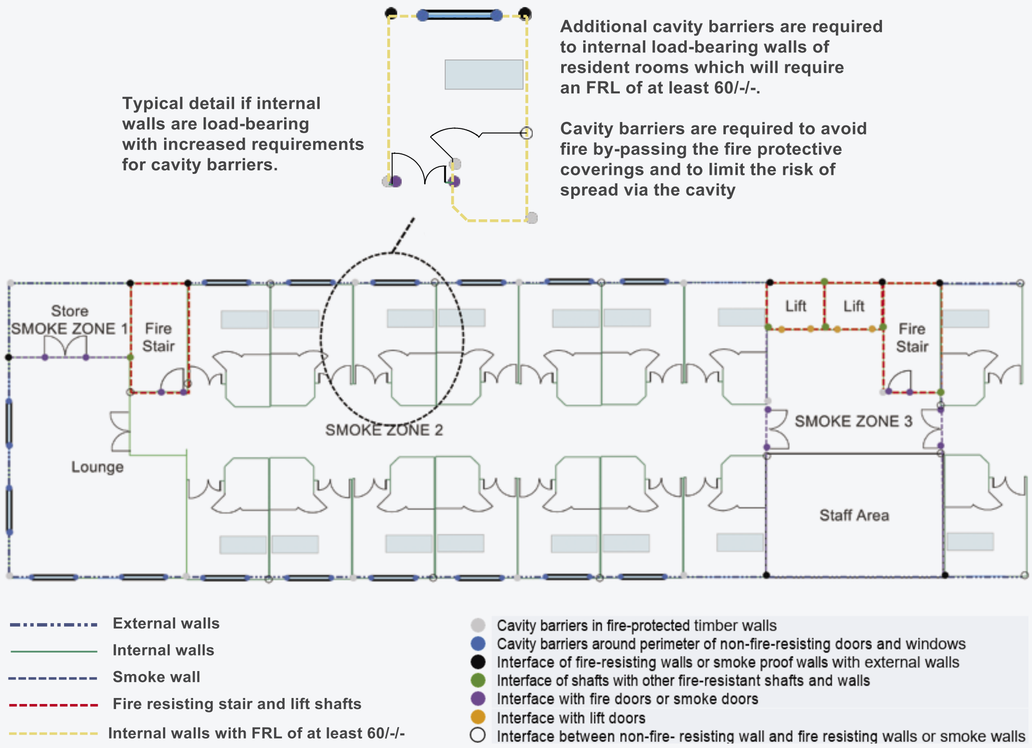


Figure 6.11: Floor plan of residential area of a Class 9c building showing typical cavity barrier locations.

6.8.2 Specifying Cavity Barrier Requirements for Building Elements

Essentially, there are two levels of performance required for cavity barriers prescribed by the NCC.

- Cavity barriers with FRLs of –/45/45 for building elements with FRLs not exceeding 90/90/90
- Cavity barriers with FRLs of –/60/60 for building elements with FRLs greater than 90/90/90.

The FRL –/45/45 criteria should also be applied to smoke-proof walls.

For each case, the NCC prescribes Deemed-to-Satisfy Provisions based on minimum thicknesses of timber or non-combustible mineral fibre in the direction of heat flow as summarised in Table 6.7 as an alternative to evaluating proprietary systems by testing in accordance with AS 1530.4.

Table 6.7: NCC-prescribed Deemed-to-Satisfy Solutions for cavity barriers.

Prescribed solution options	Fire-protected timber FRL	
	–/60/60 or –/90/90	–/120/120, –/180/180, –/240/240
FRL for cavity barrier	–/45/45	–/60/60
Timber required minimum thickness*	45 mm	60 mm
Mineral wool required minimum thickness*	45 mm	60 mm

** Minimum thickness measured in the direction of heat flow – refer Appendix B.*

Table 6.7 is a derivative of Specification C1.13 Table 1 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

6.8.3 Interfacing fire-protected timber with external façade systems

In commercial building applications, it is common to use glazed curtain walling systems secured to floor plates particularly where the building is more than 3 m from the allotment boundary since non-loadbearing external elements do not require an FRL. In these applications, any openings (i.e. horizontal gaps, when viewed in plan, between the edge of the fire-resistant floor and the curtain wall) need to be protected to maintain the same FRL as that required for the floor system.

These perimeter seals are sometimes also referred to as cavity barriers; however, the performance levels and Deemed-to-Satisfy Solutions provided in Specification C1.13 for cavity barriers within timber-frame construction should not be used since, among other things, higher FRLs are required at the perimeter of a floor plate.

If a combustible façade is proposed using verification method CV3 as Evidence of Suitability for a performance solution, the cavity barriers forming part of the external façade system are required to be evaluated by one of the full-scale test methods nominated by AS 5113 and alternative cavity barrier systems cannot be substituted.

6.9 Lift Shafts

Some timber building designs adopt a hybrid approach and incorporate concrete or masonry shafts. Where this approach is adopted, it is important that the potential for differential movement between the timber structure and shaft be considered when detailing connections and interfaces. When designing lift shafts, the lift supplier should be involved at an early stage to ensure the shaft will satisfy their design requirements and applicable regulations.

The remainder of this Section addresses the fire safety performance of lift shafts of fire-protected timber construction.

6.9.1 Timber-framed Lift Shaft Construction

Table 6.8 shows the NCC requirements that are applicable to timber-framed lift shafts in mid-rise timber Class 9a and 9c buildings. Other commercial classes of buildings have been included for applications where a hospital building contains parts having different classifications.

Note: RIR 37401400 is available from the WoodSolutions website

Table 6.8: Requirements for fire-protected timber-framed lift shafts. The wall FRL and RISF requirements are applicable from both inside and outside the shaft.

Criteria	Required Performance		
	Class 5, 7a or 9a, b and c	Class 6	Class 7b or 8
FRL for loadbearing lift shaft walls	120/120/120	180/120/120	240/120/120
FRL for non-loadbearing lift shaft walls	–/120/120	–/120/120	–/120/120
RISF for lift shaft walls	45 mins		
FRL Lift landing doors	–/60/–		

To minimise sound transmission to adjoining areas, double stud construction may be employed and/or an independent support structure provided within the shaft.

The fire-resistance of lift landing door assemblies should be determined by fire tests in a representative wall construction type. At the time this Guide was prepared, Evidence of Suitability for lift landing doors directly fixed to timber-framed wall assemblies could not be obtained.

A practical way to address this is to transition the shaft wall construction around the door opening to a form of non-combustible construction having FRLs with which the performance of the lift door has been verified.

An example of transitioning to a steel shaft wall system from a fire-protected timber wall shaft is shown in Figure 6.12 and Figure 6.13 (refer to Step 5 for further details). These interface details have been assessed by an Accredited Testing Laboratory (Regulatory Information Report (RIR) 37401400, available from the WoodSolutions website), which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the lift landing doors up to the lesser of 120/120/120 or the FRL of the element.

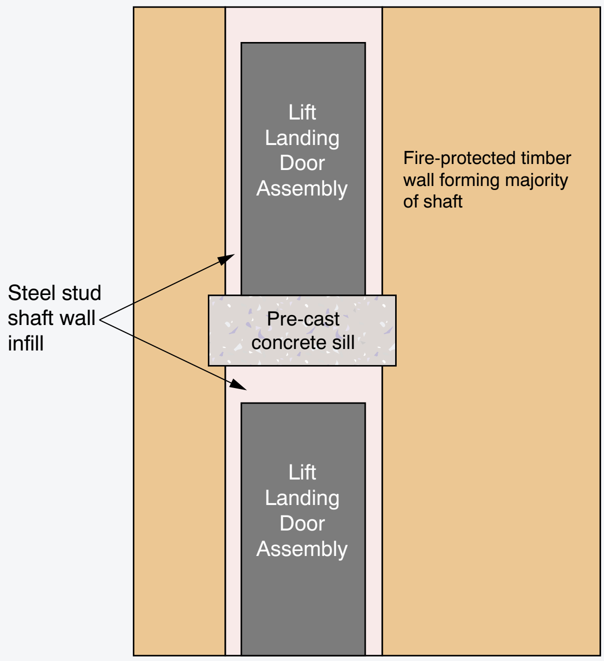


Figure 6.12 (19): Elevation showing wall transition around lift landing doors.

Evidence of Suitability for the specific proprietary lift door, steel stud shaft wall and timber shaft wall, in accordance with Clause A5.2 and A5.4 to A5.6 as appropriate of the NCC, should be submitted to the relevant regulatory authority in addition to RIR 37401400.

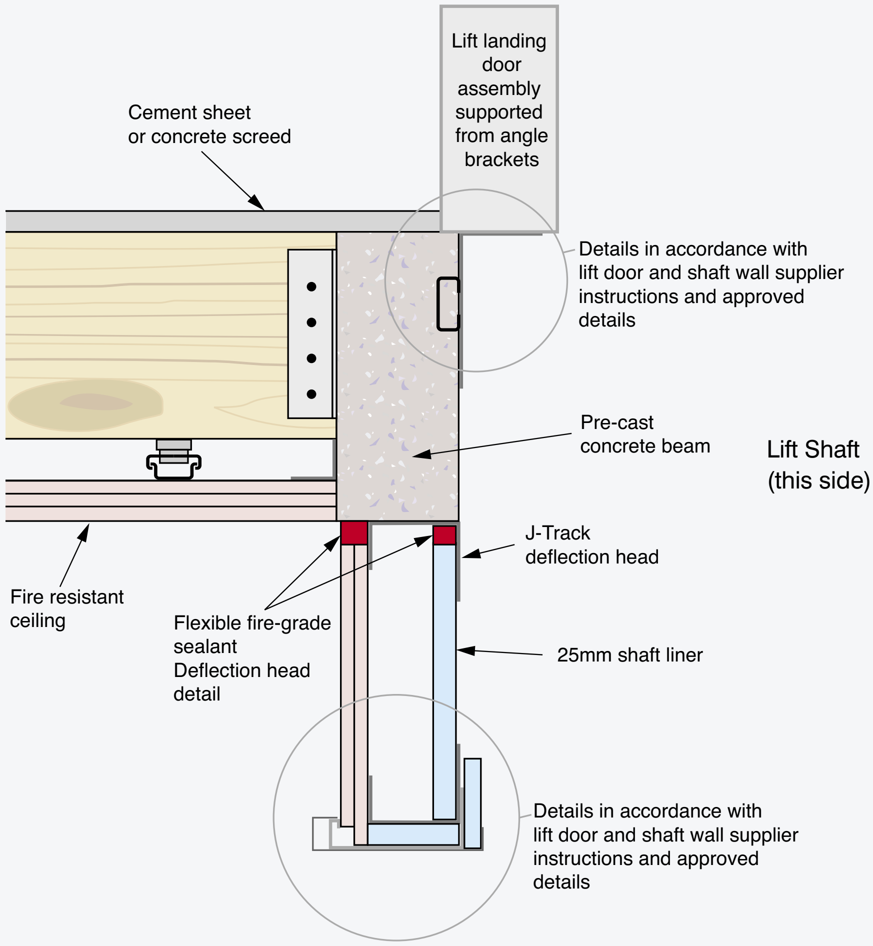


Figure 6.13: Generic detail for sill and head mounting.

6.9.2 Massive Timber Lift Shaft Construction

Table 6.9 shows the NCC requirements that are applicable to massive timber lift shafts in mid-rise Class 9a and 9c buildings. Other commercial classes of buildings have been included for applications where a hospital building contains parts having different classifications.

Table 6.9: Requirements for fire-protected timber lift shafts if using massive timber.

Criteria	Required Performance		
	Class 5, 7a or 9a, b and c	Class 6	Class 7b or 8
FRL for loadbearing lift shaft walls	120/120/120	180/120/120	240/120/120
FRL for non-loadbearing lift shaft walls	–/120/120	–/120/120	–/120/120
RISF for lift shaft walls	30 mins outside face; 20 mins inner face		
FRL Lift landing doors	–/60/–		

If utilising massive timber construction, the MRISFs are reduced from 30 to 20 minutes within the lift shaft. This relaxation reflects the lower probabilities of severe fires occurring within these areas, but a basic level of protection is retained to address the small potential of fires occurring within these areas; where fire may spread to evacuation paths which could be quickly compromised due to rapid fire spread in the early stages of a fire. The outer faces still require an MRISF of 30 minutes. This configuration is shown in Figure 6.14.

To minimise sound transmission to adjoining areas, double skin construction may be employed and/or an independent support structure provided within the shaft for a single skin option. If double skin construction is employed, it should be noted that the NCC does not permit an unfilled cavity between the massive timber skins when using the massive timber provisions. If unfilled double-skin construction is preferred, there is still an option to use the General Requirements (timber-framed construction) rather than the massive timber requirements. The General Requirements require the inner and outer faces to achieve a RISF of 45 minutes. This can be achieved by applying two layers of 13 mm thick fire-protective grade plasterboard to both the inner and outer faces of the shaft.

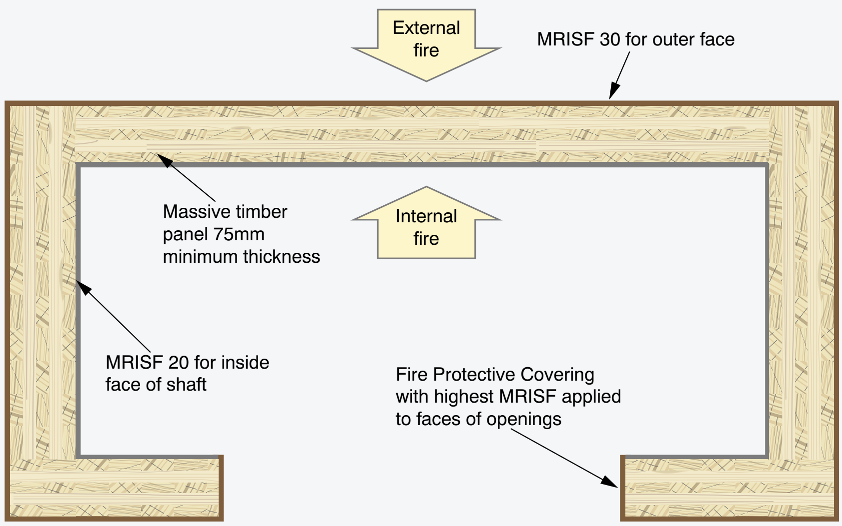


Figure 6.14: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

6.10 Fire Isolated Stairs and Passageways

The FRL, RISF or MRISF required for Fire-Isolated Stairs and Passageways are the same as those required for lift shafts described above, without the complication of lift landing doors.

6.10.1 Fire Doors to Fire-isolated Stairs or Passageways

Fire doors to fire-isolated stairs or passageways are required to achieve an FRL of –/60/30. Several proprietary fire door systems have been tested when mounted in timber construction. Installation details for fire doors capable of achieving the required FRLs should be obtained from the supplier as they may vary. Figure 6.15 shows a typical interface detail with a fire-protected timber wall. These interface details have been assessed by an Accredited Testing Laboratory (RIR 37401400) which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the fire doors up to the lesser of 120/120/120 or the FRL of the element. Evidence of Suitability for the specific proprietary door, and timber shaft wall, in accordance with Specification A5.4 of the NCC, should be submitted to the relevant regulatory authority in addition to RIR 37401400.

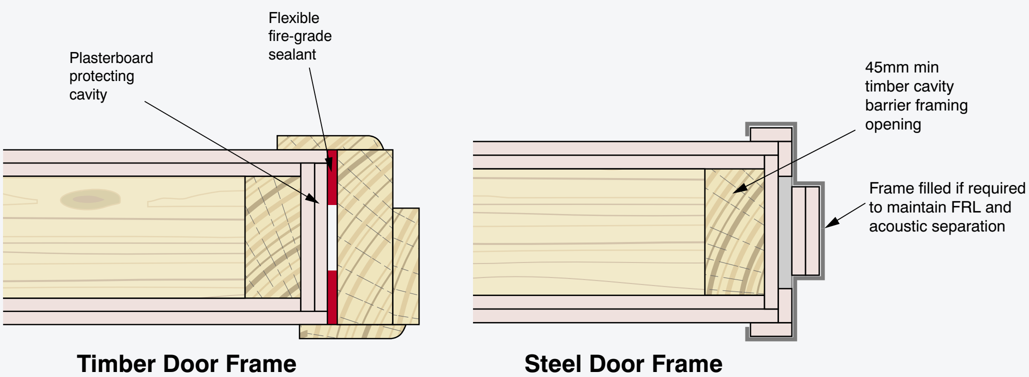


Figure 6.15: Typical fire door installation details.

6.10.2 Timber Stairways Concession

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway constructed from fire-protected timber in accordance with Specification C1.13a subject to:

- timber having a finished thickness of not less than 44 mm
- an average timber density of not less than 800 kg/m^3 at a moisture content of 12%
- the building being protected throughout by a sprinkler system complying with Specification E1.5, other than FPAA101D and FPAA101H systems, that is extended to provide coverage within the fire-isolated enclosure
- the underside of flights of stairs and landings at or near the level of egress or direct access to a car park being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres.

Fires starting in fire-isolated stairs are rare. When they do occur, they generally involve stored or introduced materials and often the cause is malicious. Even though it is not permitted to store goods in fire-isolated stairs and passageways, areas under the lowest flight of stairs form a convenient dry area for temporary storage. These areas may also not be secured, further increasing the risk of malicious fire starts.

While it could be argued that the extension of the sprinkler system to fire-isolated stairs and passageways addresses this issue, as an additional precaution, the underside of the lower stairs where combustibles can be stored are required to be protected by a fire-protective covering of 13 mm fire-protective grade plasterboard.

Step 5 provides further details of the requirements for timber stairways.

Careful planning and design of building services and distribution paths, at all stages of the design process, can greatly simplify construction and subsequent maintenance through the life of the building.

6.11 Building Services

6.11.1 Selection of Building Services and Distribution Paths

Building services and associated distribution paths need to be selected and refined during the design process to ensure that installation and maintenance of the services, including reinstatement of the fire and smoke compartmentation and structural fire resistance of elements if affected by the services, can be undertaken safely, causing minimal disruption to the operation of the building and its occupants. Maintaining the availability and efficacy of fire safety systems through the life of the building and managing the risk of fire ignitions and smoke and fire spread associated with services are also critical considerations.

Key principles for consideration with respect to fire safety are:

- Minimise service penetrations through fire-protected timber construction and fire-resisting construction. This can be achieved by measures such as self-contained air-conditioning systems serving each fire compartment (where practicable), and false ceilings and wall facings allowing services to run behind the non-fire-rated facing without penetrating fire-resisting elements.
- If service penetrations through fire-resisting construction cannot be avoided, the services should penetrate shaft or service duct walls rather than fire-resisting walls or floors separating occupied areas as far as practicable. This approach limits the consequences if a penetration protection system fails, as smoke and fire spread will initially be limited to the service ducts.
- Where practical, shafts, service risers and service ducts should be readily accessible to facilitate maintenance and inspection. It may be appropriate in some applications for access hatches/ panels or doors to be secured to prevent unauthorised access and tampering with fire protection systems and the services.
- If service penetrations through fire-protected timber construction cannot be avoided they should be grouped together, where practicable, and pass through a framed-out opening which is then fire-stopped using a proprietary system such as non-combustible batts, board, pillow systems or proprietary multiple transit systems. This approach substantially reduces the risk of fire spread to cavities at a point of weakness and ignition if hot works are being undertaken on the services.
- If service penetrations through fire-protected timber elements cannot be avoided, the fire resistance of the element including the service penetration system should be reinstated such that the resistance to the incipient spread of fire (RISF) or modified resistance to the incipient spread of fire (MRISF), prescribed in Specification C1.13a of the National Construction Code, is maintained.

- Services and connection details should be designed to avoid or minimise the need for hot works.
- For fire services, such as sprinkler systems, the design, installation, and distribution layout should be selected to minimise the time the system will be unavailable during maintenance and/or renovations.
- Precautions should be taken to address the fire hazards associated with the storage and distribution of medical gases. This includes consideration of the increased flammability of combustible materials in oxygen-rich environments in addition to the flammability of the gases.
- Where practical the risk of fire spread across the surfaces of services should be minimised. For example, non-combustible pipe insulation materials should be considered in lieu of more combustible options and limiting the flammability of cable insulation and sheathing can reduce the risk of fire spread.

In some applications these principles may conflict with other project drivers and constraints, but with careful design, suitable solutions can be developed in most cases. For example, the use of CPVC piping for sprinkler systems can reduce hot works but if the pipework needs to be adjusted the system will be unavailable while the glue sets. A better option to avoid hot works and minimise down time may be to use mechanical joiners for metal pipes.

Further details on the application of some of the above principles are provided in the following sections.

6.11.2 Evidence of Suitability for Systems Protecting Service Penetrations

Clause C3.15 of the NCC generally requires that openings for service installations penetrating a building element required to have an FRL, with respect to integrity or insulation or a Resistance to the Incipient Spread of Fire (RISF), be protected such that installation must achieve the required FRL or RISF.

For most applications, Evidence of Suitability with this requirement can be demonstrated by a report from an Accredited Test Laboratory complying with the requirements of AS 4072.1 and AS 1530.4, as appropriate, that indicates the installation has achieved the required FRL or resistance to the incipient spread of fire (RISF/MRISF) in a fire-resistance test; or if it differs from a prototype, the modified installation will achieve the nominated FRL, RISF or MRISF with the nominated variations.

Further explanations of the test procedures are provided in Appendix A.

Typical solutions to address RISF/MRISF criteria include:

- boxing out openings with plasterboard or other non-combustible board achieving the required RISF, MRISF and FRL performance
- filling the area around the service penetration with non-combustible mineral fibre insulation
- transitioning to a different wall type where service penetrations are required.

Some typical examples are provided in Step 5.

6.11.3 Minimising Service Penetrations and Grouping Service Penetrations

Large volumes of building services run throughout Class 9a and 9c buildings for medical purposes and to provide facilities for patients and residents. Class 9a and 9c buildings are generally broken down into fire and smoke compartments in ward areas of Class 9a buildings and resident areas in class 9c buildings. It is prudent to group services together, as far as practical, and use multi-penetration sealing systems to protect locations where they penetrate fire and smoke compartment boundaries. These locations should be located in readily accessible positions with access provided by access panels to facilitate safe and easy access for reinstatement and inspection.

It can be relatively straightforward to apply these principles to electrical and communication systems in Class 9a and 9c buildings where one or more structural cores are used to achieve both lateral load resistance and stability. These cores are commonly used to house lift and stair shafts, service risers and kitchens and toilets which are included in the core or located next to the core. It is then relatively easy to distribute power, lighting communications and install the sprinkler system piping for a whole floor of the building above a false ceiling so that it is not necessary to penetrate the fire protective coverings or fire-resisting elements except for floor to floor penetrations at the building core positions and at fire and smoke compartment boundaries. A schematic of this arrangement is shown in Figure 6.16.

Often patients or residents are provided with their own room with a dedicated ensuite. It can be impractical to fully consolidate services such as drain, waste and vent (DWV) pipes around the central core due to the need for minimal falls on horizontal runs and therefore service shafts may also need to be distributed around the floor within a compartment for these applications. Individual arrangements may also be required for penetrations of the HVAC systems, active smoke control systems and the distribution of medical gases.

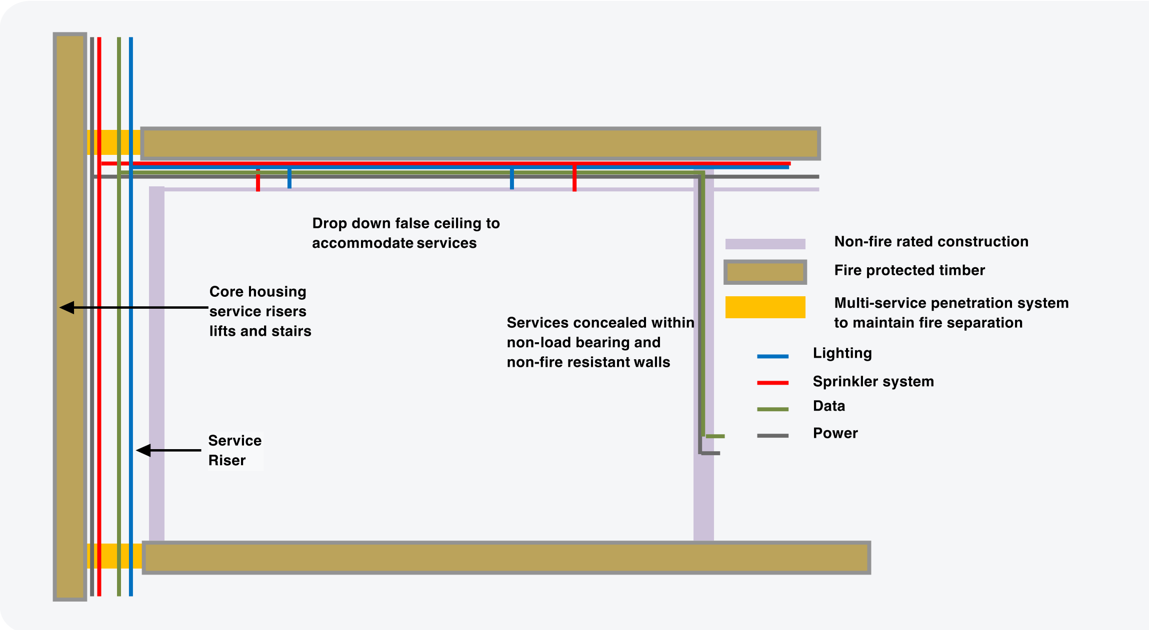


Figure 6.16: Schematic showing distribution of services from a central core.

6.11.4 Service Shaft Construction

The requirements for fire-protected timber service shafts used for ventilation, pipes, garbage or similar purposes are summarised in Table 6.10 for buildings of Type A construction.
Shafts must also be enclosed at the top and the bottom with a floor/ceiling system of the same FRL and RISF ratings as the walls; except where the top of the shaft is extended beyond the roof or the bottom of the shaft is of non-combustible construction laid directly on the ground.

Table 6.10 Requirements for fire-protected service shafts in mid-rise timber buildings

Criteria	Required Performance				
	Class 9a	Class 9c	Class 5, 7a or 9b	Class 6	Class 7b or 8
FRL for loadbearing service shaft walls	120/90/90	60/–/–[1]	120/90/90	180/120/120	240/120/120
FRL for non-loadbearing service shaft walls	–/90/90	–/60/–[1]	–/90/90	–/120/120	–/120/120
RISF for service shaft walls	45 minutes				
MRISF for service shaft walls (Massive Timber)	30 minutes				

Note 1 The NCC is open to interpretation in relation to treatment of service shafts. Check interpretation with the appropriate authority before implementing.

In many instances, it is more practical to construct non-loadbearing shafts from laminated board systems (i.e. unframed non-combustible board systems achieving the required FRL) or steel frame shaft wall construction in lieu of fire-protected timber construction. For non-loadbearing shaft walls in Class 9a and 9c buildings, the required FRL is reduced to –/90/90 or –/60/–, which can make this form of construction more attractive

Details on how to construct shafts in timber-framed construction and how to interface fire-protected timber walls with laminated board shafts or steel shaft wall construction are given in Step 5 and below.

6.12 Interfacing with Other Forms of Construction

There can be advantages in adopting hybrid forms of construction in buildings. For example, ground floor and basement areas may be constructed from concrete to minimise the risk of water penetration, minimise potential damage in flood-prone areas or address the risk of termites.

The relatively lighter weight of timber structures also makes timber construction ideally suited to the upward extension of existing buildings facilitating infill developments and recycling existing buildings. For example, it may be possible to add wards above existing Class 9a or 9c buildings without having to undertake extensive foundation works.

There can also be advantages in using forms of construction other than timber for elements such as shafts under certain circumstances.

6.12.1 Separation of Different Classes of Buildings

The NCC addresses the separation of classifications within a building in Clauses C2.8 and C2.9.

For different classifications on the same storey, parts having different classifications should be separated by a fire wall having the higher FRL of the two, in accordance with Specification C1.1.

For different classifications in different storeys in a Type A building (most mid-rise buildings), the floor between the adjoining parts must have an FRL not less than that prescribed by Specification C1.1 for the lower storey.

A typical building layout is shown in Figure 6.17 with concrete-framed construction for the carpark below timber healthcare levels. For the fire-protected timber concession to apply, the whole building must be sprinkler protected in accordance with NCC Specification E1.5; excluding FPAA101D and FPAA101H systems.

Retail use is assigned to Class 6 buildings unless considered a minor ancillary area within a Class9a part for example. From Table 3 of Specification C1.1, the floor separating the retail and office levels would require an FRL of 180/180/180 therefore it is important to clarify the classifications throughout the design process.

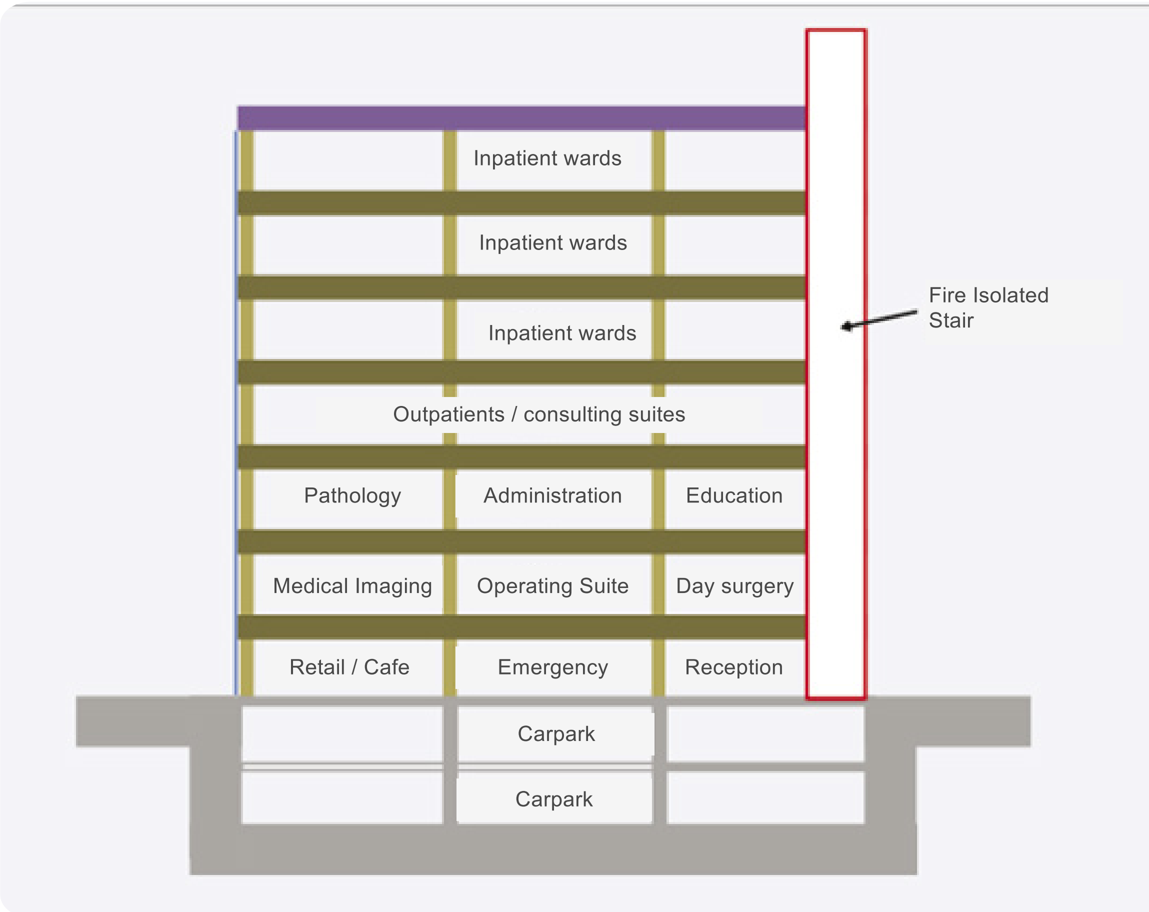


Figure 6.17: Example of multi-class building.

6.12.2 Use of Alternate Forms of Construction for Shafts

In addition to the provision of fire protected timber loadbearing shafts, there are other design options that can be adopted.

For example, concrete or masonry shafts may be adopted for the structural core of a building; particularly for taller mid-rise buildings to provide resistance to lateral loads in which case the core may be used to house lift and stair shafts. There may be the potential for differential settlement to occur between concrete and masonry shafts and the remainder of a timber building and this will need to be addressed in the detailing of the interfaces.

Other options include:

- The use of massive timber framing for the shafts for structural purposes in conjunction with non-loadbearing infill sections (this may be compatible with a post and beam design for the structural frame). The infill sections may be timber-framed or of non-combustible construction.
- The use of a self-supporting lift assembly or internal framing to support the lift and enclosure of the shaft in non-loadbearing timber-frame construction or other forms of non-combustible non-loadbearing shaft construction

In all the above cases the structural design of the building may influence the selection of shaft construction.

6.13 Special Fire Issues

When designing, constructing, commissioning, and using mid-rise timber healthcare buildings, special issues arise, particularly as the buildings become larger and more complicated. Some issues are addressed through technical provisions and regulations other than the technical provisions of the NCC and some matters may only be partially addressed through the NCC (e.g. fire precautions during construction).

Additional fire hazards may be introduced associated with procedures being undertaken and the healthcare services provided, such as the storage and distribution of medical gases.

Although this Guide does not attempt to provide information to suit all circumstances, general information is provided relating to some typical examples that are highlighted in the following sections and are relevant to timber construction. This list is not comprehensive and it is strongly recommended to involve key stakeholders and the design team in a Design Brief process at an early stage of the project to ensure all project drivers and constraints have been identified; to identify essential design criteria before the proposed design is prepared. Refer Figure 1.2 for a flow chart showing a typical process.

6.13.1 Fire Precautions During Construction

Fires may occur on building construction sites due to the nature of the works. Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Timber construction covered with fire-protective linings is a safe and economical building system. The fire-protective coverings play an important role in providing this fire safety but, due to the construction sequencing, there may be a period where the timber is not protected. This is when timber buildings are at their highest risk from construction fires.

The NCC requires a suitable means of fire-fighting to be installed in a building under construction to allow initial fire attack by construction workers and for the fire brigade as detailed below.

A building under construction that is less than 12 metres in effective height must have one fire extinguisher to suit Class A, B and C fires, as defined in AS 2444, and electrical fires provided at all times on each storey adjacent to each exit, or temporary stairway or exit.

Note: The regulatory requirements for health-care buildings to address the bushfire risk vary between the States and Territories and currently the NCC does not provide requirements for Class 9a and Class 9c buildings. It is likely that provisions for Class 9a and 9c buildings may be introduced in future editions and users should check the current status. Notwithstanding the above, if the building is located in a high bushfire risk area, additional mitigation measures should be considered to provide adequate protection of vulnerable occupants.

After the building has reached an effective height of 12 metres, the following additional measures are required:

- the required fire hydrants and fire hose reels must be operational in at least every storey that is covered by the roof or the floor structure above, except the two uppermost storeys
- any required booster connections must be installed.

In this instance, 'required' means satisfying the NCC performance requirements as if the building had been completed using either the performance or Deemed-to-Satisfy pathways.

As the scope of the NCC does not fully address Work Health and Safety (WHS) issues, and the NCC prescribes minimum levels of compliance, builders and building owners need to consider what is actually required for the building site. Typical matters that should be considered include:

- progressive installation of fire-fighting services
- progressive installation of fire-protective grade covering of timber members (i.e. installation of fire-protective coverings) and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

6.13.2 Bushfire-prone areas

The requirements for healthcare buildings to address the risk of bushfires vary between States and Territories and may fall under different jurisdictions to standard building works. The need to consider bushfire exposures should be determined early in the design processes and addressed accordingly.

The NCC requires external walls to be of non-combustible or fire-protected timber construction in mid-rise buildings. The fire-protected timber provisions require timber elements to be protected by non-combustible fire- protective coverings providing a good basis for the building to resist bushfire attack.

6.13.3 Lightweight Construction Structural requirements – Specific Applications

The NCC requires elements that have Fire Resistance Levels (FRLs), or that form a lift, stair shaft, an external wall bounding a public corridor, non-fire-isolated stairway or ramp, comply with Specification C1.8, if they are made out of lightweight materials such as timber-framing faced with plasterboard.

Specification C1.8 defines a structural test for lightweight construction and, in most parts, is directly related to the performance of the linings used. Appropriate Evidence of Suitability should be obtained from suppliers of lining materials used to verify compliance during the design phase.

6.13.4 Medical Gases

Medical gas storage, distribution and use are required for the day-to-day operation and provision of healthcare services in hospitals and may also be provided within aged care facilities.

The storage, distribution and use presents a significant fire hazard. For example, the use of oxygen can significantly increase the flammability of materials, increasing the risk of ignition and accelerating fire spread. If compressed gas cylinders are exposed to heat, an over pressure condition may occur potentially further increasing the consequences from a fire.

Although specific mitigation measures relating to medical gases are not included in the NCC, work health and safety legislation, among other things, requires that appropriate mitigation measures be taken to address the hazard.

These mitigation measures can include operational controls such as:

- removal/reduction as far as practicable of potential fuel and ignition sources from enclosures where an oxygen-rich environment may form
- avoidance of any form of lubricant/oil coming in to contact with oxygen fittings and hoses
- ensuring gas cylinders are not placed near heat sources

AS 2896-2011 provides guidance on the installation and testing of medical gas pipeline systems.

This includes requirements for the storage of medical gases summarised below:

- A recommendation that the quantity of gas in use and storage should be as low as possible.
- Where oxygen is stored in liquid form, the area around the depot (where liquid oxygen is likely to be spilled) shall be paved with smooth concrete.
- Controlled access should be provided with measures to limit the risk of vehicular impact and the storage areas must be ventilated or in the open air.
- Where storage and manifolds are located outdoors, vertical and lateral separation from other buildings and other fire hazards should be according to the requirements of the local fire authority.
- Where cylinders are stored inside a building and/or manifolds provided within a building, the following measures are prescribed:
  - cylinders and manifolds in a separate area constructed from impervious non-combustible materials with an FRL of 120/120/120.
  - ventilation provided at high or low levels directly to the outside of the building
  - fire doors provided that comply with AS 1905.1 having an FRL of −/60/30
- the stored gases and manifolds must not be located:
  - close to exposed electrical conductors, transformers, or other source of electrical arcing, or
  - close to oil storage tanks
  - in an area where temperatures may exceed 55°C
- the floor should be of concrete construction with a trowelled finish
- appropriate signage should be provided
- appropriate access to facilitate safe delivery and removal of cylinders should be provided.

For mid-rise timber buildings, practical implementation of the above requirements can be achieved by:

- locating the storage and manifolds in a separate location outside the hospital building or
- extending the concrete sub-ground structure to the ground floor level such that the storage/ manifold enclosure can be constructed from masonry and/or concrete.

Pipelines for distribution of medical gases should as far as practicable be separated from ignition sources and combustible materials. To achieve this in a fire-protected timber building, medical gas distribution pipes should:

- Not be run through cavities within fire-protected timber construction. (Note: in addition to direct fire safety issues, some timber fire-retardant treatments may accelerate pipe corrosion if the pipework is in contact with the fire-retardant treated timber).
- As far as practical, be run within dedicated shafts or ducts constructed from impervious non-combustible materials with adequate ventilation but pipe entries and other penetrations should be sealed to prevent gas escape by routes other than the vents or openings into the user space. The FRLs for the shafts should as a minimum be in accordance with the NCC requirements for service shafts.
- Have an allowance for free expansion of the pipes.

Where pipes pass through walls, partitions or floors, they should be provided with sleeves of copper pipe that will be required to be protected where the pipes penetrate fire-resisting or smoke-proof elements. The fire protection system of the penetration should be non-combustible and any openings through fire protected elements must be framed out to further reduce the risk of fire spread or leakage of gases into the fire-protected timber element.

Pipe jointing, when undertaken in an occupied facility, can represent a significant fire hazard because of the risk of increased flammability due to excess oxygen in the area where hot works may be undertaken, the risk of contamination with oils and similar materials that may readily ignite within an oxygen rich atmosphere and high temperatures required for brazing. A risk assessment should be undertaken, and appropriate measures implemented to address these hazards which may include purging pipelines with inert gases.

The framing out of openings in fire-protected timber and use of non-combustible fire protection systems for medical gas service penetrations will reduce the risk of ignition and spread through cavities in fire protected timber construction.

6.13.5 Robust Structural Design

The NCC, under Part B1 Structural Provisions (BV2), provides a verification method for structural robustness as a means of verifying compliance with performance requirement BP1.1(a)(iii). The Verification Method states:

Compliance with BP1.1(a)(iii) is verified for structural robustness by:

(a) assessment of the structure such that upon the notional removal in isolation of -

- (i) any supporting column; or
- (ii) any beam supporting one or more columns; or
- (iii) any segment of a loadbearing wall of length equal to the height of the wall, the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high-risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The structural design of mid-rise timber buildings must comply with these requirements and the design guidance provided in *WoodSolutions Design Guide #39 Robustness in Structures* to ensure the building is adequately robust in the event of localised failure of elements during a fire.

7

Step 5 Integrate Architectural, Structural and Building Service Designs (Detailed Design)

This step brings together the previous Steps 1 to 4 to develop an integrated design. A hospital with basement carparking and retail at ground level is used to demonstrate the process, including interfacing a fire-protected timber building with other forms of construction and parts of a building with a different Class.

A key focus of this Step is coordinating the various design disciplines so that:

- Timber elements and protection systems are optimised to satisfy the NCC requirements in a practical and cost-effective manner by focusing on the synergies between elements designed to satisfy the following criteria:
  - fire-protected timber
  - sound transmission and insulation
  - thermal resistance
  - weatherproofing
  - structural tests for lightweight construction.
- Interfaces between building services and the structure, fire-protected timber elements and acoustic barriers are designed:
  - to minimise building service penetrations through fire-protected timber elements and acoustic barriers as far as practical
  - such that where services have to penetrate fire-protected timber elements the fire safety performance of the element is not compromised, and fire separation is maintained
  - so that if services have to penetrate acoustic barriers the positions are selected to minimise negative impacts on amenity
  - so that service penetration systems can accommodate any differential movement between elements
  - to allow for maintenance and additions/modifications to the building services.
- Structural design is efficient and robust.
- Other fire safety principles for mid-rise buildings are satisfactorily implemented including:
  - cavity barriers
  - automatic fire sprinkler systems.
- Other design requirements are addressed such as termite management and resistance to ground water/moisture penetration.

7.1 Optimising the Performance of Elements of Construction

Elements of construction in a modern building may have to serve a number of functions including:

- restricting fire spread
- limiting sound transmission from adjacent enclosures (and in some instances external noise)
- limiting heat loss and/or heat gain through external elements
- weather resistance of external facades and roofs
- impact resistance to reduce the risk of damage to lightweight construction.

The elements also need to achieve levels of durability appropriate for the application. Further advice on durability is provided in: *WoodSolutions Timber Design Guide #5 Timber service life design – Design guide for durability.*

Efficient designs can be achieved by selecting combinations of materials and configurations that work together to satisfy the design objectives summarised in the following Sections.

Typical examples include:

- Cavity barriers required by the NCC Deemed-to-Satisfy for mid-rise timber buildings to reduce the risk of fire spread through concealed spaces can also be used to minimise flanking noise transmission around the perimeters of elements of construction and reduce heat loss via leakage through the structure.
- Non-combustible cavity insulation will:
  - reduce the risk of fire spread through cavities
  - reduce sound transmission through elements of construction
  - reduce heat loss and/or gain through external walls.

7.1.1 Fire-protected Timber

Fire-protected timber has timber structural members with non-combustible fire-protective coverings. The fire-protective coverings:

- prevent or delay the ignition of the timber members so that the response to an enclosure fire will be similar to non-combustible elements such as masonry or concrete during the growth period and prior to fire brigade intervention
- ensure the fire-protected timber element achieves the Fire Resistance Level (FRL) prescribed for the particular element.

Any insulating materials provided within cavities must be non-combustible to reduce the risk of fire spread through cavities and voids.

The NCC contains some Deemed-to-Satisfy (DTS) Solutions for fire-protective grade plasterboard coverings but there are many opportunities for the use of optimised proprietary systems. For example, combinations of high-performance non-combustible fire-resisting claddings and mineral fibre insulation could provide lighter-weight, more cost-effective options.

The NCC DTS Solutions recognise that massive timber panels have a relatively high inherent fire resistance and, if there are no concealed cavities or voids, the risk of fire spread through concealed spaces will be substantially reduced or removed. Therefore, provided the minimum dimensions prescribed for massive timber panels are satisfied and there are no internal cavities and voids, the NCC allows some relaxations to the requirements for fire-protective coverings (see Section 6.3.2).

Note: The use of timber cladding and other combustible fire protection systems such as intumescent paints in lieu of non-combustible fire-protective coverings is not permitted under the NCC DTS Solutions for mid-rise timber buildings due to the potential increase in risk of fire spread to the structural element as the combustible materials are consumed.

7.1.2 Cavity Barriers

The primary objective of cavity barriers is to prevent uncontrolled spread of fire through cavities in the low probability the protective covering fails, or fire starts within the cavity.

The NCC provides DTS Solutions using solid timber or mineral fibre but also specifies FRLs for cavity barriers encouraging the development of proprietary systems optimised for specific applications.

Careful detailing can provide opportunities for efficient design. Typical examples include:

- in a single leaf, timber-framed stud wall, the top and bottom plates can be dimensioned such that they can act as cavity barriers
- if a cavity is filled with non-combustible mineral fibre insulation to achieve a nominated R-value or enhanced acoustic separation, the mineral fibre may also satisfy the requirements for a cavity barrier.

7.1.3 Sound Transmission and Insulation

In timber construction, airborne and impact sound requirements are primarily achieved using one or more of the following principles:

- ***Increasing mass (e.g. increasing the thickness of wall linings).*** This can be particularly useful in reducing airborne sound transmission. For instance, like fire-grade linings, the greater the number of layers, the greater the increase in R_w (Note: extra factors are involved in increasing R_w+C_{tr}).
- ***Isolating one side of a wall from the other*** (e.g. using double stud cavity wall construction). This is also known as decoupling (discontinuous construction) and can reduce both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.
- ***Avoiding rigid connections between the opposing sides of isolated (decoupled) elements.*** This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at changes in floor level (Figure 26).
- ***Using absorptive materials to fill wall and floor cavities*** (glass fibre or mineral wall) can reduce airborne sound transmission. The NCC requires absorptive material to be non- combustible.
- ***Sealing sound leaks*** at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

There are also simple techniques that can be incorporated into the building design that can dramatically improve the sound performance of timber wall and floor/ceiling systems. The following systems provide examples that can be used to enhance sound performance of walls and floors.

Note: Timber wall and floor/ceiling systems shown in this Guide are typically of generic systems. Actual systems must comply with the required fire and sound provisions in the NCC for the class of building being considered.

Wall Systems

***Battened-out walls in wet area*.** In wet area construction, fire/sound rated walls can be compromised where bath and shower base units need to be recessed into the wall. A simple means of achieving this is to batten out the wall (after fire/sound resisting linings have been applied) and then provide an additional lining over the top (Figure 7.1). The bath can then be installed into the batten space without affecting the fire- and sound-rated wall. In such instances, it is best to have at least 35 mm batten space and to place insulation into the wall cavity. This arrangement also substantially reduces the risk of compartmentation being compromised during refurbishment activities. For example, if the additional lining boards are removed or replaced, the fire-protective covering can be left in place, maintaining the required fire separation.

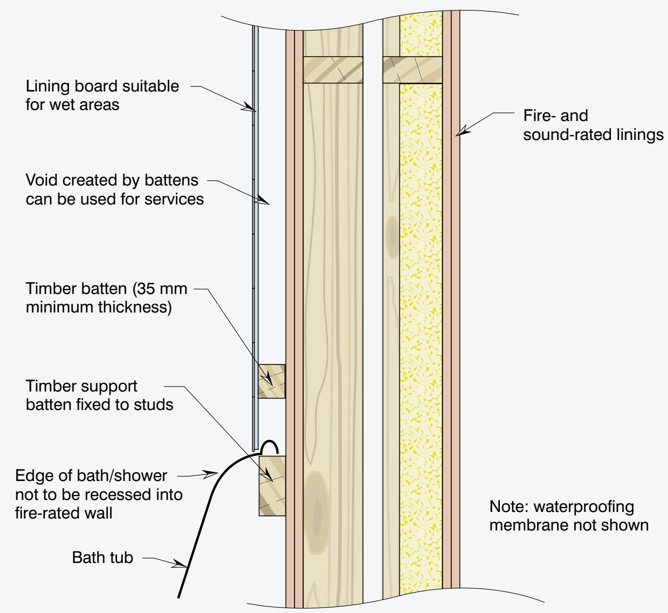


Figure 7.1: Batten detail of wet area walls – elevation view.

Floor Systems

Floor joists parallel to sound rated wall. By running floor joists parallel rather than perpendicular to the sound rated wall, the ability of impact sound from the floor being transferred across the wall to the adjoining SOU is lessened (Figure 7.2).

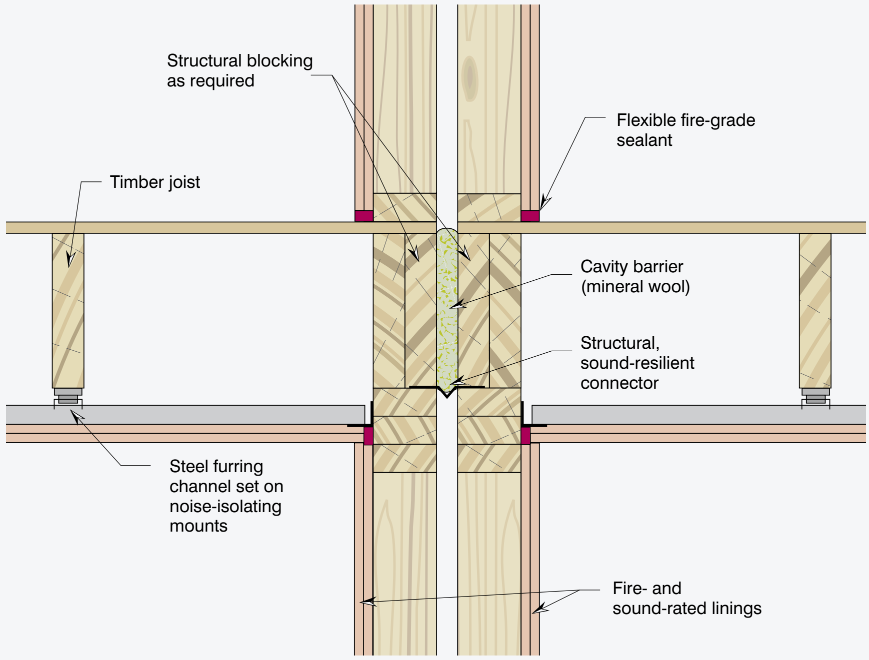


Figure 7.2: Joists running parallel to bounding wall – elevation view.

Upgrade sound-resilient ceiling mounts.

Ceiling mounts are commonly used to reduce the transmission of noise from the floor to the ceiling below. They help reduce sound transfer between the bottom of the floor joist and the ceiling lining. To improve performance, some ceiling mounts now provide an isolating and damping effect (Figure 7.3). They typically force the sound energy through a rubber component that deforms slightly under load as the sound passes from the joist to ceiling sheet. Sound-resilient mounts are not all the same and different systems have different performance.

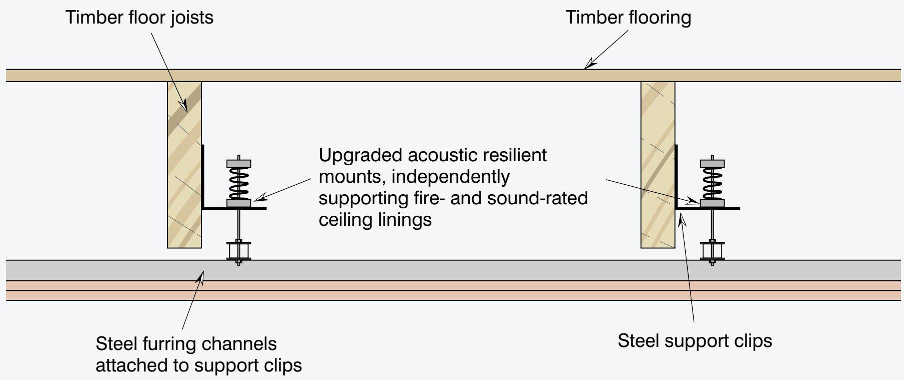


Figure 7.3: Upgraded sound-resilient ceiling mounts – vertical section.

Increase mass of the top layer of floor systems.

Increasing the mass of the top surface of the acoustic floor system is one of the best ways to improve acoustic performance. There are three common ways – sand, concrete topping or additional floor sheets.

Quantifying the improvement is difficult as the acoustic performance aims to improve the low frequency performance of the floor, a phenomenon not measured by tested systems. Unless evidence of suitability is available it is suggested that the base floor system be designed to comply with the NCC's sound requirements, and the additional floor mass provides enhanced performance.

Note: When height is added to a floor, consideration of the effect this has in other areas (such as wet areas, corridors, stairs, doors and windows) is needed at the planning stage.

Sand: This increases the mass of the upper layer of the floor element. The air spaces between the sand particles help reduce the vibration and energy created by impact sound from footfalls. Typically, this is achieved by placing 45 mm thick (high) timber battens directly over a normal acoustic floor system at typical 450 or 600 mm centres (dependent on floor sheet spanning capacity). A dry sand layer is placed between the battens and levelled just below the surface of the final floor sheet. The final floor sheet is fixed in the normal manner, and desired floor covering placed on this (Figure 7.4).

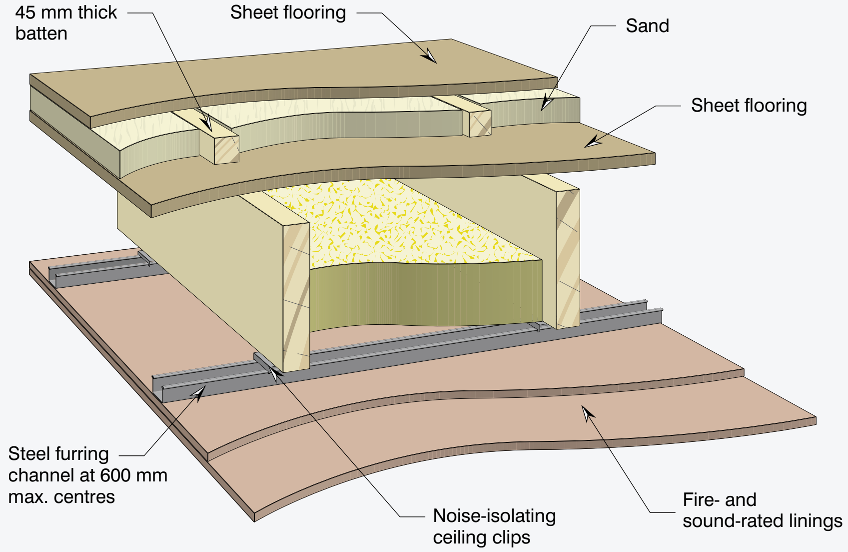


Figure 7.4: Adding mass to floor system – sand.

Concrete topping: This increases the sound performance of the floor system, and typically can be achieved with a 35 to 45 mm thick layer of concrete placed over an isolating acoustic mat. Care is required to turn the isolating acoustic mat up at the perimeter of the topping adjacent to the wall, otherwise the effect of the topping is negated (Figure 7.5).

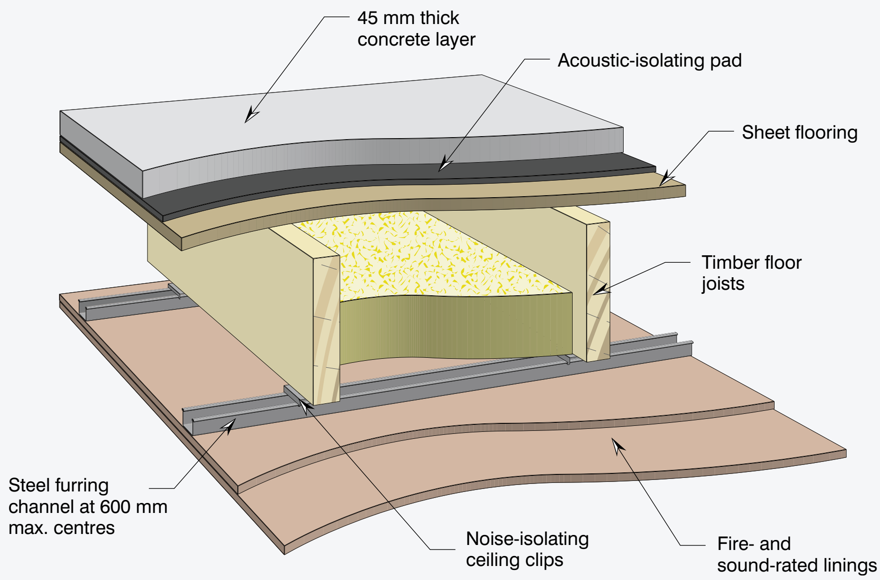


Figure 7.5: Adding mass to floor system – concrete topping.

Extra sheet flooring: This method utilises standard sheet flooring on an isolating mat. This system does not perform as well as the higher mass products, sand or concrete (Figure 7.6).

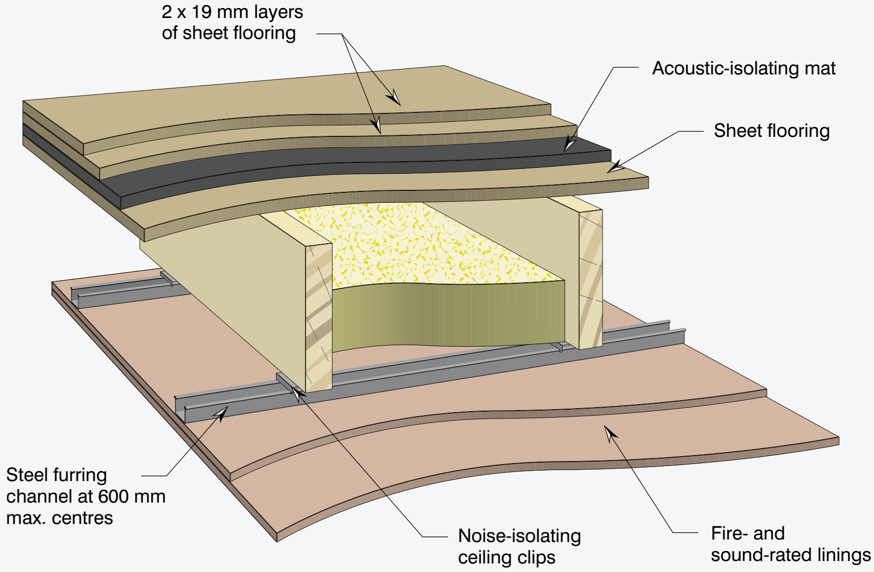


Figure 7.6: Adding mass to floor system – additional floor sheets.

Separate floor and ceiling frame.

By having two sets of joists (separate floor and ceiling joists) that are nested between but not touching each other, it is possible to isolate the two structures, thereby minimising the transference of impact sound through the structure. Care is needed with this approach to prevent flanking noise running along the floor joists and into the walls below. This can be improved by sitting the ceiling joists onto strips of acoustic isolating mat (Figure 7.7).

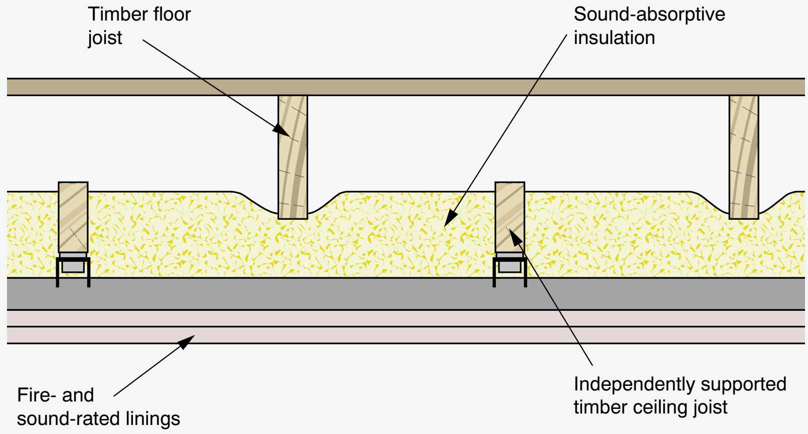


Figure 7.7: Separate ceiling and joist construction.

7.2 Establish Architectural Layout

The basic architectural layout of a building is determined by considering many drivers and constraints, the relative importance of which will vary from project to project. Typically, these include:

- the project brief
- site conditions
- sustainable construction
- aesthetics
- economics
- planning, building and other regulations.

The design should then be refined with input from the various disciplines involved in the design team in consultation with other stakeholders.

This process is demonstrated for a mid-rise hospital building with basement car parking and ground level retail as shown in Figure 7.8.

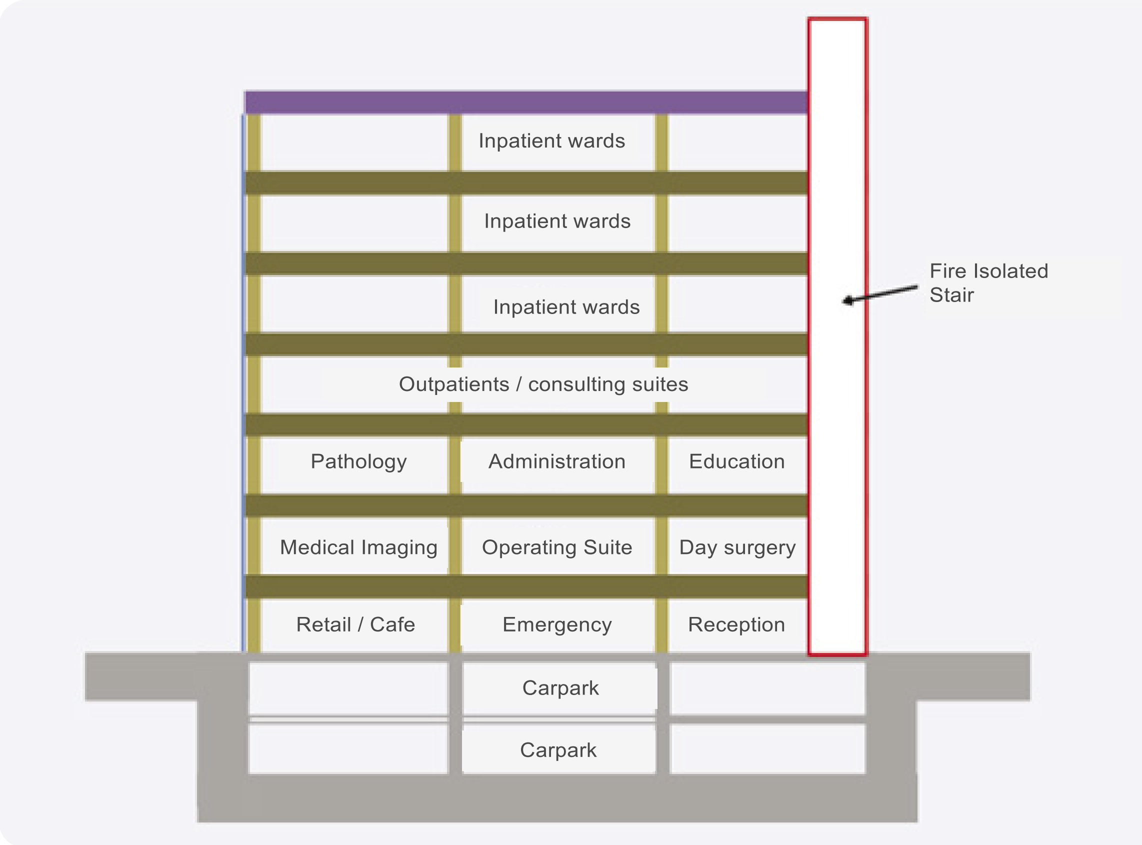


Figure 7.8: Typical Schematic Section through mid-rise hospital building.

A typical ward level plan is shown in Figure 7.9.

7.2.1 Optimising Building Layout – Provisions for Escape

As part of the strategy for evacuation of occupants during a fire emergency, the NCC prescribes DTS provisions for escape in Part D1.

The following is a brief overview of the DTS requirements that are relevant to the optimisation of the building layout of a ward-level in a Class 9a hospital building. Similar principles apply to resident levels in Class 9c Residential Care Buildings. Refer to part D of the NCC for further details.

While some mid-rise buildings (effective height less than 25 m) with a small footprint are permitted to provide one exit under the DTS provisions, Clauses D1.2(d) and D1.2(e) (summarised below) apply to Class 9a and Class 9c buildings which generally require at least two exits from each floor and in many cases more than two exits are required.

> *D1.2 Number of exits required*
>
> *(d) Class 9 buildings — In addition to any horizontal exit, not less than 2 exits must be provided from the following:*
>
> *(i) Each storey if the building has a rise in storeys of more than 6 or an effective height of more than 25 m.*
>
> *(ii) Any storey which includes a patient care area in a Class 9a healthcare building.*
>
> *(iii) Any storey that contains sleeping areas in a Class 9c building.*
>
> *(iv) Each storey in a Class 9b building used as an early childhood centre.*
>
> *(v) Each storey in a primary or secondary school with a rise in storeys of 2 or more.*
>
> *(vi) Any storey or mezzanine that accommodates more than 50 persons, calculated under D1.13.*
>
> *(f) Exits from Class 9c buildings and patient care areas in Class 9a healthcare buildings — In a Class 9a healthcare building and a Class 9c building, at least one exit must be provided from every part of a storey which has been divided into fire compartments in accordance with C2.2 or C2.5.*

Clause D1.4 also applies limits to the distance to a point on the floor where travel in different directions to two exits is available and on the maximum distance to one of the exits, which in some circumstances may require additional exits to be provided..

For Class 9c buildings (similar to commercial buildings), the NCC DTS provisions require no point on a floor to be more than 20 m from an exit or a point of travel in different directions, in which case the maximum distance to one of those exits must not exceed 40 m.

For Class 9a patient care areas (including wards) the following additional criteria apply:

- no point on the floor must be more than 12 m from a point from which travel in different directions to two of the required exits is available; and
- the maximum distance to one of those exits must not be more than 30 m from the starting point.

Clause D1.5 applies the following additional criteria to the distances between alternative exits:

- the exits are distributed as uniformly as practicable in positions where unobstructed access to at least two exits is available from all points of the floor including lift lobby areas
- the exits are not less than 9 m apart and not more than 45 m in patient care areas in healthcare buildings (in other areas this is increased to the default value of 60 m)
- paths of travel to exits do not converge within 6 m of each other.

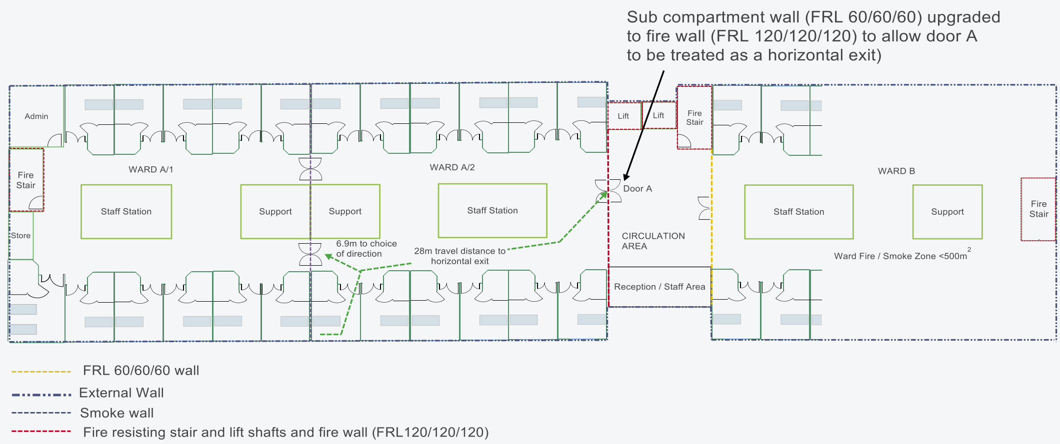


Figure 7.9: Typical ward level of a Class 9a Building.

The fire and smoke compartmentation requirements for Class 9a and 9c buildings are more extensive than most classes of building to facilitate progressive horizontal evacuation where practicable; particularly from ward, patient care and resident areas in Class 9c buildings. This may allow patients and residents close to a fire who require high levels of assistance to be quickly moved to an adjacent fire or smoke compartment to await further assistance. A fire and smoke compartmentation arrangement compliant with the NCC is shown in Figure 7.9.

One fire-isolated exit is to be incorporated into the structural cores adjacent to the lifts with additional stairs at the end of each wing addressing the need for exits to be distributed around the floor.

It is then necessary to check travel distances to exits. The critical ward enclosure was identified in Figure 7.9 and the nearest fire stair door (exit) was found to be more than 30 m away. An additional stair would be costly and a much more effective solution using a DTS Solution is to increase the FRL of the ward wall from 60/60/60 to 120/120/120 so that Door A can be treated as a horizontal exit, reducing the distance to the nearest exit to approximately 28 m (i.e. less than the maximum of 30 m that is permitted).

This highlights the need to consider compartmentation, structural design and egress provisions in combination to identify practical solutions that are compatible with the evacuation strategy for the building.

7.3 Select Structural Form

There are several structural forms that can be adopted to suit the preferred architectural layouts, including various timber forms, other materials and hybrid systems.

Where more open forms of construction are preferred, or the option to change internal layouts is needed, post and beam type construction may be preferred over lightweight framed construction.

In some cases, a mix of structural forms may be the most practical solution. For example, post and beam timber construction may be selected as the primary structural elements with lightweight timber-frame construction being used for pre-fabricated floor cassettes and lightweight timber framing for wall systems carrying lower loads or non-loadbearing wall systems.

While subsequent sections consider both options, the proprietary nature of massive timber panels, columns and beams manufactured from engineered products such as Laminated Veneer Lumber (LVL) and Cross-Laminated Timber (CLT) limits the number of generic details that can be included in this Guide.

For this example, timber is the preferred structural material for all levels above the carpark. It may have been selected for many reasons, including:

- lightweight construction (useful if ground conditions are difficult)
- speed of construction
- sustainable construction
- prefabrication of elements.

The primary reasons for selecting reinforced concrete construction for the basement were to address potential ground water penetration and also as part of the strategy to manage termite risk. In some instances, there may be advantages to extending the concrete construction to the ground floor. Typical examples include:

- Where parts of the building ground floor level are close to or below the external ground level the extension of concrete construction may be appropriate to address moisture penetration and manage termite risk.
- Part of the carpark extended to the ground floor (it is usually practicable to retain concrete construction for all levels that include carparking).
- Medical gas storage is provided within the building instead of being stored in a separate structure/ compound (see 6.13.4 Medical Gases).
- If higher levels of fire resistance are required for the ground floor or part of the ground floor compared to other parts of the building and concrete trades are on site.

7.3.1 Determine the Fire Resistance Levels Required for Structural Elements within Fire Compartments

Generally, each part of a building must be classified according to its purpose and comply with all the appropriate requirements for its classification.

The Structural Adequacy component of the FRLs required for the various fire compartments should be derived from Specification C1.1 of the NCC based on the required Type of Construction and Building Class for the relevant part of the structure. The distance from the allotment boundary and adjacent buildings also needs to be considered for external walls, external columns and other elements that could be exposed to fire from adjacent buildings. This generally defines the highest FRLs required within a fire compartment with relaxations or concessions being applied to some elements of construction particularly with respect to the integrity or insulation criteria.

It is common for hospital buildings to include parts to which different classifications apply. A simple example is shown in the schematic section in Figure 7.10.

If there are different classifications in different storeys in a Type A building, the floor between the adjoining parts must have an FRL not less than that prescribed by NCC Specification C1.1 for the lower storey (NCC C2.9).

If there are different classifications on the same storey the parts must be separated in that storey by a fire wall having the higher FRL required by specification C1.1 (NCC C2.8)

The example in Figure 7.10 shows the general layout and required structural adequacy levels for each of the floors and fire compartments within a floor having different classifications. The example building includes Class 9a (hospital), 6 (retail) and Class 7 (carpark) parts. The retail part of the building requires maximum FRLs up to 180/180/180 compared to 120/120/120 required for the hospital and carpark parts.

Integrity and insulation criteria depend on the role of the element of construction.

The requirements may require further adjustments to account for other provisions and concessions within the NCC such as the support of another part requirements in NCC Specification C1.1 Clause 2.2.

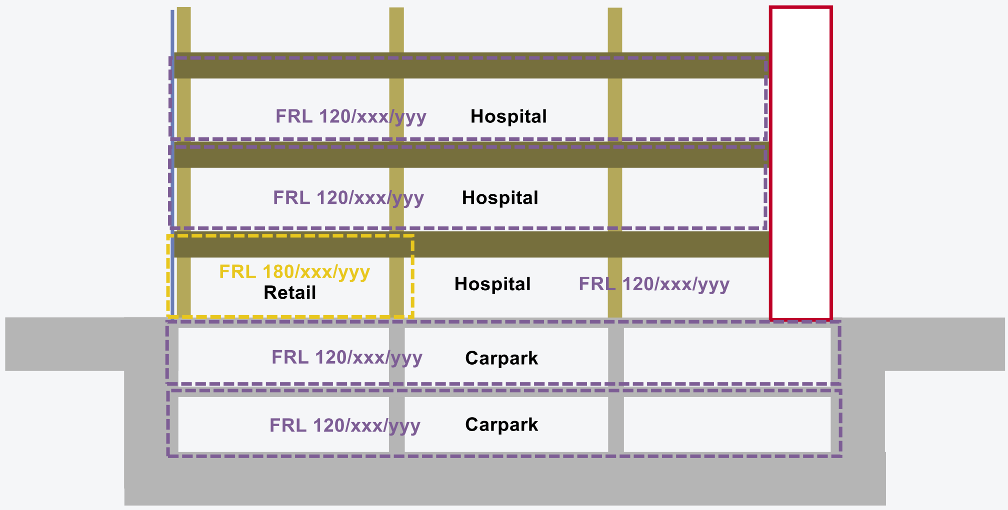


Figure 7.10: Example of a hospital building with parts of different classifications.

There is an exemption provided in NCC A6.0 where a part of a building has been designed, constructed or adapted for a different purpose and is less than 10% of the floor area of the storey it is situated on, where the classification of the other part of the storey may apply to the whole storey.

In the above example, if the retail area on the ground level was less than 10% of the floor area it can be deemed to be an ancillary area within a Class 9a building and not a separate part of the building. Under these circumstances, the FRLs within the retail area can be reduced from a maximum of 180/180/180 to 120/120/120 and a fire wall need not be provided to separate the retail and hospital parts.

__Note: It is critical to liaise with the regulatory authority in the early stages of a project to obtain a clear understanding of the classifications that will apply to the structure since it may impact on the structural form and other fundamental design decisions which will be costly to change later in the project.__

7.3.2 Select Basement and Ground Level Structural Forms

The NCC 2019 Deemed-to-Satisfy Provisions allow the use of fire-protected timber in all building Classes or parts of buildings with an effective height not greater than 25 m. However, concrete is commonly selected for construction below ground level to address ground water penetration and as part of the system providing protection from termites as noted in the previous sections.

The Class 6, 7 and 9 parts should be fire-separated in accordance with Clause C2.9 (or Clause C2.8 if different classes share the same floor).

The general Structural Adequacy FRL requirements for the carpark levels are 120 minutes and an FRL 120/120/120 fire separation is required by Specification C1.1 between the carpark level and retail level or hospital area on the ground floor.

From Specification C1.1, the retail (Class 6) ground floor level requires a general Structural Adequacy FRL of 180 minutes and an FRL of 180/180/180 for firewalls and the floor above unless the retail area can be treated as an ancillary area (typically less than 10% of the floor area) . Figure 7.11 and Figure 7.12 show typical options for fire separation from the carpark and between the retail level and hospital parts.

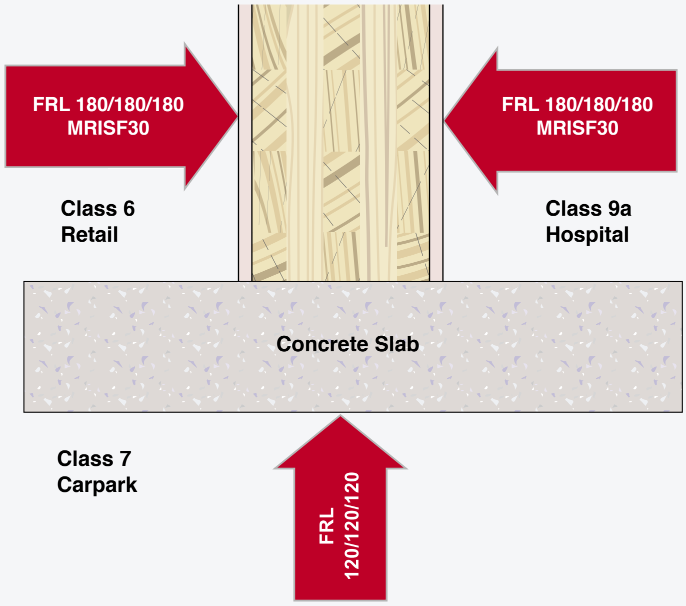


Figure 7.11: Typical massive timber detail for separation of different parts of the example building.

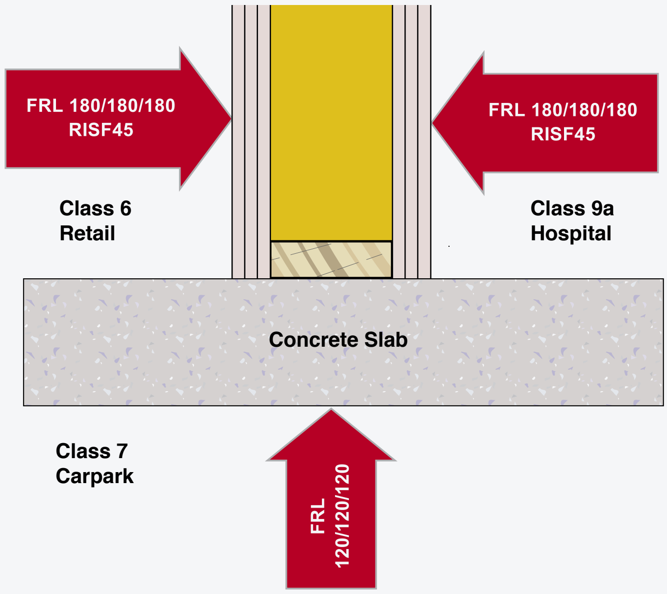


Figure 7.12: Typical timber frame detail for separation of different parts of the example building.

Refer EWFA report RIR 55945800.1B, available from the WoodSolutions website, for the assessment of timber frame wall systems for FRLs up to 180/180/180 or evidence of suitability for other proposed systems

For the massive timber fire wall option, due to the large inherent fire resistance of the massive timber, the fire protective coverings are provided to comply with the minimum MRISF requirements of 30 minutes with the outer face exposed to fire and, if necessary, to supplement the inherent fire-resistance performance of the massive timber so that an FRL of at least 180/180/180 can be achieved with an appropriate thickness and design of the massive timber element. Evidence of suitability should be obtained from material suppliers.

For the timber-frame fire wall option, substantially greater reliance is placed on the fire-protective coverings which greatly exceed the minimum RISF requirements of 45 minutes and supplementary protection within the frame cavities may be required. Further details of a system that has successfully achieved an FRL 180/180/180 and RISF of 120 minutes are provided in report RIR EWFA 55945800.1B and is available from the WoodSolutions website.

For the floor separating the retail area from the hospital areas above, an FRL of 180/180/180 is required, which normally leads to the selection of massive timber elements to provide a practical timber option. A fire protective covering is required to satisfy the minimum prescribed MRISF of 30 minutes for massive timber elements, but this may need to be increased to achieve an FRL 180/180/180 depending on the massive timber system that is selected.

Substantial simplifications and standardisations to the design can be made if the retail area is sufficiently small and treated by the authority having jurisdiction as an ancillary area. Then, other than the parking levels, the example building will be treated as Class 9a, allowing the fire wall separating hospital and retail areas on the ground floor to be deleted and the FRL of the floor element above the retail area to be reduced to an FRL 120/120/120.

7.3.3 Select Structural Forms and Fire-resistance Construction for Hospital Parts of the Building

The structural form selected for the upper levels in the example hospital building is fire-protected timber construction using a DTS Solution. This will require a maximum FRL 120/120/120 in addition to the need to comply with the RISF and MRISF requirements.

These FRLs can be satisfied using lightweight timber-framed construction or massive timber construction depending upon the specific circumstances; as shown in Figure 7.13 and Figure 7.14. Depending on the applied loads and availability of evidence of suitability, the thicknesses of the fire-protective coverings for the massive timber option may be determined based on the required MRISF; whereas for the lightweight timber-frame option the dominant requirement is likely to be the FRL.

Obtain evidence of suitability from product suppliers and WoodSolutions website, for FRLs up to 120/120/120 and RISF of 45 mins or MRISF of 45 mins as appropriate.

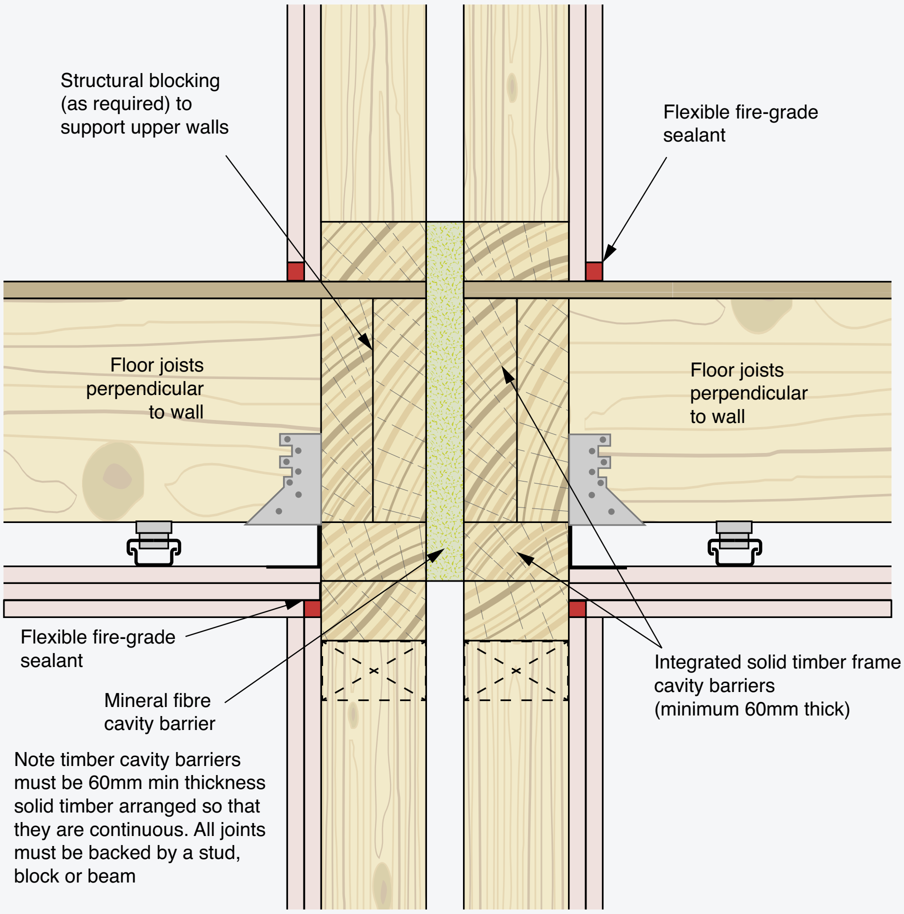


Figure 7.13: Typical FRL 120/120/120 Light-weight timber frame walls and floors.

Obtain evidence of suitability from product suppliers and WoodSolutions website, for FRLs up to 120/120/120 and RISF of 45 mins or MRISF of 45 mins as appropriate.

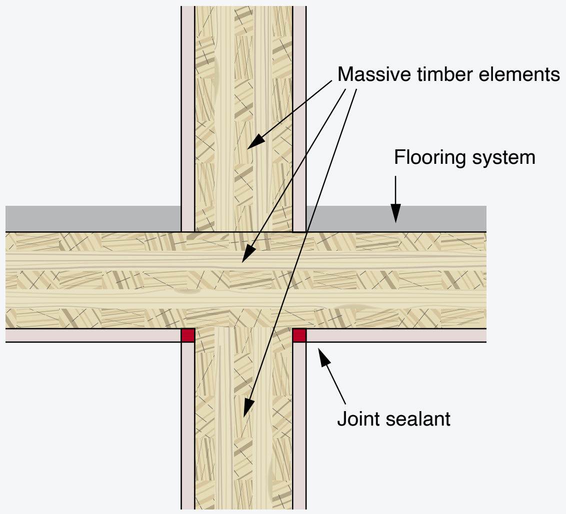


Figure 7.14: Typical FRL 120/120/120 Massive timber walls and floors.

If this detail is subjected to high vertical loads, reinforcement of wall/floor interfaces may be required.

7.3.4 Select Lift and Fire Stair Shaft Construction

Lift and fire stair shafts in mid-rise timber buildings can be of timber, masonry or concrete construction. The choice will depend on the structural design of the building and numerous other factors.

If concrete or masonry shaft construction is adopted, it is important that the detailing can accommodate the possibility of differential movement between the timber structure and masonry/concrete shafts. Further information relating to masonry and concrete shaft construction lies outside the scope of this Guide.

Fire-protected timber shafts can be timber-framed construction or massive timber panel systems. Both options will be considered in subsequent sections.

An independent structural frame can be provided within the shaft as part of the lift installation, effectively isolating the lift system from the shaft walls and providing adequate acoustic separation.

If an independent steel frame is used within a fire-protected timber shaft, the possibility of differential movement between the timber and steel frame will need to be addressed. Typically, this can be addressed if the lift system can be readjusted for differences in floor levels.

7.3.5 Structural Design

Issues that should be considered in the structural design of mid-rise timber buildings include:

- The design must comply with the relevant NCC requirements including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1(a)(iii) (structural robustness). A first principles performance pathway can also be adopted that addresses both fire and structural performance. Further guidance is provided in *WoodSolutions Technical Design Guide #39 Robustness in Structures.*
- The lighter mass of timber to that of masonry/concrete construction – greater attention needs to be given to resistance against overturning.
- The greater effect from wind loads than expected on smaller structures. This is due to a greater height-to-width ratio, resulting in a need for attention to resistance to overturning.
- Potential for movement (and differential shrinkage in buildings of hybrid construction) in taller timber buildings. Movement can be minimised by:
  - using seasoned timber or engineered timber
  - constructing bearers and joists in the same plane
  - detailing to avoid differential shrinkage between dissimilar materials, e.g. steel to timber; timber to masonry or allowing articulation to absorb the differential movement
  - allowing for differential movement with respect to plumbing and other services.

A professional structural engineer with appropriate skills will be needed to ensure the above issues and structural performance in general are adequately addressed. Guidance is provided in *WoodSolutions Technical Design Guide #50 Mid-rise Timber Building – Structural Engineering*.

The following standards should be called on where appropriate:

- AS 1170.0 – Structural design actions – General Principles
- AS 1170.1 – Structural design actions – permanent, imposed and other actions provides the basis for determination of appropriate dead, live design loads and loads combinations
- AS 1170.2 – Structural design actions – wind actions – which provides the basis for wind loads
- AS 1170.4 – Structural design actions – Earthquake actions in Australia – which provides guidance and design procedures for earthquake forces
- AS 1720.1 – Timber structures – Design methods
- AS 1720.5 – Timber structures – Nail plated timber roof trusses

In addition:

- Select details that minimise the effects of shrinkage (especially since differential shrinkage may have an adverse impact on the function of fire-resisting wall and floor elements).
- Check walls and columns are capable of supporting multi-storey load paths from above. Enlist internal fire-resisting walls and columns if required.
- Check that any elements supporting loads (including bracing elements) are treated as fire-resisting construction and designed accordingly.

7.4 Establish Service Plant Areas, Service Runs, Risers and Shafts

7.4.1 Service Plant Areas

Service plant rooms are generally located away from public areas, either in basements or on roof tops.

Clauses C2.12 'Separation of Equipment' and C2.13 'Electricity Supply System' generally require certain types of equipment to be fire separated from the rest of the building by construction having an FRL 120/120/120 with doorways protected by self-closing fire doors with an FRL not less than –/120/30.

For hospital buildings, some plant areas and related storage areas such as medical gas storage and distribution systems have additional safety requirements that lie outside the scope of the NCC and the preferred option may be for these services to be in a different building/compound. If they have to be stored within the building, the most practical solution would be to house plant rooms in the basement with areas such as medical gas storage located at ground level to provide safe access. The concrete sub-ground structure should then be extended so that these areas can be constructed from concrete as appropriate. Refer Section 6.13.4 Medical Gases for further information relating to medical gases.

Emergency power generators may also need to be located in plant areas and particular attention needs to be paid to fuel storage and supply. A preferred option may be for fuel storage to be located in a dedicated building or compound independent of the main hospital building.

7.4.2 Service Runs

In fire-protected mid-rise timber buildings, the timber elements are protected by fire-protective coverings and services tend to be concealed in a similar manner to conventional building designs using service risers, ducts and fitting cabling and pipes behind false wall and ceiling linings.

While the use of cavities within fire-protected timber construction to run cables and pipes can appear to be a simple solution, this choice presents several issues including:

- difficulty in maintaining the RISF or MRISF ratings of the elements at points of service penetrations
- risk of acoustic separation being compromised
- risk of fire protection systems not being correctly installed after modifications or additions to existing installations
- risk of disruption of concealed cavity barriers during modifications or additions
- risk associated with medical gases (e.g. oxygen which can increase the flammability of materials significantly). Refer Section 6.13.4 Medical Gases for further information relating to medical gases.

A more reliable option is to plan the layout of required services and potential future services carefully utilising service risers, service shafts and ducts, and additional (false) linings to conceal services minimising penetrations through fire-protected timber elements as far as practicable. In addition, the services should be run, and penetrations through fire and smoke resistant elements located, such that they are readily accessible for maintenance and servicing without significantly disrupting the operation of a healthcare facility. This approach is described in the following sections.

7.4.3 Service Risers and Horizontal Distribution of Services

Figure 7.15 shows part of a typical ward area of a hospital and has been used to provide a typical example of planning the location of service shafts and service runs. The same general principles apply to other healthcare facilities and parts of the hospital although the mix and complexity of services will vary.

Services such as electricity, water and telecommunications/data systems are normally distributed between floors through service risers/shafts that are commonly located close to the structural cores. These may be consolidated in a single location or distributed depending on the size and complexity of the building. Two general service riser locations adjacent to the fire stairs are shown, highlighted in green in Figure 7.15.

Fire compartmentation can be maintained by protecting the service penetrations at each floor level or using fire-resisting construction for the risers (shafts), fitting fire doors or fire-rated access panels to the risers and fire protecting each service where it penetrates the riser wall (refer Figure 7.17).

Generally, the option of protecting the service penetrations at each floor level is the most practical solution. This can be achieved by forming an opening in the floor such that no timber is exposed, the FRL and RISF or MRISF is not compromised and services can be run through the opening. The opening can be protected by a multi-service penetration system such as a pillow, mineral fibre batt, or other proprietary fire protection system that can be readily reinstated if additional services need to be run. A typical example is shown in Figure 7.16.

Using a framed opening also avoids the need to expose cavities and timber members if additional services need to be run substantially reducing the risk of cavity fires and premature ignition of timber members. Refer to Section 7.13 Service Penetration Treatments for further details on the selection of service penetration systems.

For the hospital building example, the services have been distributed from the risers above a false ceiling as shown schematically in Figure 7.17.

Services can be run through internal walls within a fire or smoke compartment without the need for protection provided the wall is not required to be of fire-resisting or of smoke-proof construction.

Most loadbearing members are required to achieve an FRL even if they are not part of a fire-resisting wall.

Adding/modifying services is simplified if services are protected at each floor level by framing out the opening and using a multi-service penetration system that can be easily reinstated

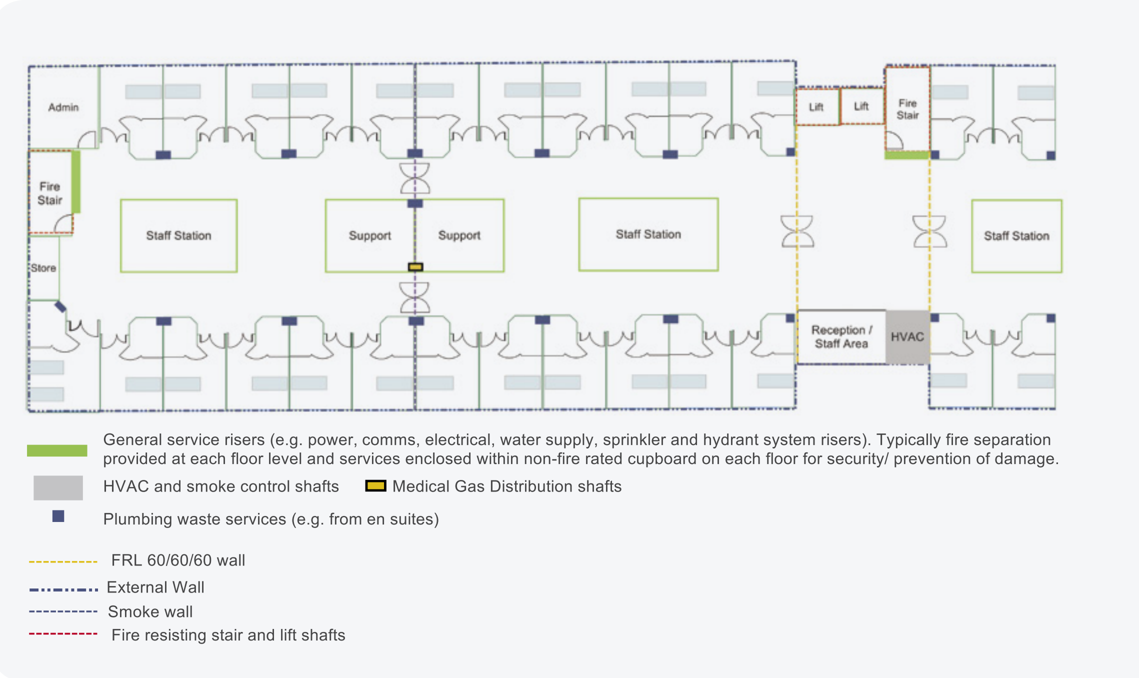


Figure 7.15 Typical riser locations for a ward level in a hospital

The face of openings within fire-protected timber elements need to be protected so that no timber is exposed. The required RISF or MRISF and the FRL of the fire-protected timber must also be maintained. Continuing fire protective coverings around the opening is a typical solution.

Walls that are not required to achieve an FRL or be smoke-proof can be a practical option for the location of services such as power outlets.

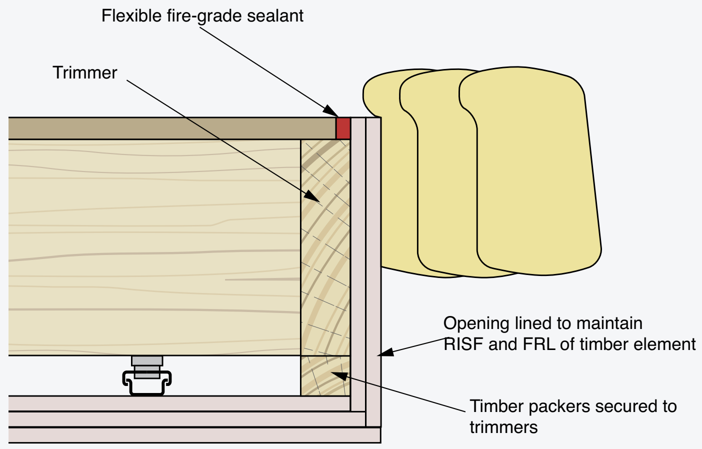


Figure 7.16: Typical riser penetration detail through a fire-protected timber floor.

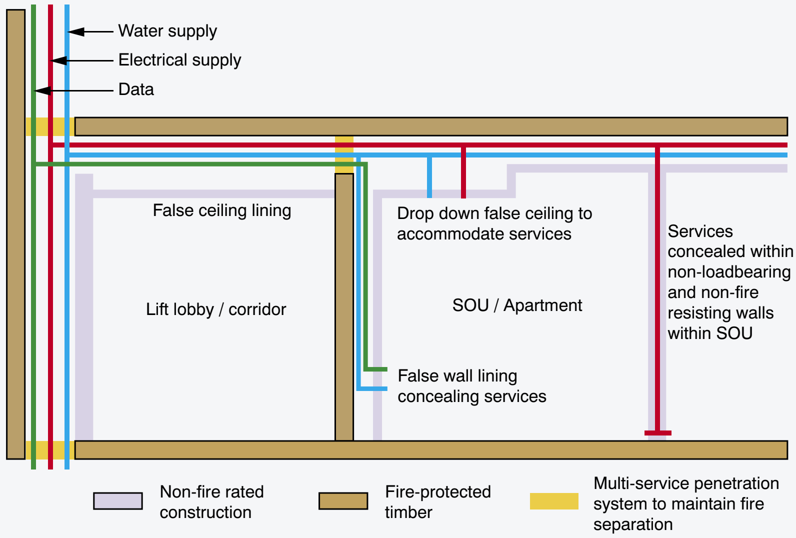


Figure 7.17: Typical distribution of general services.

The use of false ceiling linings to conceal services can provide a number of advantages including a significant improvement in the reliability of fire-protection systems by:

- avoiding large numbers of individual service penetrations within fire-protected timber elements for services such as power outlets, lighting and plumbing services, including fire sprinklers
- concealing pipework and cable runs
- allowing reconfiguration of services without disrupting fire-protected timber elements
- reducing the risk of cavity fires during maintenance activities and the risk of fire spread to cavities if the fire protection of services is not reinstated after reconfiguration or repair to services
- enabling services to be grouped together and protected by a single multi-service penetration system fitted above the false ceiling (access panels can be provided to facilitate access for inspection and/or adding or modifying existing services).

To manage waste services from ensuites and other areas, where pipe falls are required to ensure adequate flow, it may be impractical to use small numbers of centralised shafts and distributing shafts close to ensuites can present a practical solution as shown shaded dark blue (plumbing waste services) in Figure 7.15. For this application, services are normally fire-protected at the point where the shaft wall is penetrated rather than providing protection at floor level and therefore the shaft is required to have an FRL of –/90/90 if non-loadbearing and 120/90/90 if loadbearing.

Refer Section 6.13.4 Medical Gases for further discussion and seek specialist advice regarding requirements of the healthcare building operators and regulatory requirements applicable within the relevant jurisdiction.

HVAC and smoke hazard management systems are generally run within service shafts, but the horizontal distribution details will vary depending on the application, as will the distribution of shafts across a floor. For simple HVAC systems that are required to shut down in the event of a fire emergency, horizontal distribution can be achieved above a false ceiling with fire and/or smoke dampers being provided where fire-resisting walls and smoke-proof walls are penetrated. For complex smoke management systems reference should be made to the NCC and AS 1668.1.

Treated of medical gases requires consideration of additional protection measures that lie outside the scope of the NCC and additional requirements relating to combustibility of shafts and ducts together with requirements for ventilation of shafts and ducts, controls of materials used for distribution and service penetration fire protection systems, etc. In some instances, it may be practicable to provide medical gas shafts to serve each smoke and fire compartment and, if the horizontal pipe runs are to be covered, enclosing them in dedicated ducts with required ventilation.

7.4.4 Service Shafts

There are advantages (space, economy and design options and reduced risk to critical structural elements) in ensuring that service shafts are non-loadbearing.

The integrity and insulation criteria for non-loadbearing service shafts are –/90/90 for Class 9a and 9c buildings but an FRL of 120/90/90 is required if the shaft is loadbearing; further encouraging the use of non-loadbearing service shafts.

Where timber framing is used to support the shaft linings, fire-protective coverings must be applied to both sides of the timber frame and service penetrations will be required to satisfy the RISF criteria. A more practicable alternative is to use laminated plasterboard or shaft wall systems. These systems are proprietary, developed by fire-protective covering manufacturers, and reference to their details is required. The number of layers, type and thickness of plasterboard (or other fire-protective covering) and fixing methods selected depend on the required FRL and Evidence of Suitability that is available.

Figure 7.18 and Figure 7.19 show details of typical fire-resisting laminated shaft details and their connection to fire-protected timber elements.

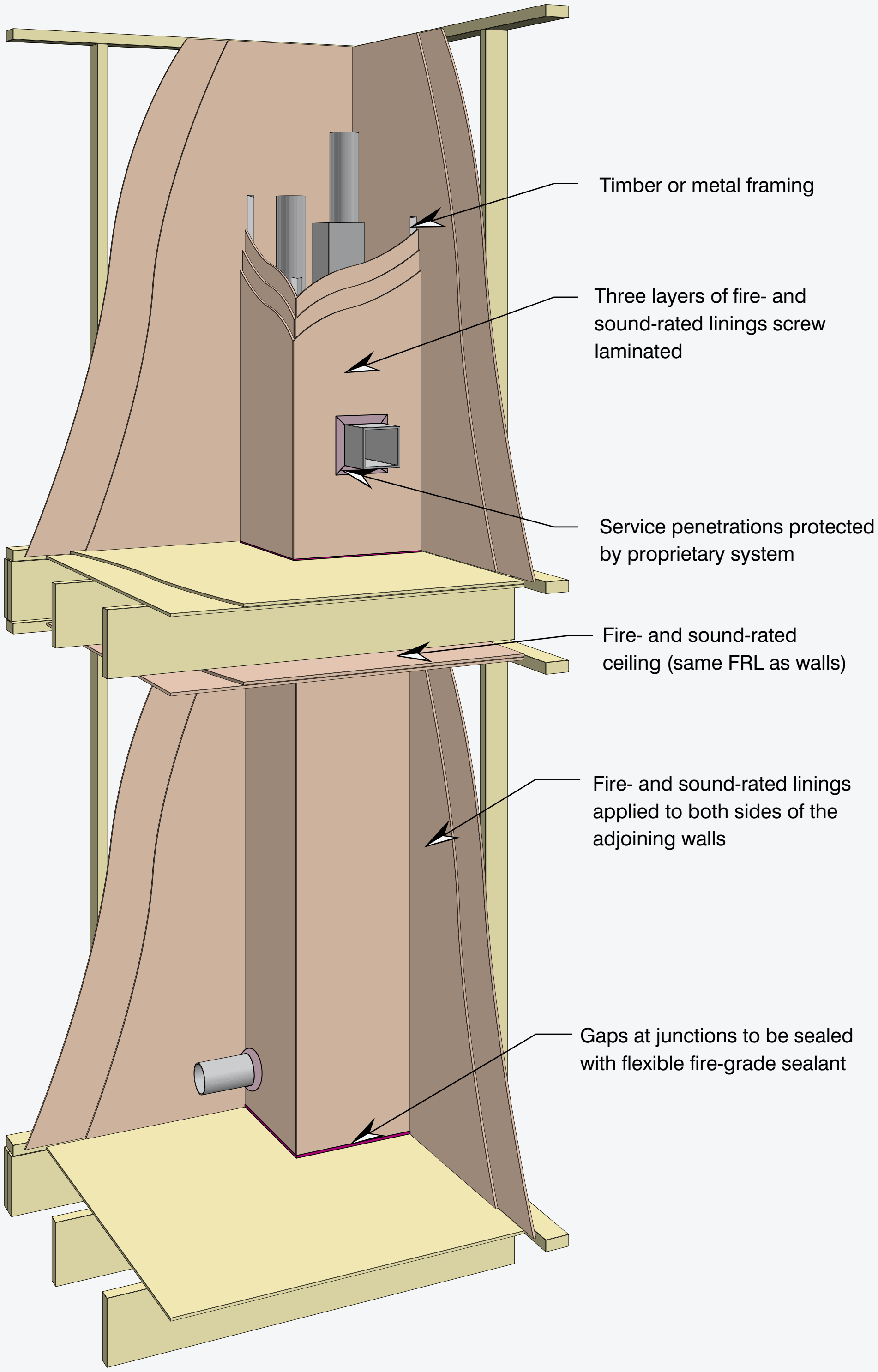


Figure 7.18: Typical fire-resisting service riser/shaft.

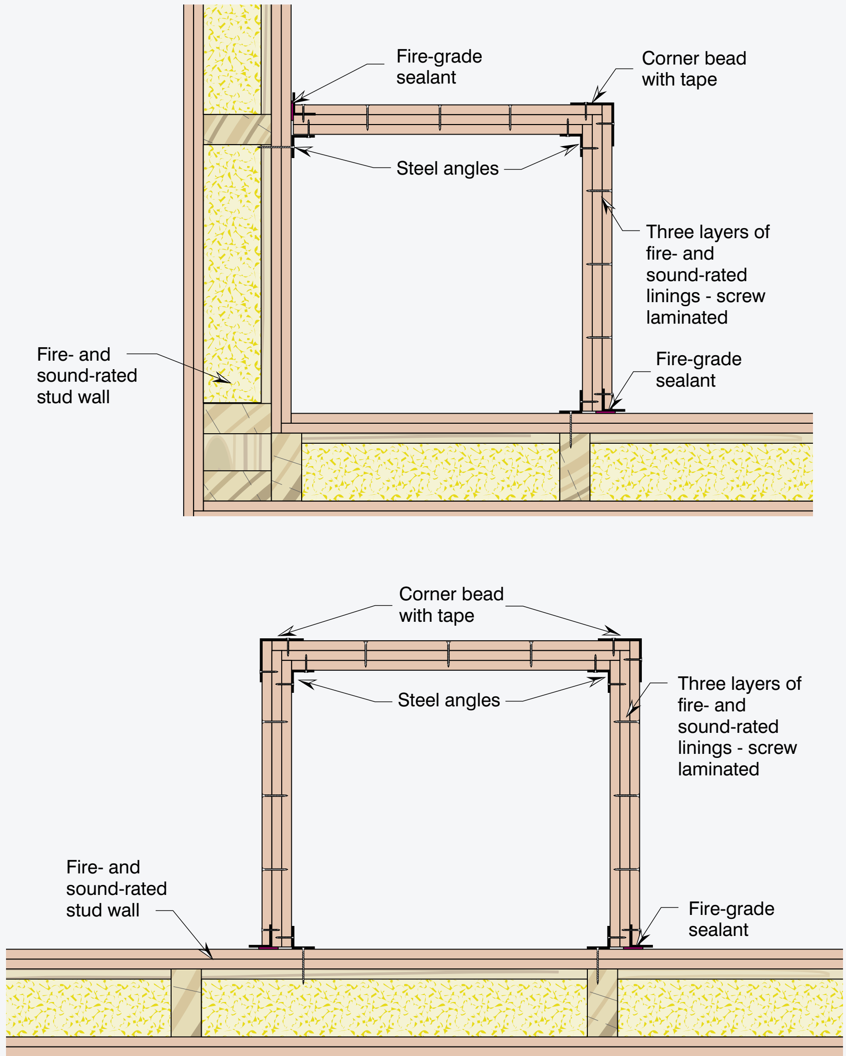


Figure 7.19: Laminated fire-grade plasterboard used to create shafts.

7.5 External Walls

External walls must be designed to satisfy a range of criteria including:

- fire performance
- structural performance (for safety and serviceability)
- weather resistance (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation (the control of transmission of sound from external sources is not required by the NCC but may be part of a design brief or planning control).

7.5.1 Fire Performance of Fire Protected Timber External Walls

The external face of the wall may form the fire-protective covering of a fire-protected timber element. An example is the brick veneer construction shown in Figure 7.20.

If this option is used, the specification will need to address, among other things, the installation of cavity barriers to ensure correct placement and so moisture is not transported from the internal brickwork face to the timber frame through the cavity barrier.

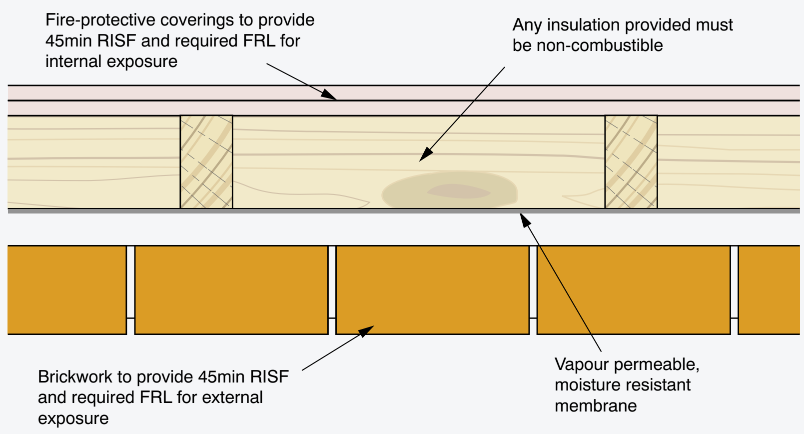


Figure 7.20: Fire-protected timber brick veneer external wall.

Alternatively, a cladding system may be fixed to a fire-protected timber element to prevent water penetration and serve other non-fire related functions. The cladding system could be a direct fix system or ventilated systems as shown schematically in Figure 7.21 and Figure 7.22 for lightweight timber-frame and massive timber construction, respectively. These figures may not show all components that form part of proprietary systems.

Many massive timber panels are proprietary products. Fire (and other) properties depend on the adhesives used and manufacturing processes, which are currently not fully standardised. Evidence of Suitability for massive timber external wall systems will tend to be product specific in most instances and configurations will tend to vary to satisfy the relevant NCC and other design requirements.

Fixings for the cladding system must be detailed so that the performance of the fire-protective coverings is not compromised.

The NCC DTS provisions require the external walls to be of non-combustible construction or of fire-protected timber construction. Therefore, any cladding systems applied to fire-protected timber external walls in mid-rise buildings must be non-combustible to comply with the NCC DTS provisions.

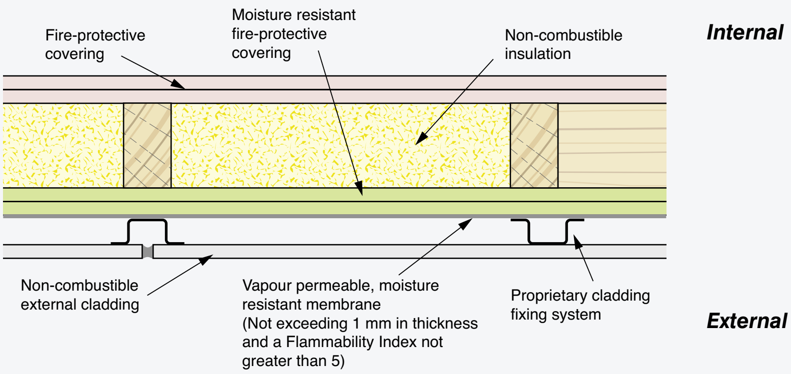


Figure 5.19: Fire-protected timber frame external walls with lightweight cladding.

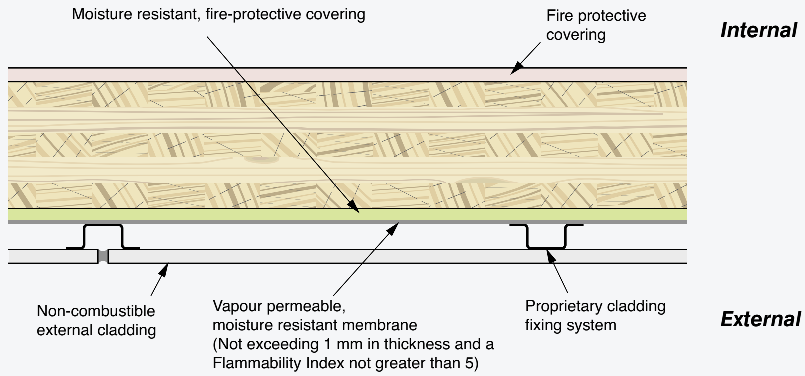


Figure 5.20: Fire-protected massive timber external wall with external lightweight cladding.

If combustible cladding systems are to be used, the Performance Solution pathway must be adopted to demonstrate compliance of the external wall system with the relevant NCC performance requirements. Verification method CV3, in conjunction with verification methods CV1 and CV2, and the classification standard AS 5113 define an appropriate method for demonstrating compliance in most States and Territories.

Table 7.11 summarises the required FRLs and RISF or MRISF based on the distance from the boundary for Class 9 buildings. Refer to NCC Specification C1.1 Table 3 for other classes.

Table 7.11: Fire-resistance requirements for external walls for a Class 9 building of Type A construction

Distance from fire source feature	**FRL – Structural Adequacy /Integrity/ Insulation – minutes**		**General Timber**	**Massive Timber**
	Loadbearing	Non-Loadbearing	RISF (minutes)	MRISF (minutes)
≤1.0 m	120/120/120	–/120/120	45	45 external 30 internal
<1.5 m	120/120/120	–/120/120	45	30
≥1.5 and <3 m	120/90/90	–/90/90	45	30
≥3 m	120/60/30	–/–/–	45	30
External Columns	120/–/–-	–/–/–	45	30

Table 7.11 is a derivative of Specification C1.1 Table 3 and Specification C1.13a of the National Construction Code 2019 Amendment 1 provided by **https://www.abcb.gov.au/** *Australian Building Codes Board © 2020.*

While there are significant reductions in the required FRLs for non-loadbearing elements as the distance from the fire source feature increases, the design of the external walls will not vary significantly because the required RISF or the MRISF, in combination with the minimum thickness requirement of 75 mm for massive timber, will become the dominant design factors.

If the subject building is of massive timber construction and is not more than 1 metre from a fire source feature, the required MRISF is increased to 45 minutes externally to minimise the risk of fire spread from adjacent structures.

Table 7.11 indicates that an FRL of –/–/– is applicable to an external wall system that is non-loadbearing and more than 3 m from a fire source feature. In these applications, a common solution is to use curtain walling system (which must be non-combustible) secured to floor plates Any openings between the floor plate and the curtain walling systems need to be protected to maintain the same FRL as that required for the floor system. These perimeter seals are sometimes also referred to as cavity barriers, but the performance levels and Deemed-to-Satisfy Solutions provided in Specification C1.13 for cavity barriers within timber frame construction should not be used since, among other things, higher FRLs are required at the perimeter of a floor plate. For the example building, the opening between the edge of the floor plate and curtain walling system would need to be protected by a system capable of achieving an FRL –/120/120. These are available from some specialist passive fire protection system suppliers but the evidence of suitability should be checked for each application.

7.5.2 External Noise

Currently, there are no NCC requirements to provide external noise attenuation for buildings. However, Government authorities have regulatory or legislative powers to require control of noise entering buildings and market forces may generate requirements.

WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise provides examples of lightweight external wall systems that can be used as guidance.

7.5.3 Weatherproofing

At the time of writing there were no Deemed-to-Satisfy Provisions in the NCC in relation to the weatherproofing of external walls and so suppliers of waterproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). A weatherproofing Verification Method (FV1.1) is described in the NCC to enable compliance with FP1.4 via a tested prototype. It is important that installed waterproofing membranes/systems for timber construction are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate through to the structural timber building elements (moisture resistant).

7.6 Fire-protected Timber Floors

Floor systems must be designed to satisfy a range of criteria including:

- structural performance (for safety and serviceability)
- fire performance
- acoustic separation
- durability.

Common structural elements used for timber floors include:

- solid timber beams
- LVL beams
- I-section beams with OSB or plywood webs
- parallel chord steel web trusses
- parallel chord timber web truss
- I-section with Steel Web
- massive timber panel systems (e.g. CLT or LVL).

These structural members can be used with a range of flooring systems, internal insulation systems and soffit/ceiling lining systems in keeping with the building finishes and to achieve the required fire and acoustic performance.

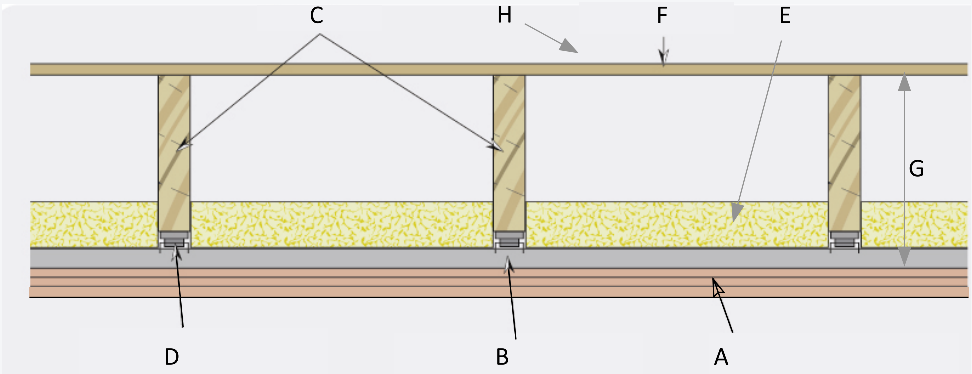
7.6.1 Fire Performance of Flooring Systems Protected by Typical Ceiling Systems

Typical floor systems that satisfy the fire related NCC DTS fire requirements for typical Class 9a or 9c healthcare building are shown in Figure 7.23 for lightweight timber-frame construction and Figure 7.24 for a CLT floor system for applications where a considerable reliance is placed on the fire-resistance contribution from the ceiling.

The system incorporates a ceiling system comprising three layers of 16 mm thick fire-protective grade plasterboard secured to steel furring channels supported from the structural element. Since the ceiling provides the largest contribution to the FRL of the floor/ceiling systems, and the performance will be largely independent of the structural members prior to structural failure, the results can be applied to a large range of combinations of structural element, cavity insulation and flooring systems to which additional finishes may be applied, provided compliance with other NCC requirements is not compromised.

For lightweight timber-framed floor systems, the required thickness of the fire-protective coverings tend to be dominated by the FRL criteria rather than the RISF criteria of 45 minutes and the above construction could be considered a practical solution.

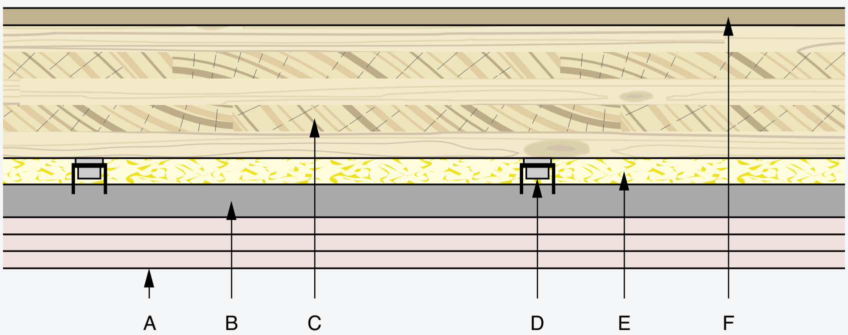
The fire performance of CLT systems depends on many factors, including adhesives, number of layers and thicknesses of lamella, manufacturing process, timber species and grade, and applied loads. For massive timber panel floor systems and beams, thicknesses greater than 150 mm are commonly used to achieve adequate structural performance. Under these circumstances it is likely the required thickness of fire-protective coverings will be dominated by the MRISF or RISF criteria rather than FRL criteria because of the high inherent fire resistance of the massive timber panels. Since variables such as adhesives and maximum gaps between lamella are not fully standardised in Australia, evidence of suitability in the form of fire-resistance test reports from Accredited Testing Laboratories should be obtained from the suppliers of the specific CLT system to confirm the FRL of fire-protected CLT members if a significant reliance is placed on the inherent fire resistance of the panels.



Construction
A Linings 3 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 120/120/1200 (verify with supplier and/or WF report FAS190034-RIR 1.0)
RISF 120 mins (verify with supplier and/or WF report FAS190034-RIR 1.0) or RISF 45 mins (NCC Spec C1.13a DTS)

Figure 7.23: Typical timber-framed floor – FRL 120/120/120.



Construction
A Ceiling linings 3 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Massive Timber Panel (min 75 mm thick)
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Cavity minimum 40 mm (must be filled with non combustible insulation if massive timber 'concession' applied otherwise may be unfilled)
F Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 120/120/120 (verify with supplierand/or WF report FAS190034-RIR 1.0) Note higher FRLs may be achieved depending upon massive timber element design
MRISF/RISF 120 mins (verify with supplier and/or WF report FAS190034-RIR 1.0) or MRISF/RISF> 45 mins (NCC Spec C1.13a DTS)

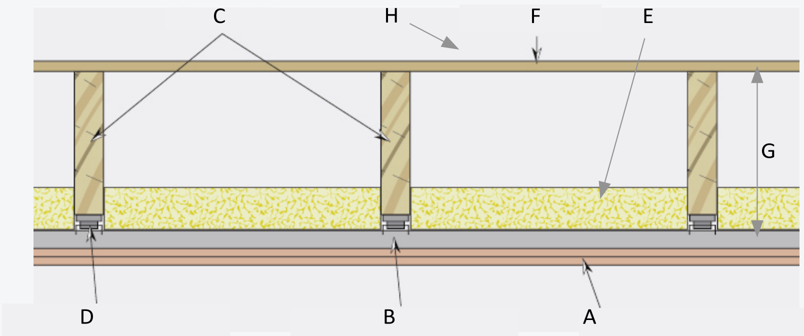
Figure 7.24: FRL 120/120/120 Massive timber floor system. Plasterboard is not direct fix and therefore massive timber concession does not apply and RISF 45 is applicable.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers.

7.6.2 Sound

The sound performance of a floor/ceiling system depends on a number of elements including: the density of the floor covering (tile, timber, carpet), isolation from the structure (acoustic underlay), ceiling insulation (density), ceiling installation (acoustic mounts) and layers and thicknesses of ceiling plasterboard. The objective is to minimise both airborne (Rw+Ctr) and impact sound (Ln,w) transmission through the floor/ceiling system and the performance of the floor/ceiling system should be verified with the plasterboard supplier for DTS Solutions.

There are a range of flooring products (e.g. timber overlay, carpet) that can be used and achieve the minimum NCC acoustic requirements. The use of a hard flooring surface will influence the impact performance (Ln,w) of the floor/ceiling system. Acoustic performance is not an NCC mandatory requirement for Class 9a buildings but is for Class 9c buildings. The following provides an example of a typical timber-framed floor/ceiling system performance.



Construction
A Linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm thick
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – thickness 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier)[xxv]
RISF 60 mins (verify with supplier)
or RISF 45 mins (NCC Spec C1.13a DTS)
Acoustics (Verify with supplier):
Above system incorporates: 10 mm overlay solid strip flooring, 4.5 mm acoustic underlay on top of flooring products (F), R2.5 ceiling insulation (E), Acoustic (resilient) mounts (D).
$R_w + C_{tr} \geq 50$
$L_{n,w} \leq 62$ (bare floor)
$L_{n,w} \leq 50$ (carpet and underlay)

Figure 7.25: Sound performance of typical timber-framed floor/ceiling system.

The acoustic performance of various tested CLT floor/ceiling system configurations can be found in *WoodSolutions Technical Design Guide #44 CLT Acoustic Performance.*

7.7 Service Shafts

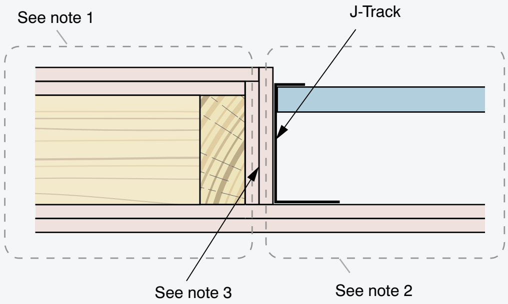
While service shafts can be constructed from fire-protected timber walls, in many instances there are substantial advantages in using either steel stud shaft wall or laminated shaft wall construction, particularly if the shafts are in locations where sound transmission is not a significant consideration.

The advantages include:

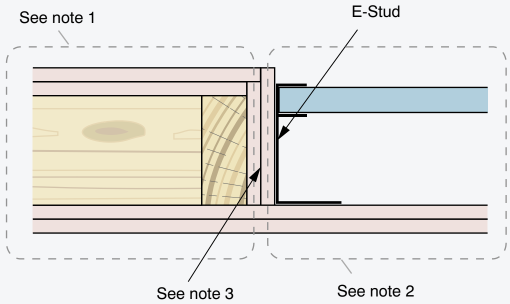
- ease of construction
- smaller footprint (more usable space)
- simplification of treatment of service penetrations
- greater selection of proprietary fire protection systems for service penetrations that already have Evidence of Suitability to demonstrate the FRLs of the systems.

It is important that the fire performance is not compromised at the interfaces between the shaft and fire-protected timber walls and floors. Figure 7.26 and Figure 7.27 shows typical interface details for steel framed shaft construction and Figure 7.28 shows typical interface details for laminated shaft construction. These interface details have been assessed by an Accredited Testing Laboratory (WFA report reference RIR 37401400) and found not to compromise the performance of the wall or shaft systems provided the FRL performance of the shaft system is not exceeded.

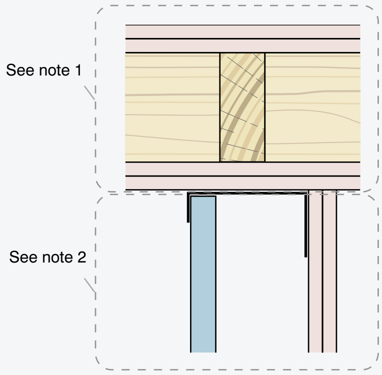
Report RIR 37401400 available from the WoodSolutions website assesses the impact of the interface details on the FRL, RISF and MRISF of the systems



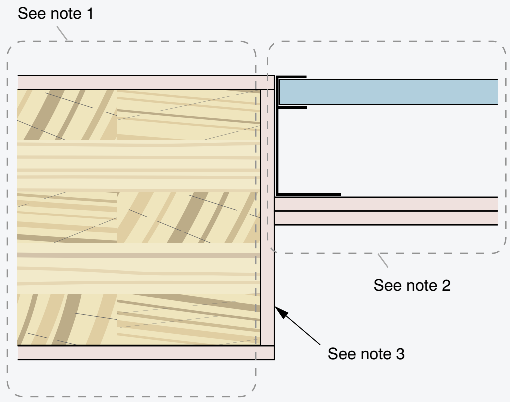
Steel stud shaft to timber stud with J-Track



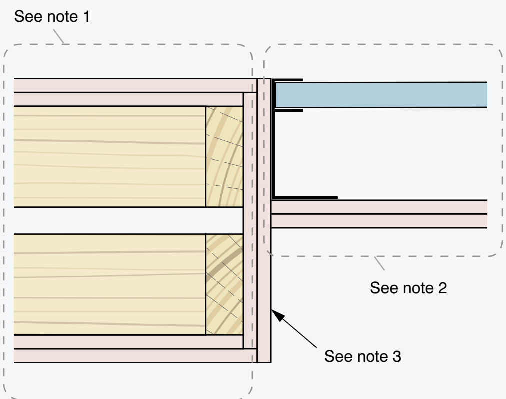
Steel stud shaft to timber stud with E-Stud



Steel stud shaft to timber stud perpendicular



Steel stud shaft to massive timber stud



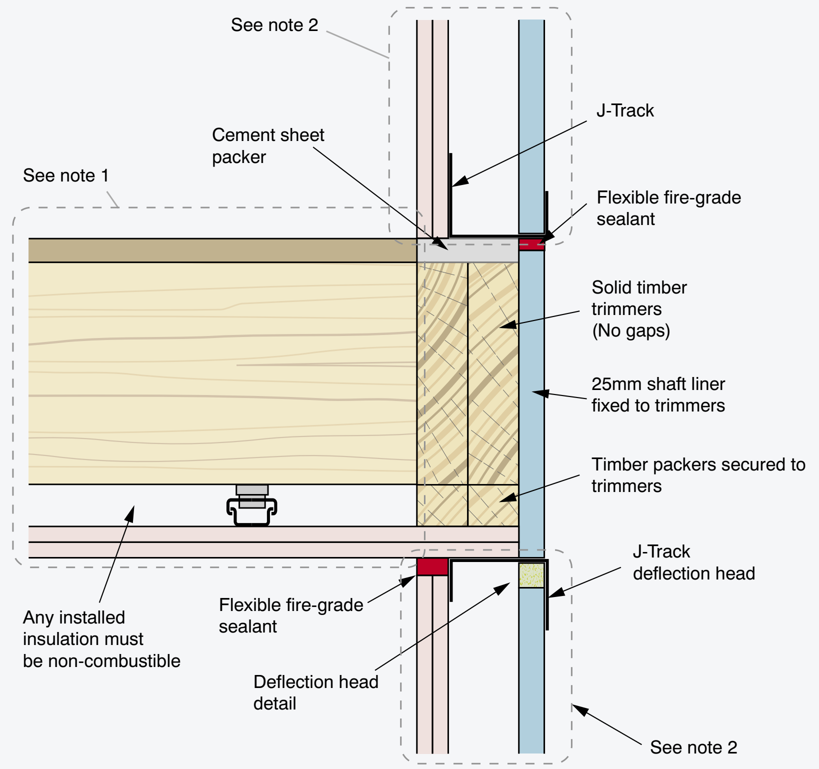
Steel stud shaft to massive timber panel

Figure 7.26: Interfaces between fire-protected timber and steel stud shafts.

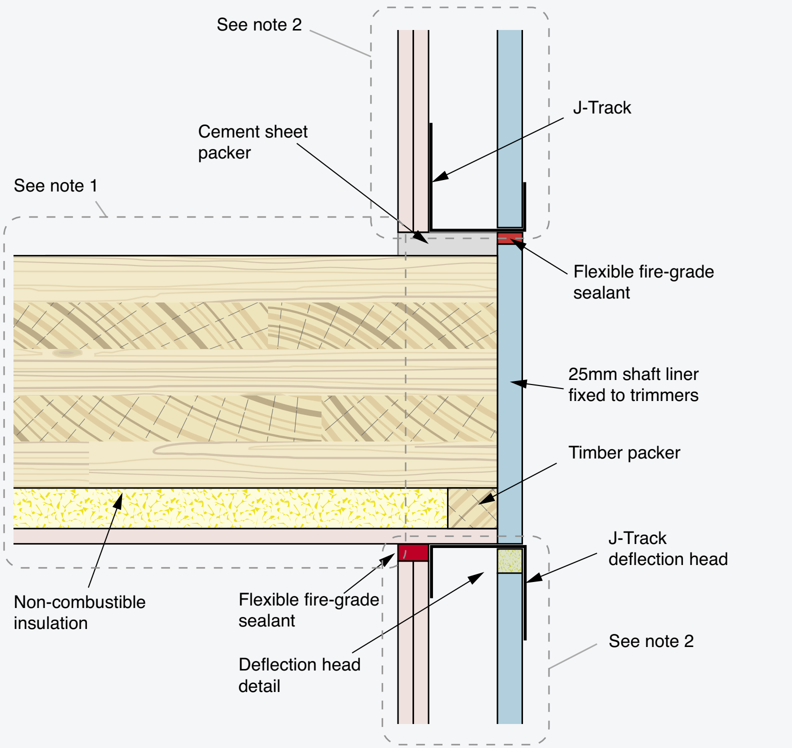
Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws



Shaft continuity at timber framed floor



Shaft continuity at massive timber panel floor

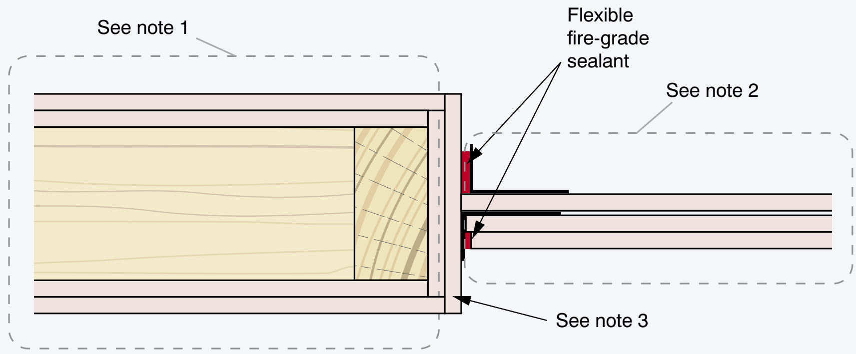
Figure 7.27: Interfaces between fire-protected timber and steel stud shafts.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

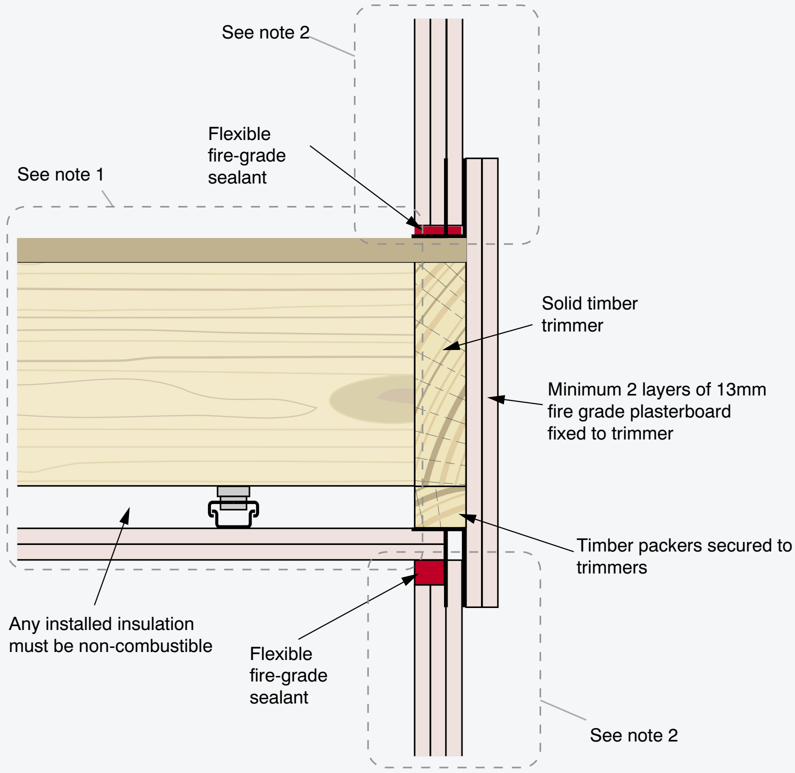
Note 2: Shaft wall construction having the required FRL.

Report RIR 37401400 available from the WoodSolutions website assesses the impact of the interface details on the FRL, RISF and MRISF of the systems

Report RIR 37401400 assesses the impact of the interface details on the on the FRL, RISF and MRISF of the systems



Laminated shaft to timber stud wall



Shaft continuity at timber framed floor opening. (Massive timber detail similar)

Figure 7.28: Interfaces between fire-protected timber and laminated board shafts..

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws.

7.8 Fire Doors in Fire-protected Timber Walls

Fire door assemblies are required to comply with AS 1905.1 as appropriate in addition to achieving the required FRL. Generally, fire doors are required to be tested when mounted in a wall of representative construction. Evidence of suitability should therefore be provided from the supplier that relates to the performance of their fire doors when mounted in representative timber elements of construction.

In addition, the fire doors must not compromise the RISF or MRISF performance of the wall. The frame-fixing details shown in Figure 7.29 have been assessed by an Accredited Testing Laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber-frame systems and 30 minutes for the massive timber panel systems (Refer report reference RIR 37401400). Other details may be adopted if appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements for fire doors and fire-protected timber elements is provided.

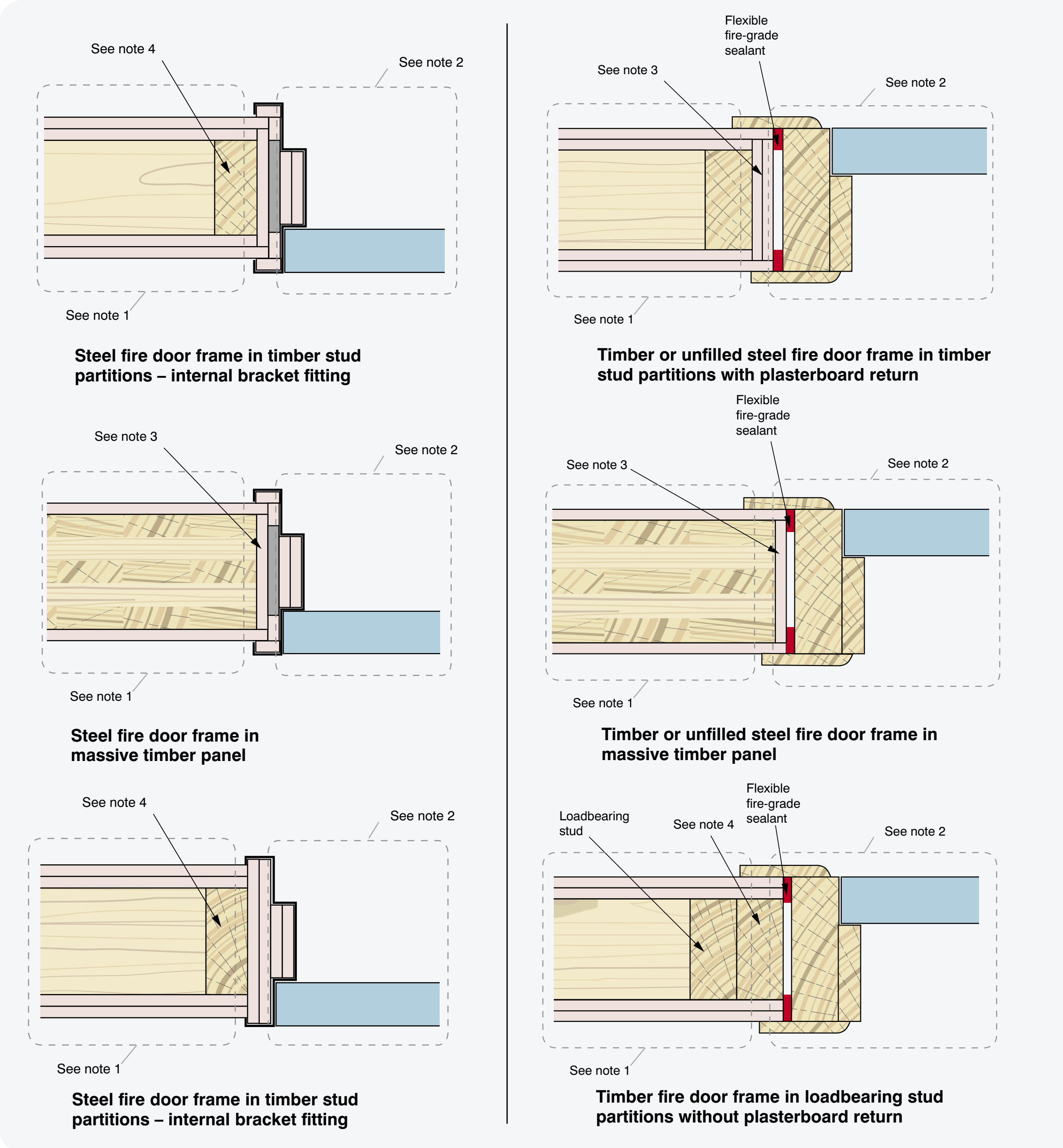


Figure 7.29: Fire door interface details.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Fire Door Assembly with the required FRL determined in accordance with AS 1530.4 and AS 1905.1 as appropriate.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

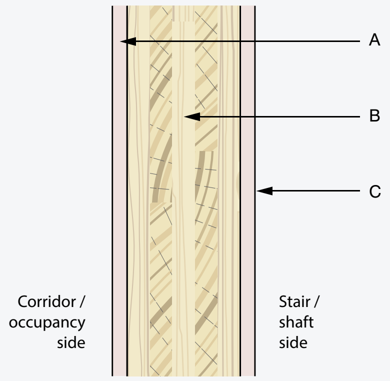
Note 4: Minimum of 45 mm thick non-loadbearing solid timber cavity barrier framing the cavity opening around the door.

7.9 Construction of Fire-isolated Shafts

Fire-isolated stair shafts can be constructed from fire-protected timber, concrete masonry and other non-combustible non-loadbearing materials or a hybrid construction may be adopted. The selection will depend on the structural design of the building, construction programming and other factors. The required FRL for fire-isolated stair shafts in healthcare buildings is 120/120/120 unless modified by other parts of the building having a different Class.

Where concrete or masonry shafts are used the design will need to account for differential movement between the shaft and timber structure.

A concession is provided for massive timber panels in that the fire-protective covering for the internal face of the shaft is permitted to achieve a MRISF of 20 minutes rather than the 30 minutes required for the outer face (see Figure 7.30 and Figure 7.31). The fire protective coverings around the face of openings for doors and access panels should be the greater of that required for the external facing or to achieve the required FRL.



Construction
A Linings min 1 x 16 mm fire-protective grade plasterboard (corridor/occupancy side)
B Massive timber min 75 mm thick ***(note greater thickness may be required to achieve required FRL unless additional fire protective coverings applied)***
C Linings min 1 x 13 mm fire-protective grade plasterboard (internal shaft side)

Performance
FRL 90/90/90 obtain Evidence of Suitability from supplier
MRISF 30 mins (1 x 16 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS
MRISF 20 mins (1 x 13 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS

Figure 7.30: Typical stair and lift shaft construction for single skin massive timber panel construction.

Although a minimum panel thickness of 75 mm is permitted, in most instances substantially greater thicknesses will be required as part of the structural design and/or to achieve the required FRLs. The FRL should be checked to ensure the load levels during the test were comparable to the loads that will be applied under fire conditions.

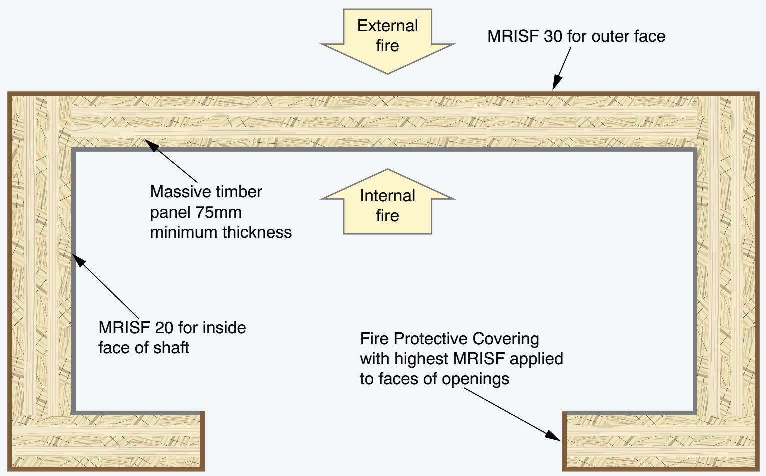


Figure 7.31: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

7.10 Construction for Stairways within Fire-isolated Stairs

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway provided the timber used:

- has a finished thickness of not less than 44 mm with an average timber density of not less than 800 kg/m^3 (at 12% moisture content).
- the building is protected throughout by a sprinkler system complying with Specification E1.5 (other than a FPAA101D or FPAA101H system) that is extended to provide coverage within the fire-isolated enclosure
- the underside of flights of stairs directly above landings providing access to ground level or car parking levels being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres (Figure 7.32).

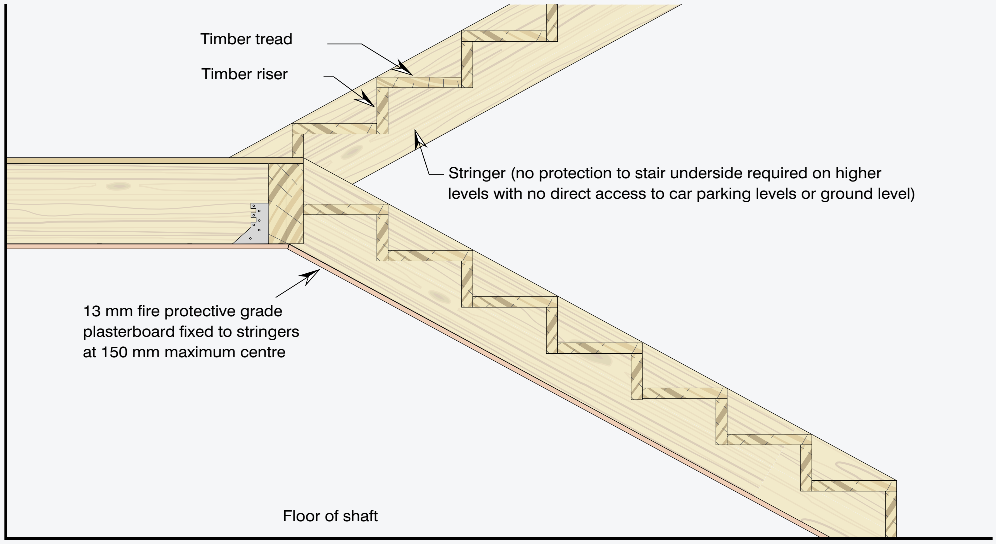


Figure 7.32: Stairway fire protection.

Impact sound from stair usage may vibrate the stair shaft walls, creating a pathway for sound transmission. A practical way to prevent this is by isolating the support for the stair structure by using stringers to support the stairs (top and bottom) rather than the wall adjoining areas requiring sound isolation (see Figure 7.33). In some instances, newel posts to support the stringers may be necessary.

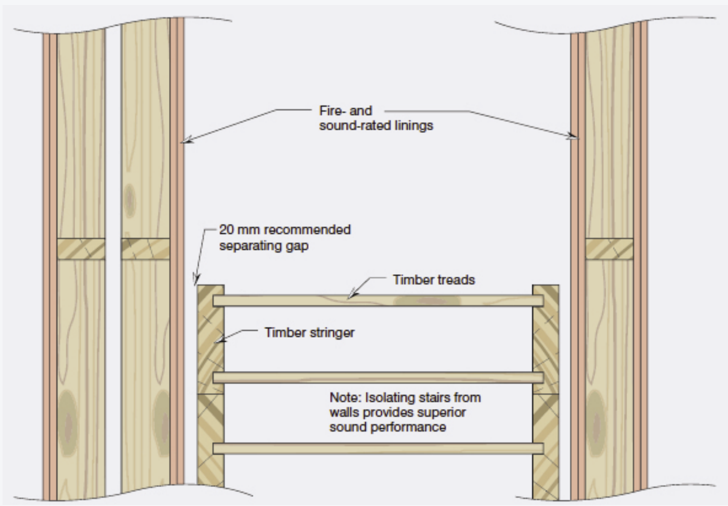


Figure 7.33: Sound isolation of stairway.

If a non-combustible stair (e.g. steel) is installed within the fire-protective timber stair shaft, the sprinkler system does not require extending to provide coverage in a fire-isolated stair.

7.11 Construction of Lift Shafts

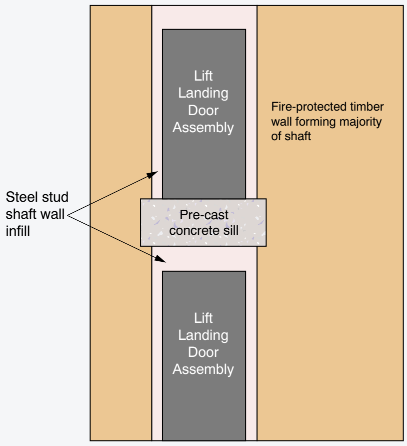
Lift shafts can be constructed in a similar manner to stair shafts as described above. Care is needed to ensure that the lift shaft is compatible with the selected lift system. Compatibility issues should be resolved early in the design process and early liaison with the lift supplier is strongly recommended.

In the short term, most lift landing door assemblies will have been fire tested in masonry/concrete or steel stud shaft wall systems. The following details provide an interface between fire-protected timber and a pre-cast concrete sill and steel shaft wall systems. This can enable lift doors to be installed within sections of the wall of steel stud /plasterboard shaft wall and concrete construction to which existing lift landing door fire-resistance test results can be applied.

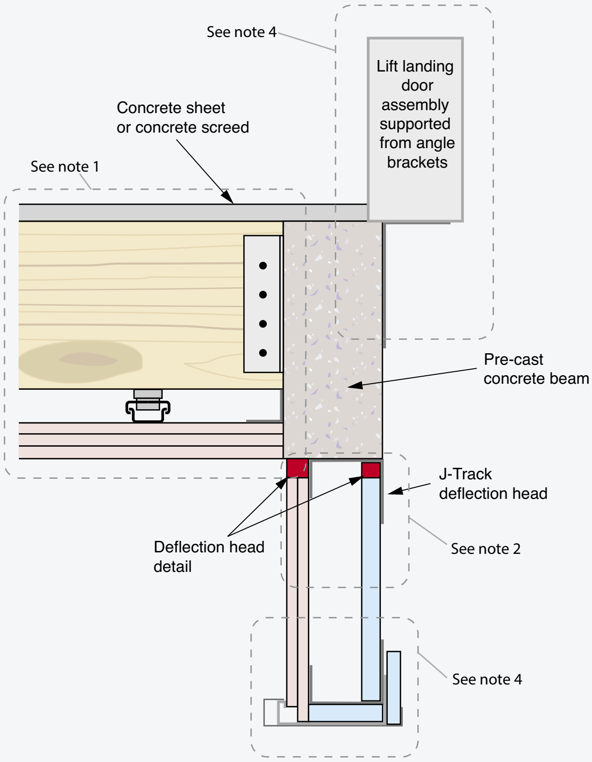
In the longer term, a larger range of lift landing doors is expected to be fire tested in fire-protected timber construction, providing simpler installation details.

The interface details in Figure 7.34 have been assessed by an Accredited Testing Laboratory (refer report RIR 37401400). The applicability of the Evidence of Suitability to an application should be checked with the authority having jurisdiction.

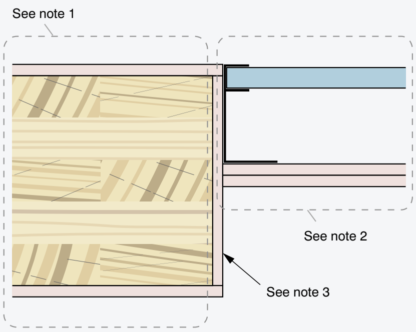
Impact sound from lift use may vibrate the lift shaft walls, creating a pathway for sound transmission. While this can be addressed to some extent using double stud wall assemblies or twin-skin massive timber panel construction utilising two layers of 13 mm plasterboard, there are other options, such as the construction of a framework within the lift shaft that supports the lift assembly independently.



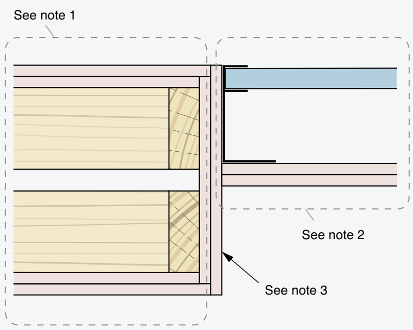
Elevation of lift doors in shaft



Head and sill detail for interfaces with shaft wall and concrete sill (timber-frame)



Side interface between shaft-wall and double timber stud shaft



Side interface between shaft-wall and massive timber panel

Figure 7.34: Typical details for shaft wall conversion for lift-landing door installation.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Lift landing door assembly having the required FRL installed in accordance with the lift door and shaft wall supplier instructions and evidence of compliance confirming the FRL.

7.12 Cavity Barriers and Junction Details

Cavity barriers are required at the junctions between fire-protected timber floor assemblies and fire-protected timber walls in framed construction. In many instances the Deemed-to-Satisfy solutions permitting the use of solid timber and/or mineral fibre enable integration of cavity barriers with typical wall and floor junction details.

7.12.1 Typical Junction Details at Intersection of Fire-Protected Timber Walls and Floors

Cavity barriers are required at the junctions between fire-protected timber floor assemblies and fire-protected timber walls in framed construction. In many instances the Deemed-to-Satisfy Solutions permitting the use of solid timber and/or mineral fibre enable integration of cavity barriers with typical wall and floor junction details.

The key design parameters are to achieve, as a minimum, the required seal thickness in the direction of potential fire spread through the cavity and ensure the seals are continuous.

Typical details for double stud walls and external walls are shown in Figure 7.35 to Figure 7.38. These details are based on a 'ring beam' design concept which can be useful in the management of the risk of disproportionate collapse. This form of construction is also compatible with the prefabrication of floor cassettes. Prefabrication can provide several advantages including:

- acceleration of the construction program
- improved quality control
- improved safety.

Although the mineral fibre cavity barrier is only required to be 45 mm or 60 mm thick (depending on the required FRL of the elements) in the potential direction of fire spread, where practical, installation of cavity barriers the full floor depth provides a more robust solution since any joins in the ring beam/blocks will also be backed by the mineral fibre.

For single stud internal walls, the detail is simplified because the top and bottom plates of the wall frame close off the cavities within the wall as shown in Figure 7.38.

Massive timber panel designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Figure 7.35 to Figure 7.38 include typical examples of joint seals to allow for movement and maintain acoustic and fire separations. The joint sealing details may vary, depending on the installation order of wall and ceiling fire-protective grade coverings, among other things. Refer to the plasterboard and/or sealant suppliers for Evidence of Suitability.

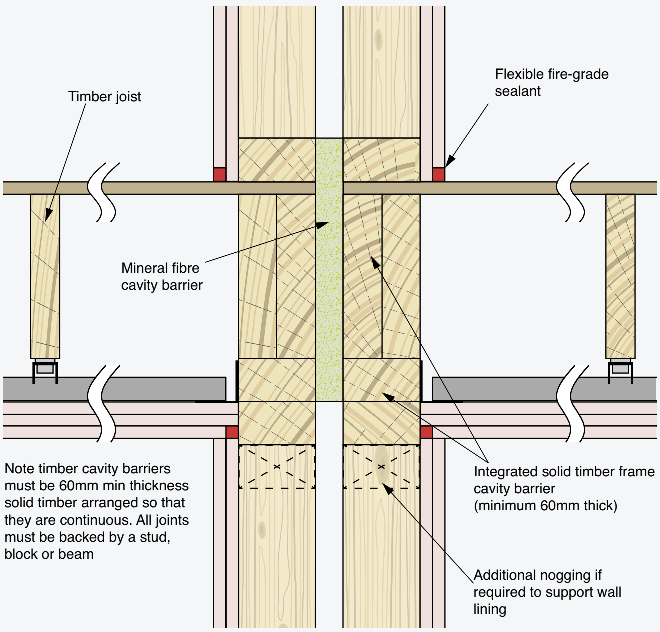


Figure 7.35: FRL 120/120/120 Fire-protected timber frame wall/floor junction with integral cavity barriers – beams parallel to wall

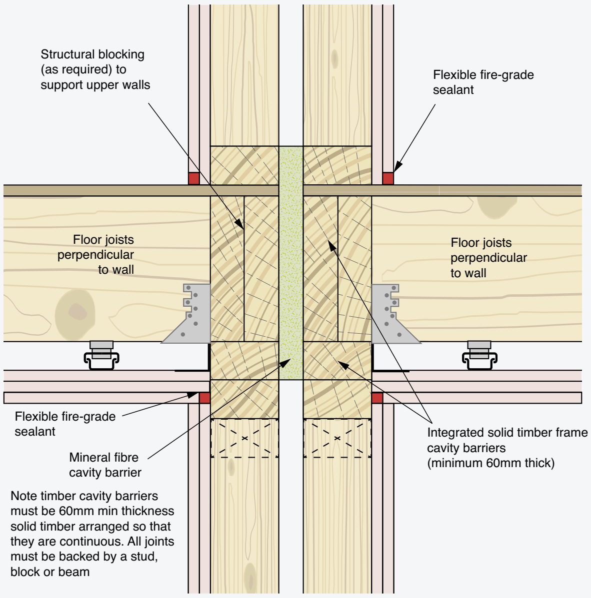
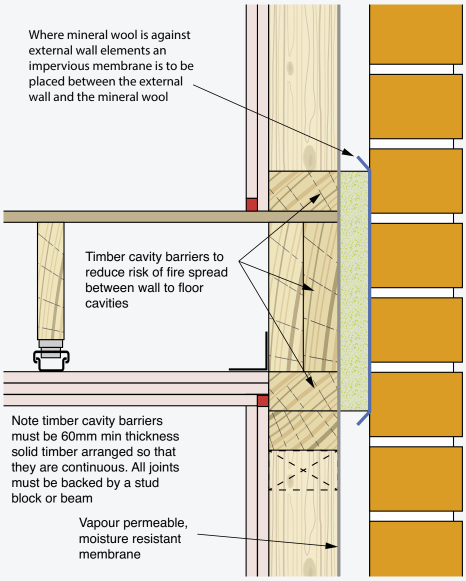


Figure 7.36: FRL 120/120/120 Fire-protected timber frame wall /floor junction with integral cavity barriers – beams perpendicular to wall.



Performance can vary dependant on stud frame depth, installed insulation, cavity insulation and masonry veneer material and thickness. Evidence of suitability must be obtained to demonstrate compliance with the required FRL.

Figure 7.37: FRL 120/120/120 Fire-protected timber frame wall/floor junction with integral cavity barriers – beams parallel to wall.

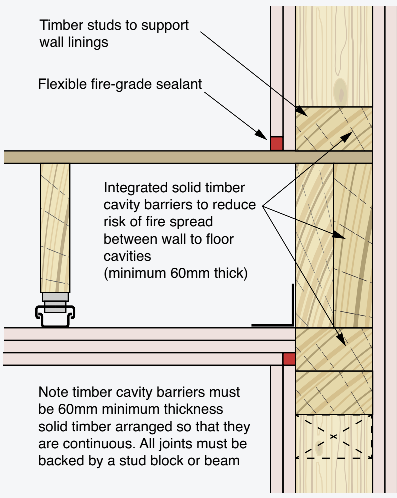


Figure 7.38: FRL 120/120/120 Fire-protected single stud timber frame wall/floor junction with integral cavity barriers – beams parallel to wall.

7.12.2 Vertical Cavity Barriers

Vertical cavity barriers are required at the intersection of walls and at 10 metres maximum horizontal centres. Typical details for double stud walls and external walls are shown in Figure 7.39 to Figure 7.42.

For double stud walls separate cavity barriers can be provided for each skin as shown in Figure 7.40 but in most instances a more practical solution is to fit a wider section spanning the full width of the intersecting wall, as shown in Figure 7.39 and Figure 7.41.

Massive timber designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Where external cladding or veneer systems form part of the fire-protective coverings (e.g. brick veneer) at cavity barrier positions an impervious membrane must be placed between the mineral fibre and cladding or veneer surface to control moisture transfer from the cladding or veneer.

An alternative approach for external walls that may avoid the risk of bridging at cavity barrier positions is to apply the fire-protective coverings to the outer face of the timber elements as well as the inner face and then fit a non-combustible external cladding system that satisfies the NCC DTS requirements.

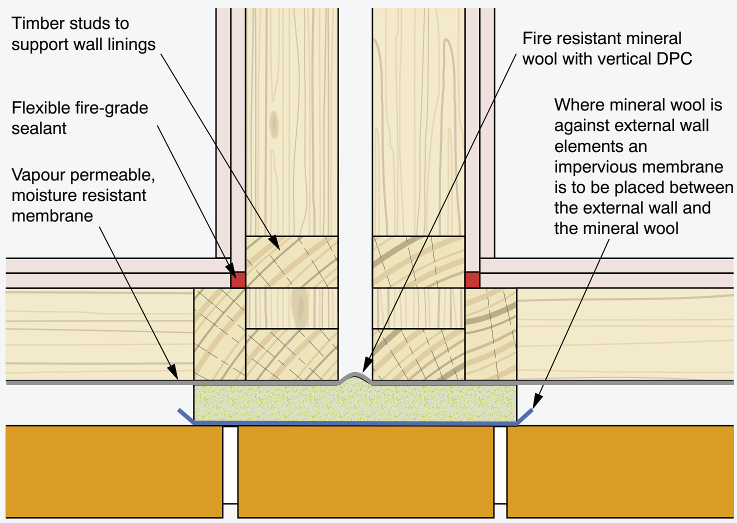


Figure 7.39: Double stud fire-protected timber internal wall intersecting a brick veneer wall.

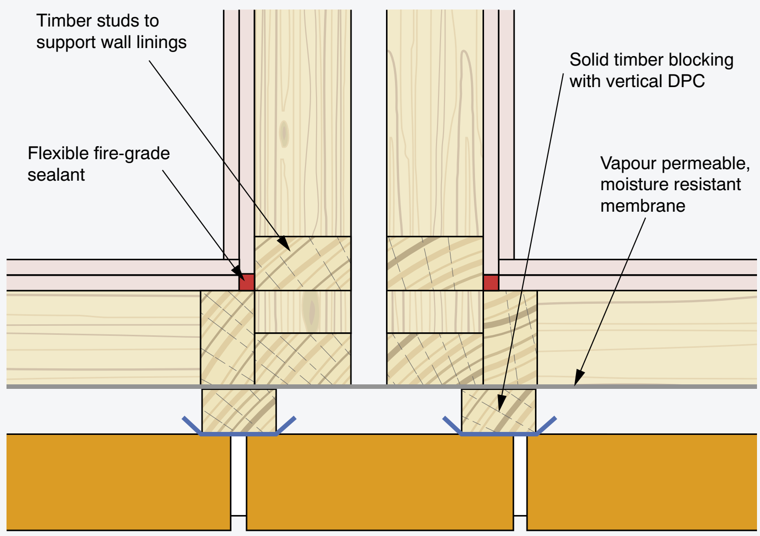


Figure 7.40: Double stud fire-protected timber internal wall intersecting a brick veneer wall with split cavity barrier system.

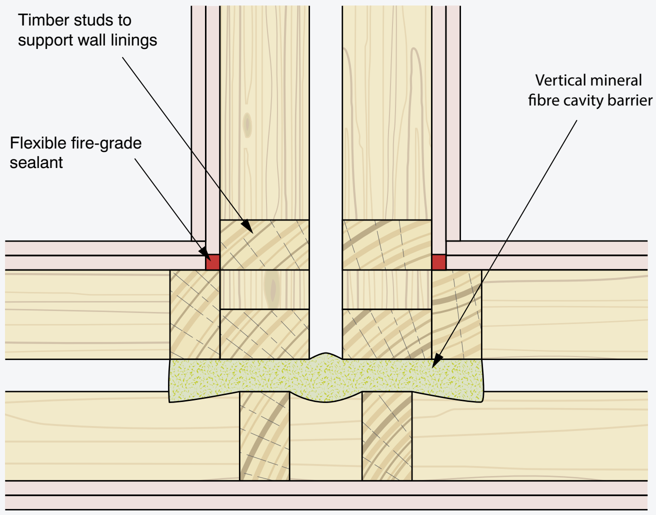


Figure 7.41: Double stud fire-protected timber internal wall intersection.

Provided the timber studs are a minimum of 45 mm thick, intermediate cavity barriers (at maximum 10 metres centres) can be fitted at a stud position as shown in Figure 7.42 if FRLs for the elements no greater than 90/90/90 are required. In elements with higher FRLs (as required for Class 9 buildings) the timber thickness must be increased to a minimum of 60 mm, which can be achieved by nailing 2 to 35 mm studs together (70 mm total thickness)

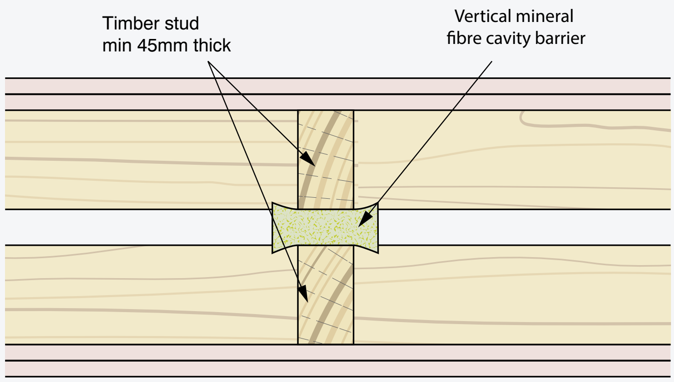


Figure 7.42: Intermediate vertical cavity barrier in double stud wall. Cavity barrier stud thickness needs to be increased to 60 mm if this detail is to be applied for FRLs greater than 90/90/90.

7.12.3 Unprotected Openings in External Walls

Cavity barriers are required around the perimeter of openings, such as unprotected windows in external walls, to prevent premature entry into the fire-protected timber cavities at these positions.

A typical example is shown in Figure 7.43 for an external wall. Timber and mineral fibre thicknesses to be selected based on required FRL as specified in NCC Spec C1.13 Table 1.

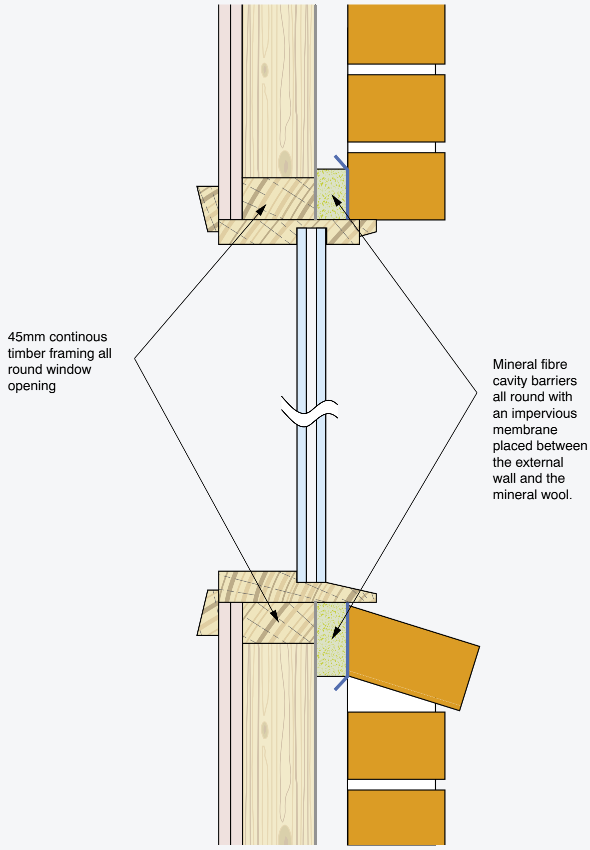


Figure 7.43: Cavity barrier around window in external wall. (Minimum thickness of timber and mineral fibre must be in accordance with NCC Spec C1.13 Table 1).

7.12.4 Intersection of Non-fire-resisting Walls with Fire-protected Timber Elements

Fire-protective coverings of fire-protected timber elements should not be interrupted at the point of intersection with non-fire-resisting walls to ensure the FRLs and RISF or MRISF are not compromised.

Typical examples are shown in Figure 7.44 and Figure 7.45. Where the non-fire-resisting element is fixed to the fire-protected element, additional framing may be required to avoid the risk of failure of the non-fire-resisting element compromising that of the fire-protected element. A typical detail for additional framing is shown in Figure 7.45.

With massive timber, the fixing point is less likely to require additional strengthening.

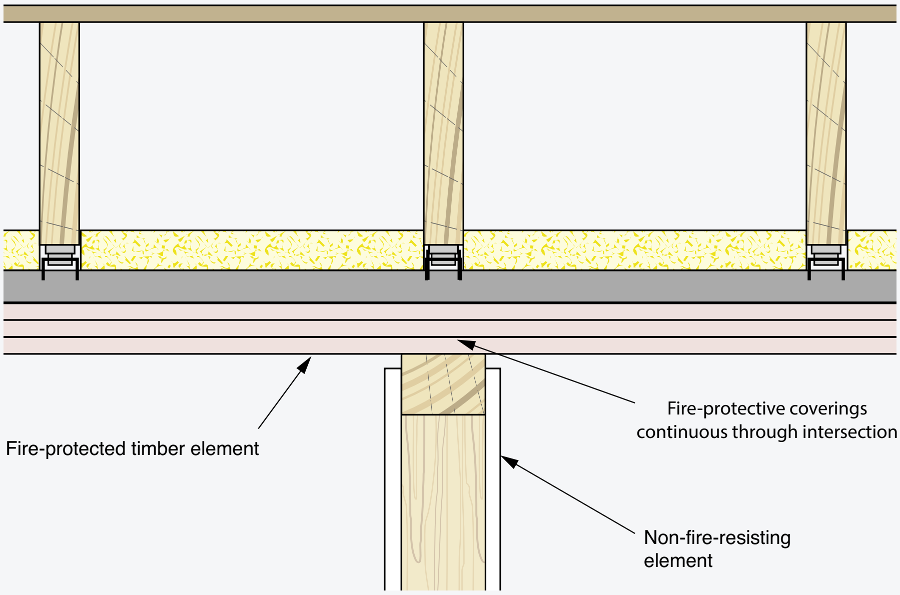


Figure 7.44: Junction of non-fire-resistant wall and fire-protected timber floor

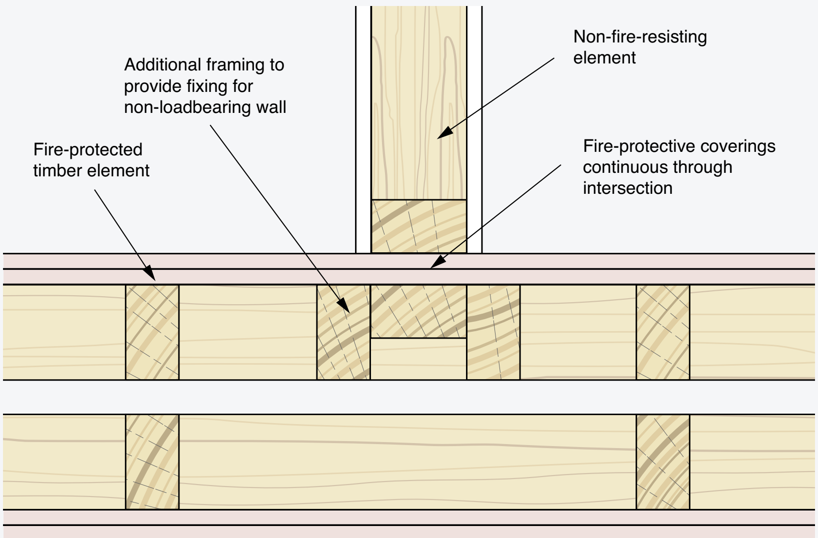


Figure 7.45: Junction of non-fire-resistant wall with fire-protected timber wall including additional framing detail.

7.12.5 Roof Space Cavity Barriers or Fire-protected Timber Wall Extension

Special attention needs to be given to the design of roof spaces to address the risk of uncontrolled fire spread if timber-frame construction is adopted, although for many applications massive timber construction may be preferred. There are generally two approaches that can be adopted for timber-frame construction.

Option 1: Extend fire-resisting timber walls to roof level

This option requires fire-protected timber walls to be continued to roof level (Figure 7.46). This has the advantage that the ceilings to the top floor do not need to be fire-resisting because the wall extension can provide the necessary fire and sound separation.

It is critical that the seal against the underside of the roof can achieve the required FRL, RISF or MRISF and that the fire separation is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect extension of the SOU boundary walls.

If this option is adopted a horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 7.46.

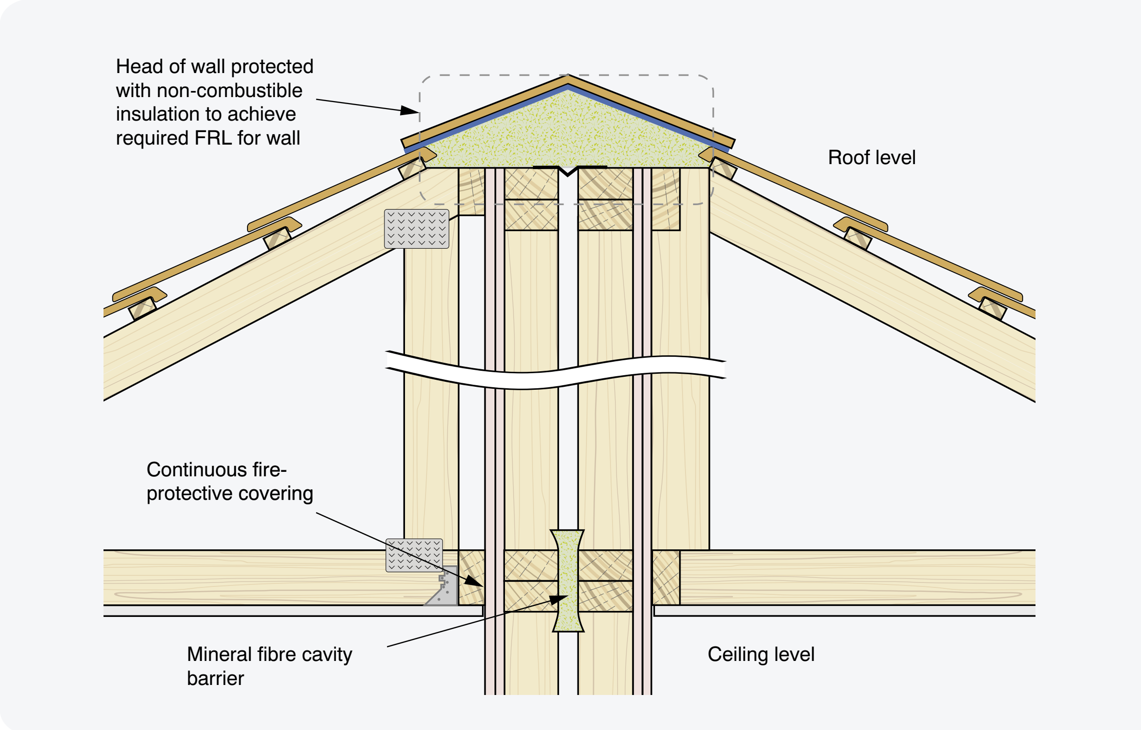


Figure 7.46: Roof space Option 1 extending SOU bounding fire-protected timber walls to roof level.

Option 2: Provide fire-protected timber ceiling and cavity barriers within roof space

If Option 2 is applied to the Class 9 example building, assuming timber-framed construction, the ceiling/roof system would require an FRL of 120/60/30 and a RISF of 45 minutes. The roof spaces would need to be divided by cavity barriers above each wall required to be fire resisting. Where the roof void is relatively deep it may be impractical to apply the Deemed-to-Satisfy Solutions of solid timber or mineral fibre and a plasterboard partition achieving the required FRL of −/60/60 minutes or a proprietary cavity barrier may provide a more practical solution as shown in Figure 7.47.

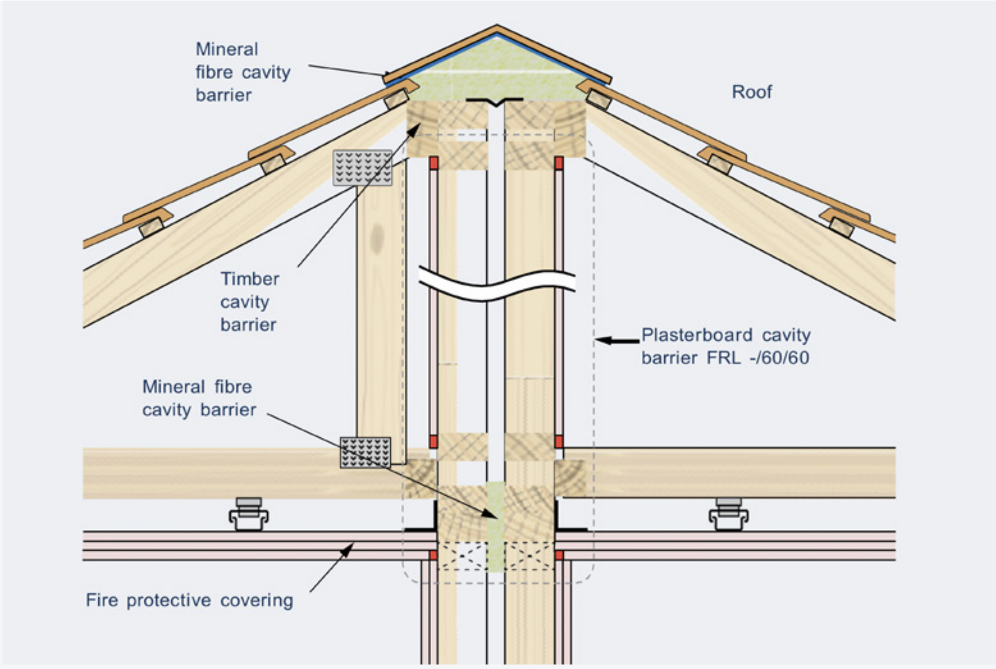


Figure 7.47: Roof space Option 2 fire-protected timber ceiling and cavity barriers within roof space.

If this option is selected it is critical that the seal against the underside of the roof is capable of achieving the required performance and the fire separation provided by the cavity barrier is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect at the extension of the fire-resistant wall.

A horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 7.47. Depending on the roof design the roof cavity height can vary from nominally 150 mm to several metres and careful consideration should be given to detailing and checking installations to ensure the design objectives are achieved.

7.13 Service Penetration Treatments

Careful detailing of services and service penetration systems during the design stages can simplify construction details and streamline the construction process as described in early chapters of this Guide. The general design approach can be expressed as three fundamental principles:

1. Select services, service locations and service runs to avoid, as far as practical, the need for service penetrations through fire-protected timber elements (e.g. the use of false walls and ceilings can substantially reduce the number of penetrations that require protecting).
2. If service penetrations cannot be avoided, where practical they should be grouped and penetrate lined openings or non-combustible shaft walls, which minimises the risk of exposing the cavity during maintenance operations. This approach also simplifies the installation of new services.
3. If service penetrations are required to pass through fire-protected timber elements, ensure the FRL and RISF or MRISF as appropriate at service penetration positions.

The following Sections provide typical generic examples. Over time, it is expected that proprietary systems will become available simplifying the installation process.

7.13.1 Multi-penetration Systems with Lined Openings

Refer report RIR 37401400 available from WoodSolutions website

Typical multi-penetration systems with lined openings are shown in Figure 7.48.

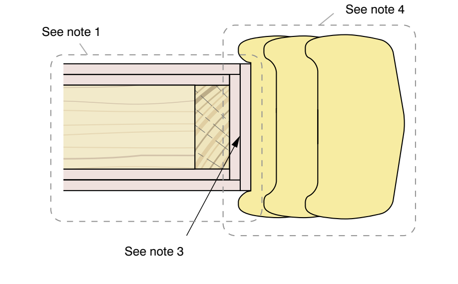
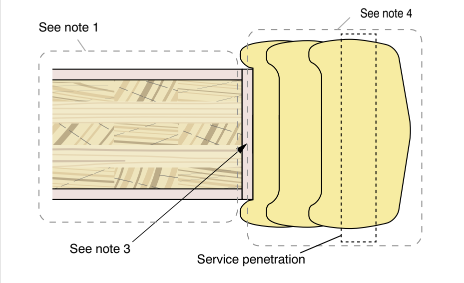
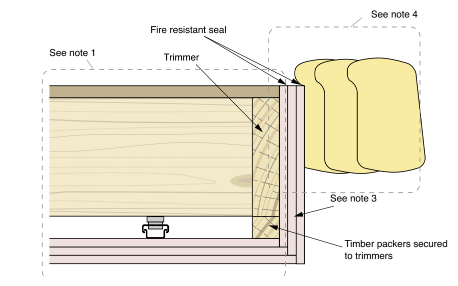
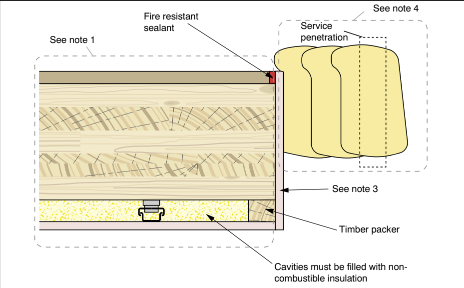
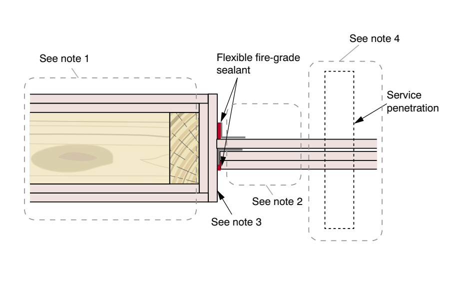
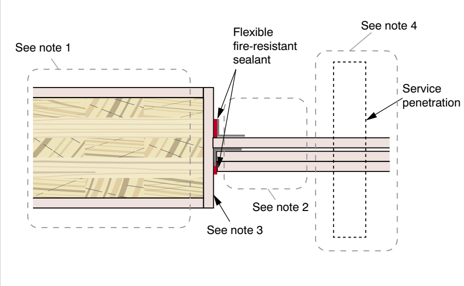
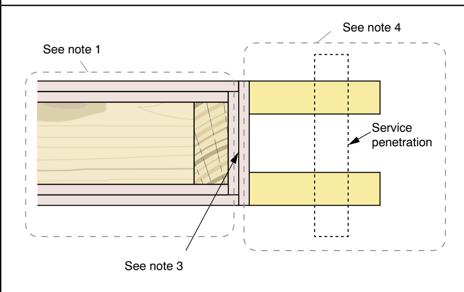
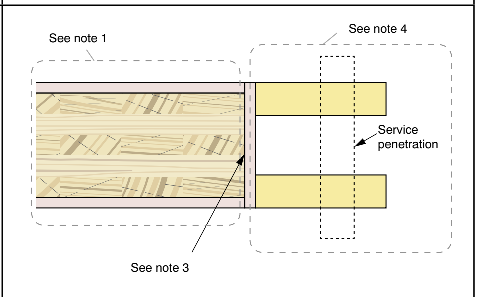
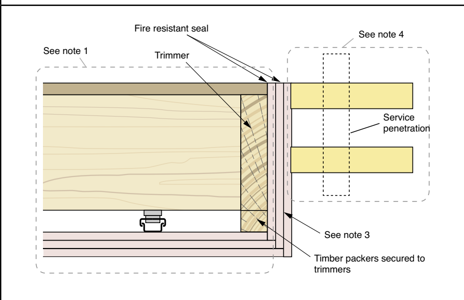
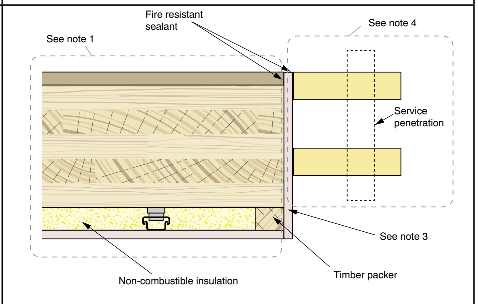
System	Timber Frame	Massive Timber Panel
Pillow in wall opening		
Pillow in floor opening		
Shaft-wall infill in wall opening		
Mineral fibre batt wall opening		
Mineral fibre batt floor opening		

Figure 7.48: Typical multi-penetration systems with lined openings.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Interface details shown in Figure 7.48 have been assessed by an Accredited Testing Laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber frame systems and 30 minutes for the massive timber panel systems (Refer Report RIR 37401400). Other details may be adopted provided appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements is provided.

Refer report RIR 37401400 available from WoodSolutions website

7.13.2 Fire Damper and Duct Penetrations

The lined opening approach can also be applied to duct and damper penetrations (Figure 7.49).

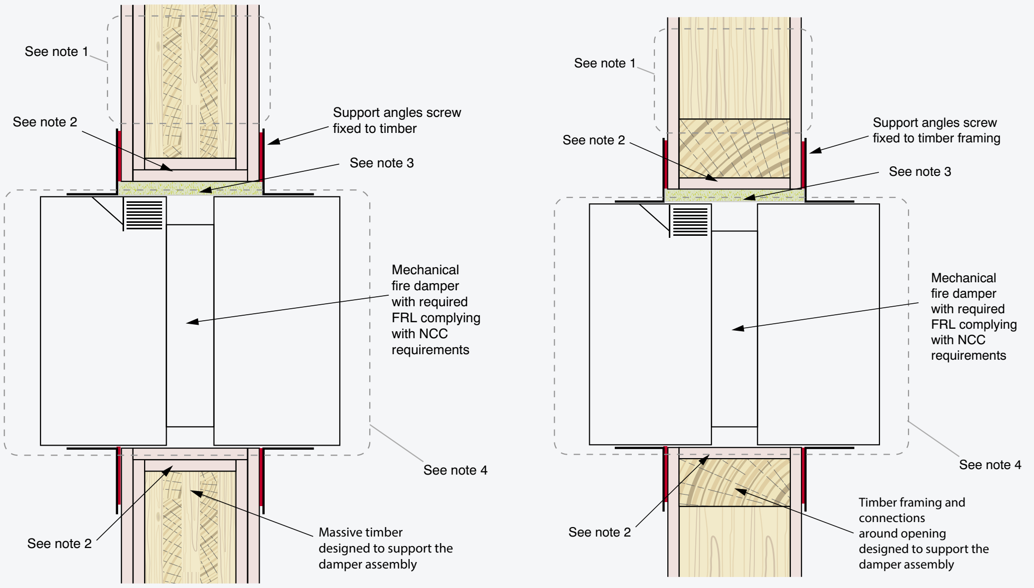


Figure 7.49: Typical details for fire damper and duct penetrations.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 3: Non-combustible mineral fibre packing may be used for fire damper penetration seal or proprietary fire damper penetration seals that achieve the required FRL with evidence of performance in the form of a report from an Accredited Testing Laboratory to be in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 4: Mechanical fire damper having the required FRL when tested in accordance with AS 1530.4 and complying with AS 1682 Parts 1 and 2 as appropriate.

7.13.3 GPO Outlets and Switches

Where practical, the need to protect GPO outlets, switches and similar penetrations should be avoided by mounting them within internal (non-fire-resisting walls) or false (decorative) linings fitted in front of fire-protected timber elements as shown in Figure 7.50.

Methods of attaching non-fire-resisting decorative linings that will not compromise the FRL, RISF or MRISF performance of wall and floor systems, such as shown in Figure 7.50, have been assessed in a report from an Accredited Testing Laboratory (refer RIR 37401400).

If it is impractical to apply an additional lining, a proprietary GPO protection system may be adopted, if it has Evidence of Suitability, demonstrating that the required FRL and RISF or MRISF for the element will not been compromised.

Alternatively, the generic systems shown in Figure 7.51 or Figure 7.52 may be adopted.

New products (e.g. skirting service ducts) also enable services to be run without penetrating fire-protective grade linings.

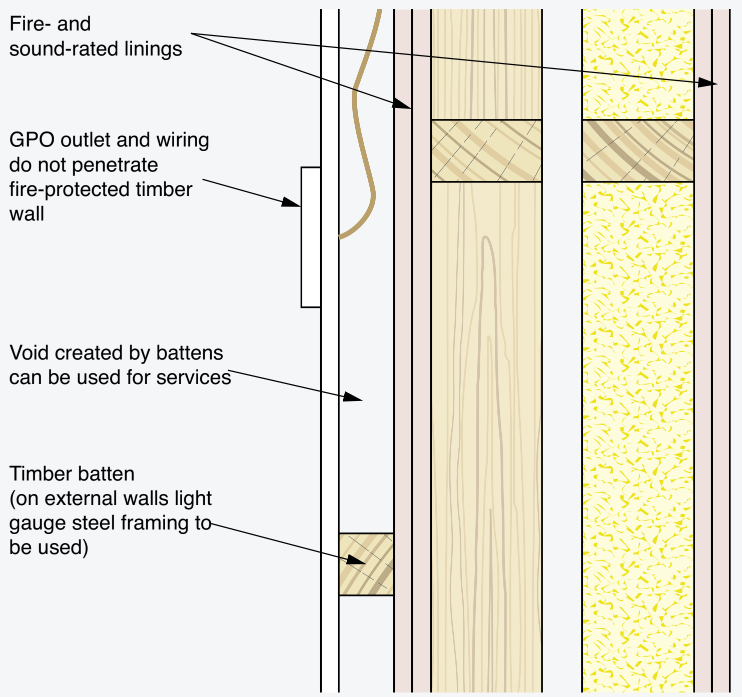


Figure 7.50: False wall system.

Refer report RIR 37401400 available from WoodSolutions website

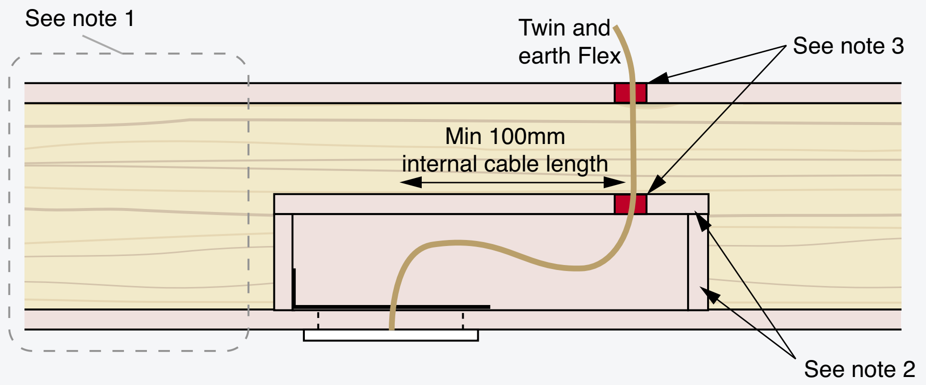


Figure 7.51: Generic GPO protection systems in massive timber construction.

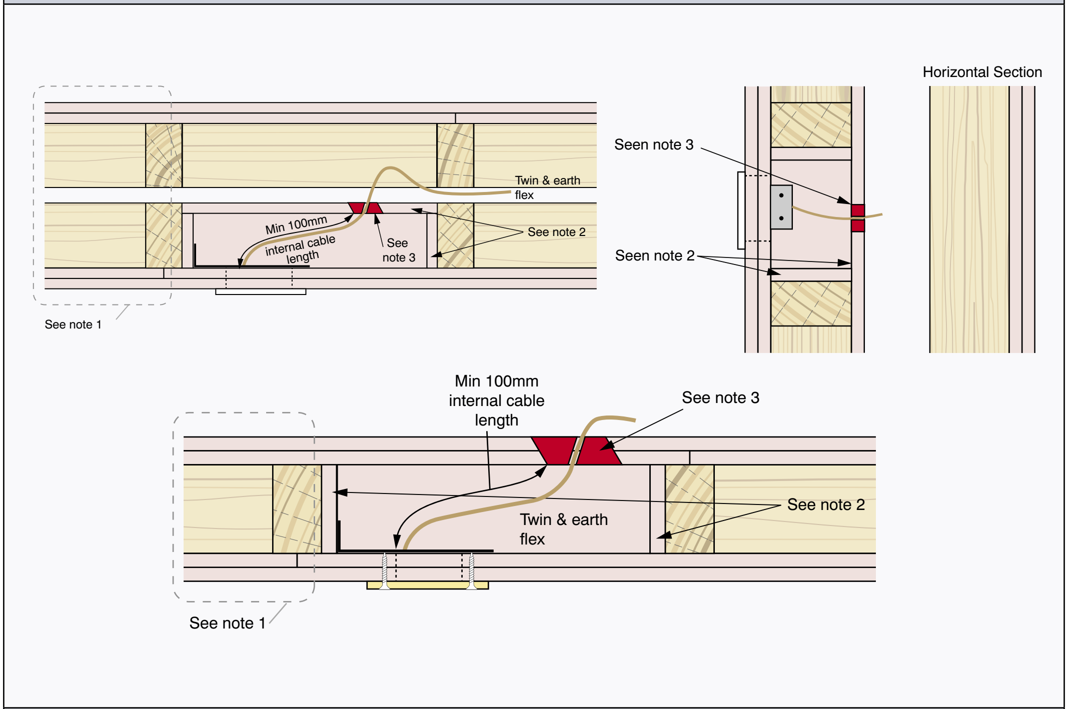
Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Aperture lined with a minimum of one layer of 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic'. The mastic should have a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve

Refer report RIR 37401400 available from WoodSolutions website

Plasterboard lined framed out protection.



Mineral fibre protection

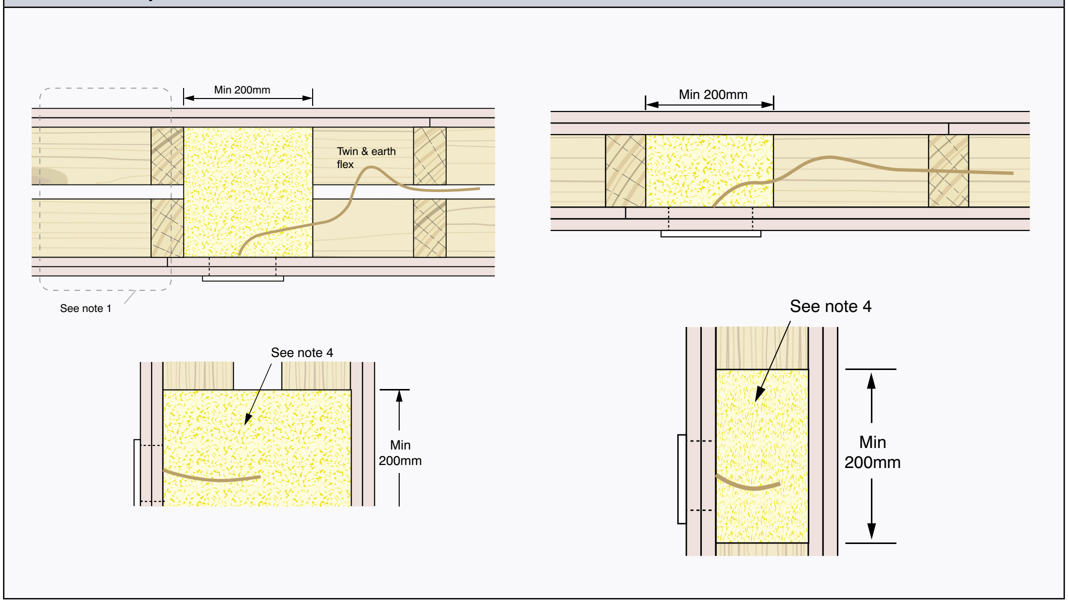


Figure 7.52: Generic GPO protection systems in timber-framed construction.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Aperture lined with a minimum of one layer of 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic'. The mastic should have evidence of performance in the form of a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve an FRL of –/60/–.

Note 4: Cavity filled full depth with mineral fibre of minimum density 60 kg/m3 for at least 100 mm to the sides and above and below the centreline of the GPO.

Note: Fire tested proprietary systems in which the RISF or MRISF performance have been determined may provide more practical options; subject to adequate Evidence of Suitability being available.

Refer report RIR 37401400 available from WoodSolutions website

7.13.4 Single Cable and Metal Pipe Penetrations

If it is impractical to apply an additional lining, a proprietary system may be adopted, if it has Evidence of Suitability, demonstrating that the required FRL and RISF or MRISF for the element will not been compromised.

Alternatively, the generic systems shown in Figures 7.53 and 7.55 may be adopted.

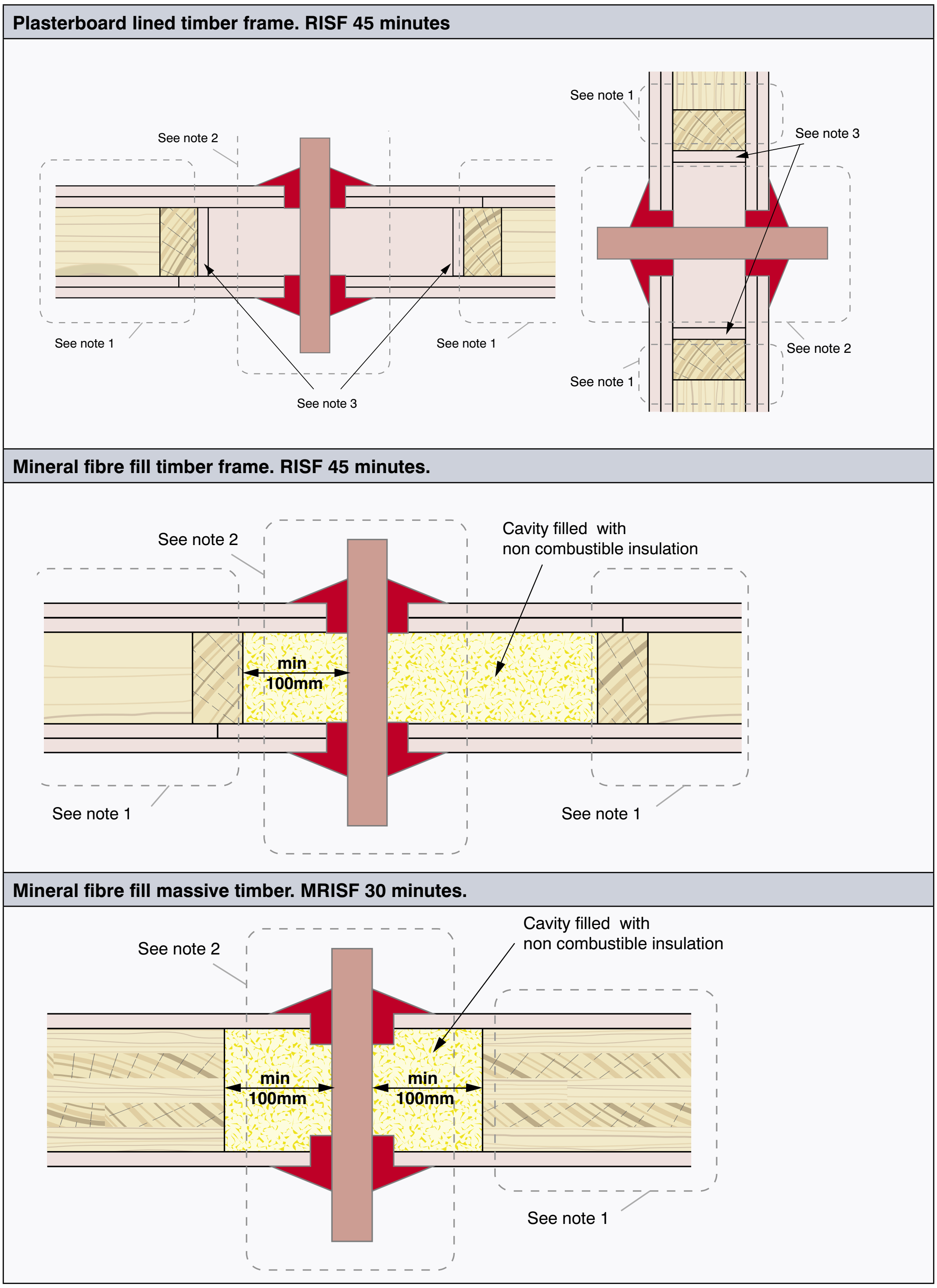


Figure 7.53: Pipe and cable penetrations through fire-protected timber

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Refer report FAS 190034, available from the WoodSolutions website, for assessment of ceiling lining detail.

The preferred option for lighting cables, sprinkler pipe penetrations and the like is to run them through the cavity above a false ceiling. A typical false ceiling detail is shown in Figure 7.54. Larger cavities can be provided above false ceilings by using suspended ceiling fixings to accommodate down lights and larger services.

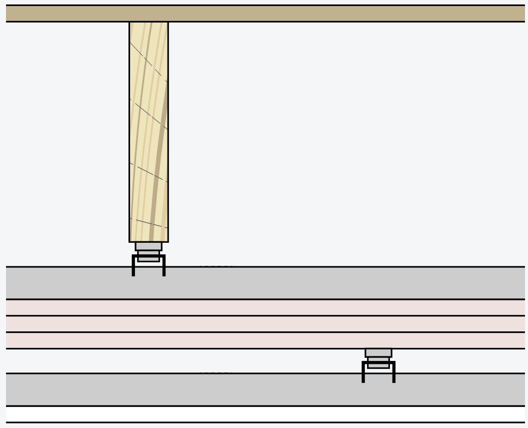


Figure 7.54: False ceiling detail for minimising service penetrations through ceiling systems.

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of back blocking system

If it is impractical to provide a false ceiling a solution for lighting cable penetrations through fire-protected timber ceilings is to use cover blocks as shown in Figure 7.55. Proprietary systems may be available to protect down-light penetrations and sprinkler pipe penetrations but access for the long-term service and maintenance of these systems and options for reconfiguration would be very limited.

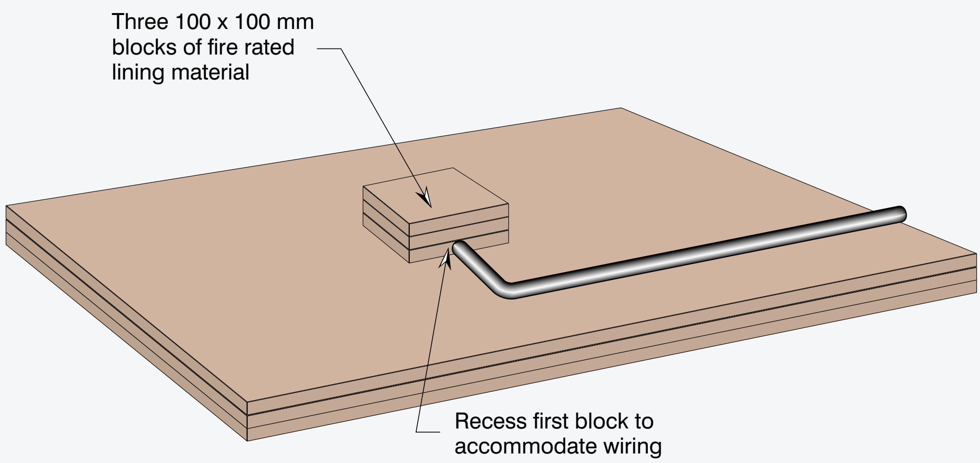


Figure 7.55: Recess block protection system for lighting cables penetrating fire-protected timber floors.

7.13.5 Rebated Ceiling Details for Housing Services

Another alternative for ceiling systems is to create a rebate to house services without penetrating a fire-protected element such as a fire-protected timber floor/ceiling system as shown in Figure 7.56. This detail has been assessed by an Accredited Testing Laboratory as achieving an FRL 120/120/120 and a RISF greater than 45 minutes. Care should be taken not to attach the rebate framing members to the floor structure to avoid short-circuiting the sound separation.

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of rebated ceiling system

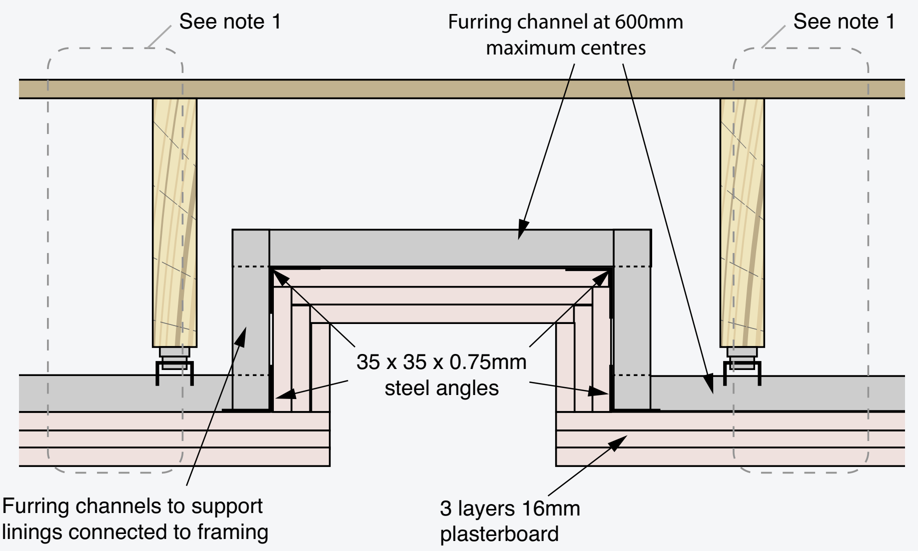


Figure 7.56: Rebated ceiling system.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

The rebate may be fitted with a grill, a section of false ceiling or may be sized to mount individual items of equipment.

7.13.6 Plastic Pipe Penetrations

Where it is impractical to adopt false wall and ceiling linings or use non-combustible shaft construction or lined opening multi-penetration systems, the following details, shown in Figure 7.57 to Figure 7.59, have been developed to maintain a RISF of 45 minutes or a MRISF of 30 minutes of the wall. The systems must have achieved the required FRL when fitted in plasterboard partitions to protect individual plastic pipe penetrations. The following notes apply:

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of Suitability to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of details to maintain the wall RISF or MRISF

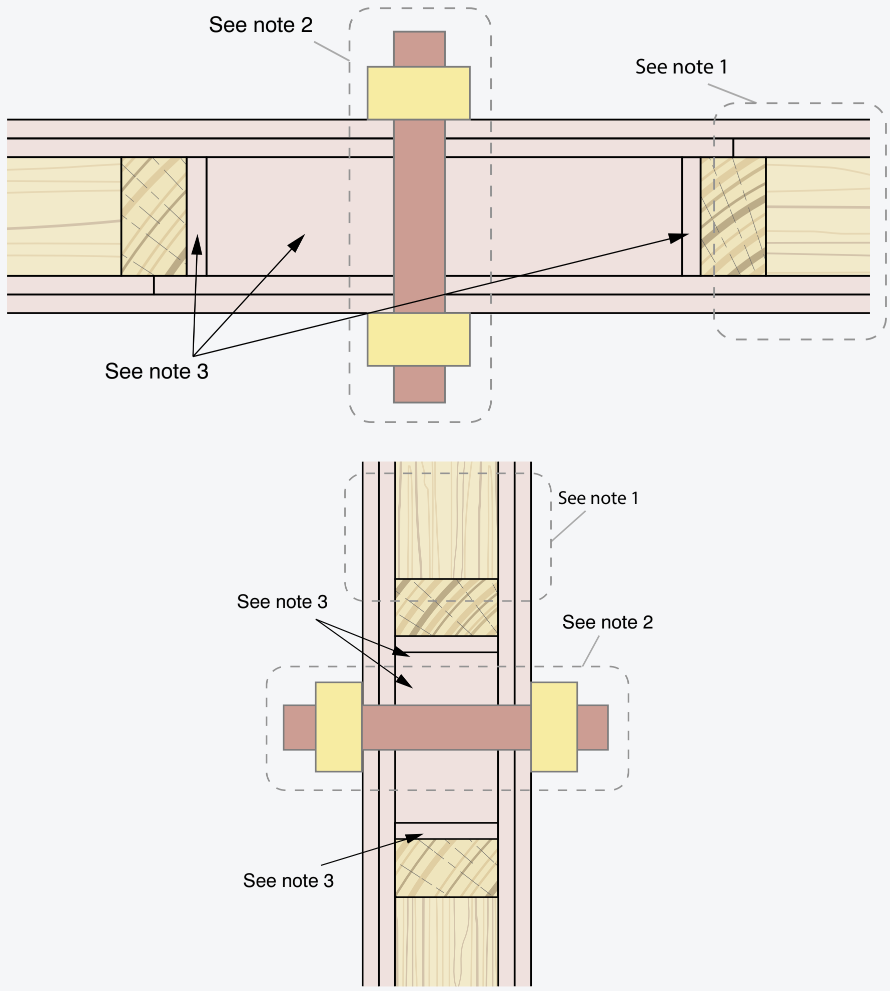


Figure 7.57: Plastic pipe penetration through fire-protected timber-framed walls with internal linings.

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of details to maintain the wall RISF or MRISF

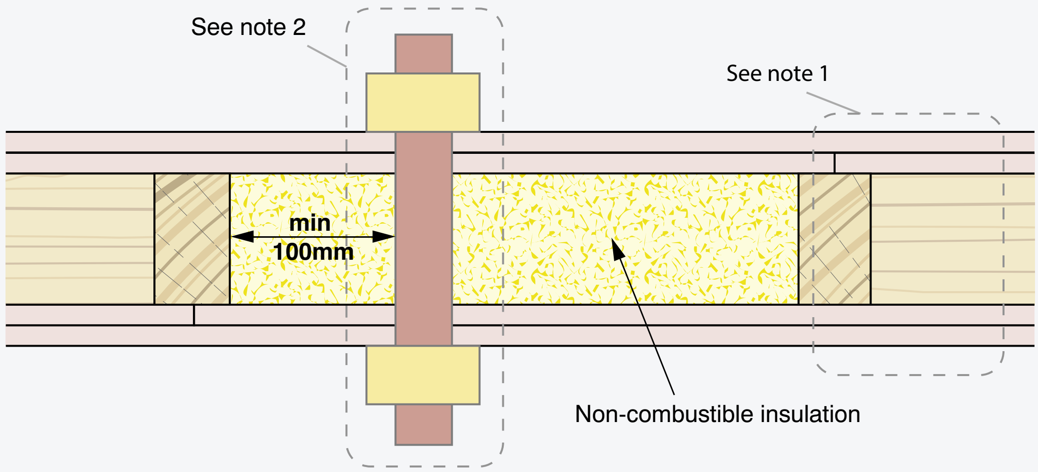


Figure 7.58: Plastic pipe penetration through fire-protected timber-framed walls with non-combustible mineral fibre insulation.

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of details to maintain the wall RISF or MRISF

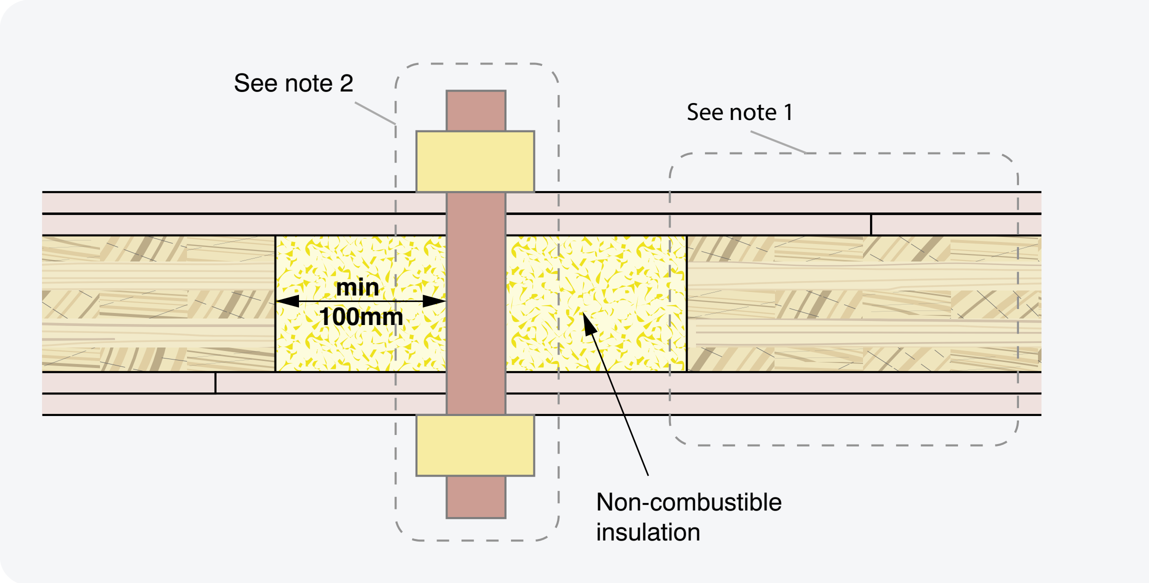


Figure 7.59: Plastic pipe penetration through fire-protected massive timber walls with non-combustible mineral fibre insulation.

Evidence of Suitability required from supplier to confirm required RISF and MRISF performance of penetrated elements is maintained in addition to the FRL for the system

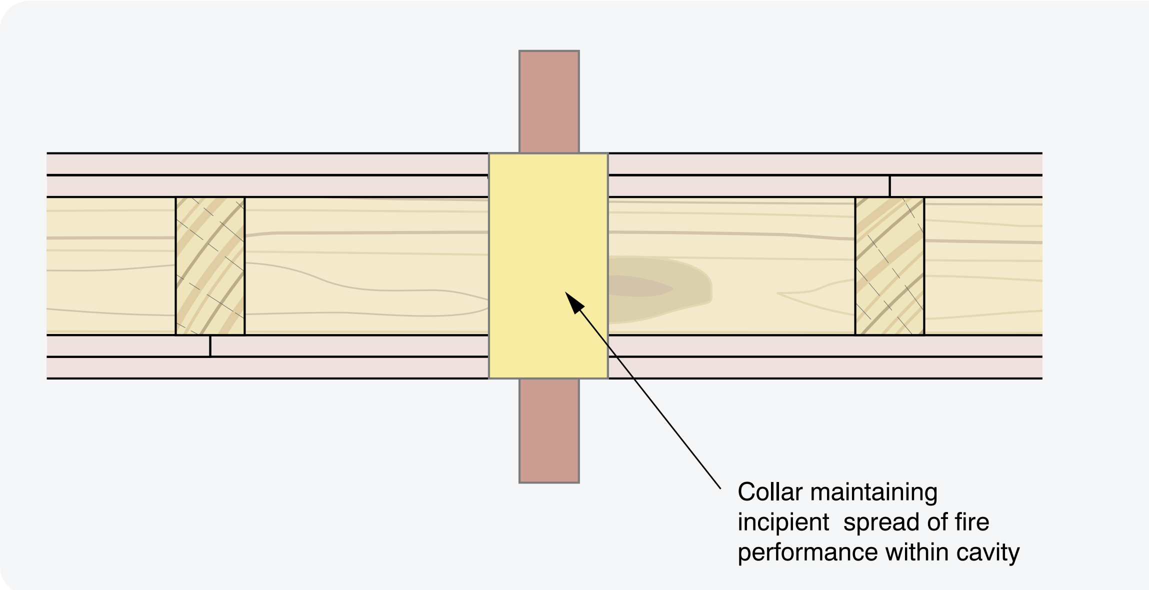


Figure 7.60: Option for a proprietary system with integral insulation protecting a plastic pipe penetration

Refer report RIR 37401400, available from the WoodSolutions website, for assessment of interface details to maintain the RISF and MRISF performance of elements penetrated by access panels as shown in the following figures.

7.13.7 Access Panels

Access panels may be used to protect openings providing access to a floor/ceiling cavity as shown in Figure 7.61 or to shafts through fire-protected timber walls as shown in Figure 7.62 and Figure 7.63.

Providing access panels will tend to compromise the sound separation and therefore they should normally be located in areas that are not sound 'sensitive'.

The following notes apply to the typical details shown in Figure 7.61 through Figure 7.63.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire-protective coverings that are applied to the fire-protected timber element face.

Note 3: Proprietary access panel system with the required FRL. For access panels providing access to ceiling cavities an RISF rating of 45 minutes or a MRISF rating of 30 minutes as appropriate is also required to be satisfied.

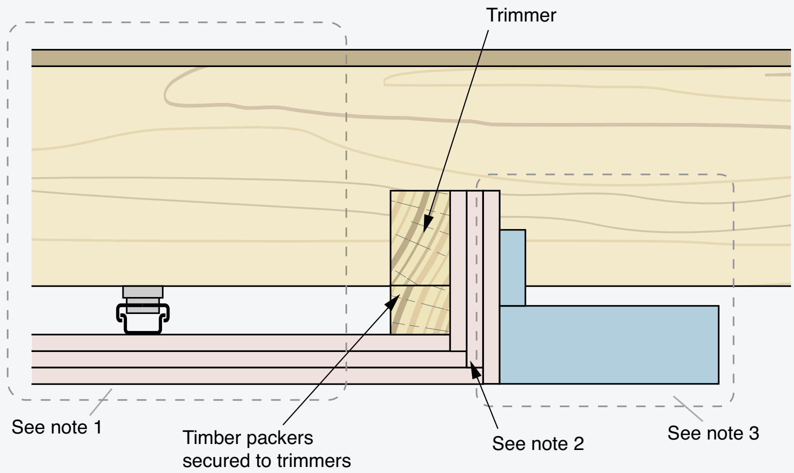


Figure 7.61: Access panel in a fire-protected floor/ceiling system.

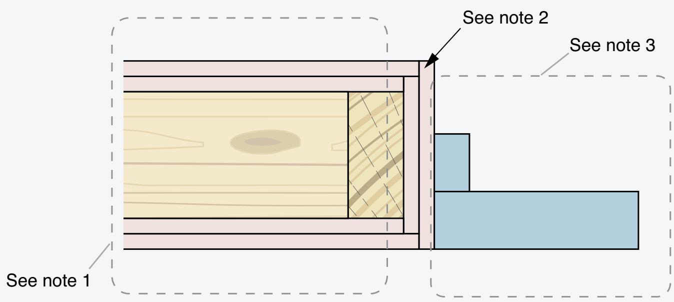


Figure 7.62: Access panel in a fire-protected timber-framed wall.

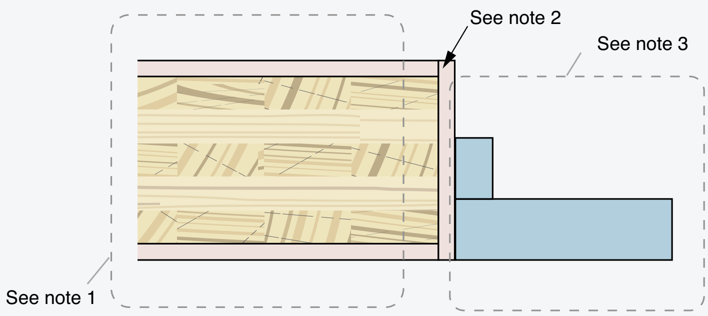


Figure 7.63: Access panel in a fire-protected massive timber wall.

7.14 Automatic Fire Sprinkler Systems

The provision of an automatic fire sprinkler system in accordance with NCC Specification E1.5 (other than a FPAA101D or FPAA101H system) is a mandatory requirement for mid-rise timber buildings if the DTS Solution pathway is adopted.

The automatic fire sprinkler system is a critical component of the fire safety design and must be designed and installed by organisations and/or individuals with appropriate competence. Detailed information about the design of automatic fire sprinkler systems is outside the scope of this Guide; however in common with all services the design needs to be integrated with the architectural, structural and passive fire protection systems. The following sub-sections highlight some key considerations, but it is not an extensive summary.

It is important that the design documentation clearly specifies the requirements for the sprinkler systems such as locations of pipe runs, types of materials and components to be used, treatment of penetrations, types of sprinkler head and positions.

7.14.1 Piping Materials and Connections

Materials for piping and connection details for fire sprinkler systems should be carefully selected to:

- comply with the NCC Specification E1.5 requirements (other than a FPAA101D or FPAA101H system)
- suit the environment
- minimise the time the system is unavailable after maintenance/repair
- minimise hot works on site such as cutting and welding metal pipes
- facilitate the reinstatement of the performance of fire-protected timber at the points of penetration by sprinkler pipes.

While plastic pipes (e.g. CPVC) can largely negate the need for hot works, if alterations are made to plastic pipes the sprinkler system could be unavailable while the adhesive sets. This is an important consideration for buildings that need to remain operational, such as healthcare buildings.

Also, the reinstatement of the performance of fire-protected timber when penetrated by plastic pipes can be more complex than metal pipe penetrations

Metal pipes may be more appropriate for some applications, but they should be pre-prepared so that, as far as practical, all on-site connections can be made without hot works. Fittings can be selected that can be adjusted on site, such as flexible sprinkler fittings minimising the need for hot works.

Once the materials and components have been selected, the pipe runs should be clearly defined to minimise the number of penetrations through fire-protected timber and that, if they cannot be avoided, they occur where the performance of the fire-protected timber can be readily reinstated.

7.14.2 Sprinkler Head Selection

Although not mandatory in AS 2118.1, fast response heads should be used where practicable and appropriate since they respond faster, reducing the risk of occupants and increasing the likelihood that the sprinkler system will suppress the fire.

Sprinkler head options include concealed and semi-recessed. Concealed heads are a common choice in public areas because they can reduce the risk of vandalism and accidental impact. However, the following issues should be considered during the selection process and appropriate mitigation measures adopted:

- Larger cut outs in ceilings are required, which can be addressed by use of a non-rated false ceiling. The false ceiling depth should be designed to allow for the fitting of the concealed heads and related pipework.
- The response time will tend to be slower; this should be checked with the manufacturer.
- Overpainting and use of sealants to retain covers can compromise the performance of a head. This should be addressed through regular inspections.

7.14.3 Monitored Isolation Valves

The reliability of an automatic fire sprinkler system can be enhanced by specifying monitored isolation valves incorporating a check valve and flow switch at each level that is permanently connected to a fire alarm monitoring service provider by a direct data link.

This approach allows the water supply to the sprinkler system on individual floors to be isolated for maintenance or reconfiguration of the system without the need to isolate the whole building. Since the valves are monitored, the risk of the water supply not being reinstated is also significantly reduced.

This arrangement is compatible with the progressive commissioning of automatic fire sprinkler systems during construction, allowing protection of the lower levels while work progresses on the upper levels and individual floors to be easily isolated for adjustments to systems. This may be adopted as part of the fire safety strategy to address fire safety during construction.

7.14.4 Fire-isolated Stairs and Passageways with Timber Stairways

The NCC allows the use of timber stairways in fire-isolated stairs and passageways subject to the automatic fire sprinkler system coverage being extended to cover the fire-isolated stair in addition to other precautions (refer Section 6.10.2 Timber Stairways Concession).

In the absence of other specifications, sprinkler heads should be provided in the following locations:

- at the top of the shaft
- under the landings at each floor level
- under intermediate landings
- providing coverage to other positions where there is a significant risk of accumulation of combustible materials.

7.15 Other NCC Requirements

This is a guide to the use of fire-protected timber for mid-rise timber buildings as a DTS Solution in the NCC. It does not address all NCC requirements that apply to mid-rise buildings nor does it address all NCC fire-related requirements (e.g. fire hazard properties of linings).

Seek advice from appropriately qualified practitioners and relevant regulatory authorities regarding compliance with the NCC for specific projects.

8

Step 6 - Further Design Assistance and Information (Appendices).

9

Appendix A – Determination of Compliance of Fire-protected Timber

Refer NCC A5.4 and Schedule 5 for FRL

Refer NCC A5.6 for RISF

Refer NCC A5.2 for non- combustibility

There are three components to the performance of fire-protected timber that need to be satisfied:

- the protected element must achieve the required Fire Resistance Level (FRL)
- the protected element must achieve the required Resistance to the Incipient Spread of Fire (RISF).
- fire-protective coverings must be non-combustible.

A1 Non-Combustible Fire-Protective Covering

Unless the NCC deems a material or element of construction to be non-combustible, non-combustible means:

- Applied to a material – not deemed combustible as determined by AS 1530.1 – Combustibility Tests for Materials.
- Applied to construction or part of a building – constructed wholly of materials that are not deemed combustible.

If the fire-protective covering is a composite or multi-layer system, each layer must be non-combustible. It is not acceptable to undertake a single combustibility test on the composite or just the facing materials and claim the fire-protective covering is non-combustible.

Typical examples of multi-layer systems are shown in Figure A1.

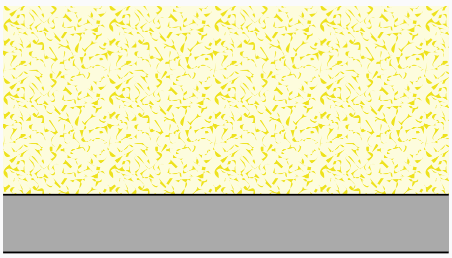
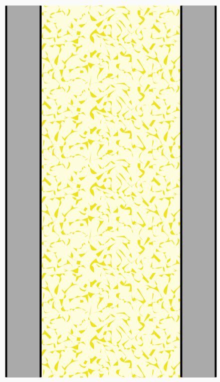
	
Multi-layer system – each layer must be non-combustible	Composite panels – each layer of the composite must be non-combustible
Commonly fire-resistant board supporting non-combustible lightweight insulation used in ceilings protecting floors/beams	Commonly non-combustible lightweight insulating core between non-combustible durable facings used for external claddings

Figure A1: Example of multi-layered fire-protective coverings (all layers).

Clause C1.9(e) of the NCC allows (deems) the following materials, though combustible or containing combustible fibres, to be used wherever a non-combustible material is required:

- plasterboard
- perforated gypsum lath with a normal paper finish
- fibrous-plaster sheet
- fibre-reinforced cement sheeting
- pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm thickness and where the Spread-of-Flame Index of the product is not greater than 0
- sarking-type materials that do not exceed 1 mm in thickness and have a Flammability Index not greater than 5
- bonded laminated materials where:
  - each laminate is non-combustible
  - each adhesive layer does not exceed 1 mm in thickness
  - the total thickness of the adhesive layers does not exceed 2 mm
  - the Spread-of-Flame Index and the Smoke-Developed Index of the laminated material as a whole does not exceed 0 and 3 respectively.

All materials forming the fire-protective covering are either permitted to be used in accordance with NCC Clause C1.9(e) or determined to be non-combustible by testing to AS1530.1.

A2 Fire Resistance Level

A fire-protected timber element must achieve the required FRL specified in the NCC for the particular application. The fire resistance of a fire-protected timber element has to be determined in accordance with Schedule 5.2(b) and (c) of the NCC.

Generally, Schedule A5.2(b) requires a prototype to be submitted to the Standard Fire Test (AS1530.4), or an equivalent or more severe test, and the FRL achieved by the prototype, without the assistance of an active fire suppression system, is confirmed in a report from an Accredited Testing Laboratory which:

- describes the method and conditions of the test and the form of construction of the tested prototype in full
- certifies that the application of restraint to the prototype complied with the Standard Fire Test;

or differs in only a minor degree from a prototype tested under Schedule 5.2(b) and the FRL attributed to the building element is confirmed in a report from an Accredited Testing Laboratory which:

- certifies that the building element is capable of achieving the FRL despite the minor departures from the tested prototype; and
- describes the materials, construction and conditions of restraint which are necessary to achieve the FRL.

The option to use AS 1720.4 char-based calculation methods to determine the fire resistance is not permitted for fire-protected timber. This is because concerns were expressed with respect to the suitability of the AS 1720.4 approach for certain types of adhesives and connections forming parts of engineered timber products. The proprietary nature of massive timber panel products and lack of standardisation of adhesives and other critical materials used in their construction meant that there was insufficient data available at the time to demonstrate the suitability or otherwise of AS 1720.4.

A3 Resistance to the Incipient Spread of Fire

A3.1 Determine Applicable Resistance to the Incipient Spread of Fire Requirements

The Resistance to the Incipient Spread of Fire (RISF) in relation to a fire-protective covering means the ability of the covering to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

The general requirement for fire-protected timber is an RISF of 45 minutes.

The NCC permits a relaxation to the RISF requirements for fire-protected timber providing both the following additional criteria are satisfied.

- the minimum timber panel thickness is not less than 75 mm
- there are no cavities between the surface of the timber and the fire protective covering or between timber members.

The 75 mm dimension relates to the inherent fire resistance achieved when using a timber panel member. If the relaxation conditions are satisfied, the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria are applicable. Typical examples of massive timber installations satisfying these conditions are shown in Figure 6.3 in the body of this Guide.

Figure A2 shows the process for determining the applicable Resistance to the Incipient Spread of Fire requirements. The general requirement for fire-protected timber is a RISF of 45 minutes.

The relaxed requirements for massive timber construction without voids and cavities is a MRISF that applies a higher interface temperature limit and the time periods for which the temperature limit applies varies according to the application in accordance with Table A1.

Table A1: Modified Resistance to the Incipient Spread of Fire required performance for applications where criteria are relaxed (massive timber construction without voids and cavities).

Application	**Modified Resistance to the Incipient Spread of Fire (MRISF)**
Inside a fire-isolated stairway or lift shaft	20 min
External walls within 1 metre of an allotment boundary or 2 metres of a building on the same allotment	45 min
All other applications	30 min

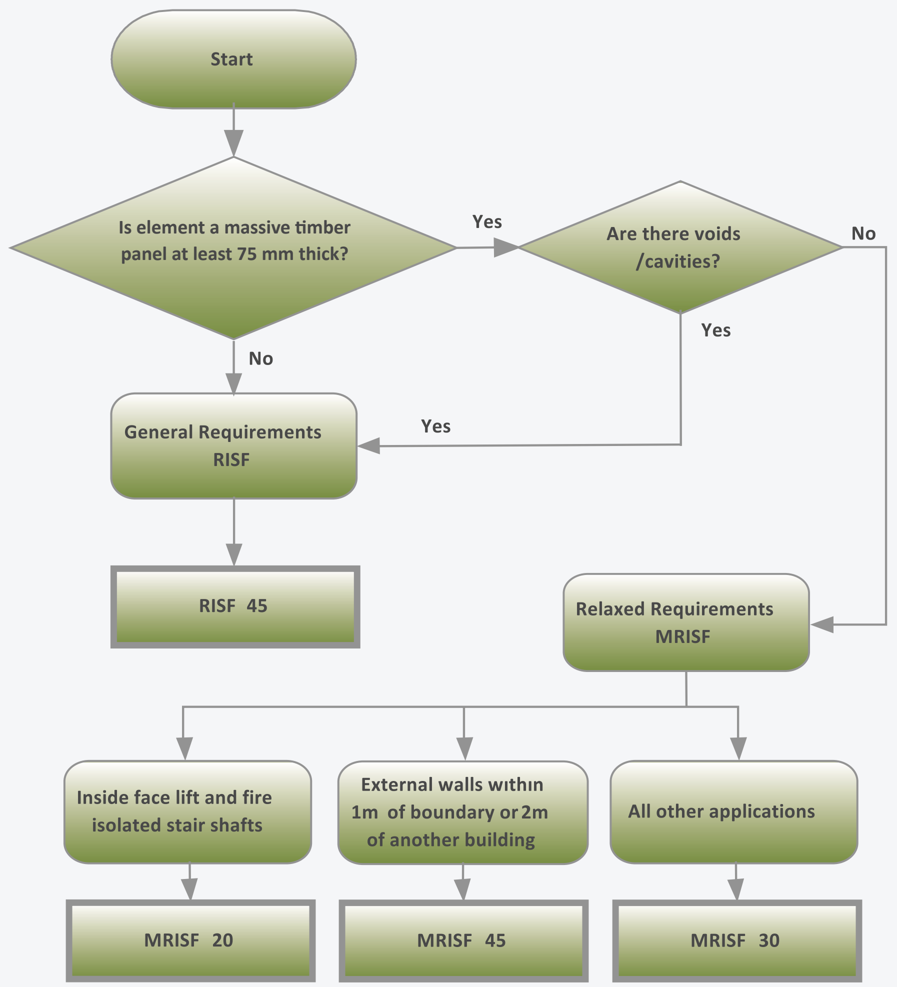


Figure A2: Determination of Resistance to the Incipient Spread of Fire acceptance requirements.

A3.2 Compliance Paths for Resistance to the Incipient Spread of Fire

Three paths are permitted to demonstrate compliance with the RISF requirements;

- simultaneous determination during a full-scale fire resistance test
- smaller-scale fire resistance test (at least 1 metre x 1 metre specimen)
- selection of Deemed-to-Satisfy fire-resisting grade plasterboard coverings.

Simultaneous determination during a full-scale fire resistance test

When a fire resistance test is undertaken to determine the FRL of an element, additional instrumentation can be included in the test to also determine the RISF or MRISF performance, providing a cost-effective approach for new protection systems.

Smaller-scale fire resistance test

There are a large number of systems that have been tested previously to determine their FRLs but in most cases insufficient data will have been recorded to determine the RISF or MRISF performance. Under these circumstances, the use of a smaller specimen (not less than 1 metre x 1 metre) is permitted to obtain supplementary data to determine the RISF or MRISF of the system in a cost effective manner. The fire-protective covering should be fitted in the same manner as that used for the original test that determined the FRL of the system.

Deemed-to-Satisfy Fire-Protective Grade Plasterboard coverings

Specification C1.13 deems fire-protective grade plasterboard facings, if fixed in accordance with the requirements to achieve the required FRL of the element, to also satisfy the requirements for Resistance to the Incipient Spread of Fire (RISF) or Modified Resistance to the Incipient Spread of Fire (MRISF). Table A2 shows the minimum requirements for plasteboard coverings.

Table A2: Fire-protective grade plasterboard coverings Deemed-to-Satisfy RISF requirements.

Requirements	Application	Performance	Minimum Deemed-to-Satisfy fire-protective grade plasterboard
General Requirements	All applications	RISF 45min	2 layers x 13 mm thick
Relaxed requirements for timber panels not less than 75 mm thick without cavities voids or cavities voids filled with non-combustible material	Inside a fire-isolated stairway or lift shaft	MRISF 20 min	1 layer x 13 mm thick
	External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	MRISF 45 min	2 layers x 13 mm thick
	All other applications	MRIFS 30 min	1 layer x 16 mm thick

Table A2 is a derivative of Specification C1.13a of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au/*** *Australian Building Codes Board © 2020.*

A3.3 Resistant to the Incipient Spread of Fire (RISF) Test Procedures

The test procedure for determining the Resistance to the Incipient Spread of Fire (RISF) of horizontal elements during a full-scale fire resistance test is provided in Section 4 of AS 1530.4 . Specification C1.13a of the NCC requires the relevant procedures from AS 1530.4 Section 4 to be applied to other elements.

AS 1530.4 requires walls to be full size or not less than 3 m high x 3 m wide and floor/ceiling systems to be full size or not less than 4 m long x 3 m wide. Floor systems are exposed to furnace heating conditions (Figure A3) from the underside and fire-resisting walls are exposed from one side. Asymmetrical walls generally require two tests to evaluate the response to exposure to fire from either side unless the side exposed to fire is specified.

Smaller-scale specimens (not less than 1 m x 1 m) can be used to retrospectively determine the RISF performance of a floor or wall system that has previously achieved the required FRL in a fire resistance test satisfying the minimum size requirements specified in AS 1530.4.

For universal application of results the minimum cavity depth should be fire tested.

To determine the RISF, five thermocouples with insulating pads as prescribed in AS 1530.4 are fixed to the inner face of the fire-protective covering system. They are placed at approximately the centre and the centre of each quarter section as shown in Figure A4.

When testing corrugated specimens, increase the number of thermocouples to six to provide an equal number of thermocouples at the maximum and minimum specimen thickness.

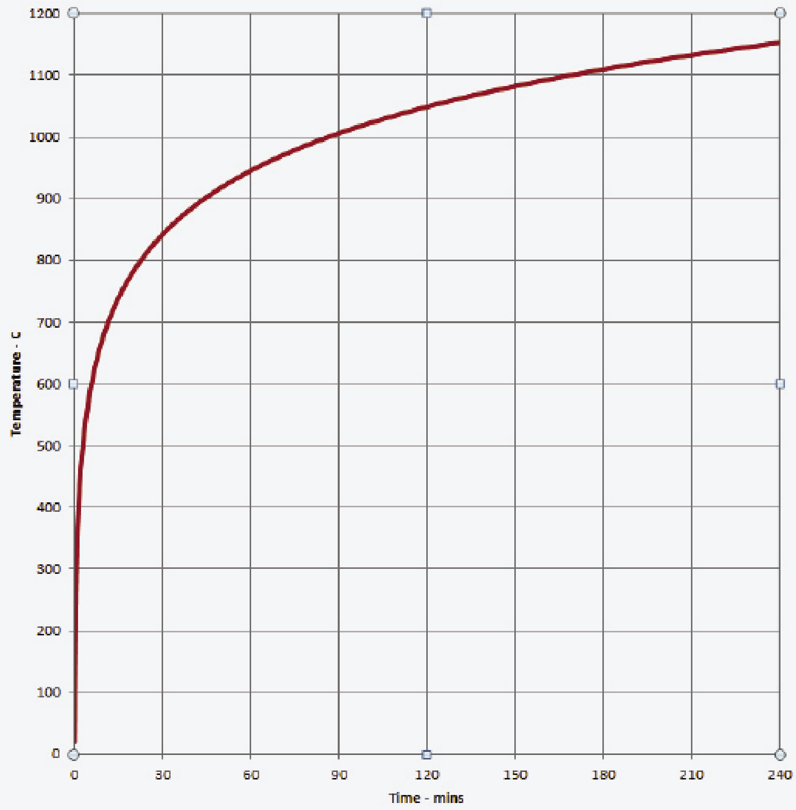


Figure A3: Standard fire resistance test heating regime.

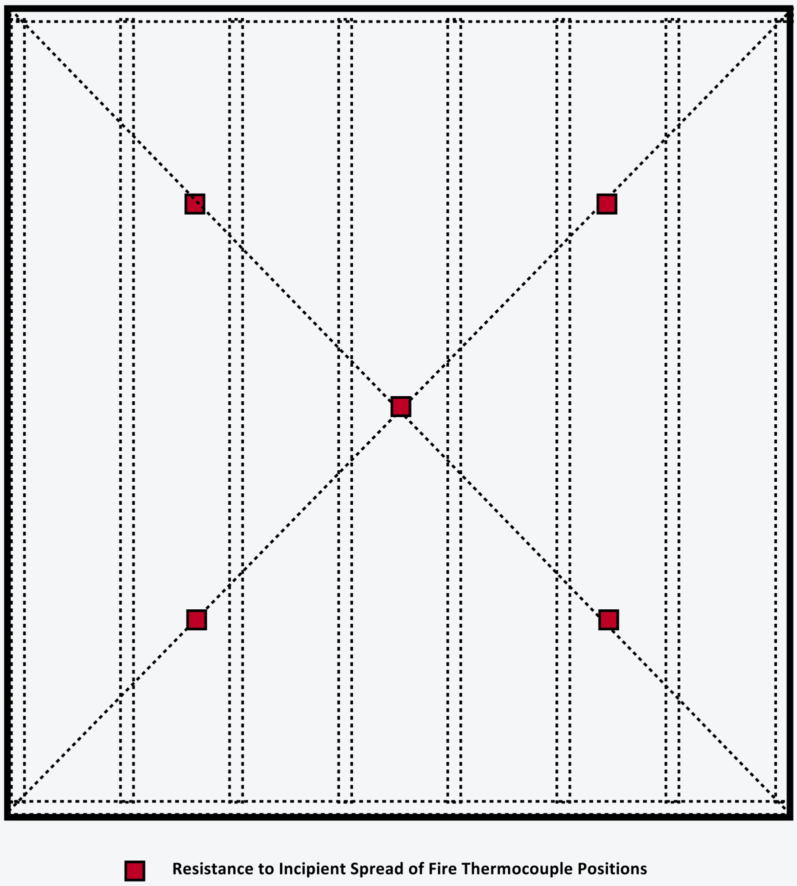


Figure A4: Elevation of a wall showing RISF thermocouple positions.

Sections through typical specimen configurations are shown in Figure A5 to illustrate the correct surfaces to apply thermocouples to determine the RISF. For fire-protected timber, the temperature has to be maintained below the prescribed temperature on the surface of the fire-protective covering facing the void and at the interface with timber elements within the wall or floor. If a wall or ceiling system is protected by a board system, for example, the temperatures are measured on the board surface within the cavity even if non-combustible insulation is applied between the timber studs or beams. However, if the non-combustible insulation forms a continuous layer between the timber elements and the board the thermocouples (t/c) should be applied to the surface of the insulation as shown in Figure A5.

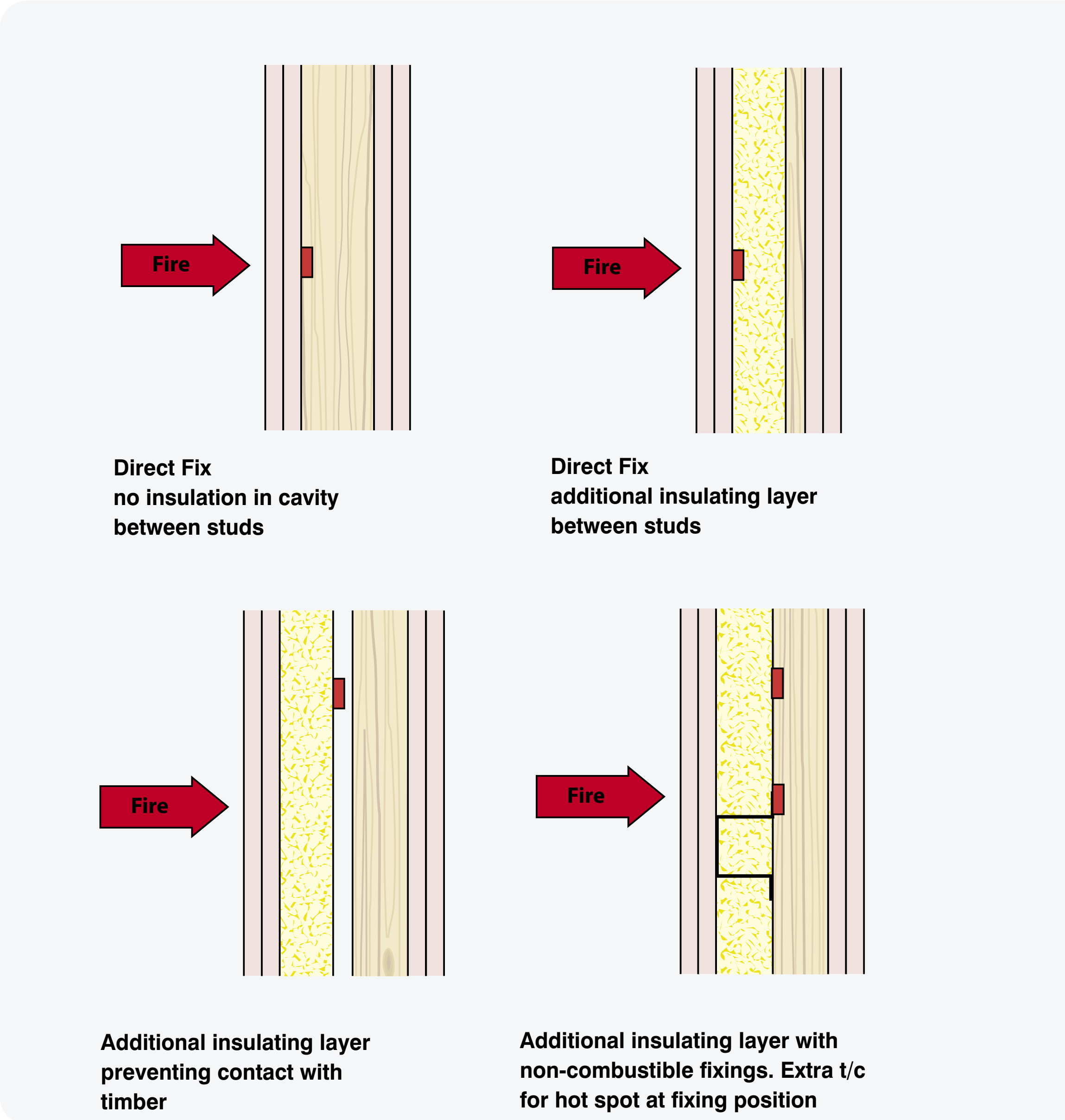
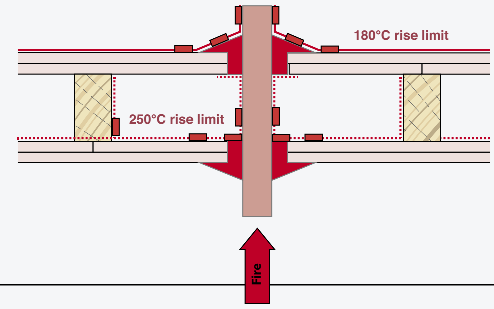


Figure A5: Resistance to the incipient spread of fire thermocouple positions for typical specimen configurations.

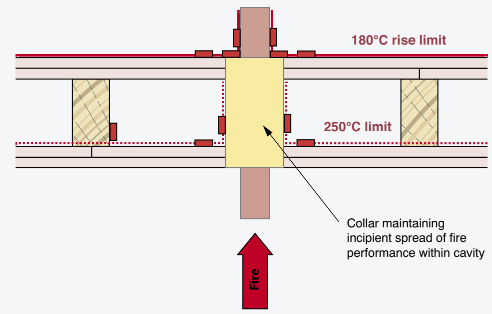
Failure in relation to the RISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 250°C.

Smaller scale specimens 1 m x 1 m can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations are shown in Figure A6. Additional thermocouples are shown to allow the simultaneous determination of the FRL of the service penetration system.

Cable/metal pipe penetration protected with fire-resistant mastic.

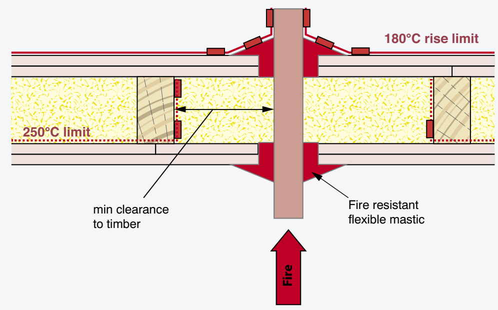


Plastic pipe protected by insulating collar system.



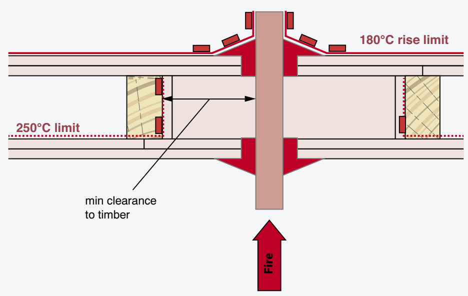
Cable/metal pipe penetration protected with fire resistant mastic and non-combustible cavity infill.

The critical interface for RISF for the service penetration system is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system



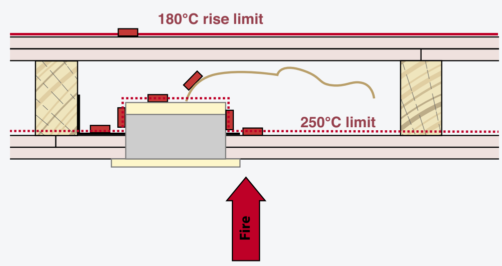
Cable/metal pipe penetration protected with fire-resistant mastic and cavity lined with non-combustible board.

The critical interface for RISF for the service penetration is the surface of the lining board where it is in contact with timber elements.



Proprietary GPO outlet protection system.

Note: Thermocouples applied to cable surface connected to the GPO, on fixing bracket and adjacent element.



GPO outlet with non-combustible cavity infill protection.

The critical interface for RISF is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system

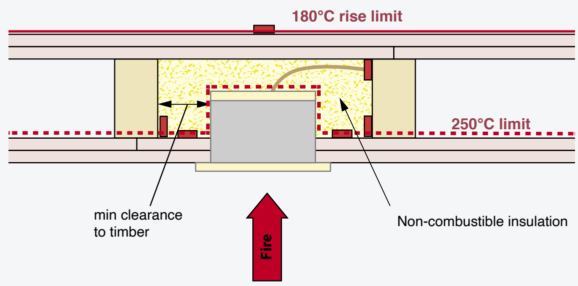


Figure A6: Typical thermocouple positions for determining the RISF of service penetrations.

The thermocouples positions must satisfy the following requirements:

- At not less than two points about 25 mm from the edge of the hole made for the passage of the service.
- Attached to adjacent structural members and those elements that support the penetrating service.
- At points on the surface of the penetrating service or its fire stopping encasement, as follows:
  - at least two thermocouples about 25 mm from the plane of the general surface of the covering and non-combustible insulation
  - where the seal or protection around the service is tapered or stepped, two additional thermocouples beyond the step or the end of any taper if it is expected that the temperatures will be higher at these points.
- Where practicable, at two points on the seal or protection around the service.
- One in the centre of the surface of the penetration nominally parallel to the plane of the fire protective covering if it terminates within the cavity (e.g GPO outlets or down lights).

Failure in relation to the RISF is deemed to occur for the service penetration when the maximum temperature of the thermocouples described above exceeds 250^{0}C.

A3.4 Modified Resistance to the Incipient Spread of fire (MRISF) Test Procedures

The MRISF is applicable to massive timber panels having a thickness not less than 75 mm if there are no voids/cavities through which fire and smoke can spread. The MRISF, amongst other things, relaxes the failure temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures are described in Section 3 of Specification C1.13a of the NCC and are summarised below:

- Tests must be carried out in accordance with AS 1530.4, or an equivalent or more severe test, on the timber element with the proposed non-combustible fire protective coverings fixed in a representative manner.
- Smaller scale specimens (not less than 1m x 1m) can be used to retrospectively determine the MRISF performance of a system that has previously achieved the required fire resistance level in a fire resistance test satisfying the minimum size requirements specified in AS 1530.4. If a fire protection system incorporates joints, the test specimens must incorporate representative joints.

To determine the MRISF interface, temperatures must be measured over the following features by a minimum of two thermocouples complying with Appendix C1 and Section 2 of AS 1530.4 as appropriate:

- at joint positions in the protection systems
- at least 200 mm from any joint
- at any other locations where, in the opinion of the Accredited Testing Laboratory, the interface temperature may be higher than the above positions.

Where the fire protective covering is not in contact with the timber (e.g. multi-layer system), the surface of the fire-protective covering is deemed to be the interface.

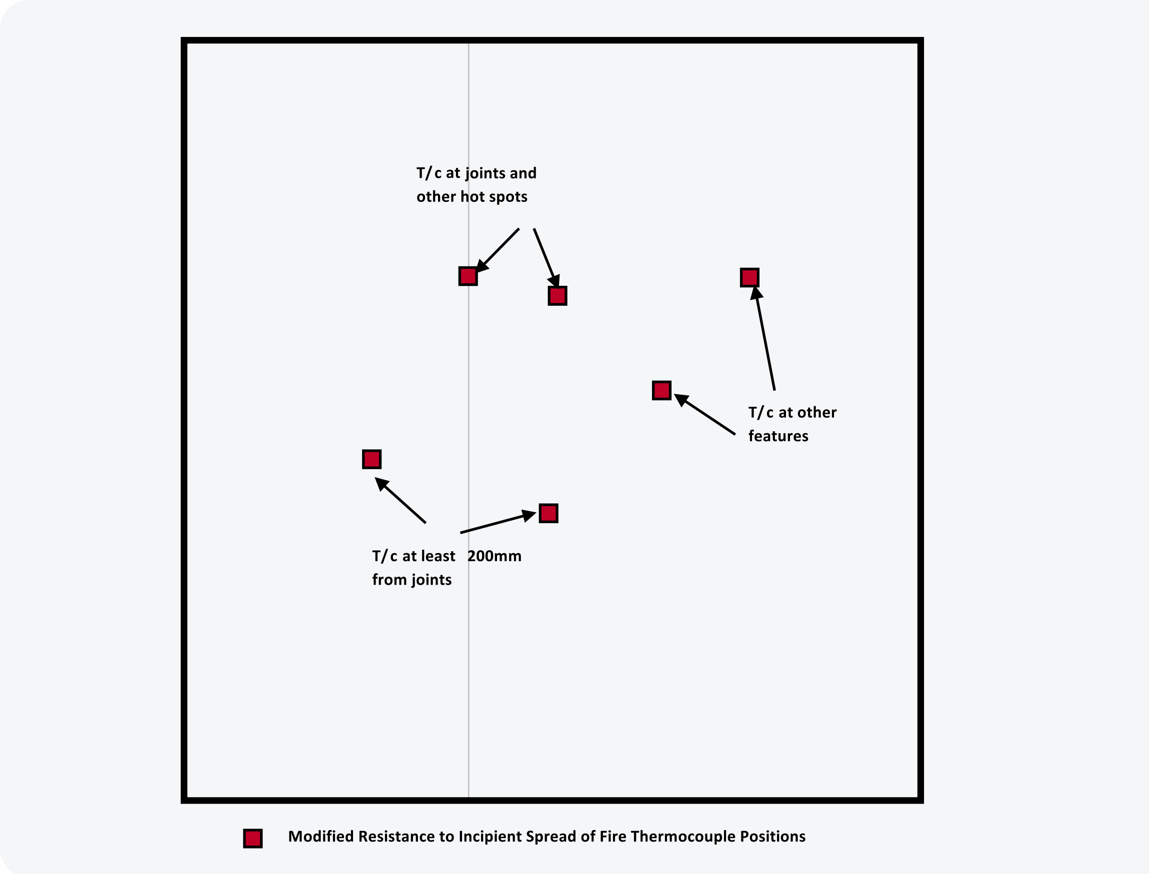


Figure A7: Elevation of a wall showing modified RISF thermocouple positions.

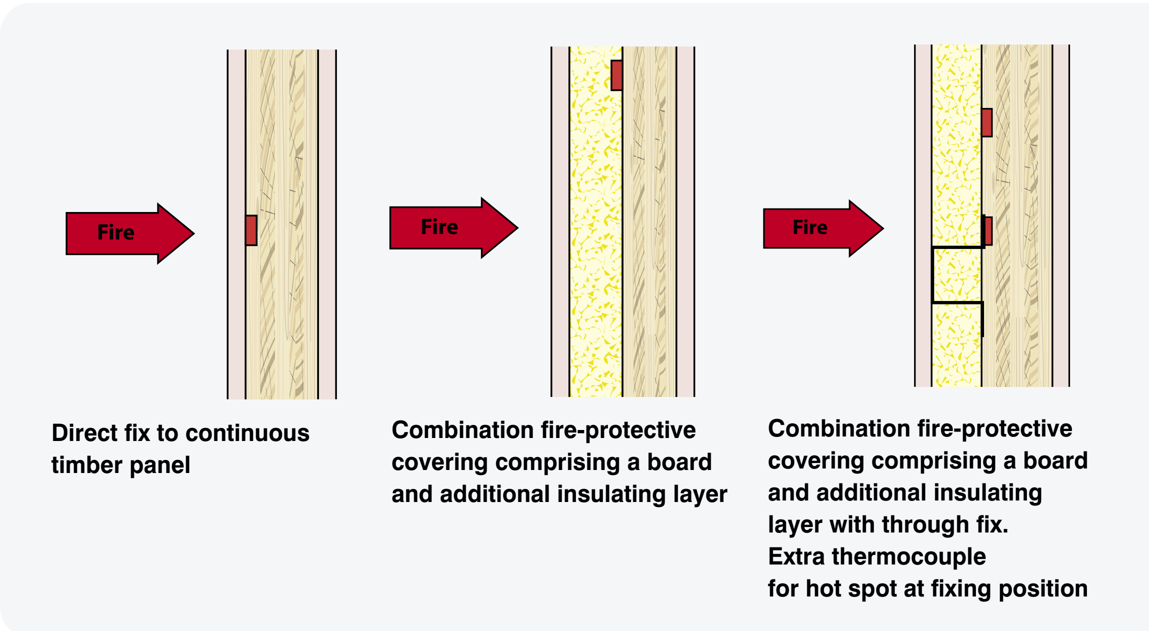


Figure A8: Modified RISF thermocouple positions for typical specimen configurations.

Failure in relation to the MRISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 300°C.

Smaller scale specimens 1 metre x 1 metre can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations to determine both the MRISF and FRLs are shown in Figure A9.

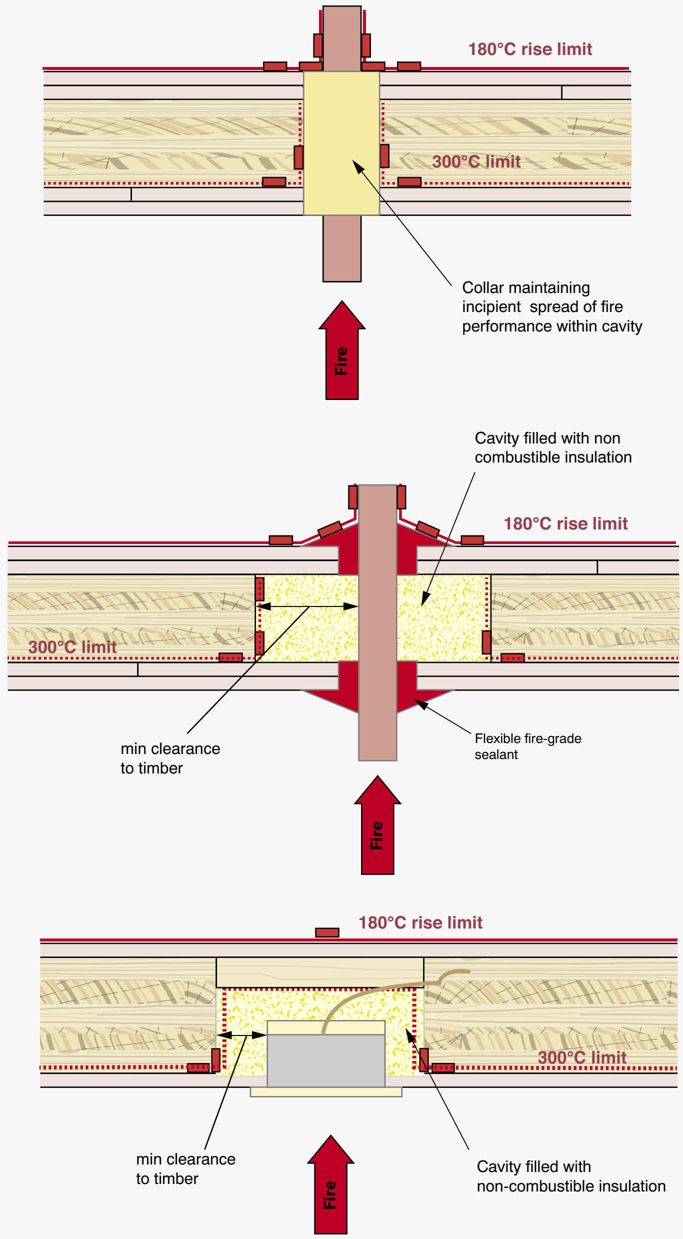


Figure A9: Typical thermocouple positions for determining the MRISF of service penetrations.

10

Appendix B – Determination of the Performance of Cavity Barriers in Fire-Protected Timber Construction

Specification C1.13 of the NCC sets out the requirements for cavity barriers in fire-protected timber construction.

The following compliance options are provided for cavity barriers:

- the cavity barrier system must achieve the FRLs specified in Table B1 when mounted in timber elements having the same or a lower density than the timber members in the proposed application or
- comprise timber of minimum thickness as specified in Table B1 or
- comprise polythene-sleeved mineral wool or non-sleeved mineral wool slabs or strips placed under compression and of minimum thickness as specified in Table B1 or
- another option is that, for cavity barriers around doors and windows, steel frames are also Deemed-to-Satisfy the requirements for cavity barriers provided that the steel frames should be tightly fitted to rigid construction and mechanically fixed. It should, however, be noted that if the windows or doors are of fire-resistant construction, the windows or door system needs to be capable of achieving the required fire resistance when mounted in the wall system, notwithstanding the requirements for cavity barriers.

Table B1: Cavity barrier requirements for fire-protected timber.

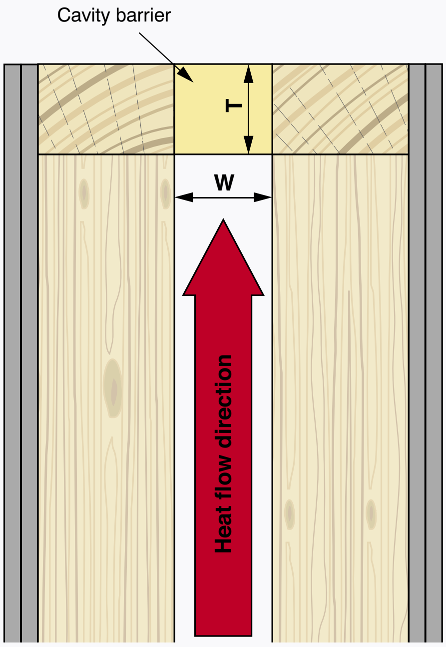
Cavity Barrier Compliance Options	FRL required for element cavity barrier is fitted to (minutes)	
	–/90/90 or less	greater than –/90/90
Cavity Barrier Required FRL – minutes	–/45/45	–/60/60
Timber required minimum thickness	45 mm	60 mm
Mineral wool required minimum thickness	45 mm	60 mm

Table B1 is a derivative of Specification C1.13 Table 1 of the National Construction Code 2019 Amendment 1 provided by ***https://www.abcb.gov.au****/ Australian Building Codes Board © 2020.*

The minimum thicknesses of protection are required to be measured in the direction of heat flow. The role of a cavity barrier is normally to prevent a fire spreading from the cavity on one side of the cavity barrier to the other. The top plate of a double stud partition (Detail A of Figure B1) is a typical example of this where the direction of heat flow for the cavity barrier would be from the underside to the upper face of the barrier.

The other role for cavity barriers is to reduce the risk of fire spread to cavities occurring around openings for doors and windows within a fire-resisting wall. This configuration is shown as Detail B in Figure B1. For this scenario, the heat flow is from the occupied area of the building through the framing to the cavity. In the Figure, the thickness dimension is identified as ‘T’.

Detail A
Cavity barrier at the head of a double stud partition



Detail B
Cavity barrier around door opening

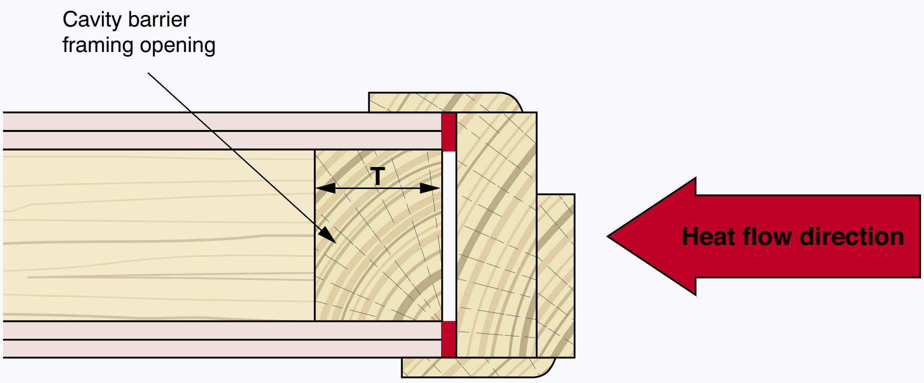


Figure B1: Heat flow direction for cavity barriers.

Proprietary cavity barrier systems may provide more practical options than the Deemed-to-Satisfy solutions for some applications. To encourage the development and use of these systems a compliance path has been provided through the specification of FRLs. For smaller cavity barriers, the performance should be determined by testing the cavity barrier as a control joint system in accordance with Section 10 of AS 1530 using timber members as the separating element. Specification C1.13 permits the results from such a test to be used for applications where the fire-protected timber is constructed from timber with a nominal density at least equal to the tested timber.

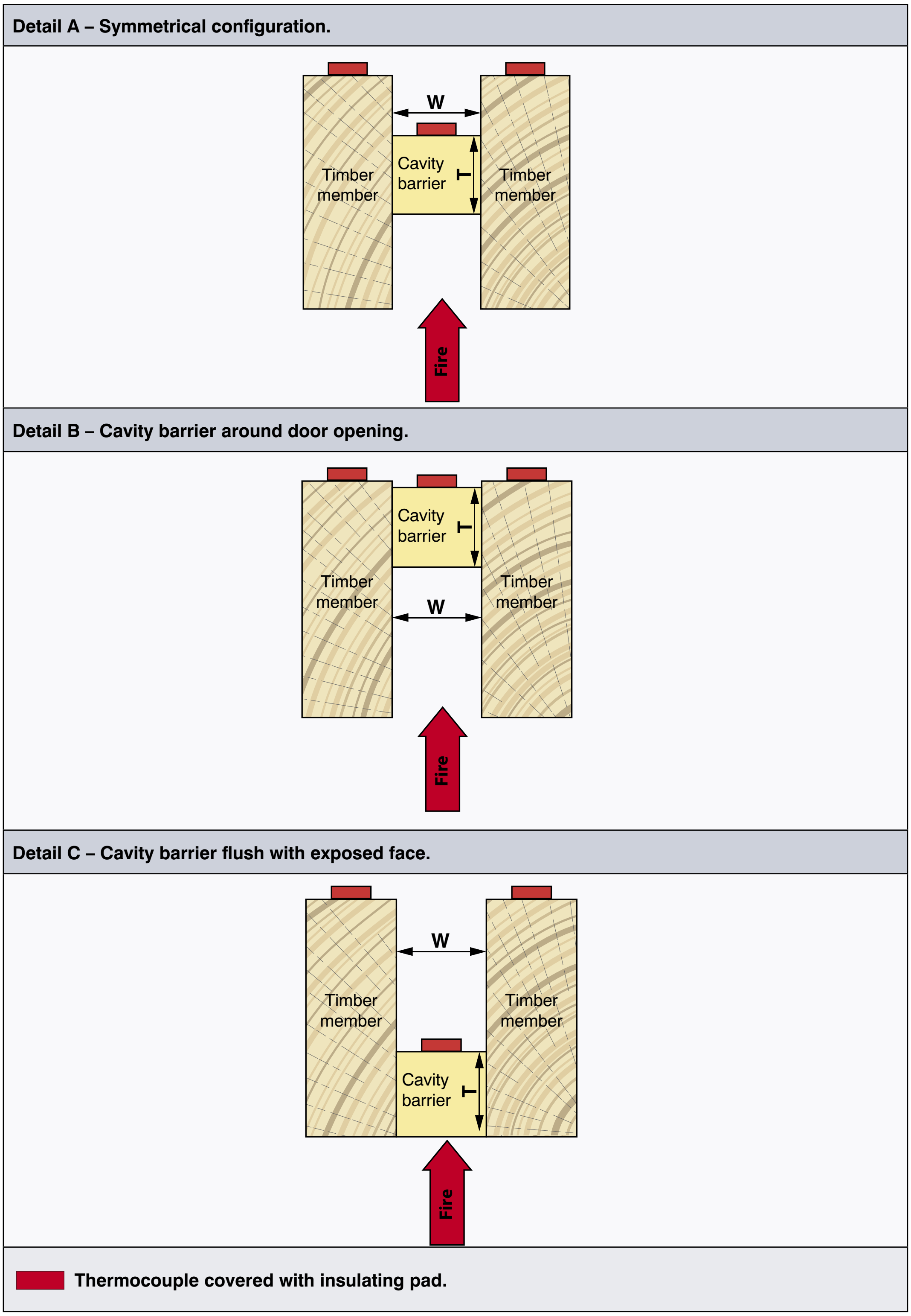


Figure B2: Typical cavity barrier test configurations.

Typical test configurations are shown in Figure B2. The selection of the test configuration(s) depends on how the cavity barrier will be mounted. If it is symmetrical (e.g. fitted at the mid-depth of a timber member), Detail A is appropriate. If the cavity barrier system is not symmetrical both details B and C should be tested unless the most onerous configuration can be determined by the test laboratory or the cavity barrier use is restricted to one configuration. A report from an Accredited Testing Laboratory should state the field of application for the cavity barrier based on the test results.

Cavity barriers can be of combustible construction and therefore a timber framed partition with exposed timber members could be used subject to the wall achieving the required FRL.

In some instances, it may be more practicable to continue the fire-resisting walls up to roof level in lieu of providing a fire-protected timber roof system with cavity barriers. This option is shown in Figure B3.

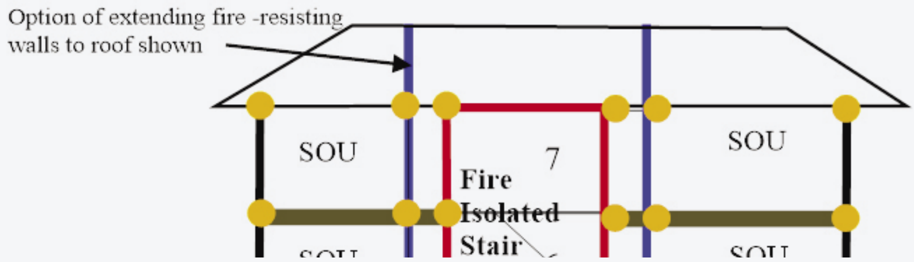


Figure B3: Design option to extend fire walls to roof level in lieu of using large cavity barriers within a fire-protected timber roof system.

11

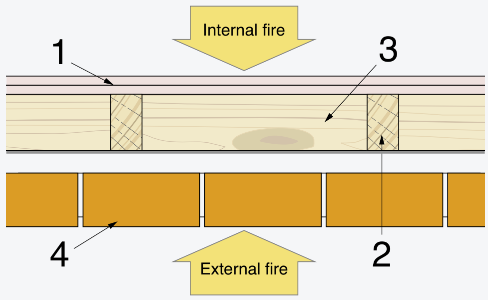
Appendix C – Example Data Sheets for an External Wall System

The following data sheet provides an example of the Evidence of Suitability required by the NCC. A brick veneer external wall system has been used because, in addition to fire and sound requirements, thermal resistance, weatherproofing and structural tests apply.

Obtain Evidence of Suitability from product supplier before specifying or installing any product or system

System External Wall 1

External Brick Veneer Timber framed wall system



1 Fire protective grade plasterboard, 2 x 13 mm thick

2.Timber framing in accordance with Evidence of Suitability

3 Cavity. – Cavity insulation may be required to achieve sound ratings and R –value (insulation must be non-combustible)

4 Outer brick veneer 90 mm thick

Typical Performance

Fire-protected timber	FRL90/90/90: RISF45: NC
Sound transmission and insulation	R_w 50: R_w+C_{tr} 50
Thermal resistance	R Value 3.3 m^{2}K/W
Damp and weatherproofing	NCC performance requirement FP1.4
Structural tests	NCC specification C1.8 Clause 3.4

Evidence of Suitability

Fire-protected timber:	
Internal Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 – (e.g.Exova Warringtonfire report 22567A-01) RISF – 45 (NCC Spec C1.13a DTS)
External Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 or design in accordance with AS 3700 RISF – 45 (AS 3700 design for insulation or test or assessment report from an Accredited Testing Laboratory)
Non-combustibility	Plasterboard NCC C1.9(e)(i) DTS Fire-protected timber NCC C1.13 Concession Cavity Insulation AS 1530.1 test report Brickwork –traditional building material

Sound Transmission and Insulation No NCC requirement for external walls in NCC 2016 but commonly specified for inner city locations. Report from a laboratory or acoustics engineer stating performance achieved.

Thermal Resistance R-Value Report complying with NCC Clause A5.2

Weatherproofing Statement of compliance with relevant requirements of AS 3700 and report confirming applicability of AS 3700 – complying with NCC Clause A5.2.

Structural tests for lightweight construction Report complying with NCC Clause A5.2 expressing results of tests in accordance with NCC specification C1.8.

Notes

Selection of systems that are fit for the purpose and the provision of Evidence of Suitability to the satisfaction of the relevant authority is the responsibility of the designers and product suppliers. Forest and Wood Products Australia Limited (FWPA) and the authors of this Guide make no warranties or assurances with respect to the fitness for purpose of the systems described in this Guide.

Ensure installation is in accordance with Evidence of Suitability, manufacturer's instructions and design drawings.

Primary Distributors

Various plasterboard distributors

12

Appendix D – Definitions and Abbreviations

12.1 NCC-related Definitions

Unless otherwise stated, the following NCC-related Definitions are based on the Definitions given in Schedule 3 of the National Construction Code 2019 Amendment 1 provided by *https://www.abcb.gov.au/* Australian Building Codes Board © 2020.

Accredited Testing Laboratory:

- an organisation accredited by the National Association of Testing Authorities (NATA) to undertake the relevant tests; or
- an organisation outside Australia accredited to undertake the relevant tests by an authority recognised by NATA through a mutual recognition agreement; or
- an organisation recognised as being an Accredited Testing Laboratory under legislation at the time the test was undertaken.

Aged care building: A Class 9c building for residential accommodation of aged persons who, due to varying degrees of incapacity, are provided with personal care services and 24-hour staff assistance to evacuate the building during an emergency.

Appropriate Authority (for the purposes of Schedule 7): The authority with the statutory responsibility to determine the particular matter satisfies the relevant Performance Requirement. The Appropriate Authority is typically the building surveyor or building certifier charged with the statutory responsibility to determine building compliance and issue the building permit/approval and occupancy certificate/ approval.

Combustible:

- Applied to a material — combustible as determined by AS 1530.1
- Applied to construction or part of a building — constructed wholly or in part of combustible materials.

Cavity barrier: A barrier placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements that complies with Specification C1.13 of the NCC, to limit the spread of fire, smoke and hot gases to other parts of the building.

Deemed-to-Satisfy Provisions: Provisions that are deemed to satisfy the Performance Requirements.

Deemed-to-Satisfy Solution: A method of satisfying the Deemed-to-Satisfy Provisions.

Discontinuous construction: A wall system typically having a minimum of 20 mm cavity between two separate wall frames (leaves) with no mechanical linkage between the frames except at the periphery intended to reduce sound transmission.

Effective height: The vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units).

Exit: Includes any of the following if they provide egress to a road or open space:

- an internal or external stairway
- a ramp complying with Section D of the NCC
- a doorway opening to a road or open space
- a fire-isolating passageway
- horizontal exit.

External wall: An outer wall of a building that is not a common wall.

Fire-isolated stair or ramp: A stair or ramp construction constructed of materials required by the NCC and located within a fire-resisting shaft or enclosure.

Fire-isolated passageway: A corridor or hallway of fire-resisting construction that provides egress to a fire-isolated stairway or ramp or to open space.

Fire-protected timber: Fire-resisting timber building elements that comply with Volume One Specification C1.13a.

Fire-protective covering:

- 13 mm fire-protective grade plasterboard; or
- 12 mm cellulose cement flat sheeting complying with AS/NZS 2908.2 or ISO 8336; or
- 12 mm fibrous plaster reinforced with 13 mm x 13 mm x 0.7 mm galvanised steel wire mesh located not more than 6 mm from the exposed face; or
- other material not less fire-protective than 13 mm fire-protective grade plasterboard, fixed in accordance with the normal trade practice for a fire-protective covering.

Fire-protective grade plasterboard: Plasterboard with glass fibre and mineral additives used to improve strength and control shrinkage under fire conditions. Typically, a lightweight loadbearing timber framed wall protected by one layer of 16 mm fire-protective grade plasterboard applied to each face would be expected to achieve an FRL of at least 60/60/60 and if protected by two layers of 13 mm fire-protective grade plasterboard on each face an FRL of at least 90/90/90.

Fire-resistance level (FRL): The grading periods in minutes determined in accordance with Schedule 5, for the following criteria, expressed in this order:

- structural adequacy
- integrity
- insulation

Note: A dash means that there is no requirement for that criterion. For example, 90/–/– means there is no requirement for an FRL for integrity and insulation, and –/–/– means there is no requirement for an FRL.

Fire-resisting (applied to a building element): Having an FRL appropriate for that element.

Fire-resisting construction: One of the types of construction referred to in the NCC.

Fire-resisting sealant: A fire-grade material used to fill gaps at joints and intersections in fire-protective linings and around service penetrations to maintain Fire Resistance Levels and Resistance to Incipient Spread of Fire performance of elements of construction. Note: The material should also be flexible to allow for movement and where required be waterproof.

Fire-source feature:

- the far boundary of a road adjoining the allotment; or
- a side or rear boundary of the allotment; or
- an external wall or another building on the allotment that is not of Class 10.

Fire wall: A wall with an appropriate resistance to the spread of fire that divides a storey or building into fire compartments.

Healthcare building: A building whose occupants or patients undergoing medical treatment generally need physical assistance to evacuate the building during an emergency and includes:

- a public or private hospital
- a nursing home or similar facility for sick or disabled persons needing full-time care
- a clinic, day surgery or procedure unit where the effects of the predominant treatment administered involve patients becoming non-ambulatory and requiring supervised medical care on the premises for some time after the treatment.

Lightweight construction: Construction that incorporates or comprises:

- sheet or board material, plaster, render, sprayed application, or other material similarly susceptible to damage by impact, pressure or abrasion
- concrete and concrete products containing pumice, perlite, vermiculite, or other soft material similarly susceptible to damage by impact, pressure or abrasion
- masonry having a width of less than 70 mm.

Loadbearing: Intended to resist vertical forces additional to those due to its own weight.

Massive timber: An element not less than 75 mm thick as measured in each direction formed from solid and laminated timber.

Non-combustible:

- Applied to a material: not deemed combustible as determined by AS 1530.1 – Combustibility Tests for Materials.
- Applied to construction or part of a building: constructed wholly of materials that are not deemed combustible.

Massive Timber 'Concession': A relaxation allowing the Resistance to Incipient Spread of Fire requirements for fire-protected timber to be modified if both the following conditions are satisfied:

- the timber is at least 75 mm thick
- any cavity between the surface of the timber and the fire-protective covering is filled with non-combustible materials.

Modified Resistance to the Incipient Spread of Fire (MRISF): The MRISF, among other things, relaxes the RISF limiting temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures for MRISF are described in Section 3 of Specification C1.13a of the NCC.

Multi-service penetration system: A service penetration system used to protect a group of services penetrating a single opening in a fire-resisting element such that the FRL, RISF or MRISF of the element is not reduced. Note: Fire protective coverings or other means may be required to be fitted around the opening to ensure that the RISF or MRISF are not reduced.

Patient care area: A part of a healthcare building normally used for the treatment, care, accommodation, recreation, dining and holding of patients including a ward area and treatment area.

Performance-based design brief (PBDB), for the purposes of Schedule 7: Means the process and the associated report that defines the scope of work for the performance-based fire safety engineering analysis and the technical basis for analysis as agreed by stakeholders.

Performance Requirement: A requirement that states the level of performance that a Performance Solution or Deemed-to-Satisfy Solution must meet.

Performance Solution: A method of complying with the Performance Requirements other than by a Deemed-to-Satisfy Solution.

Product Technical Statement: A form of documentary evidence stating that the properties and performance of a building material, product or form of construction fulfil specific requirements of the NCC, and describes:

- the application and intended use of the building material, product or form of construction
- how the use of the building material, product or form of construction complies with the requirements of the NCC
- any limitations and conditions of the use of the building material, product or form of construction relevant to (b).

Residential aged care building: A Class 3 or 9a building whose residents, due to their incapacity, are provided with physical assistance in conducting their daily activities and to evacuate the building during an emergency.

Residential care building: A Class 3, 9a or 9c building that is a place of residence where 10% or more of persons who reside there need physical assistance in conducting their daily activities and to evacuate the building during an emergency (including any aged care building or residential aged care building) but does not include a hospital.

Resident use area: Part of a Class 9c building normally used by residents, and

- includes sole-occupancy units, lounges, dining areas, activity rooms and the like
- excludes offices, storage areas, commercial kitchens, commercial laundries and other spaces not for the use of residents.

Resistance to the incipient spread of fire (RISF), in relation to a ceiling membrane: The ability of the membrane to insulate the space between the ceiling and roof, or ceiling and floor above, so as to limit the temperature rise of materials in this space to a level that will not permit the rapid and general spread of fire throughout the space.

Rise in storeys: The greatest number of storeys calculated in accordance with C1.2 of NCC.

Separating element: A barrier that exhibits fire integrity, structural adequacy, insulation or a combination of these for a period of time under specified conditions (often in accordance with AS 1530.4).

Shaft: The walls and other parts of a building bounding:

- a well, other than an atrium well
- a vertical chute, duct or similar passage, but not a chimney or flue.

Standard Fire Test: The Fire-resistance Tests of Elements of Building Construction as described in AS 1530.4.

Storey: A space within a building which is situated between one floor level and the floor level next above, or if there is no floor above, the ceiling or roof above, but not:

- a space that contains only—
  - a lift shaft, stairway or meter room
  - a bathroom, shower room, laundry, water closet, or other sanitary compartment
  - accommodation intended for not more than 3 vehicles
  - a combination of the above
  - a mezzanine.

Structural adequacy, in relation to an FRL: The ability to maintain stability and adequate loadbearing capacity as determined by AS 1530.4.

Structural member: A component or part of an assembly that provides vertical or lateral support to a building or structure.

Treatment area: An area within a patient care area, such as an operating theatre and rooms used for recovery, minor procedures, resuscitation, intensive care and coronary care from which a patient may not be readily moved.

Verification Method: A test, inspection, calculation or other method that determines whether a Performance Solution complies with the relevant Performance Requirements.

Ward area: That part of a patient care area for resident patients and may contain areas for accommodation, sleeping, associated living and nursing facilities.

12.2 General Definitions

For general terminology used to describe the typical forms of timber construction that can be adopted for mid-rise buildings refer to the Timber Construction Options for Mid-rise Buildings (Page 10)

High-rise buildings: Buildings with an effective height greater than 25 m.

Low-rise buildings: Buildings of Type B or C construction as required by the NCC. For healthcare Class 9a and Class 9c buildings low-rise buildings are typically 1-2 storeys high but may typically extend up to 3 storeys for other classes.

Mid-rise buildings: Buildings that are not low-rise and have an effective height not more than 25 m. Typically they are 3-8 storeys high. The maximum number of storeys depends on the specific site conditions and building design.

12.3 Abbreviations

CLT	Cross-laminated Timber
DTS	Deemed-to Satisfy
LVL	Laminated Veneer Lumber
MRISF	Modified Resistance to the Incipient Spread of Fire
NCC	The National Construction Code Volume One, Building Code of Australia 2019 Amdt. No. 1
RISF	Resistance to the Incipient Spread of Fire

13

Appendix E - References

13.1 WoodSolutions Technical Design Guides

#1 Timber-framed Construction for Townhouse Buildings Class 1a

#2 Timber-framed Construction for Multi-residential Buildings Class 2 and 3

#3 Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b

#4 Building with Timber in Bushfire-prone Areas

#5 Timber service life design – Design guide for durability

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)

#20 Fire Precautions during Construction of Large Buildings

#37R Mid-rise Timber Buildings Multi-Residential Class 2 and 3

#37C Mid-rise Timber Buildings Commercial and Education Class 5, 6, 7, 8 and 9b (incl.Class 4 parts)

#38 Fire Safety Engineering Design of Mid-Rise Buildings

#39 Robustness in Structures

#42 Building Code of Australia DTS Solutions for Timber Aged Care Buildings (Class 9c)

#44 CLT Acoustic Performance

13.2 Australian Standards

AS 2118.1 Automatic fire sprinkler systems – General requirements

AS 2118.4 Automatic fire sprinkler systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height

AS 2118.6 Automatic fire sprinkler systems – Combined sprinkler and hydrant systems in multi-storey buildings

AS 1170 series – Structural design actions

AS 1720.1 Timber structures – Design methods

AS 1720.4 Timber structures – Fire resistance for structural adequacy of timber members

AS 5113 Amd 1 Classification of external walls of buildings based on reaction to fire performance

AS 1905.1 Components for the protection of openings in fire-resistant walls – Fire-resistant doorsets

AS 1530.1 Methods for fire tests on building materials, components and structures – Combustibility test for materials

AS 1530.4 Methods for fire tests on building materials, components and structures – Fire-resistance tests for elements of construction

AS 1530.7-2007 Methods for fire tests on building materials, components and structures Part 7: Smoke control assemblies—Ambient and medium temperature leakage test procedure. 2007, Standards Australia: Sydney.

AS 4072.1 Components for the protection of openings in fire-resistant separating elements – Service penetrations and control joints

AS 2444 Portable fire extinguishers and fire blankets – Selection and location

AS 1682.1 Fire, smoke and air dampers – Specification

AS 1682.2 Fire, smoke and air dampers – Installation

AS 1668.1 The use of ventilation and air conditioning in buildings – Part 1 Fire and smoke control in buildings. 2015

AS 6905-2007 Smoke Doors. 2007

AS 2896-2011 Medical gas systems – installation and testing of non-flammable medical gas pipeline systems. 2011

13.3 Accredited Test Laboratory Reports

RIR 22567A-04 – The fire resistance performance of timber-framed walls lined with plasterboard if tested in accordance with AS1530.4-2005

RIR 37600400 – The Fire Resistance Level (FRL) of timber-framed floor/ceiling systems incorporating timber and metal web floor trusses or various engineered joists when tested in accordance with AS 1530.1-2014

RIR 37401400 – The Fire Resistance Level (FRL), Resistance to the Incipient Spread of Fire (RISF) and Modified Resistance to the Incipient Spread of Fire (MRISF) performance of various timber-framed and massive timber panel systems

RIR 55945800.1B – The fire-resistance performance of timber-framed walls lined with 3 layers of 16mm fire protective grade plasterboard if tested in accordance with AS 1530.4-2014 for 180 minutes

WF report FAS190034-RIR 1.0 – Timber-framed floor/ceiling systems incorporating various timber and metal web floor trusses or engineered joists with an FRL of 120/120/120

13.4 ABCB references

National Construction Code 2019 Amendment 1 Australian Building Codes Board https://www.abcb.gov.au/ Australian Building Codes Board © 2020."

Other References

National Construction Code Volume One: Building Code of Australia 2019 Amendment 1 – Australian Building Codes Board, Canberra ACT – © Commonwealth of Australia and the States and Territories 2019

International Fire Engineering Guidelines (2005) – Australian Government, Sate and Territories of Australia

Safe Design of Structures – Safe Work Australia

Exova Warringtonfire Australia Pty Ltd (EWFA) Regulatory Information Reports (RIR) issued to Forest & Wood Products Australia:

RIR 22567A-04 – The fire resistance performance of timber-framed walls lined with plasterboard if tested in accordance with AS1530.4-2005

RIR 37600400 – The fire resistance level (FRL) of timber-framed floor/ceiling systems incorporating timber and metal web floor trusses or various engineered joists when tested in accordance with AS 1530.1-2014

RIR 37401400 – The fire resistance level (FRL), Resistance to the Incipient Spread of Fire (RISF) and Modified Resistance to the Incipient Spread of Fire (MRISF) performance of various timber-framed and massive timber panel systems

Bennetts, ID *et al* Fire Safety of Hospitals - A guide for designers. 2018: Melbourne.

37 R

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Aug 2019
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Mid-rise Timber Buildings
Multi-residential
Class 2 and 3

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

WoodSolutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-43-0

Researchers

Paul England
EFT Consulting
PO Box 1288, Upwey, VIC 3158
paul.england@eftconsult.net

Boris Iskra
TPC Solutions (Aust) Pty Ltd
Level 2, 394 Little Bourke Street
Melbourne VIC 3000

Published: September 2016
Revised: August 2019

Table of Contents

Introduction

The National Construction Code Volume One, Building Code of Australia 2019 (NCC), allows the use of timber construction systems under the Deemed-to-Satisfy (DTS) Provisions for all buildings up to 25 metres in effective height ('mid-rise construction', see Figure 0.1).

The DTS provisions cover both traditional 'lightweight timber framing' and 'massive timber' products such as Cross-laminated Timber (CLT) in conjuction with the use of appropriate non-combustible fire-protective coverings – termed 'fire-protected timber' in the NCC – and the use of appropriate compliant automatic sprinkler systems. With mid-rise timber construction design, fire and sound are two of the major considerations: appropriate fire-resisting construction is critical to providing acceptable levels of fire safety, while sound or acoustic performance is essential because of its daily impact on inhabitant amenity and quality of life.

This Guide applies to Class 2 and 3 Residential Buildings or parts of buildings. It aims to assist in providing specific advice on both of these areas and is specifically written for use by designers, specifiers, builders, regulatory and certifying authorities. It is set out according to a simple step-by-step process as presented in Figure 0.2. The steps are then used as the basis for headings throughout the main body of this Guide. Details on the scope and other important aspects of the Guide are set out below.

Scope

This Guide explains how to achieve the targeted fire and sound Performance Requirements in the National Construction Code (NCC) for Class 2 (apartments) and Class 3 (e.g. hotels, motels) mid-rise timber buildings using the Deemed-to-Satisfy pathway for fire-protected timber introduced in the 2016 edition of the NCC, with further developments included in the 2019 edition.

Mid-rise timber buildings are typically 4 to 8 storeys high

Low-rise timber buildings
are buildings of:

- Type C construction (1 or 2 storey)

or

- Class 2 and 3 buildings up to 3 storeys; 4 storeys if the ground level is a concrete or masonry garage. (Timber concession)

Mid-rise timber buildings
have an effective height of not more than 25 metres

Typically, they are 4-8 storeys high (the maximum number of storeys depends on the floor-to-floor height)

High-rise timber buildings
have an effective height greater than 25 m.

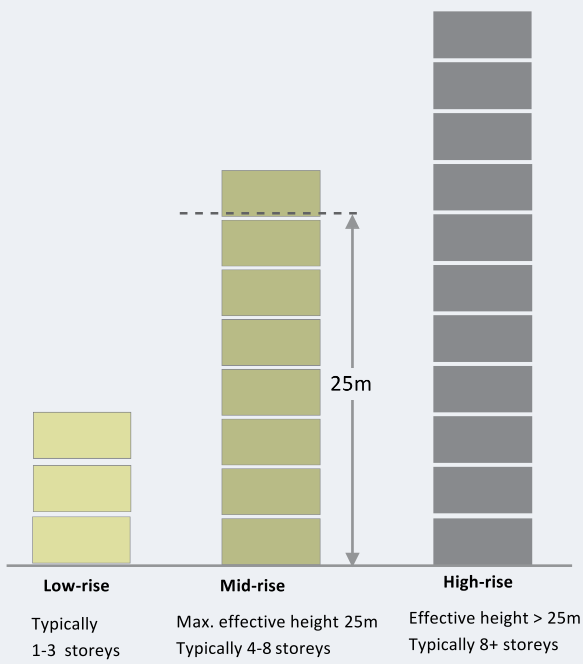


Figure 0.1: Comparison of low, mid and high-rise buildings.

The guide provides some advice relating to good practice where the NCC leaves off in areas of increasing interest to users

This Guide specifically focuses on:

- sound insulation of wall, floor and ceiling elements relevant to sole-occupancy units (SOUs) and surrounding construction in multi-residential buildings
- fire-resisting construction (including fire-protected timber provisions) of wall, floor and ceiling elements for multi-residential and office buildings
- additional fire safety measures required for mid-rise timber buildings.

In addition, this Guide provides advice on good practices to facilitate compliance, ease of maintenance and enhancements to the minimum NCC prescriptive provisions relating to fire and sound.

This Guide does not deal with all aspects of fire safety and sound insulation. Nor does it provide advice on which specific wall, floor and related systems should be used as there are many suppliers of proprietary systems and the intention is to encourage innovation. Generic details are provided for demonstration purposes. Before adopting these details, designers should check the availability of appropriate Evidence of Suitability with the material suppliers and, if necessary, modify the details accordingly.

Design Process for Sound- and Fire-Resisting Construction

Although national, some NCC provisions vary by State. It is vital to know the applicable provisions

Regulatory Differences between States and Territories

This Guide focuses on the NCC requirements of the 2019 edition. From time-to-time, State and Territory-based NCC amendments or other State legislation may vary requirements. Users of this Guide should make themselves aware of any differences and should develop a full understanding of the resulting implications. This Guide should be used on this basis.

Timber Construction Options for Mid-rise Timber Buildings

General Construction Options for Timber Buildings

A number of timber system options are available for the construction of mid-rise timber buildings with a range of possible options shown in Figure 0.2. Note: Under the NCC DTS provisons only fire-protected timber building systems are permitted, where an element is required to be of non-combustible construction or of masonry or concrete construction.

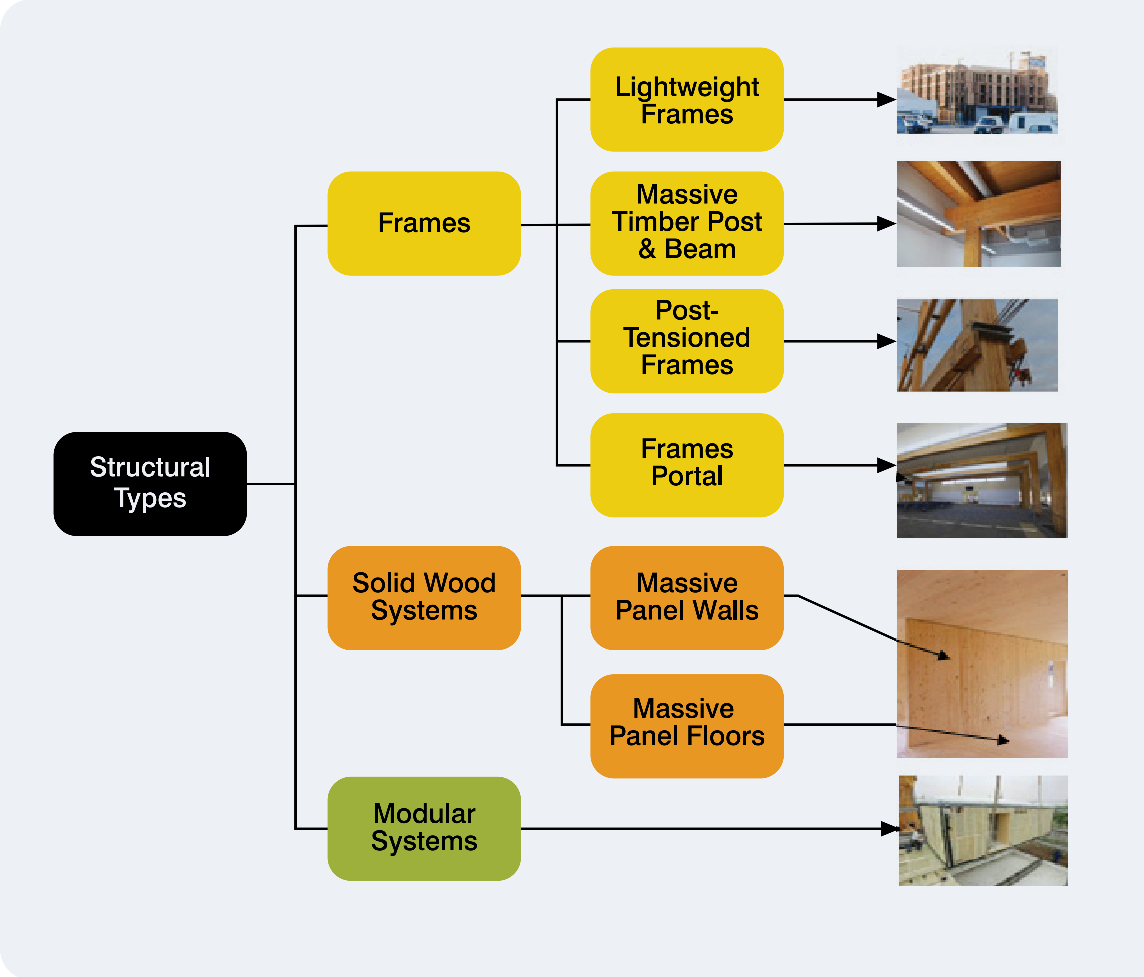
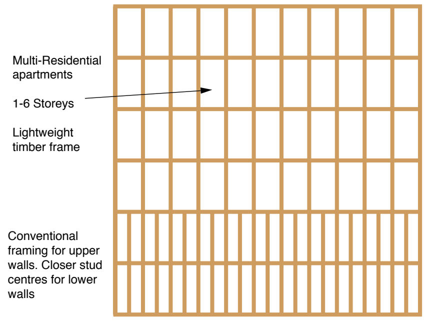
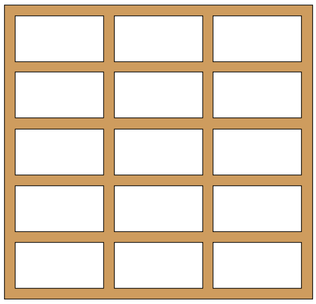


Figure 0.2: Summary of structural timber construction options.

Note: No exposed structural timber is currently permitted under Deemed-to-Satisfy Provisions where an element must be of non-combustible construction.

The most appropriate system is influenced by the function and floor plan of the building.





- In general, apartment (Class 2) and hotel/motel (Class 3) buildings tend to have sole-occupancy units (SOUs) or individual rooms with quite closely spaced walls. In effect they form a 'honeycombed' structure with many individual load paths and, as such, the use of lightweight timber-framed systems (up to around six storeys) combined with fire- and sound-rated plasterboard is an effient form of construction. Alternatively, a solid wood system, or a mixture of solid and lightweight timber construction might be considered.
- Commercial buildings by contrast generally require larger open-plan spaces for either work amenity or flexibility in fitout and as such these buildings are often constructed utilising a post and beam approach for the overall structure and non-loadbearing lightweight partitions as needed. Typically, the main columns and beams might be constructed using glued-laminated timber (Glulam) with floors being either lightweight prefabricated cassettes or solid massive panel floor plates.

There are significant efficiency, speed and cost benefits in using timber structural systems compared to alternative material such as reinforced concrete. These include:

- Reduced on-site construction infrastructure (preliminary costs) such as fixed cranes, site accommodation, storage areas, scaffolding and edge protection, hoists, etc.
- Direct savings from faster methods of construction compared to traditional steel and concrete structures due to:

  – increased scope for off-site prefabrication and panelisation, and

  – lighter and more easily manoeuvred and installed materials.
- Reduced foundation requirements due to a lighter above-ground structure.
- Significantly reduced on-site costs and Work, Health and Safety (WHS) issues, particularly with a shift to more prefabricated solutions.
- Increased ability to commence follow-on trades earlier in the construction process, reducing the overall construction time.
- Increased accessibility of the construction site and significantly lower impacts of noise and site activities on local neighbourhoods (less truck movements); a major benefit for suburban multi-residential developments.

Detailed information on the specific construction cost benefits of timber systems in different Class buildings can be found in the following WoodSolutions Technical Guides:

#26 Rethinking Office Construction – Consider Timber – a material cost comparison of a typical office building

#27 Rethinking Apartment Building Construction – Consider Timber – a material cost comparison of a typical apartment building

#28 Rethinking Aged Care Construction – Consider Timber – a material cost comparison of typical aged care accommodation

#29 Rethinking Industrial Shed Construction – Consider Timber – a material cost comparison of a typical industrial shed.

Further information on the cost benefits of using timber can be found in the Rethinking Series of WoodSolutions Design Guides

The NCC prescriptive Deemed-to-Satisfy (DTS) solution for mid-rise buildings requires non-combustible fire-protective coverings to be applied to timber elements.

Refer Section 4.3

Fire-protected Timber Options for Mid-rise Timber Buildings

Whichever timber construction option is selected the prescriptive Deemed-to-Satisfy (DTS) solutions for mid-rise timber buildings require timber members to be fire-protected, where an element is required to be of non-combustible construction or concrete or masonry construction..

The 'general timber' requirements that apply for fire-protected timber are:

- the building element must be protected to achieve the required FRL, and
- a non-combustible fire-protective covering must be applied to the timber that achieves a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

The NCC permits a 'relaxation' to the general requirements in the case of fire-protected massive timber panels if the following additional criteria are satisfied:

- the timber panel is at least 75 mm thick, and
- any cavity between the surface of the timber and the fire-protective covering is filled with non-combustible materials.

If both these conditions are satisfied it is still necessary for the fire-protected timber member to achieve the required FRL and have a non-combustible fire-protective covering. However, the thickness of the fire-protective coverings, based on the covering's RISF performance, can be modified depending on the application (e.g. internal SOU wall, external wall).

The basis for allowing specific provisions for massive timber panels is that timber with a large cross-section can achieve high fire-resistance due to the formation of a char layer that protects the timber core and allows it to continue to support an imposed load or maintain a fire separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for longer than lighter forms of construction; and by not permitting any concealed spaces between the massive timber members, or between the timber and fire-protective coverings, the risk of fire spread is addressed.

Further details relating to fire-protected timber are provided in Section 4.3.

The different timber construction systems generally use one or more of a range of different sawn or engineered timber products, including:

- Sawn timber – softwood (MGP) and hardwood (F- and A-graded).
- Engineered timber – particleboard, plywood, Oriented Strand Board (OSB), Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL), I-beams, fabricated floor and roof trusses, Glued Laminated Timber (Glulam) and Cross-laminated Timber (CLT).

Depending on the dimensions of the element and configuration, some of these construction systems may satisfy the requirements that allow the massive timber provisions to apply.

Timber-framed Construction

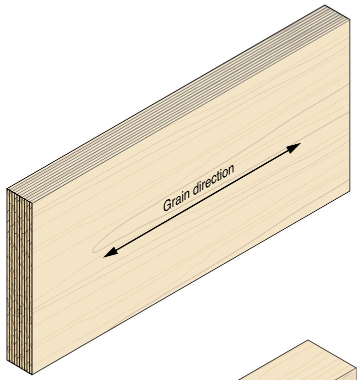
Lightweight timber-framed construction systems use commonly available structural timber framing products assembled into lightweight systems such as wall frames, floor and roof trusses, and prefabricated cassette floor modules.

Sawn Timber Products

Sawn timber products include seasoned structural softwood (MGP10, 12 & 15) or seasoned structural hardwood (typically F17 or F27). Typical thicknesses are 35 and 45 mm and depths include: 70, 90, 120, 140, 190, 240,* and 290* mm.

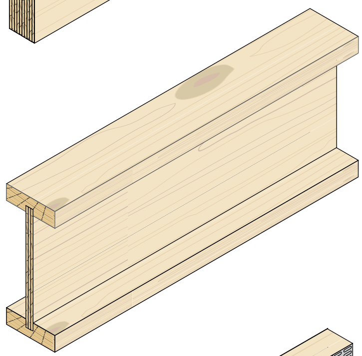
* available on order

Laminated Veneer Lumber (LVL)



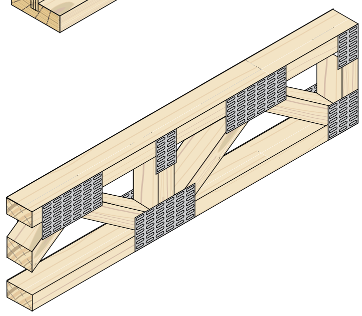
Lightweight framing elements are available in Laminated Veneer Lumber (LVL), a widely used softwood engineered wood product, available in all the standard framing sizes. LVL is manufactured by bonding together rotary peeled or thin sliced wood veneers under heat and pressure. As LVL is typically used in a beam or stud application, the grains of the veneers are all oriented in the same direction. LVL is typically manufactured in slabs 1200 mm wide, known as billets, which are then cut into the commonly available framing member depths required. LVL is typically manufactured in lengths up to 12 metres in 0.3 metres increments.

I-Beams



I -Beams are lightweight, high-strength, long-span structural timber beams. They typically comprise top and bottom flanges of LVL or solid timber – which make the distinct shape. The flanges are separated by a vertical web, usually manufactured from structural plywood, Oriented Strand Board (OSB) or light gauge steel. Typical depths are: 200, 240, 300, 360 and 400 mm and lengths are available up to 15 metres.

Parallel Chord Trusses

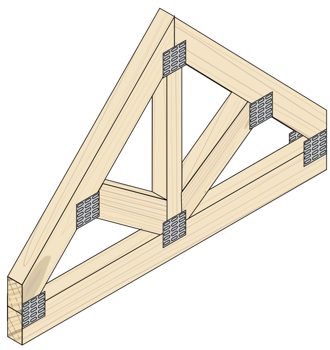


Parallel Chord Trusses are similar to I-beams in that they have top and bottom chords (flanges) of LVL or solid timber but instead of solid webs, web struts are used.

The struts may be either timber or light gauge steel and are secured to the chords typically with nailplates. The struts may be diagonal (more common for steel struts) or a mix of vertical and diagonal (more common with timber struts).

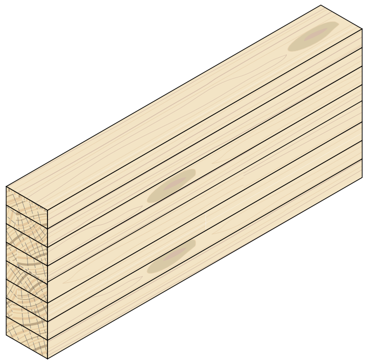
Figure 0.3: Hybrid floor system with parallel-chord trusses supported from a steel I-beam.

Roof Trusses



Timber roof trusses provide an engineered roof frame system designed to carry the roof or roof and ceiling, usually without the support of internal walls. The characteristics of a roof depend on the purpose of the building it covers, the available roofing materials and the wider concepts of architectural design and practice. Light truss roofs are formed from sawn or LVL timber elements connected with nailplates or other mechanical fixings designed and supplied by frame and truss manufacturers.

Glued-laminated Timber (Glulam)



Glued-laminated Timber (Glulam) consists of a number of strength graded, kiln-dried laminations face bonded and finger-jointed together with adhesives. Elements can be manufactured to practically any length, size or shape: beams are often manufactured with a built-in camber to accommodate dead load deflection or curved for aesthetic appeal.

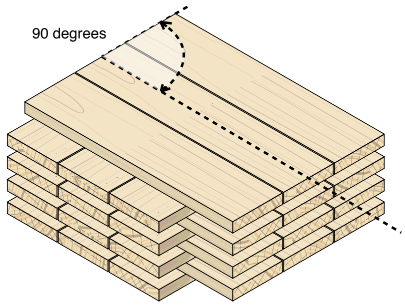
A range of GL Grades are produced in Australia or imported depending on the timber species used in manufacture: GL10 (Cypress), GL13 (Radiata Pine, Oregon), GL17 (Slash Pine, Merbau), GL18 (Tas Oak, Vic Ash), GL21 (Spotted Gum) – the GL descriptor refers to the element's Modulus of Elasticity (E), i.e. GL10 describes a Glulam member that has an E-value of 10GPa.

A wide range of depths are available in increments from 90 mm to over 1,000 mm; and thicknesses from 40 mm to 135 mm; with 65 mm and 85 mm being two commonly used. Lengths up to 18 metres are available in 0.3 metre increments from traditional suppliers and up to 27 metres in length from specialist manufacturers.

Solid Massive Wood Panel Construction Systems

Solid wood panel construction systems utilise massive timber engineered wood panels such as Laminated Veneer Lumber, Cross-laminated Timber (CLT), or Glue/nail-laminated Mass Timber panels, in minimum panel thicknesses of 75 mm when used in accordance with the NCC DTS provisions. Solid wood panels can be used to form complete floors, walls and roofs and construction methods have more in common with precast concrete panels than timber framing; except that timber panels are much lighter, more easily worked and easier to erect.

Cross-laminated Timber (CLT)



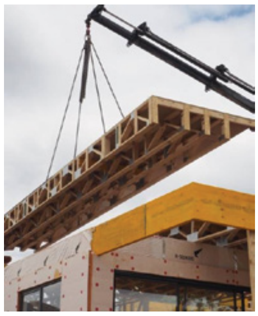
Cross-laminated Timber (CLT) utilises individual planks of timber 12-45 mm thick and 40-300 mm wide face-glued together (and edge-glued in some instances), each layer at 90° to its neighbouring lamella; effectively 'jumbo plywood'. CLT panels are typically 57 mm – 320 mm thick and made up of 3, 5, 7 or 8 layers depending on application. Panels are available in 2.2 to 2.95 metres wide and up to 11.9 metres in length (dictated mainly by shipping containers as all product used in Australia is currently imported).

More detailed information on CLT can be found at the WoodSolutions website or in *Technical Guide #16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)*, which introduces the use of CLT in construction and provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

Prefabrication and Modular Wood Construction Systems

A major benefit in utilising timber structural systems in mid-rise construction is the ability to prefabricate off-site and manufacture frames or cassettes, panelised elements or full volumetric modules to minimise the on-site construction requirements and costs.

Prefabricated Cassette Floor Systems



Prefabricated Cassette floor systems utilise a range of timber structural products, typically for flooring (particleboard, plywood or OSB panels) and for floor joists and bearers (sawn timber, LVL, OSB beams, floor trusses or I-beams). Cassettes tend to be around 3 metres wide and up to around 12 metres long (due to travel restrictions). Cassette floor systems are highly effective in mid-rise construction as they are extremely fast to install and far safer for on-site workers, dramatically reducing 'fall-from-height' risks for workers.

With mid-rise construction, where effective acoustic separation is required, a Timber/Concrete Composite Cassette Floor might be considered; the concrete screed adding mass for acoustic and vibration control as well as acting compositely with the timber components for improved structural performance. For more detailed information on this refer to *WoodSolutions Technical Guide #30, Timber Concrete Composite Floor Design Guid*e.

Panelised Elements

Fully panelised elements involve the total off-site manufacture of all components including; structural members, insulation, sarking, plumbing and electrical fittings, window and/or door installation and internal lining installation (and external cladding if appropriate).

Modular Wood Construction Systems

Modular wood construction systems (volumetric modules) utilise either light-frame systems (mainly) or solid wood panel systems. The main principle is that the entire volumetric box consisting of walls, floor and ceiling, as well as inner lining and all services are assembled in a factory and transported to the construction site for erection. To assemble the modules on top of each other a male-female connector arrangement is often used. The size of the modules is generally limited by transportation restrictions with maximum dimensions of around 4.2 metres wide, up to 13 metres long and 3.1 metres high. The overall building height is generally the same as for light-frame systems, around 6–7 storeys. Modular systems are used in a range of applications including apartments, student housing, hotels and aged care facilities.

Hybrid Construction

Mid-rise timber buildings may also use a 'mix' of materials – hybrid construction – to achieve cost-effective, practical and robust solutions.

For example, a common configuration is to use concrete construction for below ground structures, such as car parks and basements to reduce the risks to timber elements associated with groundwater and for ground floor construction to provide a physical separation from the ground as part of a termite management strategy (where required).

Other forms of hybrid construction may use timber and other structural materials within the same element of construction, for example:

- concrete toppings to timber floors with shear connections to use composite action
- floors constructed with a mix of steel beams and timber beams to manage a mix of spans as shown in Figure 0.3 (LVL timber beams are increasingly used to replace steel beams for this application).

Robust Structural Design

The structural design of mid-rise timber buildings must comply with the relevant NCC requirements, including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1(a)(iii).

Further guidance is provided in ***WoodSolutions Technical Design Guide #39 Robustness in Structures.***

Options for fire precautions during construction should be considered as part of the overall design process

Fire Precautions during Construction

Mid-rise timber buildings, when complete, provide a high level of safety because of the combination of automatic fire sprinklers and fire-protected timber, among other things.

While the use of timber significantly reduces a number of risks during construction, the fire risk can be increased as a result of the increased volumes of unprotected timber. *WoodSolutions Technical Design Guide #20 Fire Precautions during Construction of Large Buildings* provides advice relating to fire precautions during construction to help building professionals and organisations with responsibilities for fire safety on a construction site reduce the risk of fire.

Other Design Considerations

Designers need to take account of a broad range of design considerations to ensure that a building is fit-for-purpose and complies with all requirements of the NCC and other legislation. These include:

- structural design (for safety and serviceability)
- weatherproofing
- safe access and egress
- light and ventilation (including condensation control)
- energy efficiency
- durability (including termite management)
- design in bushfire-prone and flood-prone areas.

Some sources of information on these matters are referenced in the Appendices of this Guide.

1

Step 1 – High-Level NCC Design Issues (Schematic Design)

The National Construction Code (NCC) is the regulatory framework for determining the minimum design and construction requirements for buildings in Australia. This Step covers a selection of high-level design issues relating to fire-resisting and sound-insulating construction.

1.1 Determine the Type of Construction Required

Refer NCC Volume One A6 for details of all classes of building

The NCC contains mandatory performance requirements that apply to 10 primary classes of building that are determined by the building's purpose. The classes directly relevant to this Guide are:

- **Class 2 buildings** – buildings containing two or more sole-occupancy units each being a separate dwelling, e.g. apartment buildings
- **Class 3 buildings** – a residential building that is a common place of long-term or transient living for a number of unrelated persons, including:
  - a boarding-house, guest house, hostel, lodging-house or backpackers accommodation
  - a residential part of a hotel, motel, school, detention centre or health-care building (where accommodating members of staff)
  - accommodation for the aged, children or people with disabilities.

The building class in conjunction with the building height, expressed in terms of the rise in storeys, and the maximum size of fire components are used to determine the type of construction required.

Refer NCC Volume One C1.2 for calculation of rise in storeys

The rise in storeys is the sum of the greatest number of storeys at any part of the external walls of the building and any storeys within the roof space:

- above the finished ground next to that part; or
- if part of the external wall is on the boundary of the allotment, above the natural ground level at the relevant part of the boundary.

The maximum size of fire compartments (floor area, volume) is defined in the NCC Clause C2.2.

Type C construction is applicable to most low-rise buildings. It is the least fire-resisting form of construction and places few fire-related restrictions on the use of structural timber members.

Type B construction, while not requiring as high FRLs as Type A construction, applies similar constraints to the use of timber.

Type A construction is the most fire-resisting and the prescriptive solutions within the NCC have, in the past, imposed severe limitations on the use of timber through the prescription of masonry and concrete construction and non-combustibility for elements required to achieve a prescribed Fire Resistance Level (FRL).

Table 1.1 shows the required types of construction specified by the NCC.

Table 1.1: Types of construction required by NCC Volume One.

Rise in storeys	Multi-residential		Office	Retail	Car Park/ Storage	Factory/ Laboratory	Hospitals/ Public assembly
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
4 or more	**A**	**A**	**A**	**A**	**A**	**A**	**A**
3	**A***	**A***	**B**	**B**	**B**	**B**	**A**
2	**B***	**B***	**C**	**C**	**C**	**C**	**B**
1	**C**	**C**	**C**	**C**	**C**	**C**	**C**

** Refer low-rise concessions (e.g. Specification in C1.1 Clause 3.10 and Clause 4.3).*

Design parameters such as the building class, rise in storeys, effective height and type of construction should be confirmed with the building surveyor/certifier.

1.2 Determine NCC Compliance Pathway

1.2.1 NCC Compliance Pathway

To comply with the NCC the relevant Performance Requirements must be satisfied, as demonstrated by means of the Assessment Methods specified in the NCC. There are two pathways that can be followed (or a combination of the two).

- For a Deemed-to-Satisfy solution, it is necessary to provide Evidence of Suitability to show that the Deemed-to-Satisfy Provisions have been met.
- For a Performance Solution (previously referred to as an 'Alternative Solution'), specific building solutions are developed for a building that may vary from the Deemed-to-Satisfy Provisions.

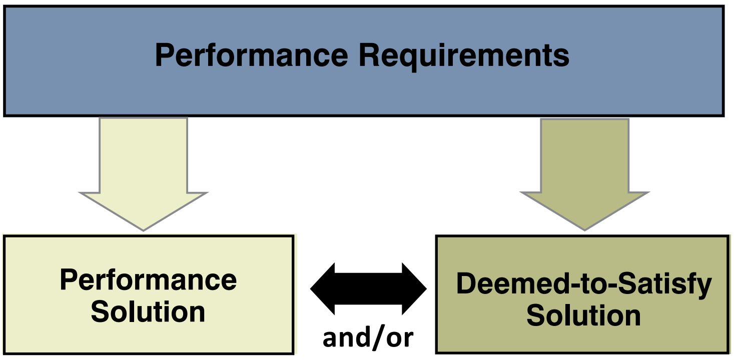


Figure 4: Pathways for demonstrating compliance with NCC performance requirements.

The construction systems and details in this Guide are based on the Deemed-to-Satisfy (DTS) Solution pathway for mid-rise timber buildings which was first introduced into the 2016 edition of the NCC for Class 2, 3 and 5 buildings and extended to all classes of buildings in the 2019 edition. This does not prevent designers using Performance Solutions but variations from the DTS Solution should ensure the fire safety strategy for the building is not adversely affected. Further guidance is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-Rise Buildings*.

1.2.2 NCC Compliance Options for Timber Buildings

In the context of this Guide, timber buildings are defined as buildings where timber is the predominant material in the structure. There are still opportunities to use timber for some structural and non-structural applications in buildings using other materials for the primary structure.

Table 1.2 summarises options for complying with the NCC Performance Requirements for Class 2 to 9 buildings with further details provided below. DTS Solutions are available for the building configurations shaded in green (light and dark green) with this Guide being directly applicable to the applications shaded in dark green. At the time of writing, Performance Solutions needed to be developed for the areas shaded in blue.

Mid-rise Class 2 and 3 buildings are addressed in this Guide

Table 1.2: Design options for timber buildings.

Rise in storeys or effective height	Multi-residential		Office	Retail	Car Park/ Storage	Factory/ Laboratory	Hospitals/ Public assembly/ Schools
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
Effective height greater than 25m	High	High	High	High	High	High	High
8[EH]	Mid	Mid	Mid	Mid	Mid	Mid	Mid
7	Mid	Mid	Mid	Mid	Mid	Mid	Mid
6	Mid	Mid	Mid	Mid	Mid	Mid	Mid
5	Mid	Mid	Mid	Mid	Mid	Mid	Mid
4	Mid[1]	Mid[1]	Mid	Mid	Mid	Mid	Mid
3	Low[1]	Low[1]	Mid	Mid	Mid	Mid	Mid
2	Low[1]	Low[1]	Low	Low	Low	Low	Mid
1	Low	Low	Low	Low	Low	Low	Low

EH: Effective height of not more than 25 metres

Note 1: Refer to Technical Design Guide #02 to check if low-rise timber concessions apply.

Low	Deemed-to-Satisfy Solution – Guide #02 or 03
Mid	Deemed-to-Satisfy – Guide #37 (this Guide)
Mid	Deemed-to-Satisfy – Guide #37C for Class 5, 6, 7, 8 and 9b commercial and educational buildings and Guide #37H for Class 9 healthcare buildings
High	Performance Solution

Low-rise timber buildings

Refer NCC Spec C1.1 Clauses 3.1 and 4.3 and WoodSolutions Design Guides #01, #02 and #03

There are relatively few fire-related restrictions on the use of structural timber members in Buildings of Type C construction irrespective of the Class of Building under the DTS Solution pathway and for domestic housing.

The NCC Volume One Deemed-to-Satisfy Solution pathway includes concessions that facilitate the use of timber-framed construction for Class 2 and 3 buildings up to a rise in storeys of 3 and, in limited cases, up to 4 storeys. Guidance in relation to construction of these low-rise options is provided in the following WoodSolutions Technical Design Guides:

#01 Timber-framed Construction for Townhouse Buildings Class 1a – information about the fire safety and sound insulation performance requirements in the NCC for Class 1a attached buildings.

#02 Timber-framed Construction for Multi-residential Low-rise Buildings Class 2 and 3 – information about the fire and sound performance requirements in the NCC for Class 2, 3 low-rise buildings.

#03 Timber-framed Construction for Commercial Low-rise Buildings Class 5, 6, 9a & 9b – information about the fire performance requirements in the NCC for Class 5, 6, 9a and 9b buildings

These buildings would normally be designed following the Deemed-to-Satisfy Solution pathway with Performance Solutions being used to address minor variations and/or unusual design circumstances.

The construction systems and details in this Guide are based on the Deemed-to-Satisfy (DTS) Solution pathway for mid-rise timber buildings as detailed in the 2019 edition of the NCC.

This does not prevent designers using Performance Solutions but variations from the DTS Solution should ensure the fire safety strategy for the building is not adversely affected. Further guidance in relation to this is provided in *WoodSolutions Design Guide #38 Fire Safety Engineering Design of Mid-Rise Buildings.*

Check with the regulatory authority that the building effective height does not exceed 25 metres if applying the mid-rise fire-protected timber solution

Mid-rise timber buildings

Mid-rise buildings are of Type A or B construction up to an effective height of not more than 25 metres. The use of timber structural members under the NCC prescriptive pathway is restricted for mid-rise buildings unless the option to use fire-protected timber in conjunction with automatic fire sprinklers is adopted. This Guide addresses Class 2 and 3 buildings applying these design principles.

Guidance on the technical derivation of the mid-rise fire-protected timber solution for Class 2 and 3 buildings is provided in *WoodSolutions Technical Design Guide #38 Fire Safety Engineering Design of Mid-rise Timber Buildings* which may assist with the development of a Performance Solution.

The NCC defines effective height as *"the vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units)"*. If there is any doubt as to whether a building's effective height does not exceed 25 metres, it is recommended that the effective height is checked with the relevant authorities.

High-rise buildings

All high-rise timber buildings (effective height greater than 25m) need to follow the Performance Solution pathway.

1.2.3 Overview of the Deemed-to-Satisfy Solution for Mid-rise Timber Buildings

The NCC 2019 includes Deemed-to-Satisfy Provisions that allow the construction of mid-rise timber buildings. The main features of the mid-rise timber building DTS Solutions are:

- the building has an effective height of not more than 25 metres
- fire-protected timber complying with Specification C1.13a of the NCC is used for loadbearing timber elements, non-loadbearing timber walls required to achieve an FRL and for elements of construction required to be non-combustible
- the building has a sprinkler system, other than a FPAA101D or FPAA101H system, complying with Specification E1.5 of the NCC throughout
- any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible
- cavity barriers are provided in accordance with Specification C1.13 of the NCC.

These fire safety precautions aim to provide a robust building solution on the following basis:

Automatic sprinkler suppression system: Objective is to suppress a fire before the structure is threatened and greatly reduce the risk to people and property.

Fire-protected timber (NCC prescribes FRLs AND non-combustible fire protective coverings): Objective is to prevent or delay ignition of the timber structural members so that the response to an enclosure fire will be similar to non-combustible elements, masonry or concrete during the growth period and prior to fire brigade intervention.

Cavity barriers: Objective is to prevent uncontrolled spread of fire through cavities in the low probability events of either failure of the protective covering or fire starting within the cavity.

Non-combustible insulation: Objective is to minimise the risk of fire spread through cavities by removing a potential source of fuel, i.e. combustible insulating materials.

1.2.4 Performance Solution Options

For high-rise timber buildings, the Performance Solution pathway has to be adopted. Refer to the following WoodSolutions Technical Guides for further information.

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT) – introduces the use of CLT in construction, outlining the history, environmental performance and mechanical properties. Also provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

#38 Fire Safety Engineering Design of Mid-rise Timber Buildings – reference source describing methods and supporting data that can be used for the fire safety engineering design of mid-rise timber buildings based on the research undertaken to develop and justify the changes to the NCC 2016.

1.2.5 Evidence of Suitability

The NCC requires every part of a building to be constructed in an appropriate manner to achieve the performance requirements using materials and construction that are fit for the purpose for which they are intended, including safe access for maintenance.

The NCC Volume One specifies requirements for Evidence of Suitability in Clause A5.2 but there are the following additional specific requirements that apply to certain aspects of fire safety under NCC prescriptive requirements:

- NCC Clause A5.4 Fire-Resistance of Building Elements
- NCC Clause A5.5 Fire Hazard Properties
- NCC Clause A5.6 Resistance to the Incipient Spread of Fire.

In most instances, for the materials and systems considered in this Guide, the Evidence of Suitability for the fire resistance or Resistance to the Incipient Spread of Fire of an element of construction will be a report from an Accredited Testing Laboratory.

If a Performance Solution is proposed, compliance should be demonstrated using the procedures prescribed in Clauses A5.4 and A5.6 of the NCC as appropriate.

1.3 Determine Schematic Building Layout

1.3.1 Mixed Class Buildings

Evidence of Suitability for fire resistance and Resistance to the Incipient Spread of Fire should be a report from an Accredited Testing Laboratory as prescribed in the NCC

The NCC DTS Solution for Class 2 and 3 mid-rise buildings using fire-protected timber in conjunction with automatic fire sprinklers can also be applied to the Class 2 and 3 parts of mixed class buildings, provided the different classes are adequately fire separated (refer Step 4) and the entire building is protected by an automatic fire sprinkler system complying with NCC Volume One Specification E1.5 other than a FPAA101D or FPAA101H system.

This provides added flexibility for the design of new buildings and facilitates the recycling of existing buildings. For example, fire-protected timber apartments (Class 2) could be constructed above existing concrete-framed carpark levels minimising the increase in foundation loads as shown in Figure 1.2.

Fire-protected timber can be used in conjunction with other forms of construction in mixed class buildings

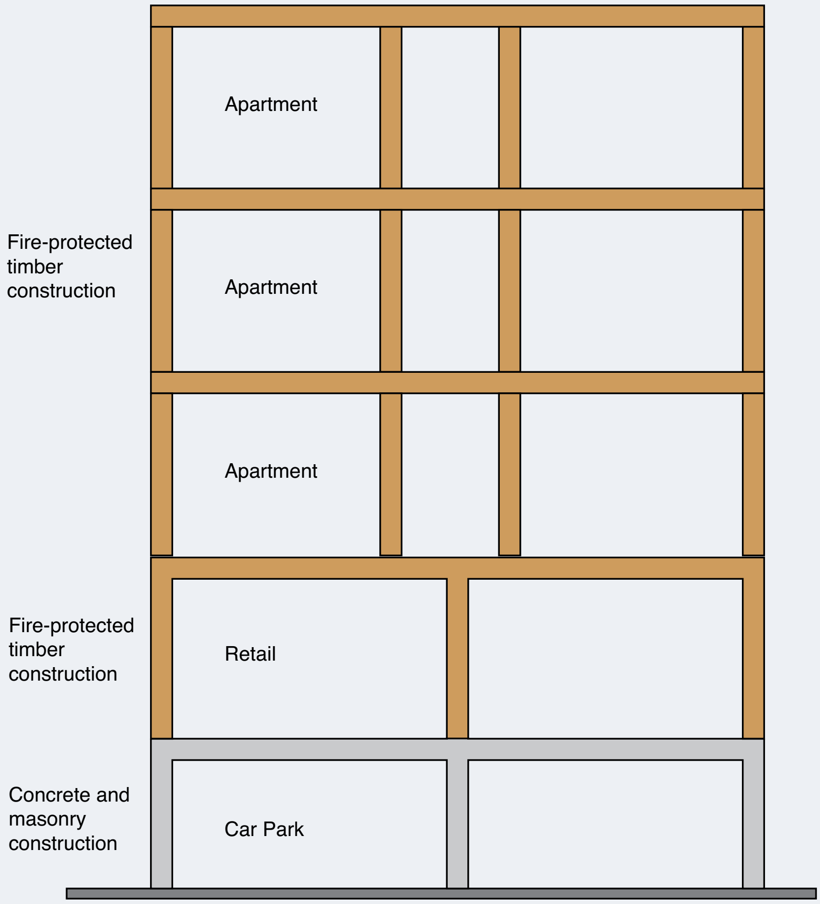


Figure 1.2: Multi-class and mixed forms of construction.

Note: Sprinkler protection required throughout entire building development.

1.3.2 Definition of Sole Occupancy Units

A Sole Occupancy Unit (SOU) is a room or other part of a building for occupation by an owner, lessee, tenant or other occupier to the exclusion of others.

The concept of an SOU is central to addressing many issues concerning fire and sound performance in Class 2 and 3 buildings. For these buildings, the SOU boundaries are used to separate a given building into manageable units for dealing with fire and sound performance.

The wall and floor/ceiling elements that bound an SOU are important in achieving NCC sound and fire performance, but specific requirements vary depending on whether the SOUs are:

- side-by-side
- stacked on top of each other (as well as side-by-side)
- adjoining rooms of a different type or space (such as a public corridor)
- adjoining rooms of similar usage back-to-back, e.g. back-to-back habitable areas or back-to-back service rooms such as laundries or kitchens.

Note: Though bounding wall and floor elements of a SOU identify the main sound and fire-rated elements, it is also highly likely that certain internal walls and floors will also need to be fire-rated when they are supporting fire rated walls/floors located above.

1.3.3 Check Compliance with Fire Compartment Size Limits

For Class 2 and 3 buildings, additional size limits are not needed because of the requirements to provide fire-separating boundaries to SOUs, which generally have relatively small areas and volumes.

The fire safety strategy should be specified at the start of the project and refined as the design progresses

1.3.4 Determine Schematic Fire Safety Design Strategy

The preliminary specification of a fire safety strategy for a building is important since it may impact significantly on the building layout. This is applicable irrespective of the compliance pathway chosen (Performance Solution or DTS Solution) since, even within the DTS pathway, options have to be selected that affect the building layout, detailed design and on the use of the building through its life cycle.

The schematic fire safety design strategy should at the preliminary stage provide as a minimum:

- A summary of the fire safety objectives.
- Building uses that the design needs to address.
- Occupant characteristics that the design addresses.
- Approach to demonstrating compliance with the NCC (Performance Solution, DTS solution or a combination).
- Where a DTS Solution is specified it is still necessary to provide details of the DTS options selected as detailed below.
- Schematic drawings and brief descriptions as appropriate indicating
  - design requirements for automatic fire sprinkler systems
  - design requirements for detection and alarm systems
  - general layout showing fire / smoke resistant compartmentation and structural elements
  - active smoke control measures if provided
  - means of egress during a fire emergency including travel distances to exits, discharge of exits, door operations, etc
  - evacuation strategy and associated emergency warning and intercom system (EWIS)
  - means to alert the fire brigade and equipment to facilitate fire brigade intervention
  - any other fire protection measures.
- An implementation plan stating who is responsible for ensuring compliance and measures that will be in place to facilitate compliance (e.g. inspection schedule).
- Protection measures required during construction.
- Fire safety management measures after completion to ensure ongoing effectiveness of the fire safety strategy through the life of the building.

With a DTS solution critical decisions still need to be made such as the application of concessions (e.g. Specification E1.5a) that impact on the layout options for a building

This preliminary strategy should be regularly reviewed and updated with further details added as the design develops.

Consider building services throughout the design process

1.3.5 Determine Building Services and Preliminary Layout

The preliminary selection of building services and service locations should be considered when determining the general building layout and provision allowed for safe maintenance, modification and addition of services without compromising fire safety and sound separation.

Typical matters for consideration include:

- in residential buildings, will each SOU be serviced by self-contained heating and air-conditioning systems or will centralised systems be provided?
- locate service shafts to minimise nuisance noise and facilitate fire compartmentation
- minimise service penetrations through fire barriers (e.g. mount ceiling lights in false ceilings)
- group service penetrations together and select treatments that minimise the risk of fire spread to cavities
- avoid the need for hot works (e.g. welding, grinding) on services at the position of penetrations through fire-resistant elements
- provide for maintenance and additional services
- select services and materials that minimise the need for hot works (e.g. welding, grinding, soldering, etc).

Consider fire safety during construction throughout the design process

1.4 Safe Design

Safe design principals should be applied throughout the design process and consider the entire building life cycle. Some general guidnce in relation to mid-rise timber buildings is given in this section.

1.4.1 Building Life Cycle

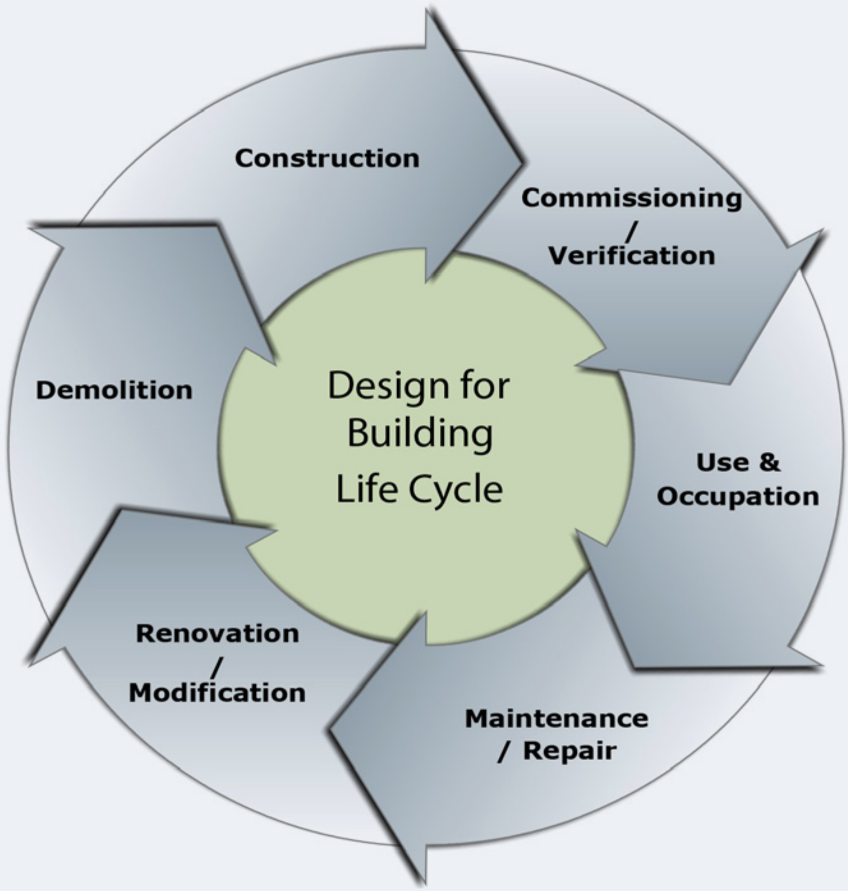


Figure 1.3: Typical building life cycle.

It is important to consider the impacts of design decisions on all phases of the cycle.

For example, the NCC Deemed-to-Satisfy Provisions may require a particular fire safety feature to be incorporated into a building. During the design process it is necessary to determine:

- how the provision can be installed/constructed safely to achieve its required performance
- how the feature will be commissioned and its performance verified
- that the feature will not present a hazard during occupation of a building
- how the feature can be maintained and repaired safely
- measures to be taken to ensure the feature does not present a hazard during renovation/modification or demolition and to ensure that the performance of the feature is not compromised during the renovation/modification process.

Many of these matters lie outside the scope of the NCC but they are addressed through State and Territory Building Acts and regulations and workplace health and safety (WHS) legislation.

For further details on how to address WHS requirements refer to the Safe Work Australia Code of Practice on Safe Design of Structures

1.4.2 Responsibilities for Safe Design

While this Guide focuses on NCC 2019 requirements relating to Deemed-to-Satisfy solutions for mid-rise timber buildings, the NCC provides a uniform set of technical provisions for the design and construction of buildings and other structures throughout Australia. The NCC does not regulate matters such as the roles and responsibilities of building practitioners and maintenance of fire safety measures that fall under the jurisdiction of the States and Territories.

State and Territory building legislation is not consistent in relation to these matters, with significant variations with respect to:

- registration of practitioners
- mandatory requirements for inspections during construction
- requirements for maintenance of fire safety measures.

Workplace Health and Safety (WHS) legislation requires safe design principles to be applied. A *Code of Practice – Safe Design of Structures* published by Safe Work Australia provides guidance to persons who design structures that will be used, or could reasonably be expected to be used, as a workplace. It is prudent to apply these requirements to all buildings as they are generally a workplace for people doing building work, maintenance, inspections and the like.

The Code defines safe design as: *"The integration of control measures early in the design process to eliminate or, if this is not reasonable practicable, minimise risks to health and safety throughout the life of the structure being designed."*

It indicates that safe design begins at the start of the design process when making decisions about:

- the design and its intended purpose
- materials to be used
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal
- what legislation, codes of practice and standards need to be considered and complied with.

The Code also provides clear guidance on who has health and safety duties in relation to the design of structures and lists the following practitioners:

- architects, building designers, engineers, building surveyors, interior designers, landscape architects, town planners and all other design practitioners contributing to, or having overall responsibility for, any part of the design
- building service designers, engineering firms or others designing services that are part of the structure such as ventilation, electrical systems and permanent fire extinguisher installations
- contractors carrying out design work as part of their contribution to a project (for example, an engineering contractor providing design, procurement and construction management services)
- temporary works engineers, including those designing formwork, falsework, scaffolding and sheet piling
- persons who specify how structural alteration, demolition or dismantling work is to be carried out.

In addition, WHS legislation places the primary responsibility for safety during the construction phase on the builder.

The design team in conjunction with owners/operators and the builder have a responsibility to document designs, specify and implement procedures that will minimise risks to health and safety throughout the life of the structure being designed.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides further guidance

1.4.3 Applying Safe Design Principles

A key element of safe design is consultation to identify risks, practical mitigation measures and to assign responsibilities to individuals/organisations for ensuring the mitigation measures are satisfactorily implemented.

This approach should be undertaken whichever NCC compliance pathway is adopted and applies to all forms of construction.

Some matters specific to fire safety are summarised below:

- The NCC and associated referenced documents represent nationally recognised standards for fire safety for new building works.
- The NCC's limited treatment of fire precautions during construction focuses on manual fire-fighting, egress provisions and fire brigade facilities. Additional precautions are required to address WHS requirements such as fire prevention and security. Refer to Section 1.4.4 and *WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings,* for further information.
- Minimising service penetrations through fire-resisting construction.
- Grouping of service penetrations through fire-resisting walls with safe access for installation, inspection and maintenance.
- Detailed design of fire safety measures to optimise reliability and facilitate safe installation, maintenance and inspection where practicable. Special attention should be given to protection of service penetrations and cavity barriers.
- Documentation of procedures and allocation of responsibilities for determining Evidence of Suitability for fire safety measures.
- Documentation of procedures and allocation of responsibilities for the verification and commissioning of all fire safety installations.
- Provision of specifications and drawings of all fire safety measures within the building, Evidence of Suitability, commissioning results and requirements for maintenance and inspection to the owner as part of the fire safety manual. (Note: Some State and Territory legislation contains minimum requirements for inspection of fire safety measures).
- The fire safety manual should also provide information on how to avoid compromising fire safety through the life of a building (e.g. preventing disconnection of smoke detectors or damage to fire resisting construction).

1.4.4 Fire Precautions during Construction

Refer NCC Volume One Cl E1.9 for NCC precautions during construction

Fires may occur on building construction sites due to the nature of the works.

Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Mid-rise timber buildings complying with the NCC 2019 Deemed-to-Satisfy Provisions offer a safe and economical building option. The addition of the fire-protective coverings plays an important role in providing this fire safety and, due to the construction sequencing, there may be a period where the timber is not fully protected and/or automatic fire sprinkler protection is not fully operational. During this period timber buildings are at their highest risk from construction fires.

The builder and design team needs to consider fire precautions during construction. The scope of the NCC is limited to specifying minimum requirements for fire hydrants, hose reels and extinguishers and egress provisions (NCC Clause E1.9).

Consider fire safety during construction throughout the design process

Addressing WHS requires a broad holistic approach that considers the building layout and site layout throughout the construction process to minimise the fire risk at a time when the building could be at its most vulnerable. Typical matters that should be considered include:

- progressive installation of services
- progressive installation of fire-protective grade covering of timber members and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works
- security provisions (to address arson)
- safe access for maintenance of equipment and minimising the down time of fire safety equipment during maintenance
- detailing service penetration and construction interfaces to minimise the risk of cavity fires during installation.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

2

Step 2 – Define NCC Design Requirements for Sound, Thermal Resistance, Damp and Weatherproofing and Structural Tests

Timber building systems can be designed to meet the regulatory requirements of the National Construction Code Volume One (NCC). From a performance perspective, the NCC sound provisions tend to govern the choice of timber building systems more so than the fire provisions due to the lightweight nature of these systems. The design of sound-resisting construction systems involves understanding the NCC Performance Requirements and Deemed-to-Satisfy (DTS) Provisions, then selecting the appropriate timber building systems to meet these requirements. The NCC Provisions are minimum requirements and further consideration may be given as to whether or not the NCC DTS Provisions will meet the expectations of the building occupants.

2.1 Utilising the Deemed-to-Satisfy Provisions for Sound

Part F5 of the NCC is concerned with safeguarding '*occupants from illness or loss of amenity as a result of undue sound being transmitted*'. The NCC Performance Requirements for Class 2 and 3 buildings focus on limiting the transmission of both airborne and impact-generated sound via floor and wall building elements bounding sole-occupancy units (SOUs) where separating:

- adjoining SOUs
- SOUs from a common space (e.g. public corridor)
- SOUs from another building classification within the building.

The sound performance of these floor and wall building elements must consider the impact of any pipe penetrations or other service elements (e.g. air-conditioning) as well as door openings on bounding construction. (Note: The provisions include the sound isolation of pumps but issues pertaining to this are not dealt with in this Guide). When interpreting these requirements, it is important to understand the difference between airborne and impact sound (Figure 2.1).

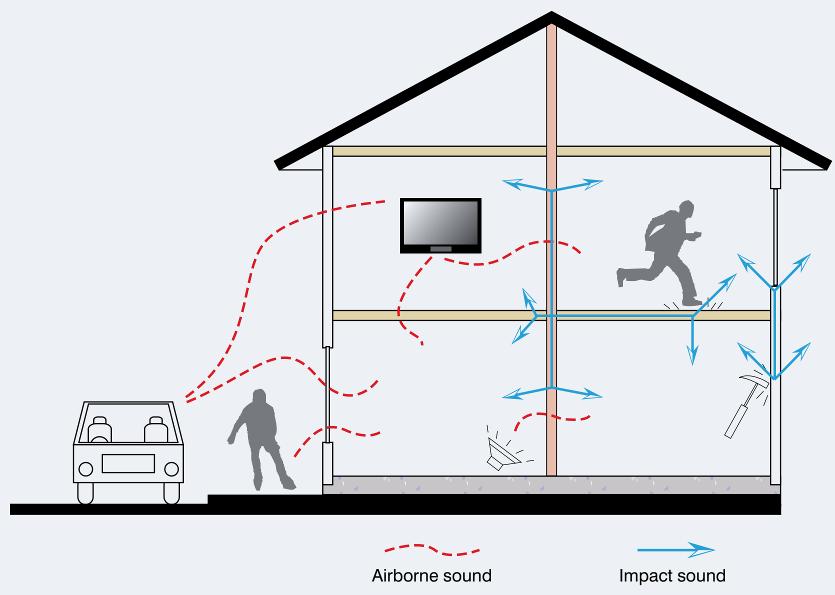


Figure 2.1: Examples of impact and airborne sound.

It is also important to understand how each type of sound is measured in order to select appropriately sound-insulated wall, floor and ceiling elements. The nomenclature used in the Deemed-to-Satisfy Provisions using results from laboratory requirements, is explained in Figures 2.2 and 2.3.
Note: Alternative methods of sound measurement also exist.

Refer to:
NCC Volume One
Cl F5.0 to F5.7

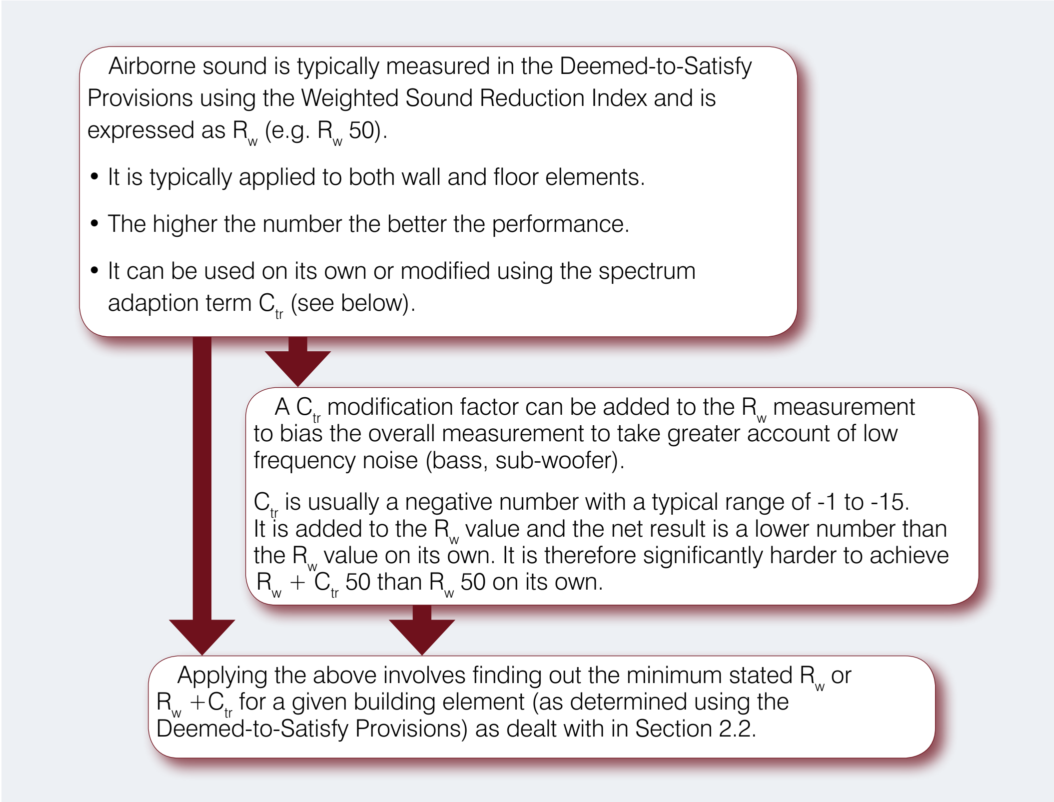


Figure 2.2: Airborne sound.

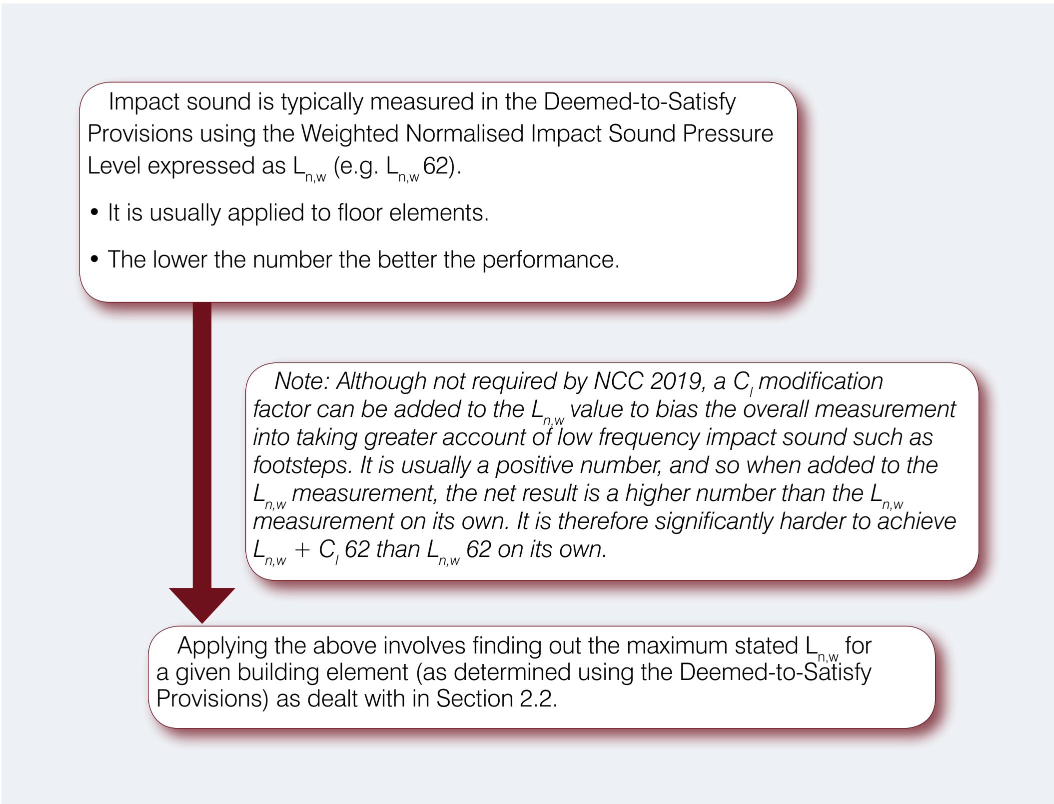


Figure 2.3: Impact sound.

2.2 Determining Sound Insulation Requirements for Individual Building Elements

As previously mentioned, the NCC specifies the minimum airborne and impact sound insulation requirements for individual wall and floor building elements. Tables 2.1 and 2.2 summarise key NCC requirements for the selection of the appropriate timber building systems.

Table 2.1: Walls – Deemed-to-Satisfy Sound Insulation Requirements in Class 2 and 3 Buildings

Wall Situation			Wall Rating	Entry Door Assembly Rating
First Space		**Adjoining space**		
SOU – generally all spaces except those noted below	Separates	**SOU – generally all spaces except those note below**	$R_w + C_{tr} \geq 50$	N/A
A bathroom, sanitary compartment, laundry or kitchen	Separates	**SOU – habitable room[1] (except kitchen)**	$R_w + C_{tr} \geq 50$ and of discontinuous[2] construction	N/A
A bathroom, sanitary compartment, laundry or kitchen	Separates	**SOU – non-habitable[3] room (including kitchen)**	$R_w + C_{tr} \geq 50$	N/A
Plant and lift shaft	Separates	**SOU – all spaces**	$R_w \geq 50$ and of discontinuous[2] construction	N/A
Stairway, public corridor, public lobby or the like or part of a different BCA building classification	Separates	**SOU – all spaces**	$R_w \geq 50$	$R_w \geq 30$ (except a part of a different NCC Building classification)

Notes:

1 Habitable room means a room used for normal domestic activities includes a bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room, home theatre and sunroom.

2. Discontinuous construction refers to walls having a minimum 20 mm gap between separate leaves and with no mechanical linkages between wall leaves except at the wall periphery.

3. Non-habitable rooms are bathroom, laundry, water closet, pantry, walk-in wardrobe, corridor, hallway, lobby, clothes-drying room, and other spaces of a specialised nature occupied neither frequently nor for extended periods. Refer to NCC definition.

Table 2.2: Floors – Deemed-to-satisfy Sound Insulation Requirements in Class 2 and 3 Buildings

Floor Situation			Floor Rating
First Space		**Adjoining space**	
SOU – all spaces	Separates	**SOU – all spaces**	$R_w + C_{tr\ (airborne)} \geq 50$, & $L_{n,w\ (impact)} \leq 62$
Public corridor or lobby or the like	Separates	**SOU – all spaces**	$R_w + C_{tr\ (airborne)} \geq 50$, & $L_{n,w\ (impact)} \leq 62$
Stair and lift shaft	Separates	**SOU – all spaces**	$R_w + C_{tr\ (airborne)} \geq 50$, & $L_{n,w\ (impact)} \leq 62$
Plant Rooms	Separates	**SOU – all spaces**	$R_w + C_{tr\ (airborne)} \geq 50$, & $L_{n,w\ (impact)} \leq 62$
Different BCA Building Classification	Separates	**SOU – all spaces**	$R_w + C_{tr\ (airborne)} \geq 50$, & $L_{n,w\ (impact)} \leq 62$

Where a wall required to have sound insulation has a floor above, the wall must continue to the underside of the floor above, or the ceiling must provide the equivalent sound insulation required for the wall. (Professional advice should be sought to upgrade the ceiling to the required wall sound insulation.)

2.3 Services

If a duct, soil, waste or water supply pipe serves or passes through more than one dwelling, it must be separated from the rooms of the dwellings by construction with an $R_w + C_{tr}$ not less than:

- 40 if it is adjacent to living areas in a dwelling
- 25 if it is adjacent to a kitchen or bathroom.

This is also required where a duct or pipe is within a wall or ceiling cavity.

2.4 The Next Step

The previous information provides an understanding of the NCC's minimum sound-insulation requirements. The next step is to:

- go to Step 3 to find out about improving and/or upgrading sound performance (e.g. beyond minimum NCC requirements); or
- go to Step 5 to select timber building systems that will comply with minimum NCC sound requirements.

Once sound-insulation requirements are satisfied, go to Step 4 Fire Design Requirements.

2.5 Other Design Considerations

There are other design considerations that need to be taken into account in meeting NCC requirements. The following are not covered in detail in this Guide but are listed as requiring consideration.

2.5.1 Thermal Resistance (R-value)

NCC Volume One, Section J provides the energy efficiency requirements that a building, including its services, must achieve. The energy efficiency provisions can be met via the Verification (calculation) Method or complying with the Deemed-to-Satisfy Provisions. These provisions will vary based on a range of factors including: the building's location (climate zone), direction of heat flow, level of external wall/window shading, size and performance of external glazing/windows and form of construction of the external building fabric.

The thermal resistance of timber building elements is dependent on the level of installed insulation (i.e. thickness), number and thickness of sheet lining layers (plasterboard, flooring), element construction (e.g. incorporating furring channels) and overall thickness of the building element.

Energy modelling software is used to simulate the potential thermal efficiency (Class 2) of the building envelope or the annual energy consumption (Class 3) of the building envelope and its services as described in the NCC. Typical external wall and floor/ceiling systems can be 'constructed' within the software packages using their internal product databases. Guidance on timber wall and floor/ceilings can be found in WoodSolutions' ***R-values for Timber-framed Building Elements***.

2.5.2 Damp and Weatherproofing

The requirements for the damp and weatherproofing of buildings are provided in NCC Volume One Part F1. The intent is to protect the building from external (rain) and internal water (e.g. laundry overflow) and the accumulation of internal moisture in a building causing unhealthy conditions for occupants and potential damage to building elements.

Key areas of consideration include:

- Internal wet areas (e.g. bathroom) need to be waterproofed in accordance with the NCC requirements and have adequate overflow systems (e.g. floor waste) in place to deal with the possibility of waste water overflow.
- External walls. There are currently no Deemed-to-Satisfy Provisions in the NCC and therefore suppliers of waterproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). It is important that installed waterproofing membranes/systems are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate (i.e. water barrier) through to the structural timber building elements.
- Roof coverings. For the purposes of this Guide, and as required by the NCC, roof coverings must be of non-combustible materials (e.g. concrete, metal, terracotta) and be fixed in accordance with, and comply with, the relevant Standard as specified in the NCC Clause F1.5.

Note

The drawings in this Guide have either omitted damp, weatherproofing and waterproofing details or provided indicative details only. Specific details may vary with climatic conditions and in many instances the only compliance pathway is a Performance Solution that may yield solutions that vary from project to project. Care should be taken to ensure that the Performance Solutions for damp, weatherproofing and waterproofing do not conflict with NCC fire safety requirements.

2.5.3 Structural Tests

NCC Specification C1.8 describes structural tests for fire-resisting, lightweight wall construction that bounds lift, stair and service shafts, fire-isolated passageways and ramps as well as external and internal walls. The test methods and criteria for compliance are stated in relation to materials, damage, deflection (under static pressure and impact) and surface indentation.

Lightweight wall systems do not require testing if designed and constructed in accordance with the relevant design and loading standards specified in the NCC Part B1 Structural Provisions.

3

Step 3 – Improve and Upgrade Sound Performance

Sound performance can often be improved by simple attention to the form and spatial arrangement of the building design. Attention to flanking noise is another important way to improve sound performance. Many end users of dwellings want higher sound performance than the minimum levels required by the NCC. As a result, this Step in the Guide focuses on ways to improve and upgrade sound performance.

3.1 Attention to Building Design to Reduce Sound Transmission

Aspects of the form and spatial design of a building that can be adapted to improve sound performance are dealt with under the following headings.

3.1.1 Room Layout

Check that the room layout is beneficial rather than detrimental to sound transmission. Service rooms including bathrooms, laundries and kitchens create extra sound compared to living rooms and bedrooms. For instance, water movement through plumbing pipes and the vibration from washing machines and dishwashers create sound problems. It is best for the service rooms in one dwelling to back onto the same type of rooms in an adjoining dwelling rather than habitable rooms such as bedrooms or lounge rooms. Also, try to ensure entrances to dwellings are an appropriate distance from adjacent units (Figure 3.1).

3.1.2 Windows

Windows normally have lower sound insulation than the walls around them. As a result, highly sound-rated bounding wall systems may become ineffective by virtue of nearby poorly sound-rated windows. For improvement, consider one or more of the following:

- use thicker glass or double glazing
- use fixed glazing in lieu of opening windows (this may also require sound-insulated ventilation)
- locate windows so that they do not face noisy areas
- provide adequate separation between windows in adjoining SOUs
- reduce the area of windows in the facade
- fill voids between the wall frame and window frame with an appropriate acoustic sealant
- use acoustic sealing strips/gaskets around the edges of open-able sashes.

3.1.3 Doors

As with windows, doors tend to be a weak link in sound-rated wall systems. Where sound control is desired, solid core doors should be used and be treated with soft acoustic gaskets at interfaces with door jambs. Threshold closers at the bottom of the door or air seals will also help reduce sound transmission. In most cases, achieving the required sound rating will involve the use of gaskets and seals. Sliding doors should be avoided where optimum sound-control is desired.

3.1.4 Services

The location and detailing of services are two of the most important considerations in controlling sound transmission in residential buildings.

Generally, services and service penetrations should not be located on sound-insulated walls between SOUs but rather on internal walls or dedicated sound resisting service shafts. In all instances, service pipes should be located away from noise-sensitive parts of the dwelling, such as bedrooms (Figure 3.1).

3.1.5 External Walls

There are no NCC requirements for sound ratings of external walls, but in some parts of Australia there may be state planning regulations or local government requirements for external wall sound rating. For information on the sound performance of common timber-framed external walls, refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

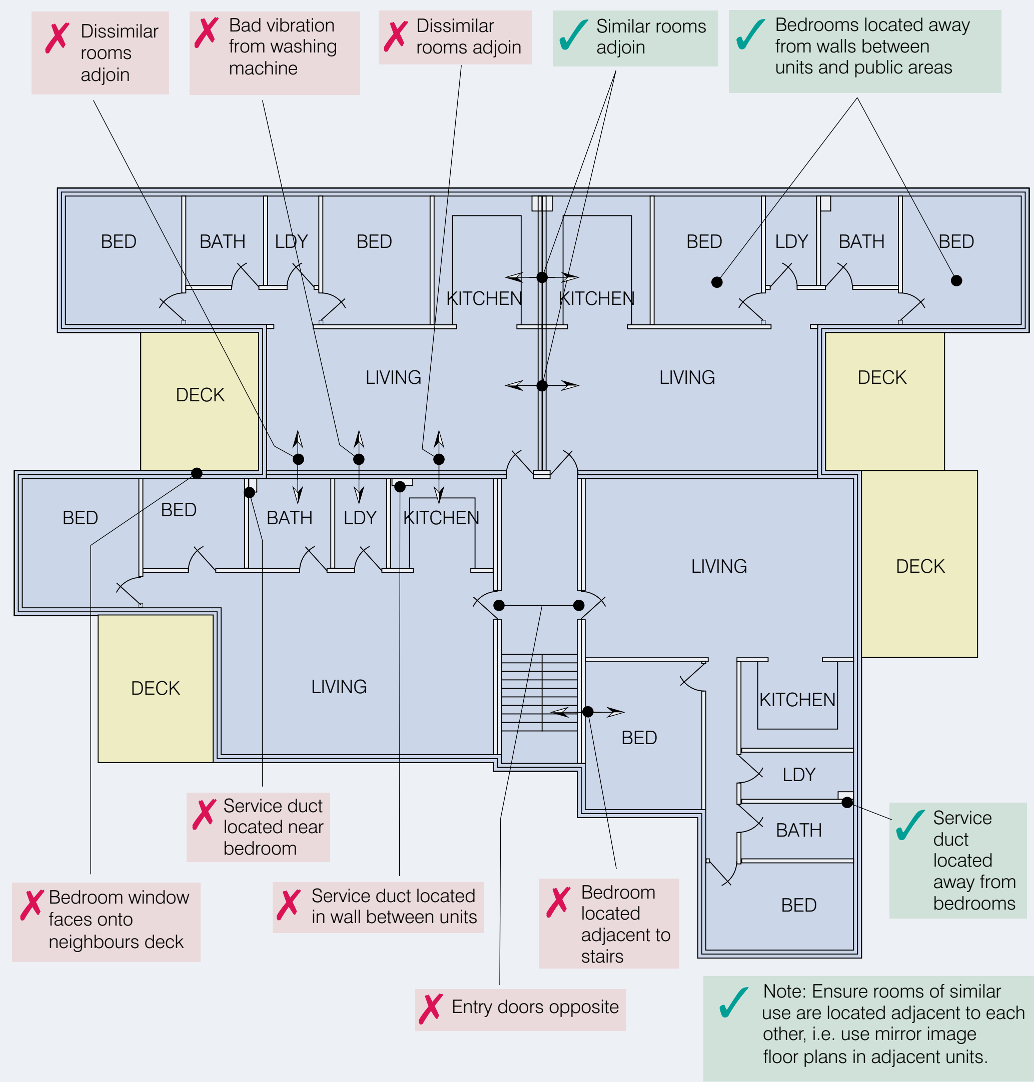


Figure 3.1: Good and bad sound design practices in building layout – plan view.

3.2 Addressing Flanking Noise

The ability to insulate against sound moving from one dwelling to the next depends not only on insulating individual wall and floor elements, but also on stopping noise from jumping or transferring from one building element to the next or, worse still, moving through the building in an uncontrolled way. As a result, the effectiveness of sound-insulated construction is concurrently dependent on addressing flanking noise. Flanking noise refers to sound passing around rather than through wall/floor elements, causing sound to unexpectedly manifest itself in unwanted places.

The main flanking routes around wall and floor elements are shown in Figure 3.2. These routes particularly apply to walls and floors separating SOUs but may also apply to external walls and, in some instances, internal walls (within SOUs) as well.

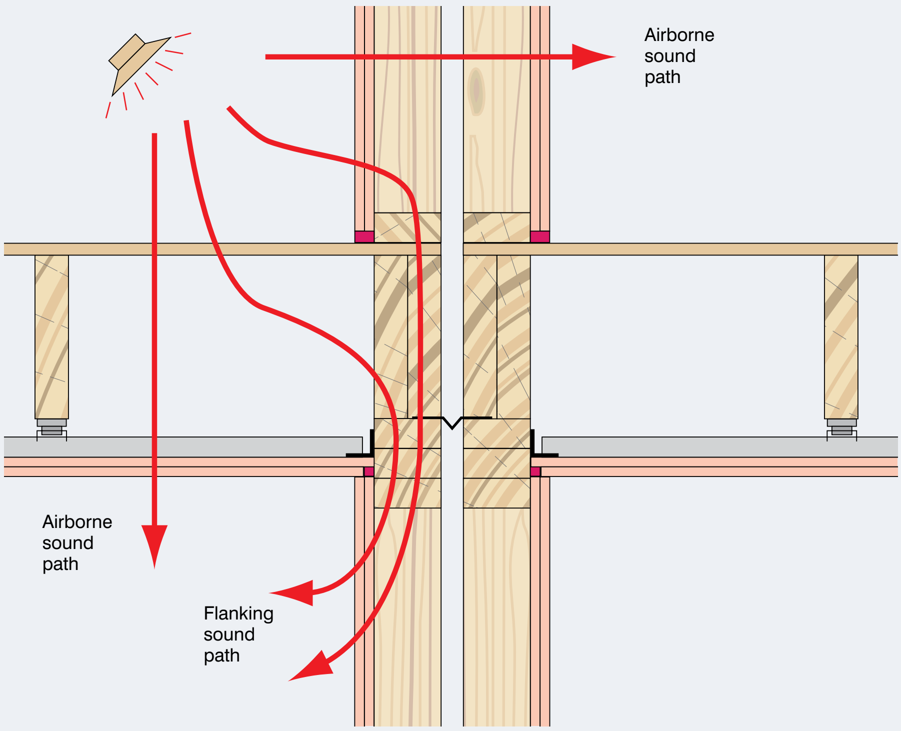


Figure 3.2: Flanking and airborne noise pathways – elevation view.

There are no minimum requirements addressing flanking noise in the NCC's Deemed-to-Satisfy Provisions, though there is an onus on designers and builders to address flanking noise in order to ensure that laboratory-tested wall and floor elements perform to their full potential in the field.

This Guide's approach is to consider reducing flanking noise paths wherever possible. The content is the result of careful thought, taking into account issues such as the limits on what could be achieved in reducing flanking because of their affect on fire and structural integrity. Even though direct reference to reducing flanking noise has not been made, many of the details incorporate elements within them.

An example of reducing flanking noise can be seen in the standard detail for floor joist and flooring over bounding walls where the joist and flooring are not continuous (Figure 3.3).

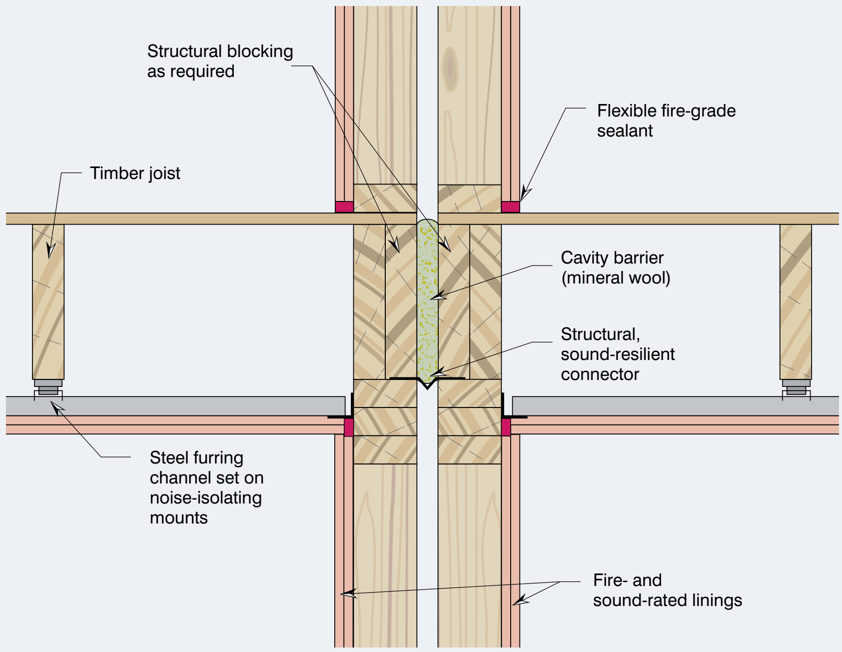


Figure 3.3: Discontinuous floor joist and floor sheeting – elevation view.

There are two main approaches used for addressing flanking noise in timber-framed buildings:

- Limit the ability of the noise to migrate from one element to another, e.g. dampening and isolation at junctions between elements (Figure 3.3).
- Limit the noise getting into wall/floor element, e.g. carpet, floating floors (Figure 3.4).

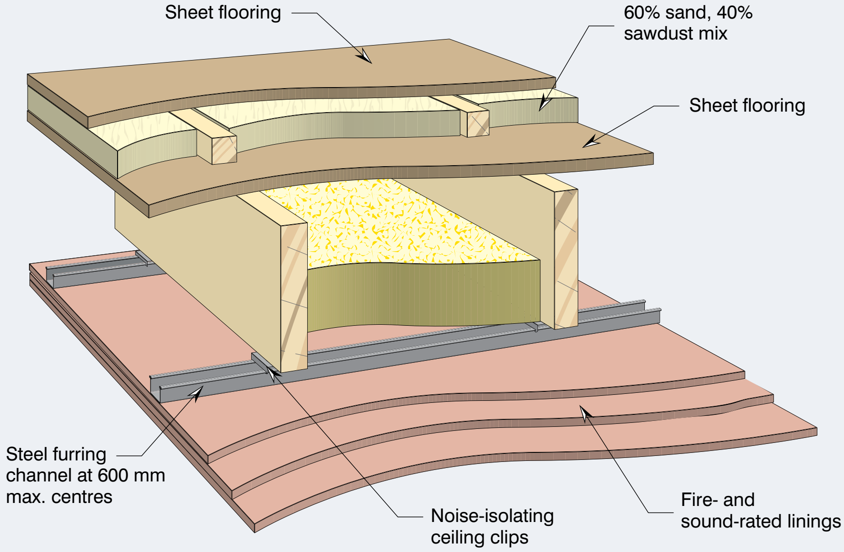


Figure 3.4: Acoustic isolating pad to reduce flanking noise.

In addition to these, timber-framed construction details orientated to improving flanking sound are provided in Section 3.3 and include:

- discontinuous elements at walls, floors and ceilings
- cavities within sound rated elements blocked or travel path increased to reduce noise
- introduced isolating elements, e.g resilient mats or brackets
- platform flooring discontinuous over double stud walls.

3.3 Strategies for Upgrading Sound Performance in Construction

Building occupants often desire higher sound performance than the NCC's minimum requirements. This is especially the case for impact sound and the related issue of vibration from footsteps, water movement through pipes, water hammer and sources such as washing machines, air conditioning units and dishwashers. Other scenarios not dealt with in the NCC include acoustic requirements for home entertainment areas, noise transfer within a dwelling and noise from outside the building (e.g. busy roads, trains, aircraft noise). Options for upgrading typical construction are provided below. Using a combination of options is more likely to give the best performance.

Isolating one side of a bounding construction from the other (e.g. using double stud cavity wall construction). This is also known as decoupling and can be useful in reducing both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.

Avoiding rigid connections between the opposing sides of isolated (decoupled) elements. This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at floor or ceiling level.

Using absorptive materials to fill wall and floor cavities (non-combustible glass fibre or mineral wool) can reduce airborne sound transmission.

Sealing sound leaks at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

For information for the upgrade of external walls refer to *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise.*

3.3.1 Walls

Extra mass on the walls – the addition of mass is a simple yet effective way to improve sound performance in timber construction. In its simplest form, it involves adding extra layers of material such as plasterboard to the outer layer of the sound-rated wall system.

Use a 90 mm rather than 70 mm wall studs – The wider the wall, the better its sound performance. This is particularly the case where trying to improve C_{tr} scores (being the modification factor for low frequency bass noise applied to R_w scores). The simplest way to do this is to use 90 mm wide studs instead of 70 mm wide studs in a double stud wall system.

Upgrade batts in the wall/floor – There are many types and grades of non-combustible insulation batts in the market place. Sound insulation specific batts are best and high-density materials tend to outperform low-density materials. Always refer to the supplier's documented recommendations; some systems require insulation or linings to affect different frequencies and therefore may have differing advice.

3.3.2 Floors

Extra mass on the ceilings – adding mass is a simple yet important way to improve sound performance in timber-framed construction. At its simplest manifestation, this involves adding extra layers of material such as plasterboard to the sound-rated ceiling system.

Extra mass on floors – the addition of mass on floors is an effective way to address impact noise (e.g. footsteps). The additional mass can be in the form of additional layers of sheet flooring.

3.4 The Next Step

The strategies and methods shown in this Step of the Guide may involve specialist proprietary systems that go beyond the scope of this publication. As a result, the next step is:

- Go to proprietary system suppliers and ask for advice on how to integrate their systems with those discussed in this Guide. As part of this, care must be taken to ensure that the fire and sound performance of systems in this Guide are not compromised in any way;
- Go to Step 4 to find out about fire-resisting construction requirements so that these requirements can be considered in tandem with sound requirements before selecting the appropriate timber construction system in Step 5.
- Go to Step 5 to select timber construction that will comply with minimum NCC sound and fire requirements.

4

Step 4 – Define NCC Fire Design Requirements (Design Development)

Designing fire-resisting construction involves a process of understanding how the NCC's Performance Requirements translate into the more objective and measurable Deemed-to-Satisfy Solutions for mid-rise timber buildings, prior to finalising the building layout and selecting timber construction systems that meet these requirements.

4.1 Utilising the Deemed-to-Satisfy Solutions for Fire Design

Section C of the NCC Volume One is concerned with safeguarding people if a building fire occurs. Specific attention is given to evacuating occupants, facilitating the activities of emergency services personnel, avoiding the spread of fire between buildings, and protecting other property from damage as a result of fire.

The NCC details Deemed-to-Satisfy (DTS) Solutions that satisfy the Performance Requirements under:

Part C1 – Fire-resistance and stability

Part C2 – Compartmentation and separation

Part C3 – Protection of openings

These Parts deal with a wide range of issues but it is primarily the fire-resistance of building elements and provisions that relate specifically to mid-rise timber buildings that are dealt with in this Guide. To this end, only relevant clauses from Parts C1, C2 and C3 are discussed in more detail below together with the following provisions that apply specifically to mid-rise timber buildings:

- protection of the building with an automatic fire sprinkler system complying with Specification E1.5 of the NCC
- fire-protected timber complying with Specification C1.13a of the NCC used for loadbearing internal walls, loadbearing fire walls and for elements of construction required to be non-combustible
- any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible
- cavity-barriers provided in accordance with Specification C1.13 of the NCC.

The NCC Deemed-to-Satisfy Provisions that facilitate the construction of mid-rise timber buildings are shown in Figure 4.1.

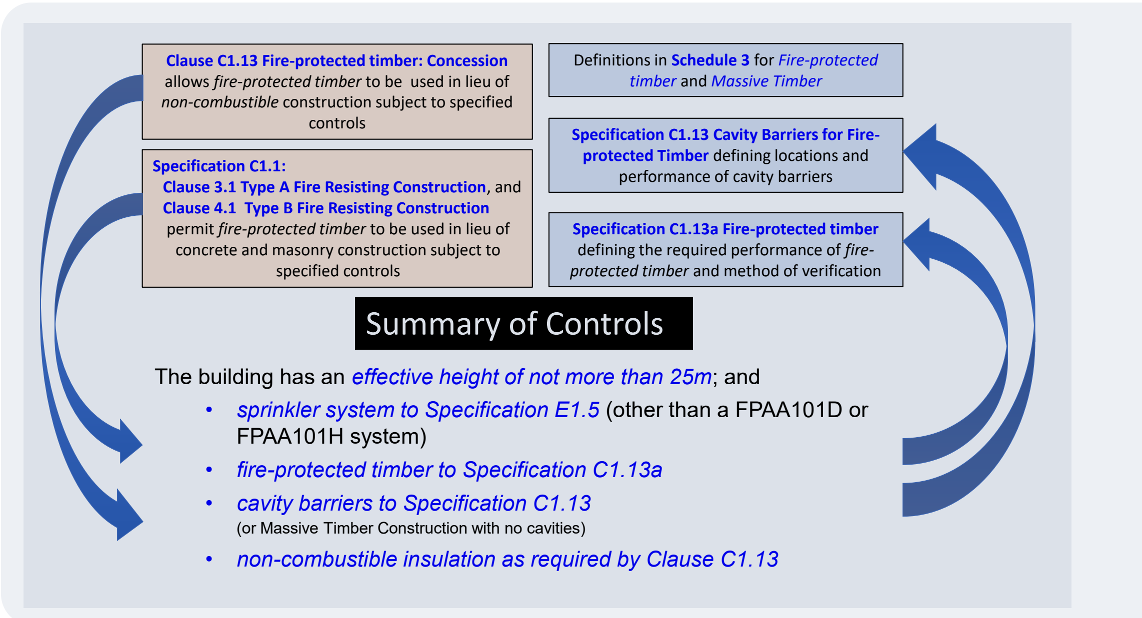


Figure 4.1: Mid-rise timber buildings overview of NCC DTS provisions.

Under Vic Spec E1.5.2 the sprinkler protection requirements are modified. Refer NCC Vic Spec E1.5.2 for projects constructed in accordance with the Victorian Acts and Regulations

4.2 Automatic Fire Sprinklers

A key fire safety feature for mid-rise timber buildings is the requirement to provide automatic fire sprinkler systems in accordance with NCC Specification E1.5 throughout the building, including any parts of the building that are not of timber construction. This requirement, in conjunction with other fire safety measures, is expected to reduce the risk from fires in mid-rise timber buildings below that in other forms of construction complying with the minimum NCC requirements that do not incorporate sprinkler systems.

4.2.1 Sprinkler Design Standards permitted by NCC Specification E1.5

Specification E1.5 allows sprinkler systems to be designed in accordance with:

- AS 2118.1: Automatic Fire Sprinkler Systems – General Requirements
- AS 2118.4: Automatic Fire Sprinkler Systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height
- AS 2118.6: Combined sprinkler and hydrant systems in multi-storey buildings.

AS 2118.4 is limited to accommodation (residential) buildings not exceeding four storeys high. Therefore, most mid-rise timber building sprinkler systems will be designed to comply with AS 2118.1 or AS 2118.6.

Note: FPAA101D and FPAA101H systems are not currently permitted to be used in conjuction with fire-protected timber systems in accordance with the NCC DTS provisions. However, the use of the FPAA systems could be considered as part of a Performance Solution.

4.2.2 Designing Fire Sprinkler Systems to Improve Their Effectiveness

There are opportunities during the design process to incorporate features that can enhance the effectiveness of an automatic sprinkler system and simplify ongoing maintenance. A few examples are described below.

Residential heads in residential SOUs and associated corridors

Both AS 2118.1 and 2118.6 allow the use of appropriately listed residential heads in residential building SOUs and associated corridor areas. These heads have a more rapid response than standard heads and are more likely to suppress rather than control a fire thus reducing the risk to occupants within the SOU of fire origin. Therefore, where appropriate, residential heads should be specified.

Monitored valves

The reliability of fire sprinkler systems can be enhanced by providing monitored components, such as main stop valves and subsidiary stop valves. Monitored stop valves on each floor, for example, enables sprinkler protection to be maintained throughout the remainder of the building while work is undertaken on part of the sprinkler system. If the valve is left closed when the work is completed, the building owner/operator can be alerted to ensure the error is quickly corrected. This minimises the time periods and extent of areas where sprinkler protection is unavailable. The progressive installation of monitored valves during construction can be used as part of the strategy to address fires during construction by facilitating the progressive commissioning of the sprinkler system.

False ceilings

If sprinkler pipes are run above a ceiling system that is required to have Resistance to the Incipient Spread of Fire (RISF), the ceiling will need to be penetrated to accommodate sprinkler heads; potentially compromising the fire performance of the ceiling if the sprinkler system fails to operate successfully.

This can be avoided by providing a false ceiling and running the pipes below the RISF ceiling and the penetrations for the sprinkler heads need only penetrate the non-fire-resisting false ceiling.

This detail also provides flexibility for lighting systems, air-conditioning and other services.

Selection of materials and pipe connections

The use of materials and pipework connections that minimise the need for hot works on site and reduce the time the sprinkler system is not in operation during maintenance should be considered.

Protection of voids/concealed spaces

Concealed spaces within fire-protected timber elements greater than 200 mm deep generally require protection in accordance with AS 2118.1 and AS 2118.6. Where these voids include elements such as beams, the void depth is measured from the soffit of the beam.

Where open web beams (trusses) or similar elements are included in the cavity, consider providing sprinkler protection where the distance between a ceiling and the bottom chord is less than 200 mm because open webs will not obstruct the sprinkler discharge to the same extent as solid beams.

4.2.3 NCC Specification E1.5a Requirements and Concessions

Overview

NCC Specification E1.5a was introduced in the 2019 edition of the NCC and sets out additional requirements for Class 2 and 3 buildings not more than 25 m in *effective height* with a *rise in storeys* of 4 or more relating to:

- the design and installation of fire sprinkler systems
- detection and alarm systems
- fire orders.

If these additional requirements are implemented the following significant concessions are permitted to be applied:

- reduced FRLs for some fire doors and service penetrations and some non-loadbearing walls
- extended distances of travel in Class 2 and 3 buildings except for residential care buildings
- modifications to requirements for internal fire hydrants
- modifications to requirements for emergency warning and intercom systems (EWIS) in residential care buildings.

The application of Specification E1.5a to fire-protected timber buildings is explained below.

Sprinkler Systems complying with AS 2118.1 or AS 2118.4 as applicable are required to be provided in mid-rise fire protected buildings and therefore the requirements of Specification E1.5a 2 (a) for sprinkler protection are satisfied.

Note: Victorian variation Vic Spec E1.5a 2(a) applies more stringent requirements to sprinkler protection of covered balconies that must be adopted if compliance with the Victorian Building Acts and Regulations is required.

Spec E1.5a 2(b) requirements reflect good practice and are recommended to be applied even if the Spec E1.5a concessions are not adopted.

Specification E1.5a 2(b) Requirements

(i) the *automatic* fire sprinkler system is permanently connected to a fire alarm monitoring system connected to a fire station or fire station dispatch centre in accordance with Specification E2.2d if –

(A) the system has more than 100 sprinkler heads; or

(B) in the case of a *residential care building*, the building will accommodate more than 32 residents; and

(ii) the *automatic* fire sprinkler system is fitted with sprinklers complying with clauses 4.4, 4.5 and 5.5.2 of AS 2118.4 in bedrooms; and

(iii) an *automatic* smoke detection and alarm system is installed in accordance with Specification E2.2a except that it need not be connected to a fire alarm monitoring system connected to a fire station or fire station dispatch centre, and in the case of a *residential care building* it must be installed in accordance with –

(A) Specification E2.2a Clause 4; or

(B) both –

(aa) Specification E2.2a Clause 3, provided Specification E2.2a Clause 3(a)(ii) is applied as if the building was not protected with a sprinkler system; and

(bb) Specification E2.2d; and

(iv) in a *residential care building*, the *automatic* smoke detection and alarm system and the automatic fire sprinkler system are connected to a local fire indicator panel provided in accordance with Specification E2.2d; and

(v) fire orders are provided in a Class 3 building in accordance with G4.9 as for a building in an *alpine area*.

It is recommended that fire orders are provided in all Class 2 and 3 buildings to facilitate a prompt evacuation.

G4.9 Fire orders states:

Every Class 2, 3 or 9 building must display a notice clearly marked “FIRE ORDERS” in suitable locations near the main entrance and on each *storey*, explaining:

(a) the method of operation of the fire alarm system and the location of all call-points; and

(b) the location and methods of operation of all fire-fighting equipment; and

(c) the location of all *exits*; and

(d) the procedure for evacuation of the building.

Specification E1.5a 3(a) Concessions

(i) The FRL for *self-closing* fire doors, as *required* by C3.8 and C3.11, may be reduced to not less than -/30/30.

C3.8 applies to doorways that open to fire-isolated stairways, fire-isolated passageways or fire-isolated ramps, and are not doorways opening to a road or open space.

C3.11 applies to:

(a) doorways providing access from a sole-occupancy unit to:

- a public corridor, public lobby, or the like; or
- a room not within a sole-occupancy unit; or
- the landing of an internal non fire-isolated stairway that serves as a required exit; or
- another sole-occupancy unit.

(b) doorways providing access from a room not within a sole-occupancy unit to:

- a public corridor, public lobby, or the like; or
- the landing of an internal non-fire-isolated stairway that serves as a required exit.

(ii) The FRL for—

(A) service penetrations through non-*loadbearing internal walls* and *shafts* constructed of *fire-protected timber,* as *required* by C3.15, may be reduced to not less than -/60/15;

Notes:

The Resistance to Incipient Spread of Fire (RISF) and Modified Resistance to Incipient Spread of Fire (MRISF) requirements for service penetrations through fire-protected timber still apply.

The base non-loadbearing fire-protected timber wall system must be capable of achieving an FRL of at least -/60/60 in addition to satisfying the RISF or MRISF requirements of Specification C1.13a.

(B) all other non-*loadbearing internal walls*, as *required* by Specification C1.1, may be reduced to -/45/45 and the FRL for service penetrations through internal non-*loadbearing* walls and *shafts*, as *required* by C3.15, may be reduced to -/45/15

Note: A useful detail in buildings of predominantly fire-protected timber construction is to construct solid non-loadbearing service shafts from laminated non-combustible boards only since this form of construction will reduce the space required for service shafts. The form of construction will need to comply with all relevant NCC requirements including acoustics and impact resistance in addition to fire related properties.

(iii) The FRL for *fire-isolated stairways* enclosed with non-*loadbearing* construction, as required by D1.3, may be reduced to –/45/45.

Note: If fire-protected timber construction is adopted it is still necessary to comply with the Resistance to the Incipient Spread of Fire and Modified Resistance to the Incipient Spread of Fire criteria as appropriate. To satisfy these requirements it is expected that an FRL of at least –/60/60 would be achieved by the wall system.

The distance of travel concession facilitates the design of more efficient floor layouts

(iv) Except in a *residential care building*, the maximum distance of travel, as *required* by D1.4(a)(i)(A), may be increased from 6 m to 12 m.

(v) The maximum distance of travel from a single *exit* serving the *storey* at the level of egress to a road or *open space*, as *required* by D1.4(a)(i)(B), may be increased from 20 m to 30 m.

(vi) The maximum distance between alternative *exits*, as *required* by D1.5(c)(i), may be increased from 45 m to 60m.

Note: The relaxations to the distance of travel requirements facilitate the design of more efficient floor layouts

(vii) Internal fire hydrants in accordance with E1.3 are not *required* where—

(A) the building is served by external fire hydrants that provide compliant coverage installed in accordance with E1.3, except that in a *residential care building* the nozzle at the end of the length of hose need only reach the entry door of any *sole-occupancy unit* to be considered as covering the area within the *sole-occupancy unit*; or

(B) a dry fire hydrant system that otherwise complies with AS 2419.1 is installed in the building and—

(aa) each fire hydrant head is located in accordance with E1.3 and fitted with a blank end cap or plug; and

(bb) the pipework is installed in accordance with E1.3 (as for a *required* fire main) except that it need not be connected to a water supply; and

(cc) a hydrant booster inlet connection is provided in accordance with E1.3; and

(dd) an external street or feed hydrant capable of providing the *required* system flow is located within 60 m of the hydrant booster connection.

(viii) An emergency warning and intercom system need not be provided in a *residential care building* in accordance with E4.9 if a warning system with an override public address facility is installed in accordance with Specification E2.2d.

4.3 Fire-Protected Timber Requirements

Refer NCC Spec C1.13a for fire-protected timber requirements

The NCC defines fire-protected timber as fire-resisting timber building elements that comply with Specification C1.13a.

4.3.1 Fire-Protected Timber – General Requirements

Specification C1.13a applies the following General Requirements to fire-protected timber:

- the building element must be protected to achieve the required FRL
- a non-combustible fire-protective covering must be applied to the timber; it must achieve a Resistance to the Incipient Spread of Fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

Note: The NCC Clause C1.9(e) permits some materials, including plasterboard and fibre-reinforced cement sheeting, to be used wherever a non-combustible material is required.

To adequately specify or check Evidence of Suitability of a fire-protected timber element, three items of information are required:

Refer NCC Schedule 5 for FRL

Refer NCC A5.2 and AS 1530.1 for non- combustibility

Refer NCC A5.6 for RISF

- Fire-resistance Level (FRL) – determined from AS 1530.4 test or an equivalent or more severe test.
- Resistance to the Incipient Spread of Fire (RISF) – determined from AS 1530.4 test or an equivalent or more severe test.
- Results from a non-combustibility test in accordance with AS 1530.1 – for materials not deemed non-combustible by the NCC.

(i) FRL is the grading period in minutes for the following three criteria expressed in the order listed below separated by forward slashes (/).

- structural adequacy – ability of a loadbearing element to support an applied load
- integrity – ability of an element of construction to resist the passage of flames and hot gases from one space to another
- insulation – ability of the surface of an element of construction, on the non-fire side of the element, to maintain a temperature below the specified limits.

For example, if an FRL of 90/60/30 is specified the element would need to satisfy the structural adequacy criteria for 90 minutes, the integrity criteria for 60 minutes and the insulation criteria for 30 minutes. A dash means that there is no requirement for that criterion, i.e. an FRL of 90/–/– means that only the criterion of structural adequacy applies for 90 minutes.

(ii) The RISF in relation to a fire-protective covering means the covering's ability to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits.

(iii) A material is classified as non-combustible if flaming is not observed and specified temperature rise limits are not exceeded when a sample of material is exposed to the heating conditions specified in AS 1530.1 or it is deemed non-combustible in accordance with NCC Clause C1.9(e).

To facilitate a consistent approach to specifying the performance of fire-protected timber the following format is recommended.

Fire-Protected Timber – FRL90/90/90:RISF45:NC

This means that the element must satisfy the structural adequacy, integrity and insulation requirements for 90 minutes; the RISF criteria for 45 minutes and the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

While individual test/assessment reports from Accredited Testing Laboratories can be used as Evidence of Suitability, it may be more practical for Accredited Testing Laboratories to provide consolidated reports stating the performance in the above format.

Further information relating to the test procedures to determine the FRL and RISF are provided in Appendix A.

__NCC Spec C1.13a includes some Deemed-to-Satisfy fire-protective covering systems based on fire-protective grade plasterboard__

Cavities are permitted within fire-protected timber elements that, without adequate measures in place, can allow fire spread through concealed spaces. The risk of fire spread from enclosure fires to the cavities is substantially reduced by the requirement for an RISF45 applied to the fire-protective covering, among other things. There is a small residual risk of fire spread to the cavity from an enclosure fire or a fire start within a cavity due to hot works, for example. The risk of fire spread via concealed spaces is further reduced by the provisions for cavity barriers and requirements for wall/ceiling cavity insulation, if present, to be non-combustible.

Specification C1.13a deems two layers of 13 mm fire-protective grade plasterboard fixed in accordance with manufacturer's system requirements to achieve equivalent performance to an RISF45:NC fire-protective covering.

The timber-framed wall system in Figure 4.2 with two layers of 13 mm fire-protective grade plasterboard either side of a cavity between studs could be classified as fire-protected timber FRL90/90/90:RISF45:NC if Evidence of Suitability (as required by the NCC) is provided for the loadbearing wall system to verify that it achieves an FRL of 90/90/90 under similar or more severe load conditions.

This evidence would normally be an AS 1530.4 fire test report from an Accredited Testing Laboratory. The RISF45 for two layers of 13 mm thick fire-protective grade plasterboard does not require further verification since it is Deemed-to-Satisfy the 45 minute requirement and plasterboard is also deemed non-combustible by Clause C1.9(e) of the NCC.

__If fire-protective coverings satisfy the NCC Deemed-to-Satisfy provisions for RISF and non-combustibility it is only necessary to provide Evidence of Suitability to verify the required FRL__

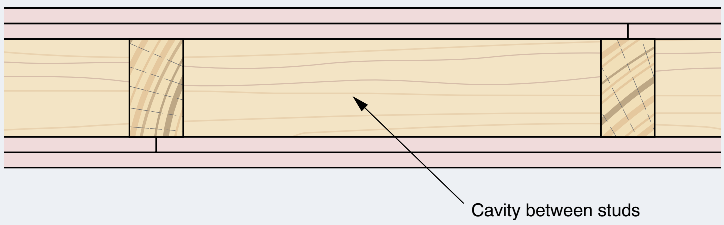


Figure 4.2: Horizontal section through typical FRL90/90/90:RISF45:NC timber stud wall showing allowable cavity.

Ensure the RISF is not compromised by service penetrations, doors and other openings and connections

The primary objective for the inclusion of the non-combustibility requirement for the fire-protective covering is so that the reaction of the fire-protected timber to external and enclosure fires is comparable to elements of construction that are non-combustible: such as reinforced concrete or steel protected with non-combustible materials.

The primary objective for the specification of RISF45 is to reduce the risk of the timber structural elements being ignited prior to burn-out of the contents or fire brigade intervention, in the unlikely event of the automatic fire sprinkler system failing. To achieve this, it is necessary that the RISF performance is not compromised by the presence of building service penetrations and openings for doors and windows. Refer to the relevant sections in this Step for further details on how the RISF performance can be maintained through appropriate penetration fire stopping systems, cavity barriers and lining of openings.

4.3.2 Massive Timber Panels

Subject to compliance with specific requirements, the RISF criteria can be 'relaxed' for massive timber panels in recognition of the higher inherent fire-resistance and mitigation of the risk of cavity fires

The NCC permits the General Requirements for fire-protected timber to be 'relaxed' if both the following additional criteria are satisfied:

- the minimum thickness of timber panels is not less than 75 mm
- there are no cavities between the surface of the timber and the fire-protective covering system or between timber members.

This 75 mm dimension relates to the minimum dimension of the dressed/finished timber member. In most instances, massive timber elements will have minimum dimensions much greater than 75 mm to meet the structural adequacy and integrity criteria of AS 1530.4.

Typical examples of massive timber installations satisfying the conditions are shown in Figure 4.3.

The rationale for allowing the 'concession' for massive timber is that it is reasonable to reduce the performance of the fire-protective covering, subject to maintaining the required FRL, because the consequences of ignition of timber structural members are significantly reduced:

- Timber with a large cross-section can achieve high fire-resistance levels due to its relatively high inherent fire resistance allowing it to continue to support an imposed load or maintain a fire separating function for significant periods. If there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for a greater period than lightweight construction.
- By not permitting any concealed spaces between the timber and fire-protective coverings or between timber members, the risk of fire spread through concealed cavities is mitigated.

If the massive timber conditions are met, the following requirements can be adopted for fire-protected timber in lieu of the General Requirements:

- The building elements must be protected to achieve the required FRL and have a non-combustible fire-protective covering applied to the timber which achieves the Modified Resistance to the Incipient Spread of Fire (MRISF) of not less than the values stated in Table 4.1 when tested in accordance with AS 1530.4.
- The Modified Resistance to the Incipient Spread of Fire is determined in accordance with Clause 3 of NCC Specification C1.13a. Further information relating to the test procedures to determine the Fire Resistance and the MRISF are provided in Appendix A.

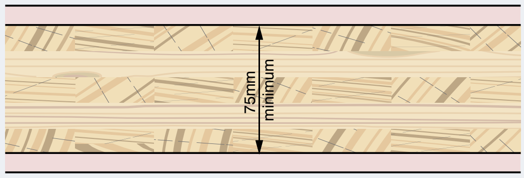
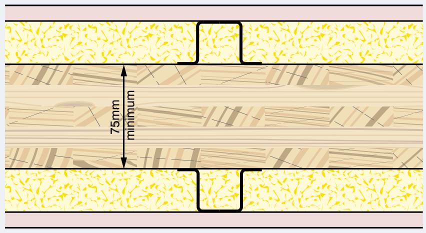
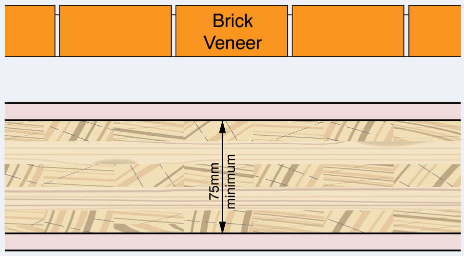
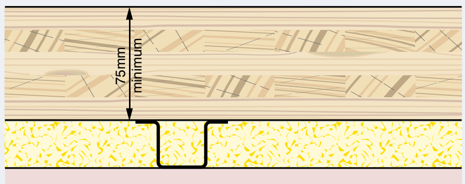
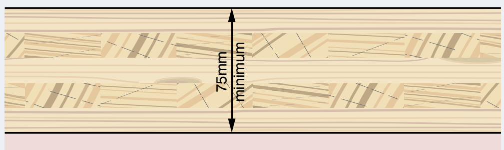
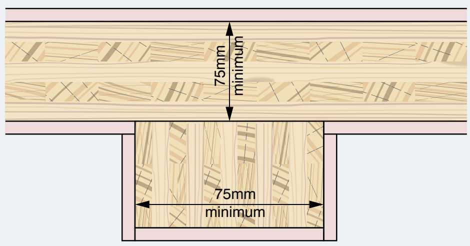
Description	Schematic section
Massive Timber Wall Panels	
Fire-protective covering direct fix to massive timber panel.	
Multi-layer fire-protective covering system direct fix to massive timber panel.	
External brick veneer wall – Note massive timber is faced on both sides with fire-protective coverings.	
Massive Timber Floor Panels	
Multi-layer fire-protective covering system direct fixed to the underside of the massive timber panel.	
Fire-protective covering direct fix to massive timber panel.	
Profiled ceiling fire-protective covering system follows the floor profile maintaining contact with massive timber element.	

Figure 4.3: Typical massive timber panel details for which the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria may be applied.

To facilitate a consistent approach to specifying the performance of fire-protected massive timber the following format is recommended.

Fire-Protected Timber – FRL90/90/90:MRISF30:NC

This means that the element must satisfy:

- the structural adequacy, integrity and insulation requirements for 90 minutes
- the Modified Resistance to the Incipient Spread of Fire criteria for 30 minutes
- the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or comply with the requirements of the NCC Clause C1.9(e).

Table 4.1: Fire-protective covering requirements – Massive timber.

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)	Minimum Deemed-to-Satisfy Fire-protective Grade Plasterboard
Inside a fire-isolated stairway or lift shaft	20 min	1 layer x 13 mm thick
External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	45 min	2 layers x 13 mm thick
All other applications	30 min	1 layer x 16 mm thick

Table 4.1 also includes Deemed-to-Satisfy fire-protective grade plasterboard minimum requirements if fixed in accordance with the manufacturer's system requirements in order to achieve the required FRL of the element for massive timber.

For example, if a non-loadbearing wall system is required to achieve an FRL of -/60/60, an appropriate specification for a massive timber element would be:

Fire-Protected Timber – FRL–/60/60:MRISF30:NC

If there is appropriate Evidence of Suitability to show a massive timber element can achieve an FRL of -/60/60 when protected by 16 mm fire-protective grade plasterboard, then no further evidence is required since the 16 mm thick plasterboard is Deemed-to-Satisfy the MRISF30 requirement and the plasterboard is also deemed to be non-combustible.

4.3.3 Fire-Protected Timber Element Requirements for Mid-Rise Class 2 and 3 Buildings of Timber Construction (General Requirements)

Mid-rise Class 2 and 3 (residential) buildings are typically more than 3 storeys high and are therefore required to be of Type A construction as designated by the NCC Volume One.

Having determined the Type of Construction for the building, it is possible to determine the fire-protected timber requirements for various wall, floor, ceiling and other building elements.

A typical mid-rise timber apartment building layout is shown in Figure 4.4 and Figure 4.5 with fire-protected timber elements. The FRLs and RISF requirements for these elements are summarised in Table 4.2.

Specification E1.5a allows significant concessions to be applied if supplementary requirements are satisfied as described in Section 4.2.3. Since the additional requirements are relatively minor and reflect good practice it is likely that in many applications the concessions will be adopted. Reductions in the FRLs of fire-protected timber constructions permitted by specification E1.5a are shown in brackets in Table 4.2.

For external walls refer to Section 4.6.

Refer NCC Volume One Specification C1.1 for required FRLs and Specification C1.13a for RISF requirements

Refer NCC Volume One Specification E1.5a for Concession option for sprinkler protected buildings

Table 4.2: FRL and RISF General Requirements for timber-framed construction in mid-rise Class 2 and 3 buildings.

Symbol	Description	FRL – Structural Adequacy /Integrity/ Insulation – min		Resistance to the Incipient Spread of Fire (min.)
		Loadbearing	Non-loadbearing[4]	
	Fire stair shaft	90/90/90	–/90/90 (–/60/60)	45
	Service Shaft	90/90/90	–/90/90 (–/60/60)	45
	Bounding walls – SOUs and public corridors etc	90/90/90	–/60/60	45
	Lift Shaft walls	90/90/90	–/90/90 (–/60/60)	45
	Door to fire Stair	Not applicable	–/60/30 (–/30/30)	Not applicable
	Fire Door to service shaft	Not applicable	–/60/30	Not applicable
	Door to SOU	Not applicable	–/60/30 (–/30/30)	Not applicable
	Lift door	Not applicable	–/60/–	Not applicable
	Fire doors to services risers[1]	Not applicable	–/60/30	Not applicable
	Non-loadbearing walls within an apartment	Not applicable	–/–/–	-
	Floors	90/90/90	Not applicable	45

Refer Specification C1.1 of NCC Volume One Cl 3.5 for the roof concession

Note 1: Riser doors may not require an FRL if service penetrations are fire stopped at floor level.

Note 2: Since the roof will have a non-combustible covering and mid-rise timber buildings are required to be sprinkler-protected throughout, the roof is not required to achieve an FRL.

In addition to the above requirements, the fire-protective coverings must also be non-combustible.

Note 3: Service penetrations through fire-protected timber elements permitted to achieve FRL of –/60/15 under the Specification E1.5a concession.

Note 4: Concessions for FRLs for non-loadbearing elements permitted by Specification E1.5a are shown in brackets.

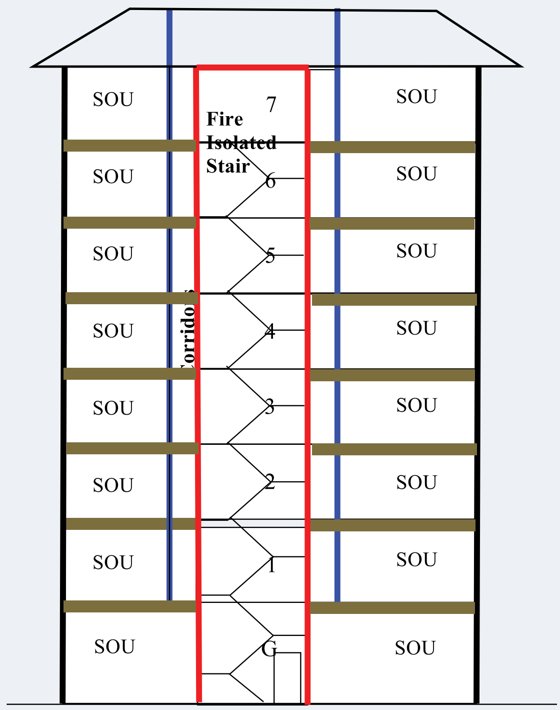


Figure 4.4: Typical section through a mid-rise apartment building.

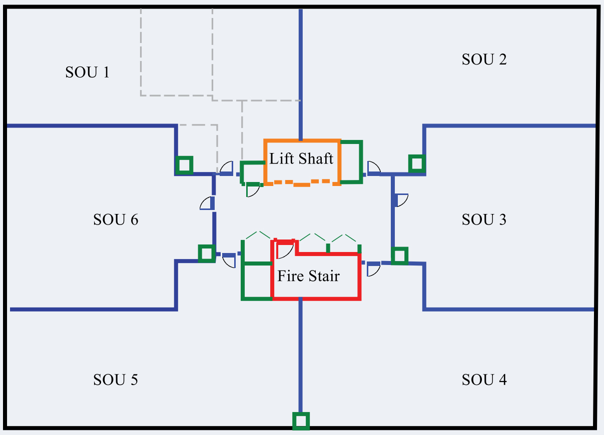


Figure 4.5: Plan of a typical apartment building floor.

4.3.4 Fire-Protected Timber Element Requirements for Mid-Rise Class 2 and 3 Buildings of Timber Construction using Massive Timber

Refer NCC Volume One Specification C1.1 for required FRLs and Specification C1.13a for MRISF requirements

The Massive Timber requirements, as summarised in Table 4.4, can only be applied if both the minimum element size and cavity restrictions are satisfied. If these conditions are not fully satisfied for an element, the General Requirements must be applied.

A typical mid-rise timber apartment building layout is shown in Figure 4.4 and Figure 4.5 with fire-protected timber elements. The FRLs and MRISF requirements for these elements are summarised in Table 4.3 for massive timber applications. For external walls refer to Section 4.6.

Table 4.3: FRL and MRISF requirements for massive timber construction in mid-rise Class 2 and 3 buildings.

Symbol	Description	FRL – Structural Adequacy /Integrity/ Insulation – min		Modified Resistance to the Incipient Spread of Fire (min.)
		Loadbearing	Non-Loadbearing[4]	
Red solid line	**Fire stair shaft**	90/90/90	–/90/90 (–/60/60)	30 outside 20 inside
Green solid line	**Service Shaft**	90/90/90	–/90/90 (–/60/60)	30
Blue solid line	**Bounding walls – SOUs and public corridors etc**	90/90/90	–/60/60	30
Orange solid line	**Lift Shaft walls**	90/90/90	–/90/90 (–/60/60)	30 outside 20 inside
Red door symbol	**Door to fire Stair**	Not applicable	–/60/30 (–/30/30)	Not applicable
Green door symbol	**Fire Door to service shaft**	Not applicable	–/60/30	Not applicable
Blue door symbol	**Door to SOU**	Not applicable	–/60/30 (–/30/30)	Not applicable
Orange dashed line	**Lift door**	Not applicable	–/60/–	Not applicable
Green riser door symbol	**Fire doors to services risers**[1]	Not applicable	–/60/30	Not applicable
Grey dashed line	**Non Loadbearing walls within an apartment**	Not applicable	–/–/–	–
Brown thick line	**Floors**	90/90/90	Not applicable	30

Note 1: Riser doors may not require an FRL if service penetrations are fire stopped at floor level.
Note 2: Since the roof will have a non-combustible covering and mid-rise timber buildings are required to be sprinkler-protected throughout, the roof is not required to achieve an FRL.

In addition to the above requirements, the fire-protective coverings must also be non-combustible.

Note 3: Service penetrations through fire-protected timber elements permitted to achieve FRL of –/60/15 under the Specification E1.5a concession.

Note 4: Concessions for FRLs for non-loadbearing elements permitted by Specification E1.5a are shown in brackets.

Refer NCC Volume One Clause C1.13(d) and Specification C1.1 Clause 3.1(d) and Clause 4.1(e)

4.4 Cavity Insulation Requirements

Combustible cavity insulation can facilitate ignition of cavity fires and the rapid spread of fire through cavities. Therefore, if cavity insulation is provided within fire-protected timber elements, it is required to be non-combustible.

Typical solutions include mineral fibre or glasswool insulation with very low organic binder contents. It is therefore important to check that Evidence of Suitability in the form of a current AS 1530.1 report from an Accredited Testing Laboratory is available for the specific products selected.

Cavity barriers are an essential component to mitigate the risk of fire spread through cavities

4.5 Cavity Barrier Requirements

Cavity barriers are barriers placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements that comply with Specification C1.13.

They are required to be provided by the following clauses as part of a Deemed-to-Satisfy solution:

- Clause C1.13
- Clause 3.1d(iii) of Specification C1.1
- Clause 4.1e(iii) of Specification C1.1.

The spread of fire, smoke and hot gases to other parts of the building is limited by cavity barriers in conjunction with other measures such as the use of non-combustible cavity insulation.

The risk of fire spread via cavities and voids in designs that use massive timber is addressed by prohibiting designs that incorporate cavities and voids and hence the level of protection to the timber element can be reduced under certain circumstances.

4.5.1 Determining the Positions of Cavity Barriers

Cavity barriers are required at the following positions:

- junctions between fire-resisting floor/ceiling assemblies and fire-resisting walls
- junctions between fire-resisting floor/ceiling assemblies and fire-resisting external walls
- junctions between fire-resisting walls and external walls
- around the perimeters of door and window openings in fire-resisting construction.

Horizontal barriers must be provided at each floor level up to a maximum distance of 5 metre centres.

Vertical barriers must be provided in walls up to a maximum distance of 10 metre centres.

Typical positions of cavity barriers are shown for an apartment building in Figures 4.6 and 4.7. Table accompanying Figure 4.7 includes a key describing the position and types of interface being protected. Typical details are shown in Section 5.

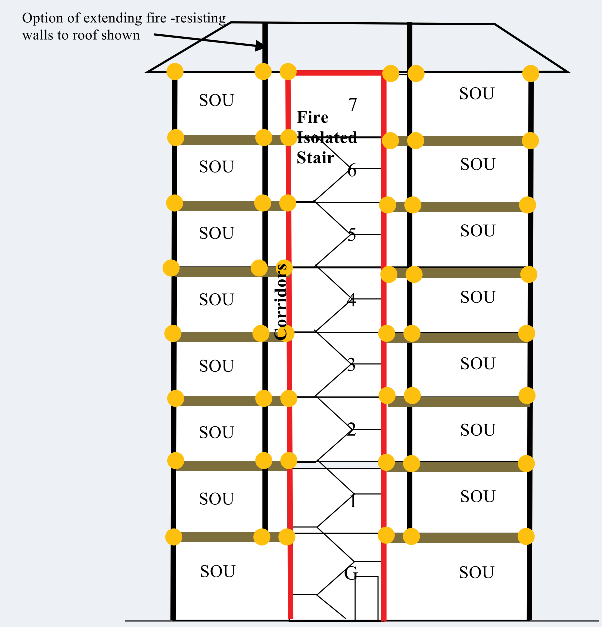


Figure 4.6: Vertical section of an apartment building showing typical cavity barrier positions.

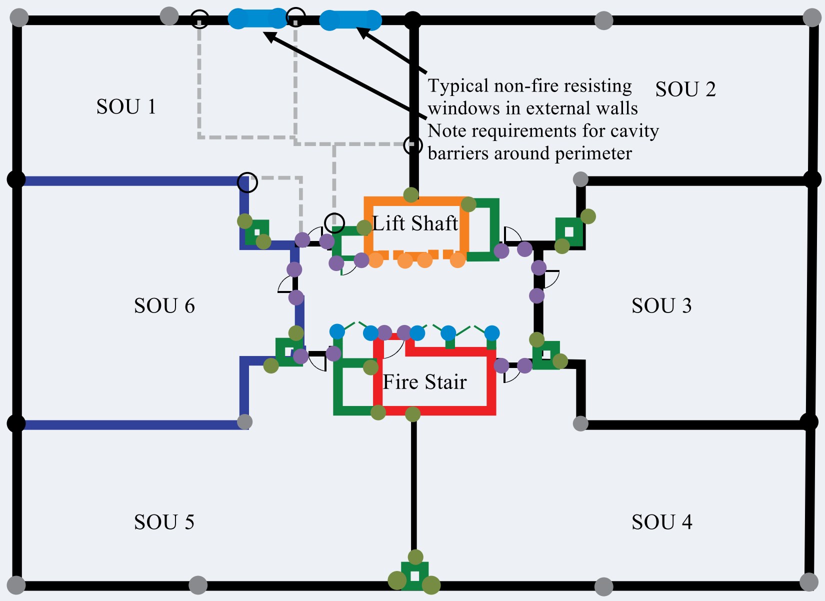


Figure 4.7: Floor plan section of an apartment building showing typical cavity barrier positions.

Symbol	Description	Comments
● (yellow)	Horizontal cavity barriers around perimeter of floors	If the floor to floor height is greater than 5 m intermediate horizontal barriers in walls would be required
● (grey)	Cavity barriers in fire-protected timber walls	Vertical cavity barriers are required at maximum 10 m centres
● (blue)	Cavity barriers around perimeter of non-fire-resisting doors and windows	Required to prevent entry of fire into cavity when non-fire-resisting elements fail
● (black)	Interface of fire-resisting walls with external walls	Can be incorporated as part of a standard detail
● (green)	Interface of shafts with standard walls	Can be incorporated as part of a standard detail
● (purple)	Interface with fire doors	Normally part of the standard detail for installation since the doorset is required to maintain the fire resistance of the wall
● (orange)	Interface with lift doors	In some instances it may be more practical to interface with other forms of construction around lift doors
○	Interface between non-fire-resisting wall and fire-resisting walls	Continuity of the fire protective coverings should be maintained at the point of penetration

Note 1: Riser doors may not require an FRL if service penetrations are fire stopped at floor level.

4.5.2 Specifying Cavity Barrier Requirements for Building Elements

Essentially there are two levels of performance required for cavity barriers prescribed by the NCC.

- Cavity barriers with FRLs of -/45/45 for building elements with FRLs not exceeding 90/90/90.
- Cavity barriers with FRLs of -/60/60 for building elements with FRLs greater than 90/90/90

For each case, the NCC prescribes Deemed-to-Satisfy Provisions based on minimum thicknesses of timber or non-combustible mineral fibre in the direction of heat flow as summarised below in Table 4.4.

For Class 2 and 3 buildings most elements have FRLs of 90/90/90 or less. Higher FRLs may be required for multi-class buildings.

Table 4.4: NCC prescribed Deemed-to-Satisfy solutions for cavity barriers.

Prescribed solution options	Fire-protected timber FRL	
	90/90/90 or less	>90/90/90 to 240/240/240
FRL for cavity barrier	-/45/45	-/60/60
Timber required minimum thickness*	45 mm	60 mm
Mineral wool required minimum thickness*	45 mm	60 mm

* Minimum thickness measured in the direction of heat flow - refer Appendix B.

For fire-protected timber with large cavities, which may occur in floor and roof cavities for example, it may be more practical to construct cavity barriers from plasterboard supported from timber framing (refer Figure 5.49)

4.6 External Walls/Building Facades

In addition to maintaining loadbearing capacity when subjected to fires within a building, the external walls also need to address the risk of fire spread via the building facade under the following scenarios:

- Fire spread from adjacent buildings (or the fire source feature as defined in the NCC) to the subject building. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by means of specification of minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Fire spread from the subject building (or the fire source feature as defined in the NCC) to adjacent buildings. Under the DTS Solution pathway for mid-rise timber buildings this is addressed by specifying minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.
- Fire spread from an external fire source adjacent to the facade other than adjacent structures; including balcony fires. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Vertical fire spread between openings from a fully developed fire within the subject building. Under the DTS Solution pathway for mid-rise timber buildings, this is addressed by specifying fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and by providing automatic fire sprinklers.

The measures described above are considered in more detail in the following Sections.

4.6.1 Fire-Protected Timber Requirements for External Walls

The FRLs required for external walls are nominated in NCC Specification C1.1 and are dependent on the building use (Class of Building), Type of Construction and proximity to the boundary (fire source feature) or other buildings. Mid-rise residential buildings (Class 2 and 3) are required to be of Type A construction.

The Resistance to the Incipient Spread of Fire (RISF) or, if massive timber is used, the Modified Resistance to the Incipient Spread of Fire (MRISF), requirements are nominated in NCC Specification C1.13a.

The requirements for Class 2 and 3 buildings of Type A construction are summarised in Table 4.5 and Table 4.6.

Table 4.5: FRL and RISF general requirements for timber-framed mid-rise residential building external walls (Type A construction).

Distance from fire source feature	FRL – Structural Adequacy /Integrity/Insulation – minutes		Resistance to the Incipient Spread of Fire (minutes)
	Class 2 and 3 (Residential)		
	Loadbearing	Non-Loadbearing	
≤1.0 m	90/90/90	–/90/90	45
<1.5 m	90/90/90	–/90/90	45
≥1.5 and <3 m	90/60/60	–/60/60	45
>3 m	90/60/30	–/–/–	45
External columns	90/–/–	–/–/–	45

Even though non-loadbearing external walls do not require an FRL if more than 3 metres from a fire-source feature, the fire-protective coverings must be applied and are required to achieve a RISF of 45 minutes since the external wall is required to be non-combustible. This is to address the risk of external fires on balconies or external areas adjacent to the building and the risk of vertical fire spread through openings if a fully developed fire occurs.

Table 4.6: FRL and MRISF requirements for timber mid-rise residential external walls for massive timber (Type A construction).

Distance from fire source feature	FRL – Structural Adequacy /Integrity/Insulation – minutes		Modified Resistance to the Incipient Spread of Fire (minutes)
	Class 2 and 3 (Residential)		
	Loadbearing	Non-Loadbearing	
≤1.0 m	90/90/90	–/90/90	45 external 30 internal
<1.5 m	90/90/90	–/90/90	30
≥1.5 and <3 m	90/60/60	–/60/60	30
>3 m	90/60/30	–/–/–	30
External Columns	90/–/–	–/–/–	30

Even though non-loadbearing external walls do not require an FRL if more than 3 metres from a fire-source feature, the fire protective coverings must be applied and are required to achieve a MRISF of 30 minutes.

For buildings within 1 metre of the boundary (or 2 m of an adjacent building on the same allotment), an MRISF of 45 minutes for the external surfaces is required to minimise the risk of ignition from fires in adjacent buildings but the internal face need only achieve a MRISF of 30 minutes.

Refer NCC Volume One Clause C2.6(b)(iii)

4.6.2 Vertical separation of openings in external walls

The NCC DTS Provisions for external walls requires vertical separation of openings to be addressed for Type A buildings to reduce the risk of fire spreading between floors if a fully developed fire occurs.

This can be achieved by providing spandrel panels or horizontal projections but the NCC waives these requirements if an automatic fire sprinkler system is provided in accordance with NCC Specification E1.5. This recognises that early suppression or control of an internal fire by an automatic fire sprinkler system is an effective means of minimising the risk of fire spread between floors via the facade provided fire-protected timber or non-combustible construction is adopted.

Overcoming the need to provide additional vertical separation by, for example, spandrel panels simplifies construction and provides greater design flexibility.

4.6.3 External Wall/Facade Systems

External walls form the building facade and the NCC requires them to serve a number of functions in addition to addressing fire safety, including:

- structural performance (for safety and serviceability)
- weather resistance (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation.

The external face of the wall may form part of the fire-protective covering (e.g. brick veneer construction) or may cover a fire-protective covering to prevent water penetration and serve other non-fire related functions (e.g. rain screen). In both cases, the NCC requires the external walls to be of non-combustible construction and therefore all these coverings must be non-combustible.

Refer Verification Methods CV1, CV2 CV3, AS 5113 and relevant State or Territory Regulations

Typical details of brick-veneer construction or fixing of non-fire-resistant coverings, such as rain screens, are shown in Section 5.

If the building design specifies combustible cladding systems, the performance pathway could be adopted subject to it being possible to demonstrate compliance of the wall system with the relevant NCC performance requirements.

NCC Verification Method CV3, in conjunction with Verification Methods CV1 and CV2, defines an appropriate method which requires, among other things:

- the external wall system to achieve the EW classification in accordance with AS5113.
- enhancements to automatic sprinkler protection.

Note: Some States and Territories may apply additional controls on the use of combustible cladding systems.

When designing lift shafts it is important to involve the lift supplier at an early stage to ensure the shaft will satisfy their design requirements and applicable regulations

4.7 Lift Shafts

Some designs of timber buildings adopt a hybrid approach and incorporate concrete or masonry shafts. Where this approach is adopted it is important that the potential for differential movement between the timber structure and shaft be taken into account when detailing connections and interfaces.

When designing lift shafts it is important to involve the lift supplier at an early stage to ensure the shaft will satisfy their design requirements and applicable regulations.

The remainder of this Section addresses the fire safety performance of lift shafts of fire-protected timber construction with respect to NCC compliance.

4.7.1 Timber-framed Lift Shaft Construction

Table 4.7 has been derived from Section 4.3 to show the NCC requirements that are applicable to timber-framed lift shafts in mid-rise residential timber buildings.

EWFA RIR 37401400 is available on the WoodSolutions website

Table 4.7: Requirements for fire-protected timber-framed lift shafts.

Criteria	Residential Buildings (Class 2 and 3)
FRL for loadbearing walls	90/90/90
FRL for non-loadbearing walls	–/90/90 (–/60/60)[2]
RISF for walls	45
Lift landing doors	–/60/–

Note1: The wall FRL and RISF requirements are applicable from both inside and outside the shaft.
Note 2: Concessions for FRLs for non-loadbearing elements permitted by Specification E1.5a are shown in brackets.

To minimise sound transmission to adjoining areas, double stud construction may be employed and/or an independent support structure provided within the shaft.

The fire-resistance of lift landing door assemblies should be determined by fire tests in a representative wall construction type. At the time of preparation of this Guide evidence of suitability for lift landing doors directly fixed to timber-framed wall assemblies was unable to be obtained.

A practical way to address this is to transition the shaft wall construction around the door opening to a form of non-combustible construction having FRLs with which the performance of the lift door has been verified.

An example of transitioning to a steel shaft wall system from a fire-protected timber wall shaft is shown in Figures 4.8 and 4.9 - refer to Section 4.10 and Section 5 for further details. These interface details have been assessed by an Accredited Testing Laboratory (EWFA Regulatory Information Report (RIR) 37401400), which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the lift landing doors up to the lesser of 120/120/120 or the FRL of the element. Evidence of Suitability for the specific proprietary lift door, steel stud shaft wall and timber shaft wall, in accordance with Clause A5.2 and A5.4 to A5.6 as appropriate of the NCC, should be submitted to the relevant regulatory authority in addition to RIR 37401400.

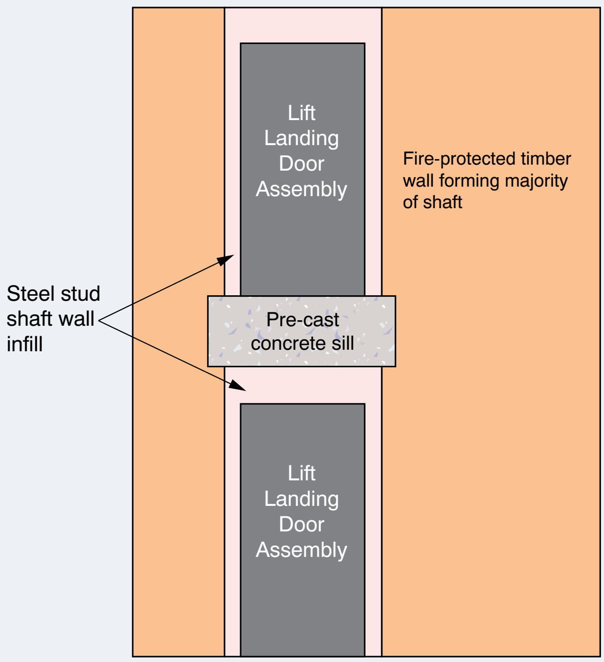


Figure 4.8: Elevation showing wall transition around lift landing doors.

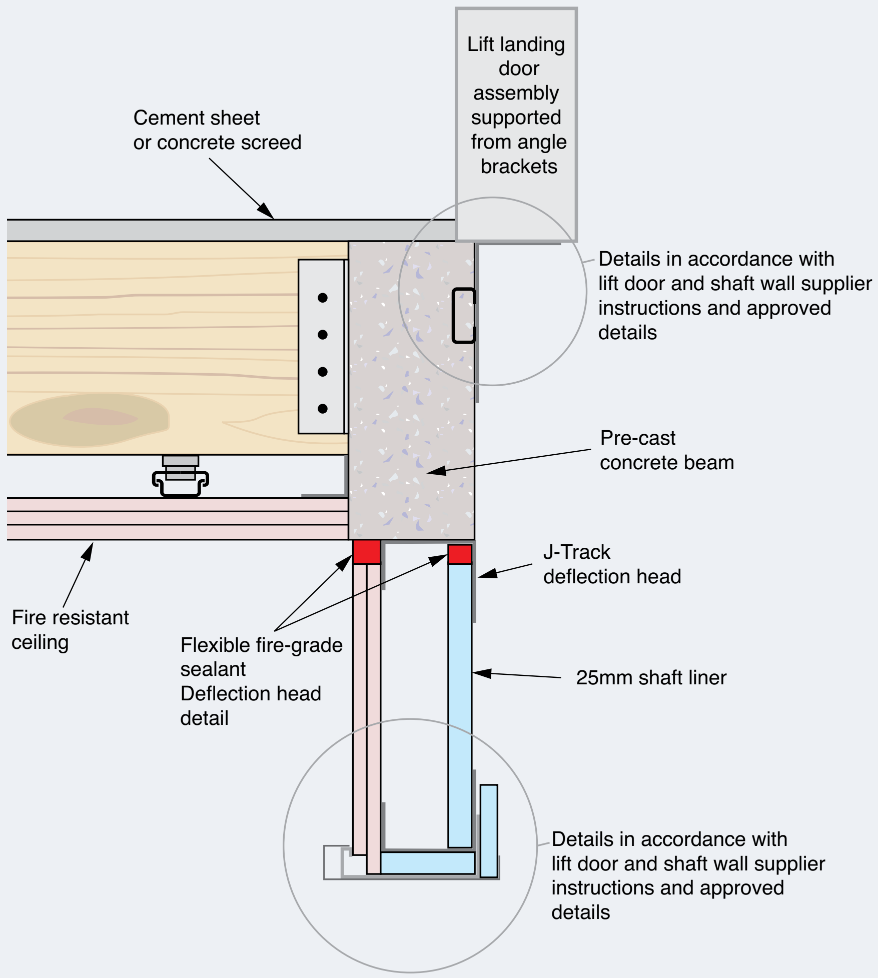


Figure 4.9: Generic detail for sill and head mounting.

4.7.2 Massive Timber Lift Shaft Construction

Table 4.8 has been derived from Section 4.3 to show the NCC requirements that are applicable to timber lift shafts in mid-rise timber buildings if using massive timber.

Table 4.8: Requirements for fire-protected timber lift shafts if using massive timber.

Criteria	Residential Buildings (Class 2 and 3)
FRL for loadbearing walls	90/90/90
FRL for non-loadbearing walls	–/90/90 (–/60/60)[2]
MRISF for walls	30 outside face 20 inner face
Lift landing doors	–/60/–

Note1: The wall FRL and MRISF requirements are applicable from both inside and outside the shaft unless otherwise indicated.
Note 2: Concessions for FRLs for non-loadbearing elements permitted by Specification E1.5a are shown in brackets.

If utilising massive timber construction, the MRISFs are reduced from 30 to 20 minutes within the lift shaft. This relaxation reflects the lower probabilities of severe fires occurring within these areas but a basic level of protection is retained to address the small potential of fires occurring within these areas, where fire may spread to evacuation paths which could be quickly compromised due to rapid fire spread in the early stages of a fire. The outer faces still require an MRISF of 30 minutes. This configuration is shown in Figure 4.10.

To minimise sound transmission to adjoining areas, double skin construction may be employed and/or an independent support structure provided within the shaft for a single skin option. If double skin construction is employed it should be noted that the NCC does not permit an unfilled cavity between the massive timber skins when using the massive timber provisions. If unfilled double-skin construction is preferred, there is still an option to use the General Requirements (timber-framed construction) rather than the massive timber requirements. The General Requirements require the inner and outer faces to achieve a RISF of 45 minutes. This can be achieved by applying two layers of 13 mm thick fire-protective grade plasterboard to both the inner and outer faces of the shaft.

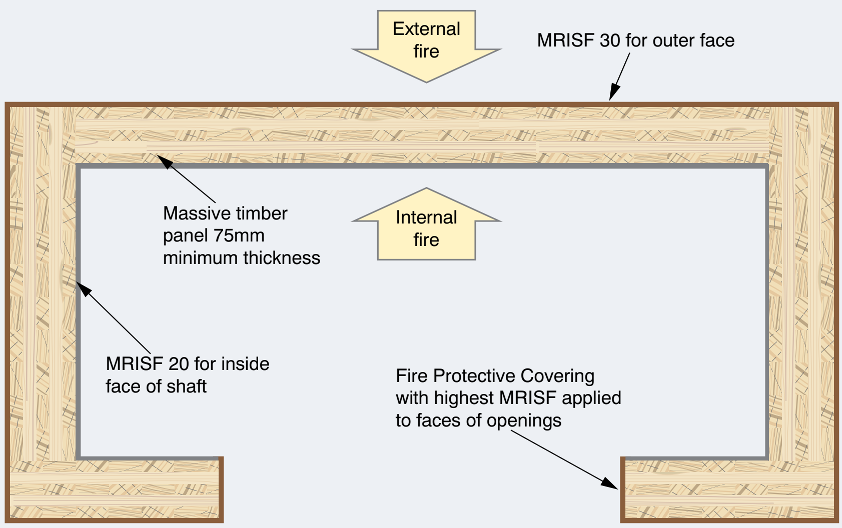


Figure 4.10: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

EWFA Regulatory Information Report (RIR) 37401400 is available on the WoodSolutions website

4.8 Fire Isolated Stairs and Passageways

The FRL, RISF or MRISF required for Fire-Isolated Stairs and Passageways are the same as those required for lift shafts described in Section 4.7 without the complication of lift landing doors.

Fire doors to fire-isolated stairs or passageways are required to achieve an FRL of –/60/30 or –/30/30 if the concessions of Specification E1.5a are applicable. Several proprietary fire door systems have been tested when mounted in timber construction. Installation details for fire doors capable of achieving the required FRLs should be obtained from the supplier as they may vary. Figure 4.11 shows a typical interface detail with a fire-protected timber wall. These interface details have been assessed by an Accredited Testing Laboratory (EWFA RIR 37401400) which determined that the interface details will not reduce the FRL, RISF or MRISF of the base wall system or the fire doors up to the lesser of 120/120/120 or the FRL of the element. Evidence of Suitability for the specific proprietary door, and timber shaft wall, in accordance with Specification A5.4 of the NCC, should be submitted to the relevant regulatory authority in addition to EWFA RIR 37401400.

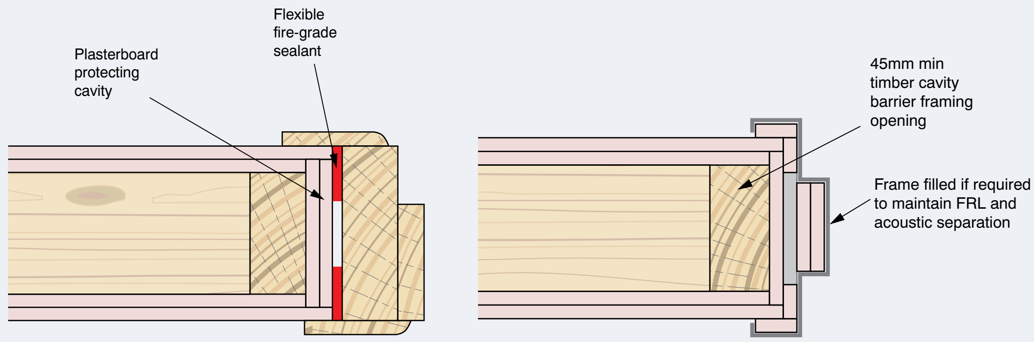


Figure 4.11: Typical fire door installation details.

4.8.1 Timber Stairways Concession

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway constructed from fire-protected timber in accordance with Specification C1.13a subject to:

- timber having a finished thickness of not less than 44 mm
- an average timber density of not less than 800 kg/m^3 at a moisture content of 12%
- the building being protected throughout by a sprinkler system complying with Specification E1.5, other than FPAA101D and FPAA101H systems, that is extended to provide coverage within the fire-isolated enclosure, and
- the underside of flights of stairs and landings at or near the level of egress or direct access to a car park being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres.

Fires starting in fire-isolated stairs are rare. When they do occur, they generally involve stored or introduced materials and often the cause is malicious. Even though it is not permitted to store goods in fire-isolated stairs and passageways, areas under the lowest flight of stairs form a convenient dry area for temporary storage. These areas may also not be secured, further increasing the risk of malicious fire starts.

While it could be argued that the extension of the sprinkler system to fire-isolated stairs and passageways addresses this issue, as an additional precaution, the underside of the lower stairs where combustibles can be stored are required to be protected by a fire-protective covering of 13 mm fire-protective grade plasterboard.

Section 5 provides further details of the requirements for timber stairways.

4.9 Building Services

4.9.1 Selection of Building Services and Distribution Paths

Careful planning and design of building services and distribution paths at all stages of the design process can greatly simplify construction and subsequent maintenance

The building services and associated cable and pipe runs need to be selected and refined during the design process to ensure the installation of the services and associated fire protection is efficient and reliable, with access to ensure the systems can be maintained or expanded without compromising fire safety.

Key points for consideration with respect to fire safety and acoustics are:

The number of service penetrations through fire-protected timber construction and fire-resisting construction generally should be minimised as far as practicable. This can be achieved by measures such as self-contained air-conditioning systems serving each sole-occupancy unit (SOU), and false ceilings and wall facings allowing services to run behind the non-fire-rated facing without penetrating the fire-resisting elements.

Generally, services and connection details should not require hot works. For fire services, such as sprinkler systems, the time they will be unavailable during maintenance and modification should be minimised. In some instances these requirements may conflict. For example, the use of CPVC piping for sprinkler systems can reduce hot works but if the pipework is adjusted the system will be unavailable while the glue sets, potentially overnight. Another option to avoid hot works may be to use mechanical joiners for metal pipes.

If service penetrations through fire-resisting construction cannot be avoided, the services should penetrate shaft or service duct walls rather than fire-resisting walls or floors separating occupied areas. This reduces the acoustic impact as well as limiting the consequences if a penetration protection system fails; as smoke and fire spread will initially be limited to the service ducts.

Where practical, shafts, service risers and service ducts should be readily accessible from public parts of the building to facilitate maintenance and inspection but access hatches/panels or doors should normally be secured to prevent unauthorised access.

Where practical, service penetrations through fire-protected timber construction should be grouped together and penetrate framed out openings that are then fire stopped with proprietary systems such as non-combustible batts, board or pillow systems. This approach substantially reduces the risk of fire spread to cavities at a point of weakness and ignition if hot works are being undertaken on the services.

In residential buildings, each SOU is effectively a fire compartment and includes bathrooms and kitchens and, in many instances, it is impractical to consolidate services such as drain, waste and vent (DWV) pipes around the central core and therefore service shafts are distributed around the floor. For apartment buildings, the use of self-contained heating, ventilation and air-conditioning (HVAC) systems tends to be preferred; in hotels and the more institutional style buildings, centralised HVAC systems may be preferred requiring duct penetration of walls and floors to be addressed.

Typical details are provided in Section 5.

4.9.2 Service Shaft Construction

The requirements for fire-protected timber service shafts used for ventilation, pipes, garbage or similar purposes are summarised in Table 4.9.

Shafts must also be enclosed at the top and the bottom with a floor/ceiling system of the same Fire Resistance Levels and Resistance to the Incipient Spread of Fire ratings as the walls; except where the top of the shaft is extended beyond the roof, or the bottom of the shaft is of non-combustible construction laid directly on the ground.

The shaft is also required to be sound rated if it passes through more than one SOU and must have a R_w+C_{tr} of 40 if the adjacent room is habitable and R_w+C_{tr} of 25 if it is a kitchen or non-habitable room.

Table 4.9: Requirements for fire-protected service shafts in mid-rise timber buildings.

Criteria	Residential Buildings (Class 2 & 3)
FRL loadbearing elements	90/90/90
FRL non-loadbearing elements	-/90/90 (–/60/60)
RISF	45
MRISF (Massive Timber)	30

Note: Concessions for non-loadbearing elements permitted by Specification E1.5a are shown in brackets. These concessions also reduce requirements for the FRLs of service penetrations to -/60/15 through fire- protected non-loadbearing timber shafts.

In many instances it is more practical to construct non-loadbearing shafts from laminated board systems or steel shaft wall construction in lieu of fire-protected timber construction.

If the shaft wall is non-loadbearing and of non-combustible construction, and the Specification E1.5a concessions are applicable, the FRL of the shaft can be reduced to –/45/45 with service penetrations required to achieve an FRL of –/45/15.

Details on how to construct shafts in timber-framed construction and how to interface fire-protected timber walls with laminated board shafts or steel shaft wall construction are given in Section 5.

4.9.3 Protection of Service Penetrations

The NCC requires service penetration systems to comply with AS 4072.1 and AS 1530.4. For services penetrating fire-protected timber elements there is an added complication that the Resistance to the Incipient Spread of Fire (RISF) or Modified RISF criteria have also to be satisfied in addition to the integrity and insulation criteria applied to the non-fire side.

Further explanations of the test procedures are provided in Appendix A.

Typical solutions to address RISF performance criteria include:

- boxing out openings with plasterboard
- filling the area around the service penetration with non -combustible mineral fibre insulation
- transitioning to a different wall type where service penetrations are required.

If Specification E1.5a concessions apply the insulating criteria for service penetrations is reduced to 15 minutes.

Examples are provided in Section 5.

4.10 Interfacing With Other Forms of Construction

There can be advantages in adopting hybrid forms of construction in buildings. For example, ground floor and basement areas may be constructed from concrete to minimise the risk of water penetration, minimise potential damage in flood-prone areas or address the risk of termites.

The relatively lighter weight of timber structures also makes timber construction ideally suited to the upward extension of existing buildings facilitating infill developments and recycling existing buildings. For example, it may be possible to add apartments above existing retail buildings without having to undertake extensive foundation works.

4.10.1 Separation of Different Classes of Buildings

Refer NCC Volume One Clause C2.8 for further details

Refer NCC Volume One Clause C2.9 for further details

The NCC addresses the separation of classifications within a building in Clauses C2.8 and C2.9.

For different classifications on the same storey, parts having different classifications should be separated by a fire wall having the higher FRL of the two, in accordance with Specification C1.1.

For different classifications in different storeys in a Type A building (most mid-rise buildings), the floor between the adjoining parts must have an FRL not less than that prescribed by Specification C1.1 for the lower storey.

Refer NCC Clause C1.3 for determining the type of construction required for a multiple classification building

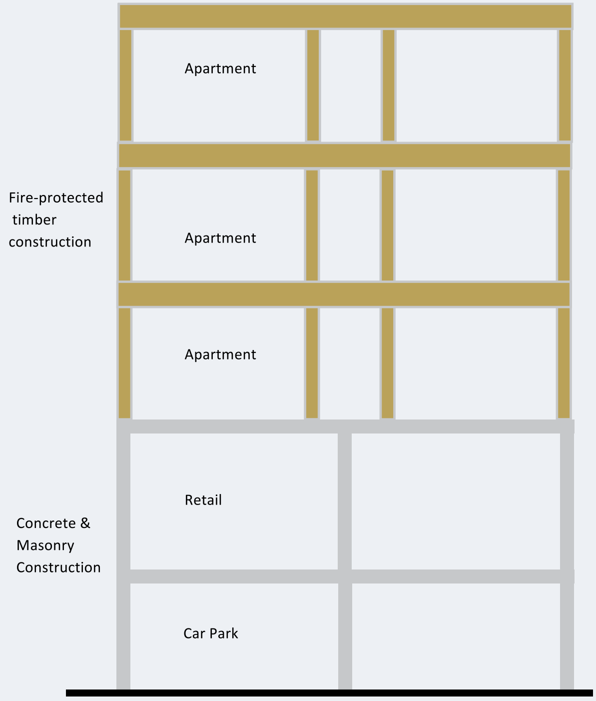


Figure 4.12: Example of multi-class building.

A typical building layout is shown in Figure 4.12 with a retail part of concrete framed construction below timber apartment levels. For the fire-protected timber concession to apply, the whole building must be sprinkler protected in accordance with NCC Specification E1.5 excluding FPAA101D and FPAA101H systems.

Retail use is assigned to Class 6 buildings. From Table 3 of Specification C1.1, the concrete slab separating the retail and apartment levels would require an FRL of 180/180/180.

4.11 Special Fire Issues

In constructing Class 2 and 3 mid-rise timber buildings, special issues arise as buildings become larger and more complicated. Although this Guide does not attempt to provide information to suit all circumstances, information is provided where there is relevance to timber construction practices.

Refer NCC Volume One Clause C1.13 for the Fire-protected timber Concession

4.11.1 Smoke-Proof Walls

For Class 2 and 3 buildings, the NCC requires that public corridors greater than 40 metres long be divided by smoke-proof walls at intervals of not more than 40 metres. These walls must be built from non-combustible materials and extend to the floor above, roof covering or Resistance to the Incipient Spread of Fire (RISF) ceiling.

Smoke-proof walls can be constructed from fire-protected timber if the RISF of 45 or MRISF of 30 (for massive timber construction) is achieved. Where the smoke-proof wall is also required to achieve an FRL (e.g. the wall is loadbearing) the fire-protected wall will need to meet the required FRL.

Where there is no requirement for an FRL, the Deemed-to-Satisfy plasterboard-based solutions can be adopted. For timber-framed construction, two layers of 13 mm fire-protective grade plasterboard should be applied to both sides of the framing; and if massive timber construction is used, the timber panel should be faced on both sides with one layer of 16 mm fire protective grade plasterboard.

4.11.2 Fire Precautions During Construction

Fires may occur on building construction sites due to the nature of the works. Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Timber construction covered with fire-protective linings is a safe and economical building system. The fire-protective coverings play an important role in providing this fire safety but, due to the construction sequencing, there may be a period where the timber is not protected. This is when timber buildings are at their highest risk from construction fires.

The NCC requires a suitable means of fire-fighting to be installed in a building under construction to allow initial fire attack by construction workers and for the fire brigade.

A building under construction that is less than 12 metres in effective height must have one fire extinguisher to suit Class A, B and C fires, as defined in AS 2444, and electrical fires provided at all times on each storey adjacent to each exit, or temporary stairway or exit.

After the building has reached an effective height of 12 metres, the following additional measures must be operational:

- the required fire hydrants and fire hose reels must be operational in at least every storey that is covered by the roof or the floor structure above, except the two uppermost storeys
- any required booster connections must be installed.

In this instance, 'required' means satisfying the NCC performance requirements in the complete building using either the performance or Deemed-to-Satisfy pathways.

As the scope of the NCC does not fully address Workplace Health and Safety (WHS) issues, and the NCC prescribes minimum levels of compliance, builders and building owners need to consider what is actually required for the building site. Typical matters that should be considered include:

- progressive installation of fire-fighting services
- progressive installation of fire-protective grade covering of timber members (i.e. installation of fire-protective coverings) and compartmentation of the building
- prefabrication and delivery to site with full or partial encapsulation of timber
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works.

WoodSolutions Technical Design Guide #20 Fire Precautions During Construction of Large Buildings provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

4.11.3 Bushfire-prone Areas

The requirements for Class 2 and 3 buildings to address the risk of bushfires vary between States and Territories and may fall under different jurisdictions to standard building works. The need to consider bushfire exposures should be determined early in the design processes and addressed accordingly.

The NCC requires external walls to be of non-combustible construction in mid-rise buildings and the fire-protected timber provisions requires timber elements to be protected by non-combustible fire-protective coverings providing a good basis for the building to resist bushfire attack.

Further guidance is provided in ***Woodsolutions Technical Design Guide #4 Building with Timber in Bushfire-prone Areas,*** which includes design solutions for BAL levels from 12.5 to BAL-FZ. However, for mid-rise timber buildings, some modifications may be required to satisfy the fire-protected timber requirements

4.11.4 Lightweight Construction Structural Requirements – Specific Applications

The NCC requires elements that have Fire Resistance Levels (FRLs), or that form a lift, stair shaft, an external wall bounding a public corridor, non-fire-isolated stairway or ramp, to comply with Specification C1.8, if they are made out of lightweight materials such as timber-framing faced with plasterboard.

Specification C1.8 defines a structural test for lightweight construction and, in most parts, is directly related to the performance of the linings used. Appropriate Evidence of Suitability should be obtained from suppliers of lining materials used to verify compliance during the design phase.

4.11.5 Robust Structural Design

The NCC, under Part B1 Structural Provisions (BV2), provides a verification method for structural robustness as a means of verifying compliance with performance requirement BP1.1(a)(iii). The Verification Method states:

Compliance with BP1.1(a)(iii) is verified for structural robustness by -

(a) assessment of the structure such that upon the notional removal in isolation of -

(i) any supporting column; or

(ii) any beam supporting one or more columns; or

(iii) any segment of a loadbearing wall of length equal to the height of the wall, the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The structural design of mid-rise timber buildings should comply with these requirements and the design guidance provided in *WoodSolutions Design Guide #39 Robustness in Structures* to ensure the building is adequately robust in the event of localised failure of elements during a fire.

4.11.6 FRL Concessions that are not Applicable to Fire-protected Timber

The fire-protected timber provisions were based on the FRLs prescribed by Specification C1.1 without reductions in FRLs as permitted by the following concession:

- the residential aged care building concession specified in Clause 2.9 of Specification C1.1
- Vic H103.1 Fire safety in Class 2 and Class 3 buildings.

If a reduction in FRLs in accordance with the above concessions is being considered, the Performance Solution pathway must be adopted.

5

Step 5 Integrate Architectural, Structural and Building Services Designs (Detailed Design)

This step brings together the content of the previous Sections to develop an integrated design. A residential building with retail at ground level and basement car parking is used to demonstrate the process including interfacing Class 2 and 3 parts of a fire-protected timber building with other forms of construction and parts of a building with a different Class.

A key focus of this Step is coordinating the various design disciplines so that:

- Timber elements and protection systems are optimised to satisfy the NCC requirements in a practical and cost-effective manner by focusing on the synergies between elements designed to satisfy the following criteria:
  - fire-protected timber
  - sound transmission and insulation
  - thermal resistance
  - weatherproofing
  - structural tests for lightweight construction.
- Interfaces between building services and the structure, fire-protected timber elements and acoustic barriers are designed:
  - to minimise building service penetrations through fire-protected timber elements and acoustic barriers as far as practical
  - such that where services have to penetrate fire-protected timber elements the fire safety performance of the element is not compromised and fire separation is maintained
  - so that if services have to penetrate acoustic barriers the positions are selected to minimise negative impacts on amenity
  - so that service penetration systems can accommodate any differential movement between elements
  - to allow for maintenance and additions/modifications to the building services.
- Structural design is efficient and robust.
- Other fire safety principles for mid-rise buildings are satisfactorily implemented including:
  - cavity barriers
  - automatic fire sprinkler systems.
- Other design requirements are addressed such as termite management and resistance to ground water/moisture penetration.

5.1 Optimising the Performance of Elements of Construction

Elements of construction in a modern buildings may have to serve a number of functions including:

- restricting fire spread
- limiting sound transmission from adjacent enclosures (and in some instances external noise)
- limiting heat loss and/or heat gain through external elements
- weather resistance of external facades and roofs
- impact resistance to reduce the risk of damage to lightweight construction.

The elements also need to achieve levels of durability appropriate for the application. Further advice on durability is provided in: *WoodSolutions Timber Design Guide #5 Timber service life design – Design guide for durability.*

Efficient designs can be achieved by selecting combinations of materials and configurations that work together to satisfy the design objectives summarised in the following Sections.

Typical examples include:

- Cavity barriers required by the NCC Deemed-to-Satisfy for mid-rise timber buildings to reduce the risk of fire spread through concealed spaces can also be used to minimise flanking noise transmission around the perimeters of elements of construction and reduce heat loss via leakage through the structure.
- Non-combustible cavity insulation will:
  - reduce the risk of fire spread through cavities
  - reduce sound transmission through elements of construction
  - reduce heat loss and/or gain through external walls.

5.1.1 Fire-protected Timber

Fire-protected timber has timber structural members protected by non-combustible fire-protective coverings. The fire-protective coverings:

- prevent or delay the ignition of the timber members so that the response to an enclosure fire will be similar to non-combustible elements such as masonry or concrete during the growth period and prior to fire brigade intervention
- ensure the fire-protected timber element achieves the Fire Resistance Level (FRL) prescribed for the particular element.

Any insulating materials provided within cavities must be non-combustible to reduce the risk of fire spread through cavities and voids.

The NCC contains some Deemed-to-Satisfy Solutions for fire-protective grade plasterboard coverings but there are many opportunities for the use of optimised proprietary systems. For example, combinations of high-performance non-combustible fire-resisting claddings and mineral fibre insulation could provide lighter weight, more cost-effective options.

The NCC DTS solutions recognise that massive timber panels have a relatively high inherent fire resistance and, if there are no concealed cavities or voids, the risk of fire spread through concealed spaces will be substantially reduced or removed. Therefore, provided the minimum dimensions prescribed for massive timber panels are satisfied and there are no internal cavities and voids, the NCC allows some relaxations to the requirements for fire-protective coverings (refer Section 4.3).

Note: The use of timber blocks and other combustible fire protection systems such as intumescent paints in lieu of non-combustible fire-protective coverings is not permitted under the NCC DTS solutions for mid-rise timber buildings due to the potential increase in risk of fire spread to the structural element as the combustible materials are consumed.

5.1.2 Cavity Barriers

The primary objective of cavity barriers is to prevent uncontrolled spread of fire through cavities in the low probability the protective covering fails or fire starts within the cavity.

The NCC provides Deemed-to-Satisfy solutions using solid timber or mineral fibre but also specifies FRLs for cavity barriers encouraging the development of proprietary systems optimised for specific applications.

Careful detailing can provide opportunities for efficient design.

Typical examples include:

- in a single leaf, timber-framed stud wall, the top and bottom plates can be dimensioned such that they can act as cavity barriers
- if a cavity is filled with non-combustible mineral fibre insulation to achieve a nominated R-value or enhanced acoustic separation, the mineral fibre may also satisfy the requirements for a cavity barrier.

Battening wet areas protects fire- and sound-rated walls from compromise due to bath and shower installation and can also be used to reduce service penetrations through fire-protected timber elements

5.1.3 Sound Transmission and Insulation

In timber construction, airborne and impact sound requirements are primarily achieved using one or more of the following principles:

- **Increasing mass (e.g. increasing the thickness of wall linings).** This can be particularly useful in reducing airborne sound transmission. For instance, like fire-grade linings, the greater the number of layers, the greater the increase in R_w (Note: extra factors are involved in increasing R_w+C_{tr}).
- **Isolating one side of a wall from the other** (e.g. using double stud cavity wall construction). This is also known as decoupling (discontinuous construction) and can reduce both airborne and impact sound. Of note, it serves to limit noise vibration from one side of the element to the other.
- **Avoiding rigid connections between the opposing sides of isolated (decoupled) elements.** This limits the occurrence of sound bridges that would otherwise allow sound to transmit from one side to the other. If required for structural stability, sound-resilient connectors should be used and should generally only be used at changes in floor level (Figure 5.2).
- **Using absorptive materials to fill wall and floor cavities** (glass fibre or mineral wall) can reduce airborne sound transmission. The NCC requires absorptive material to be non-combustible.
- **Sealing sound leaks** at the periphery of wall and floor elements or where penetrations are made for electrical and plumbing services.

There are also simple techniques that can be incorporated into the building design that can dramatically improve the sound performance of timber wall and floor/ceiling systems. The following systems provide examples that can be used to enhance sound performance of walls and floors.

Wall Systems

Batten out walls in wet area. In wet area construction, fire/sound rated walls can be compromised where bath and shower base units need to be recessed into the wall. A simple means of achieving this is to batten out the wall (after fire/sound resisting linings have been applied) and then provide an additional lining over the top (Figure 5.1). The bath can then be installed into the batten space without affecting the fire- and sound-rated wall. In such instances, it is best to have at least 35 mm batten space and to place insulation into the cavity. This arrangement also substantially reduces the risk of compartmentation being compromised during refurbishment activities. For example, if the additional lining boards are removed or replaced, the fire-protective covering can be left in place, maintaining the required fire separation.

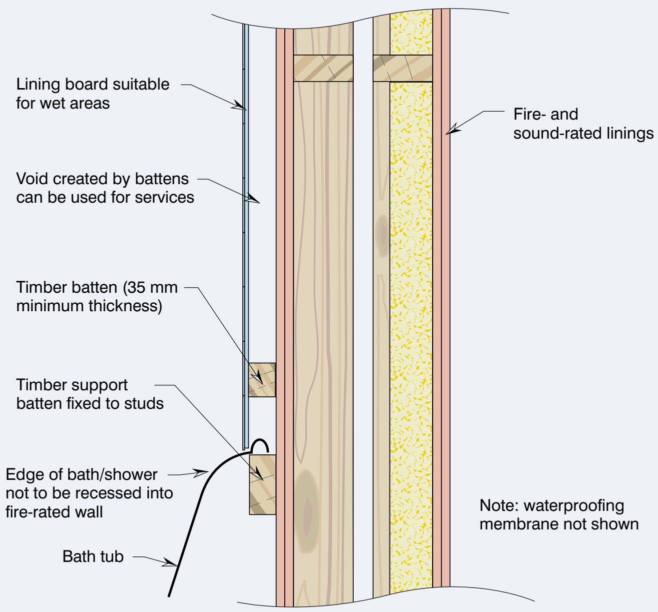


Figure 5.1: Batten detail for wet area walls – elevation view.

Floor Systems

Floor joists parallel to sound rated wall. By running floor joists parallel rather than perpendicular to the sound rated wall, the ability of impact sound from the floor being transferred across the wall to the adjoining SOU is lessened (Figure 5.2).

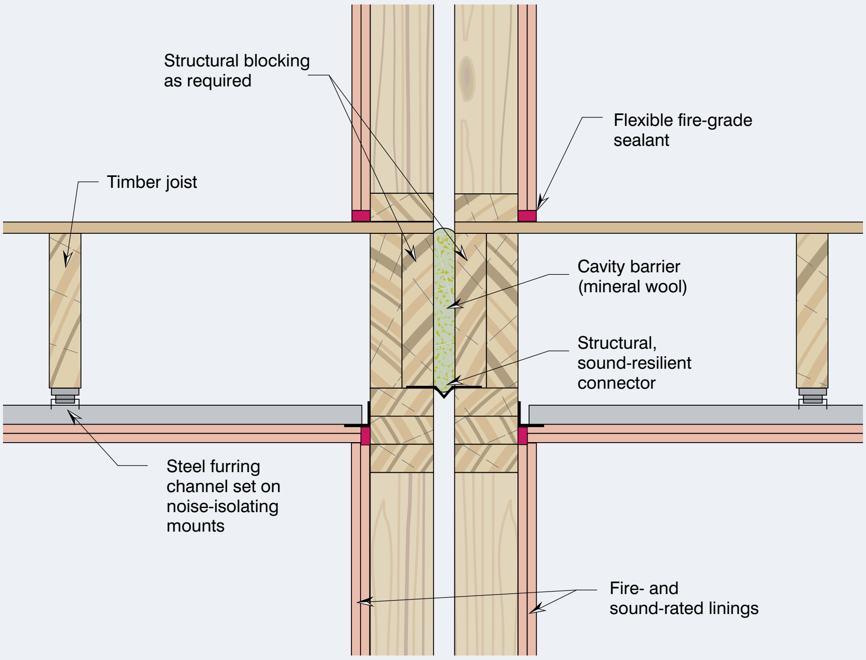


Figure 5.2: Joists running parallel to bounding wall – elevation view.

Upgrade sound-resilient ceiling mounts. Ceiling mounts are commonly used to prevent noise that gets into the floor from coming out through the ceiling below. They help reduce sound transfer between the bottom of the floor joist and the ceiling lining. To improve performance, some ceiling mounts now provide an isolating and damping effect (Figure 5.3). They typically force the sound energy through a rubber component that deforms slightly under load as the sound passes from the joist to ceiling sheet. Therefore, sound-resilient mounts are not all the same and different systems have different performance.

Time spent choosing the right sound-resistant ceiling mount can pay dividends.

Evidence of suitability must be provided to show that the required FRL of a ceiling system can be achieved using the acoustic resiliant mounts.

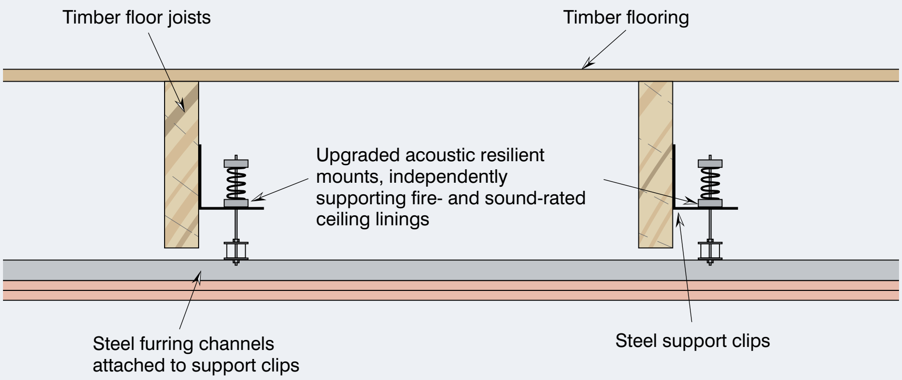


Figure 5.3: Upgraded sound-resilient ceiling mounts – elevation view.

Increase mass of the top layer of floor systems. Increasing the mass of the top surface of the acoustic floor system is one of the best ways to improve acoustic performance. There are three common ways – concrete topping, sand or additional floor sheets.

Quantifying the improvement is difficult as the acoustic performance is aimed at improving the low frequency performance of the floor, a phenomena not measured by tested systems. It is suggested that the base floor system be designed to comply with the NCC's sound requirements, and the additional floor mass provides enhanced performance unless evidence of suitability is available to quantify the improvement.

When height is added to a floor, consideration of the effect this has in other areas (such as wet areas, corridors, stairs, doors and windows) is needed at the planning stage.

Sand used to increase mass in timber floors. This increases the mass of the upper layer of the floor element. The air spaces between the sand particles help reduce the vibration and energy created by impact sound from footfalls. Typically, this is achieved by placing 45 mm battens directly over a normal acoustic floor system at typical 450 or 600 mm centres (dependent on floor sheet spanning capacity). A dry sand layer, or dry sand mixed with sawdust is placed between the battens and levelled just below the surface of the final floor sheet. The final floor sheet is fixed in the normal manner, and desired floor covering placed on this (Figure 5.4).

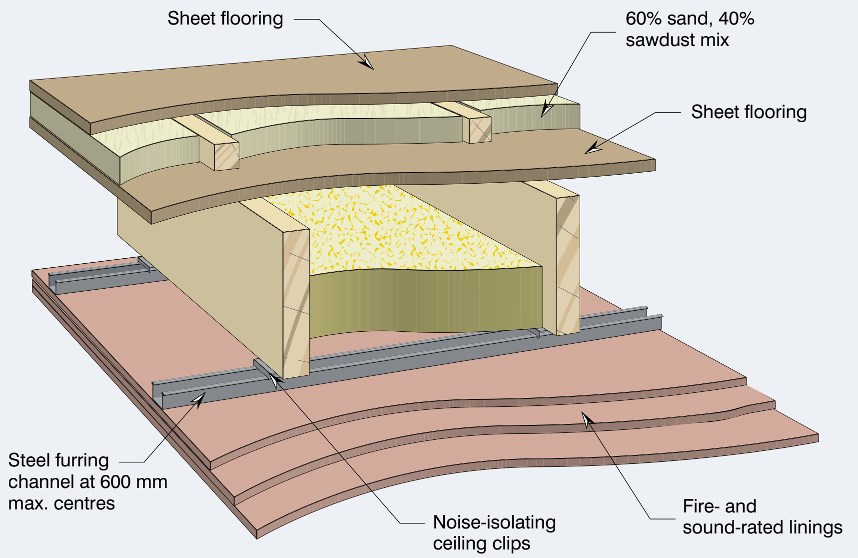


Figure 5.4: Adding mass to floor system through the use of sand top layer.

Concrete topping. This increases the sound performance of the floor system, and typically can be achieved with a 35 to 45 mm thick layer of concrete placed over an isolating acoustic mat. Care is required to turn the isolating acoustic mat up at the perimeter of the topping adjacent to the wall, otherwise the effect of the topping is negated (Figure 5.5).

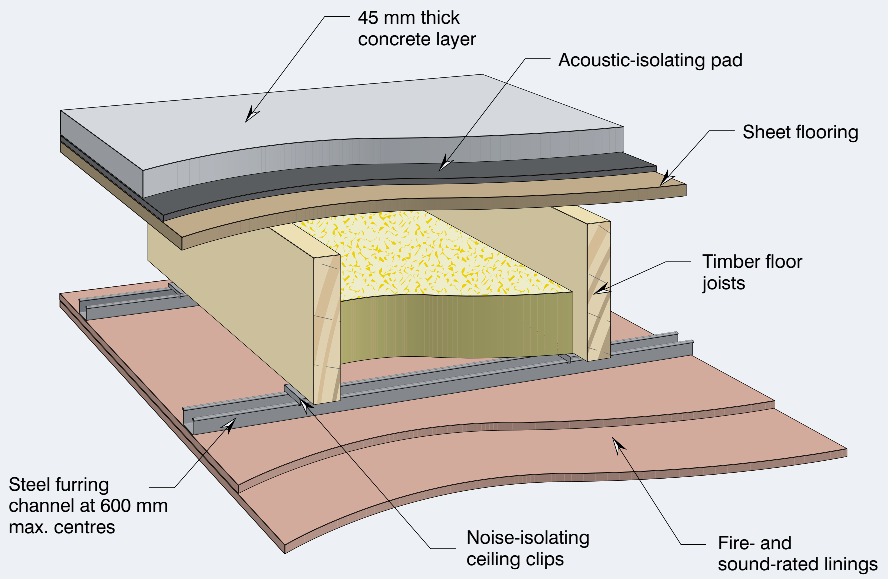


Figure 5.5: Adding mass to floor system through the use of concrete topping.

Extra sheet flooring. This method utilises standard sheet flooring on an isolating mat. This system does not perform as well as the higher mass products, sand or concrete (Figure 5.6).

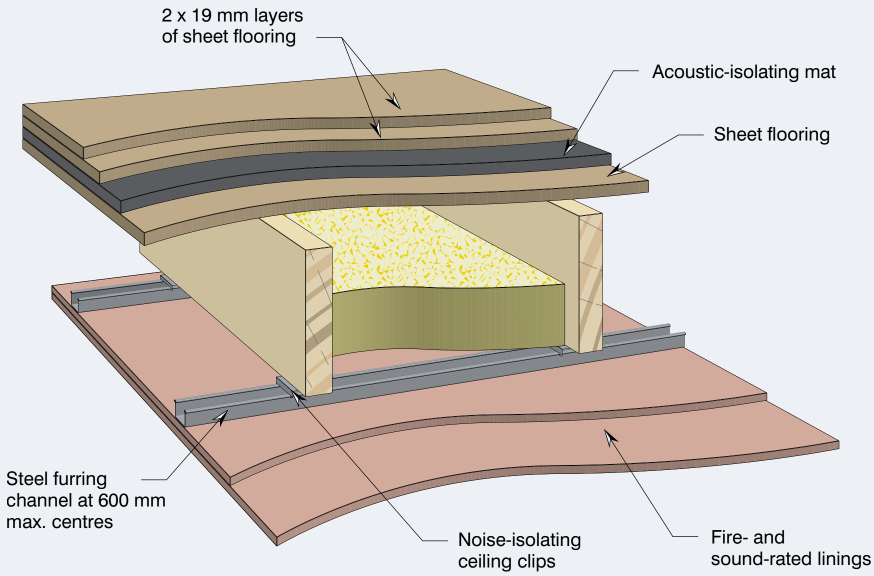


Figure 5.6: Adding mass to floor system through the use of additional floor sheets.

Separate floor and ceiling frame. By having two sets of joists (separate floor and ceiling joists) that are nested between but not touching each other, it is possible to isolate the two structures, thereby minimising the transference of impact sound through the structure. Care must be taken with this approach to prevent flanking noise running along the floor joists and into the walls below. This can be improved by sitting the ceiling joists onto strips of acoustic isolating mat (Figure 5.7).

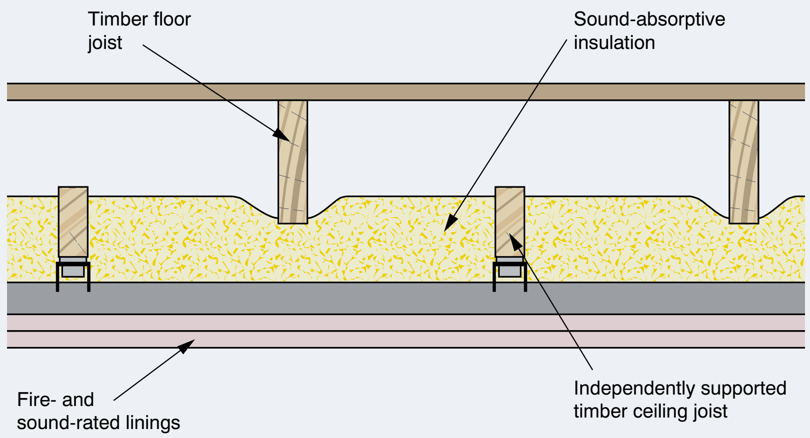


Figure 5.7: Separate ceiling and floor joist structures.

5.2 Establish Architectural Layout

The basic architectural layout of a building is determined by considering a large number of variables; the relative importance of which will vary from project to project. Typically these include:

- the project brief
- site conditions
- sustainable construction
- aesthetics
- economics
- planning, building and other regulations.

The design should then be refined with input from the various disciplines involved in the design team.

This process is demonstrated for a mid-rise timber residential building with basement car parking and ground level retail as shown in Figure 5.8.

A typical residential floor plan is shown in Figure 5.9.

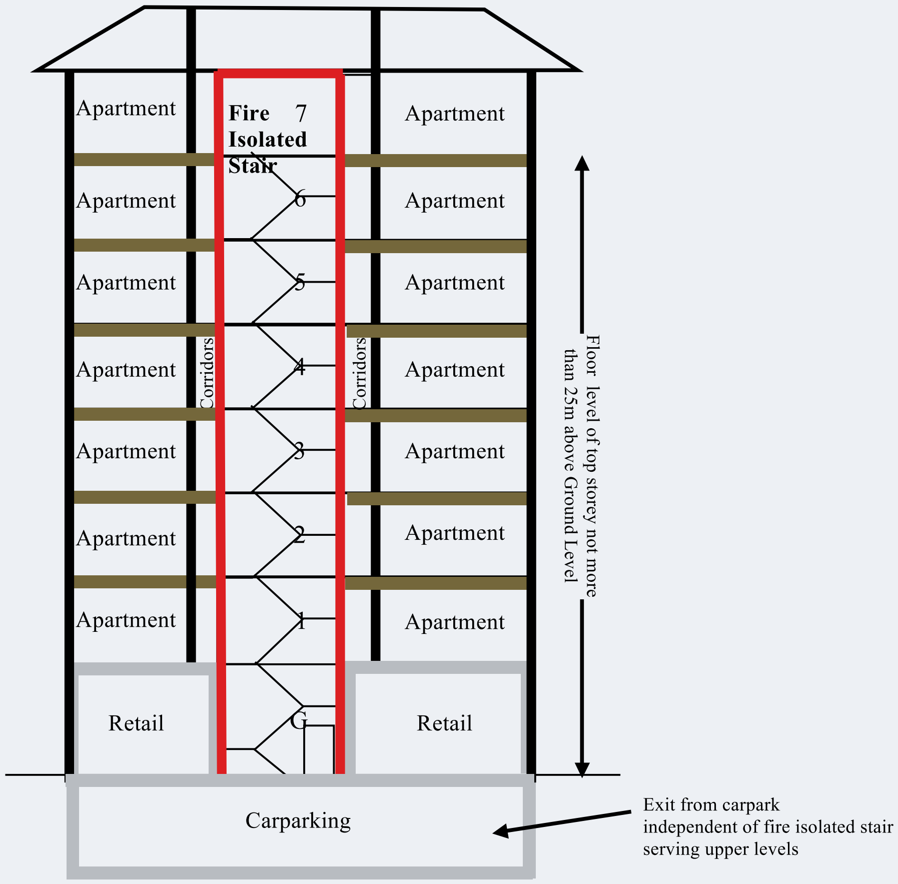


Figure 5.8: Section through a mid-rise residential building.

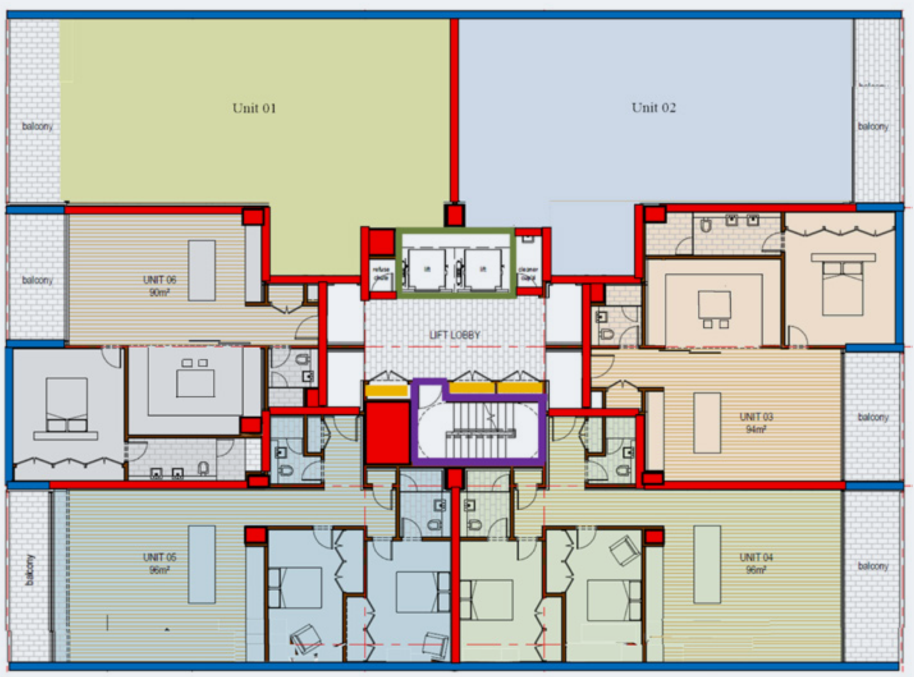


Figure 5.9: Plan of typical residential floor.

5.2.1 Optimising Building Layout - NCC DTS Distance of Travel Requirements

To address the evacuation of occupants during a fire emergency, among other things, the NCC DTS provisions prescribe maximum distances of travel from an entrance doorway of any SOU to an exit or a point from which travel in different directions to two exits is available.

These requirements for Class 2 and 3 buildings are provided in Clause D1.4 (a) of the NCC 2019 which states:

(i) The entrance doorway of any sole-occupancy unit must be not more than –

(A) 6 m from an exit or from a point from which travel in different directions to two exits is available; or

(B) 20 m from a single exit serving the storey at the level of egress to a road or open space; and

(ii) no point on the floor of a room which is not in a sole-occupancy unit must be more than 20 m from an exit or from a point at which travel in different directions to two exits is available.

Where travel to alternate exits is required, additional constraints apply to the distance between alternative exits.

These requirements are provided in Clause D1.5 and the requirements applicable to Class 2 and 3 buildings state:

Exits that are required as alternative means of egress must be—

(a) distributed as uniformly as practicable within or around the storey served and in positions where unobstructed access to at least two exits is readily available from all points on the floor including lift lobby areas; and

(b) not less than 9 m apart; and

(c) not more than—

(i) in a Class 2 or 3 building – 45 m apart; or ……..

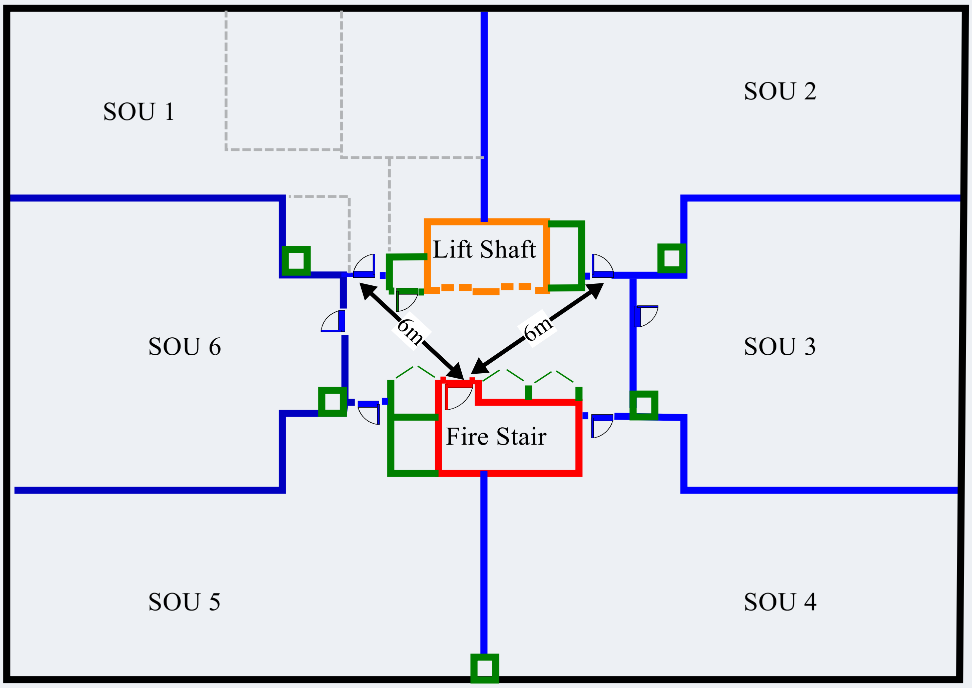
(d) located so that alternative paths of travel do not converge such that they become less than 6 m apart.

These requirements tend to dictate the architectural layout of Class 2 and 3 buildings. For example, a typical layout for a building with a single fire-isolated stair can accommodate six SOUs per floor to comply with the distance of travel requirements as shown in Figure 5.10(a);

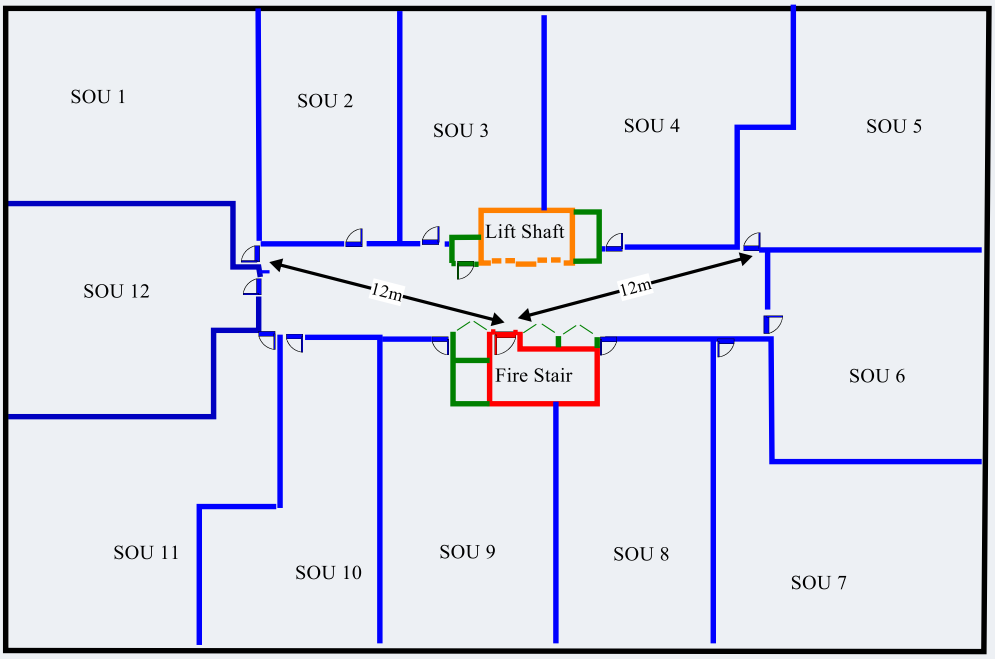
The distance of travel concession facilitates the design of more efficient floor layouts

If the requirements of NCC Specification E1.5a are satisfied (refer Section 4.2.3) a number of concessions can be applied including the following relating to distances of travel –

- Except in a residential care building, the maximum distance of travel, as required by D1.4(a)(i)(A), may be increased from 6 m to 12 m.
- The maximum distance of travel from a single exit serving the storey at the level of egress to a road or open space, as required by D1.4(a)(i)(B), may be increased from 20 m to 30 m.
- The maximum distance between alternative exits, as required by D1.5(c)(i), may be increased from 45 m to 60 m.
- These concessions facilitate greater flexibility under the DTS provisions for fire-protected timber mid-rise buildings. For example, a typical layout for a building with a single fire-isolated stair can accommodate 12 SOUs per floor to comply with the distance of travel requirements permitted in the Specification E1.5a concession as shown in Figure 5.10(b).



(a) Layout with 6 m maximum distance of travel to an exit



(b) Layout with 12 m maximum distance to an exit (refer Specification E1.5a)

Figure 5.10: Comparison of layout with different maximum travel distance to exits

5.3 Select Structural Form

For this example, the preferred structural material is timber for the Class 2 parts, which may have been selected for many reasons including:

- lightweight construction (useful if ground conditions are difficult)
- speed of construction
- sustainable construction
- prefabrication of elements.

This does not preclude the use of hybrid forms of construction.

Reinforced concrete construction was selected for the basement car park and ground floor retail areas to address ground water penetration and used as part of the termite management system.

5.3.1 Building Classes Other Than 2 or 3

The example building includes Class 6 (Retail) and Class 7 (Carpark) parts. The NCC 2019 Deemed-to-Satisfy Provisions allow the use of fire-protected timber in all building Classes or parts of buildings with an effective height not greater than 25 m:

The Class 6 and 7 parts should be fire-separated from the Class 2 part of the building in accordance with Clause C2.9 (or Clause C2.8 if different classes share the same floor) and comply with the Deemed-to-Satisfy solutions for the Class 6 and 7 parts.

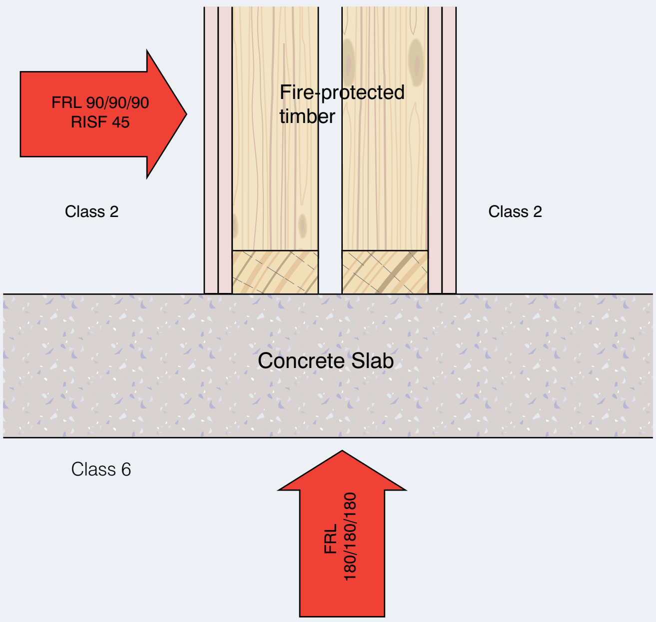


Figure 5.11: FRL requirements for fire separation between Class 6 and Class 2 parts of a building.

For the example building, the reinforced concrete slab fire separating the retail part of the building from the residential part would require an FRL of 180/180/180 if the DTS pathway is adopted while the bounding construction around apartments in the residential part typically require an FRL of 90/90/90 and an RISF of 45 minutes.

5.3.2 Select Basement and Ground Level Structural Form

Generally, reinforced concrete is selected for below ground works such as foundations and basements in conjunction with waterproofing membranes to address issues such as water penetration.

One way to manage termite risk is to extend concrete construction above ground so that termite entry can be readily detected.

Since the ground level of the example building is used for retail (Class 6), the most practical solution is to extend the concrete structure to floor level for Level 1 of the apartment building and comply with the Deemed-to-Satisfy Provisions for the basement and ground floor levels using concrete or masonry construction (Figure 5.8).

5.3.3 Select Upper Level Structural Form

All the upper levels are for residential occupancies and fire-protected timber construction has been selected as the preferred option using a DTS solution.

The mostly likely forms of construction would be timber-framed (lightweight) or massive timber panel systems such as Laminated Veneer Lumber (LVL) and Cross-Laminated Timber (CLT). While subsequent sections consider both options, the proprietary nature of massive timber panels limits the number of generic details that can be included in this Guide.

5.3.4 Select Lift and Fire Stair Shaft Construction

Lift and fire stair shafts in mid-rise timber buildings can be of timber, masonry or concrete construction. The choice will depend on the structural design of the building and numerous other factors.

If concrete or masonry shaft construction are adopted, it is important that the detailing can accommodate the possibility of differential movement between the timber structure and masonry/concrete shafts. Further information relating to masonry and concrete shaft construction lies outside the scope of this Guide.

Fire-protected timber shafts can be timber-framed construction or massive timber panel systems. Both options will be considered in subsequent sections.

An independent structural frame can be provided within the shaft as part of the lift installation, effectively isolating the lift system from the shaft walls and providing adequate acoustic separation.

If an independent steel frame is used within a fire-protected timber shaft, the possibility of differential movement between the timber and steel frame will need to be addressed. Typically, this can be addressed if the lift system can be recalibrated for differences in floor levels.

5.3.5 Structural Design

Issues that should be taken into account in the structural design of Class 2 and 3 buildings include:

- The design of mid-rise timber buildings must comply with the relevant NCC requirements including design to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage – refer NCC Clause BP1.1 (a)(iii) (structural robustness). A first principles performance pathway can also be adopted that addresses both fire and structural performance. Further guidance is provided in *WoodSolutions Technical Design Guide #39 Robustness in Structures*.
- The lighter mass of timber to that of masonry/concrete construction – greater attention needs to be given to resistance against overturning.
- The greater effect from wind loads than expected on smaller structures. This is due to a greater height-to-width ratio, resulting in a need for attention to resistance to overturning.
- Potential for movement (and differential shrinkage in buildings of hybrid construction) in taller timber buildings. Movement can be minimised by:
  - using seasoned timber or engineered timber
  - constructing bearers and joists in the same plane
  - detailing to avoid differential shrinkage between dissimilar materials, e.g. steel to timber; timber to masonry or allowing articulation to absorb the differential movement
  - allowing for differential movement with respect to plumbing and other services.

Refer NCC Volume One C2.12 and C2.13 for FRLs of construction separating equipment areas. Required FRLs may be at least 120/120/120

Avoid using cavities within fire-protected timber elements where practical

A professional structural engineer with appropriate skills will be needed to ensure the above issues and structural performance in general are adequately addressed.

The following standards should be called on:

- AS 1170.0 – Structural design actions – General Principles.
- AS 1170.1 – Structural design actions – permanent, imposed and other actions provides the basis for determination of appropriate dead, live design loads and loads combinations
- AS 1170.2 – Structural design actions – wind actions – which provides the basis for wind loads.
- AS 1170.4 – Structural design actions – Earthquake actions in Australia – which provides guidance and design procedures for earthquake forces.
- AS 1720.1 – Timber structures – Design methods.
- AS 1720.5 – Timber structures – Nailplated timber roof trusses

In addition:

- Select details that minimise the effects of shrinkage (especially since differential shrinkage may have an adverse impact on the function of fire-resisting wall and floor elements).
- Check that double stud walls bounding Sole Occupancy Units are capable of supporting multi-storey load paths from above. Enlist internal fire-resisting walls if required.
- Check that any elements supporting loads (including bracing elements) are treated as fire-resisting construction and designed accordingly. This usually includes all external walls.

5.4 Establish Service Plant Areas, Service Runs, Risers and Shafts

5.4.1 Service Plant Areas

Service plant rooms are generally located away from public areas, either in basements or on roof tops, depending on the building design

Clauses C2.12 'Separation of Equipment' and C2.13 'Electricity Supply System' generally require certain types of equipment to be fire separated from the rest of the building by construction having an FRL of 120/120/120 with doorways protected by self-closing fire doors with an FRL not less than -/120/30.

For Class 2 and 3 buildings designed to the NCC Deemed-to-Satisfy Provisions this means that some service plant areas may require to be enclosed in construction having an FRL of 120/120/120 rather than the FRL of 90/90/90 that generally applies to the rest of the structure. For applications similar to the example apartment building, the most practical solution may be to locate these service areas in the basement or ground floor where FRLs of 120/120/120 or greater are required.

Instead of using centralised plant for air-conditioning in apartment buildings, a common solution is to use self-contained units within each SOU to reduce the services that need to be distributed around the building.

5.4.2 Service Runs

In fire-protected mid-rise timber buildings, the timber elements are protected by fire-protective coverings and services tend to be concealed in a similar manner to conventional building designs using service risers, ducts and fitting cabling and pipes behind false wall and ceiling linings.

While the use of cavities within fire-protected timber construction to run cables and pipes can appear to be a simple solution, this choice presents a number of issues including:

- difficulty in maintaining the RISF or MRISF ratings of the elements at points of service penetrations
- risk of acoustic separation being compormised
- risk of fire protection systems not being correctly installed after modifications or additions to existing installations
- risk of disruption of concealed cavity barriers during modifications or additions.

A more reliable option is to plan the layout of required services and potential future services carefully utilising service risers, service shafts and ducts, and additional (false) linings to conceal services minimising penetrations through fire-protected timber elements as far as practicable. This approach is described in the following Sections.

5.4.3 Service Risers and Horizontal Distribution of Services

Adding/modifying services is simplified if services are protected at each floor level by framing out the opening and using a multi-service penetration system

Services such as electricity, water and telecommunications/data systems are normally distributed between floors through service risers that are commonly located close to the structural core (lift and stair shafts) as shown in Figure 5.12.

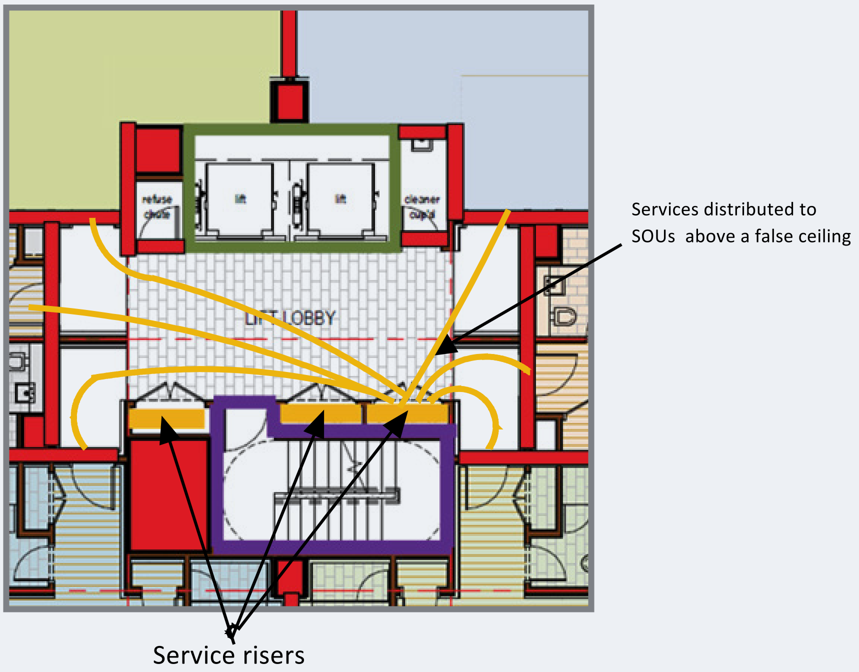


Figure 5.12: Typical position of service risers in the example apartment building.

Fire compartmentation can be maintained by protecting the service penetrations at each floor level or using fire-resisting construction for the risers (shafts), fitting fire doors or fire-rated access panels to the risers and fire protecting each service where it penetrates the riser wall.

The face of openings cut in fire-protected timber members needs to be protected so that no timber is exposed. The required RISF or MRISF and the FRL of the fire-protected timber must also be maintained. Continuing fire-protective coverings around the opening is a typical solution

Generally, the option of protecting the service penetrations at each floor level is the most practical solution. This can be achieved by forming an opening in the floor such that no timber is exposed, the FRL and RISF or MRISF is not compromised and services can be run through the opening. The opening can be protected by a multi-service penetration system such as a pillow, mineral fibre batt or other proprietary fire protection system that can be readily reinstated if additional services need to be run. A typical example is shown in Figure 5.13.

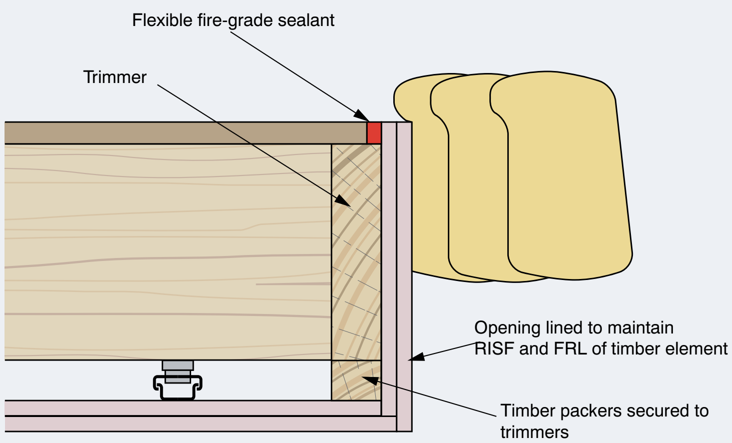


Figure 5.13: Typical riser penetration detail through a fire-protected timber floor.

Using a framed opening also avoids the need to expose cavities and timber members if additional services need to be run substantially reducing the risk of cavity fires and premature ignition of timber members. Refer to Section 5.14 for further details on the selection of service penetration systems.

For horizontal distribution of services, ducts may be created or more commonly the services can be run above a false ceiling fitted below a fire-protected timber floor/ceiling system in the lift lobby and corridors. For the apartment building example, the services could be distributed to the individual apartments from the risers as shown schematically in Figure 5.12 and Figure 5.14.

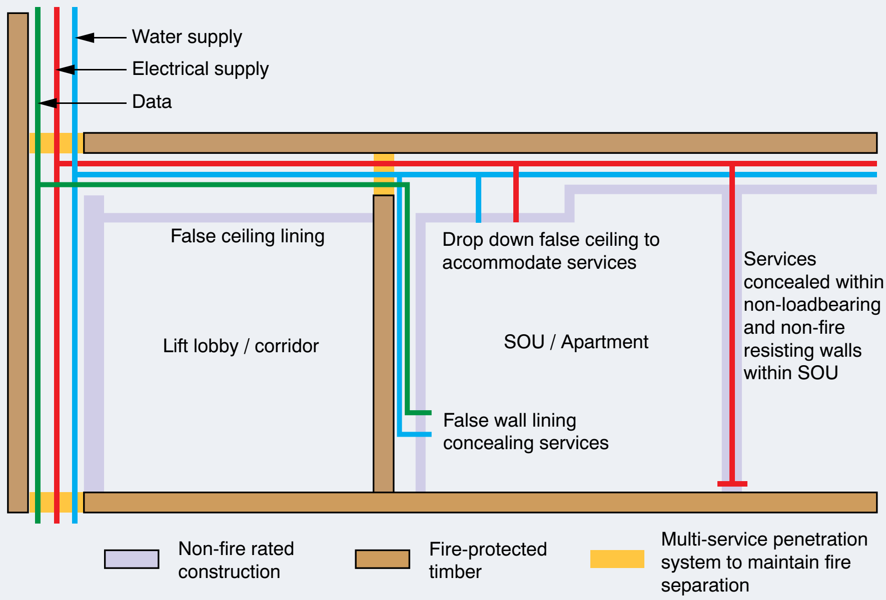


Figure 5.14: Typical distribution of services for apartment level.

The use of false ceiling and false wall linings within SOUs to conceal services can provide a number of advantages including a significant improvement in the reliability of fire-protection systems by:

Planning service runs carefully within SOUs and the use of false ceiling and wall linings can substantially reduce the number of service penetrations through fire-protected timber.

- avoiding large numbers of individual service penetrations within fire-protected timber members for services such as power outlets, lighting, plumbing services including fire sprinklers
- concealing pipework and cable runs
- allowing reconfiguration of services within an SOU without disrupting fire-protected timber elements
- reducing the risk of cavity fires during maintenance activities and the risk of fire spread to cavities if the fire protection of services is not reinstated after reconfiguration or repair to services
- enabling services to be grouped together and protected by a single multi-service penetration system fitted above the false ceiling (access panels can be provided to facilitate access for inspection and/or adding or modifying existing services).

The use of the false wall and ceiling linings can have additional benefits, such as reducing sound transmission and improving energy efficiency by reducing leakage/flanking paths and providing an additional layer of protection.

Internal walls within SOUs that are not required to achieve an FRL can be a practical option for the location of services such as power outlets.

Services can be run through internal walls within an SOU without the need for protection provided the wall is not required to be of fire-resisting construction. Note: Loadbearing members are required to achieve an FRL even if they are not part of the SOU boundary.

5.4.4 Service shafts

The use of centralised service risers is not a practical solution for some services including:

- soil and waste pipes requiring a minimum fall to avoid blockages
- waste chutes
- ventilation, exhaust and pressurisation systems.

Generally, these services require fire-resisting shafts that may be distributed across the floor plate where it is impractical to run services back to a central shaft.

The distribution of service shafts across a typical apartment floor in the example building is shown in Figure 5.15. The service shaft positions within the SOUs serve the kitchen areas and bathrooms and are generally located close to these areas to minimise penetrations through fire-resisting construction and the height required for the fall on pipes. The shaft positions should also be selected to minimise the impact of sound transmission to adjacent SOUs.

The use of false walls to conceal the service runs to the shafts is recommended as an alternative to running services through fire-protected timber walls and protecting the penetrations at the point of entry and exit.

Refer to Sections 5.8 and 5.10 for examples of appropriate shaft constructions.

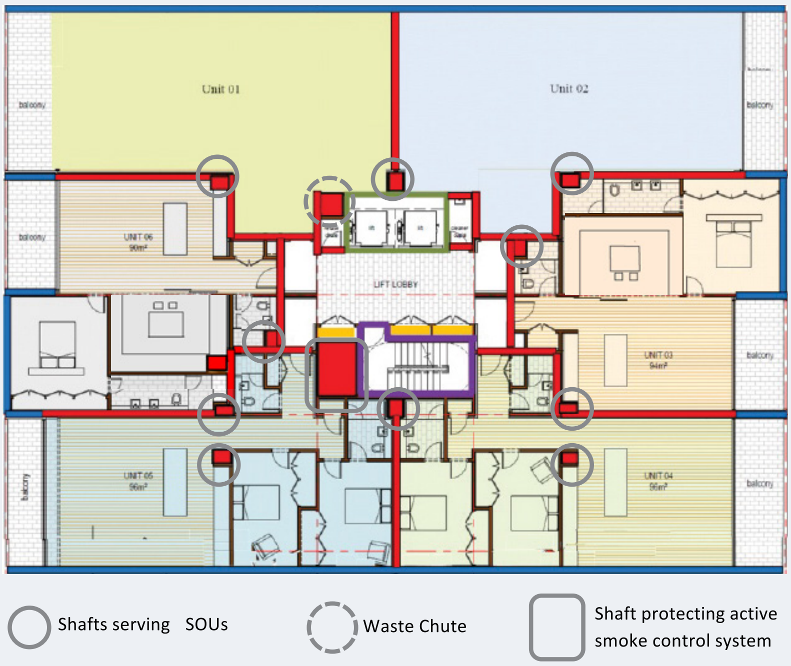


Figure 5.15: Typical shaft locations.

Shafts are generally required to have a sound rating. Where they are also required to have a Fire Resistance Level, it is best to treat the shaft like an independent compartment (Figure 5.16). Care is needed to ensure the sound rating is achieved, as many wall systems are not adequate on their own.

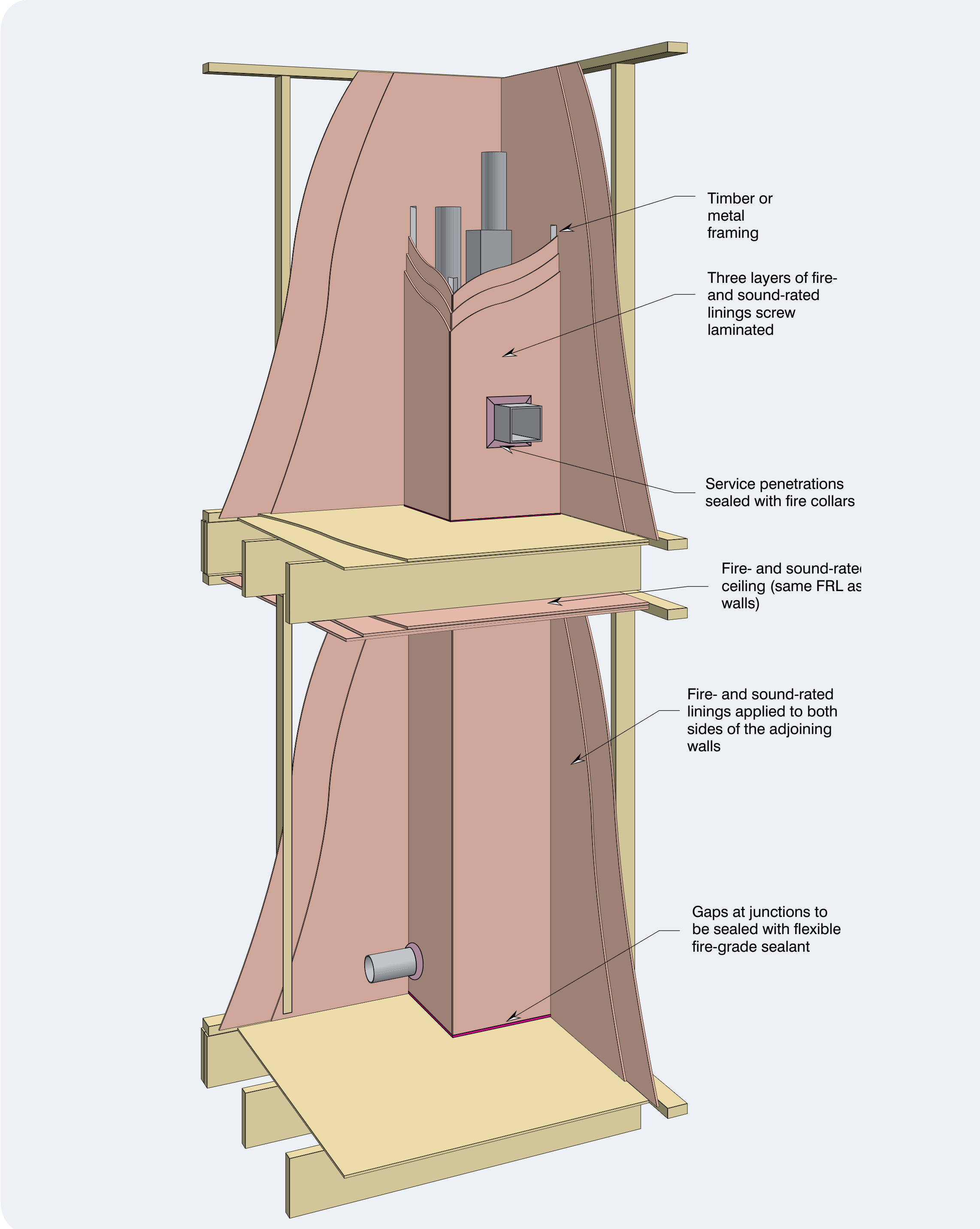


Figure 5.16: Fire-rated service duct.

Where timber framing is used to support the shaft linings it must be sheeted with fire/sound-grade linings on both sides of the shaft, including the part of the shaft that is the bounding wall of the SOU.

An alternative to using timber framing is to use laminated plasterboard or a shaft wall systems. These systems are proprietary, developed by lining manufacturers, and reference to their details is required. Refer to Figure 5.17 for an illustration of a typical fire-grade plasterboard system.

The number of layers, type and thickness of plasterboard and fixing methods selected depend on the required FRL and Evidence of Suitability that is available. Generally, for Class 2 and 3 buildings, an FRL of –/90/90 is required for non-loadbearing service shaft walls but this is reduced to –/60/60 for non-loadbearing fire-protected timber shaft walls or –/45/45 for non-combustible shaft walls if the concessions in NCC Specification E1.5a apply.

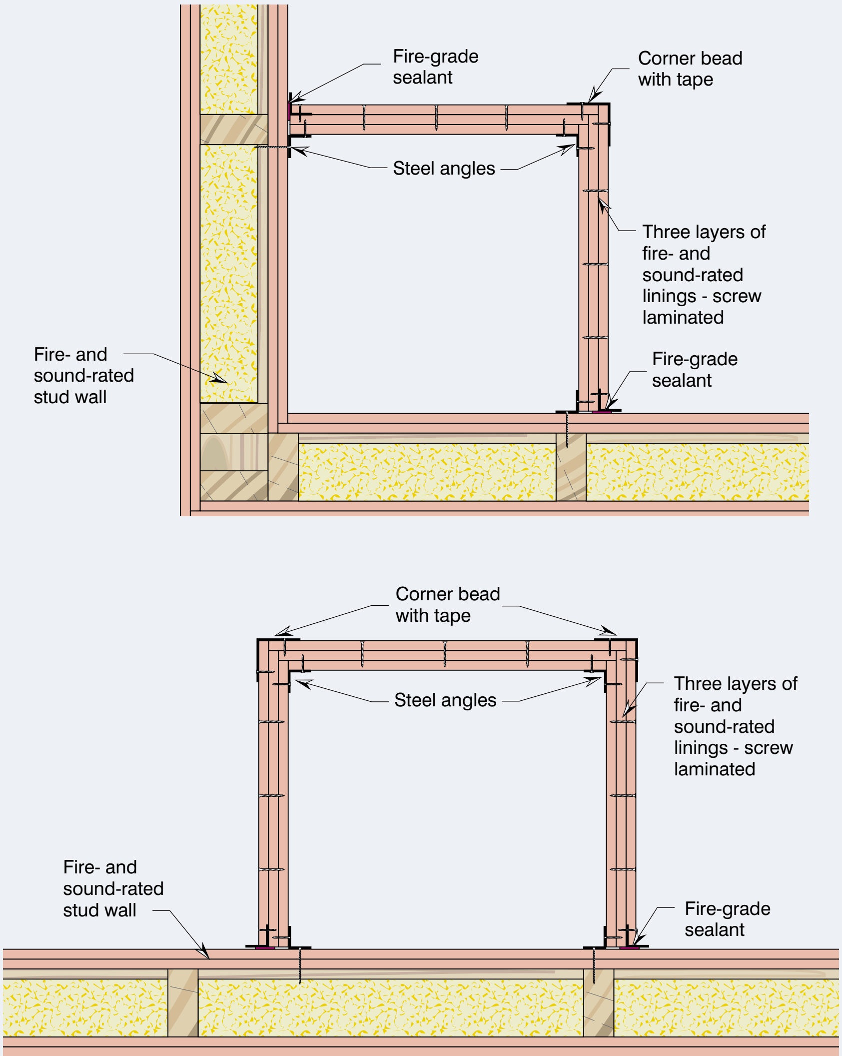


Figure 5.17: Laminated fire-grade plasterboard used to create shafts.

5.5 Fire-Protected Timber External Walls

External walls must be designed to satisfy a range of criteria including:

- fire performance
- structural performance (for safety and serviceability)
- weather resistance (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation (the control of transmission of sound from external sources is not required by the NCC but may be part of a design brief or planning control).

5.5.1 Fire Performance of External Walls

The external face of the wall may form the fire-protective covering of a fire-protected timber element, e.g. brick veneer construction as shown in Figure 5.18. If this option is used the specification will need to address the installation of cavity barriers to ensure correct placement and that moisture is not transported from the internal brickwork face to the timber frame through the cavity barrier.

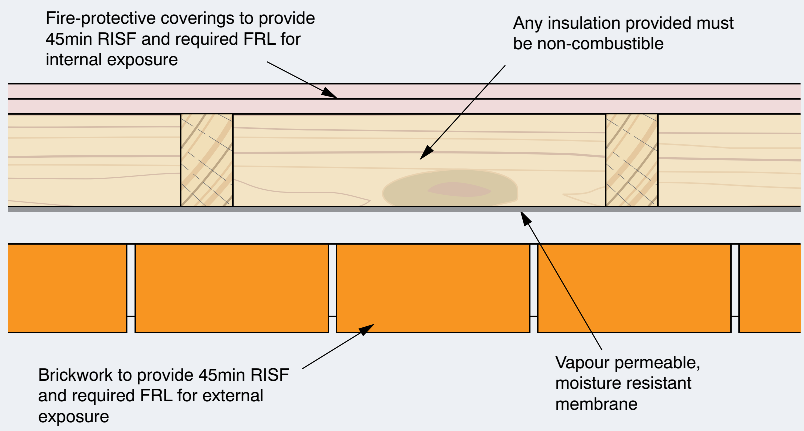


Figure 5.18: Fire-protected timber brick veneer external wall.

Alternatively, a cladding system may be fixed to a fire-protected timber element to prevent water penetration and serve other non-fire related functions. The cladding system could be a direct fix system or ventilated systems as shown schematically in Figures 5.19 and 5.20 for lightweight timber-frame and massive timber construction, respectively. These figures may not show all components that form part of proprietary systems.

Many massive timber panels are proprietary products. Fire (and other) properties depend on the adhesives used and manufacturing processes, which are currently not fully standardised. Evidence of Suitability for massive timber external wall systems will tend to be product specific in most instances and configurations will tend to vary to satisfy the relevant NCC and other design requirements.

Fixings for the cladding system must be detailed so that the performance of the fire-protective coverings is not compromised.

The NCC DTS provisions require the external walls to be of non-combustible construction. Therefore any cladding systems applied to fire-protected timber external walls in mid-rise buildings must be non-combustible to comply with the NCC DTS provisions.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers. Further details of the evidence required are provided in Appendix C.

EWFA RIR 37401400 available from the WoodSolutions website determines that non-combustible external cladding systems can be fitted to fire-protected timber walls without compromising the FRL, RISF or MRISF.

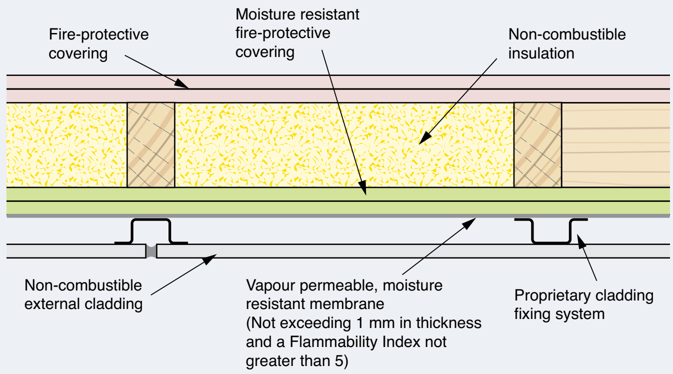


Figure 5.19: Fire-protected timber frame external walls with lightweight cladding.

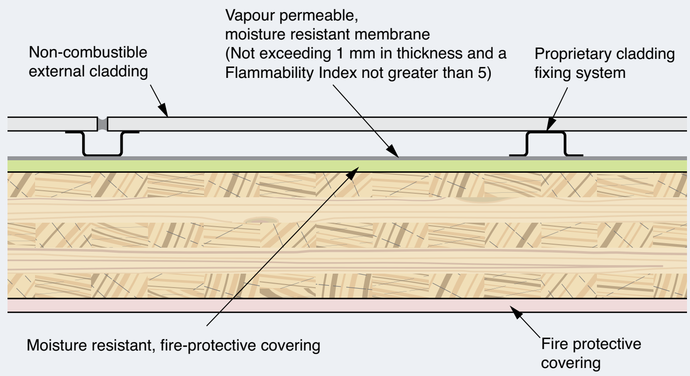


Figure 5.20: Fire-protected massive timber external wall with external lightweight cladding.

If combustible cladding systems are to be used, the performance pathway has to be adopted to demonstrate compliance of the external wall system with the relevant NCC performance requirements. Verification method CV3, in conjuction with verification methods CV1 and CV2, and the classification standard AS 5113 define an appropriate method for demonstrating compliance in most States and Territories. Further guidance is provided in Section 4.6.3.

Figure 5.21 shows the external walls, along with other wall elements, in the example Class 2 building and Table 5.1 summarises the required FRLs and RISF or MRISF based on the distance from the boundary.

While there are significant reductions in the required FRLs for non-loadbearing elements as the distance from the fire source feature increases, the design of the external walls will not vary significantly because the required RISF or the MRISF, in combination with the minimum thickness requirement of 75 mm for massive timber, will become the dominant design factors.

If the subject building is of massive timber construction and is not more than 1 metre from a fire source feature, the required MRISF is increased to 45 minutes externally to minimise the risk of fire spread from adjacent structures.

Table 5.1: Fire-resistance requirements for external walls in the example Class 2 building of Type A construction.

Distance from fire source feature	FRL – Structural Adequacy /Integrity/ Insulation – minutes		General Timber	Massive Timber
	Load bearing	Non Load bearing	RISF (minutes)	MRISF (minutes)
≤1.0 m	90/90/90	-/90/90	45	45 external 30 internal
<1.5 m	90/90/90	-/90/90	45	30
≥1.5 and <3 m	90/60/60	-/60/60	45	30
≥3 m	90/60/30	-/-/-	45	30
External Columns	90/-/-	-/-/-	45	30

5.5.2 External Noise

Currently, there are no NCC requirements to provide external noise attenuation for buildings. However, Government authorities have regulatory or legislative powers to require control of noise entering buildings; particularly residential buildings. These requirements vary around Australia and designers and specifiers should make enquiries in relation to their design/development.

The *WoodSolutions Technical Design Guide #11 Timber-framed Systems for External Noise* provides examples of lightweight external wall systems that can be used as guidance.

5.5.3 Weatherproofing

There are currently no Deemed-to-Satisfy Provisions in the NCC in relation to the weatherproofing of external walls and so suppliers of waterproofing products/membranes are relied on to demonstrate compliance with the NCC Performance Requirement (FP1.4). A weatherproofing Verification Method (FV1.1) is described in the NCC to enable compliance with FP1.4 via a tested prototype. It is important that installed waterproofing membranes/systems for timber construction are vapour permeable (i.e. allowing timber building components to breathe) but do not permit water to penetrate through to the structural timber building elements (moisture resistant).

5.6 Internal Walls Bounding SOUs and corridors

Internal walls bounding the SOUs are shown as red walls in Figure 5.21, must be designed to satisfy a range of criteria including:

- fire performance
- structural performance (for safety and serviceability)
- durability
- acoustic separation.

Figure 5.21: Example Class 2 building highlighting fire-protected timber walls.

For lightweight timber-framed construction, double stud partition systems are commonly adopted in order to achieve the required acoustic separation.

For massive timber elements it may be necessary to install a multi-layered, fire-protective covering made of non-combustible, fire-protective grade plasterboard and insulation (Figure 5.23) to improve acoustic performance.

5.6.1 Fire Performance of Walls Bounding SOUs

EWFA report 22567A -01 issued to FWPA assesses the FRL of some typical double stud wall systems

Figures 5.22 and 5.23 show typical lightweight timber frame wall systems and massive timber wall systems satisfying the NCC Deemed-to-Satisfy fire requirements for bounding construction in the example Class 2 building. While the double stud arrangement provides good acoustic separation, the central cavity will need to be protected by cavity barriers to limit the risk of fire spread through concealed spaces. Acoustic separation is not as good with a single stud system but if 90 x 45 mm studs or larger are used the timber framing members can act as in situ cavity barriers without the need for additional dedicated cavity barriers to be fitted within the wall system.

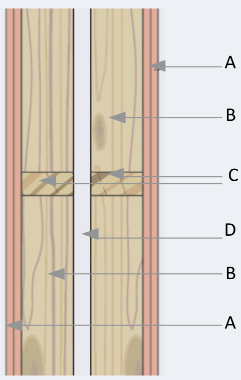
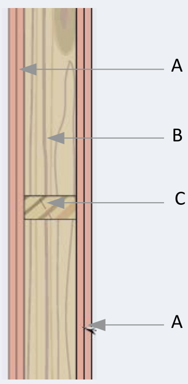
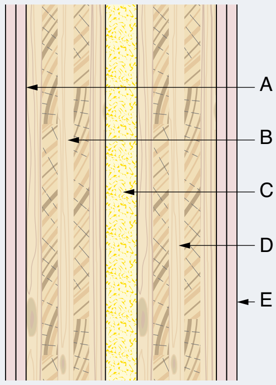
Double Stud	Single Stud
	
Construction **A** linings 2 x 13 mm fire-protective grade plasterboard **B** timber studs typically 90 x 45 **C** Noggings at mid-height **D** Cavity min 20mm Note Non Combustible Cavity Insulation may be added	
Performance FRL 90/90/90 obtain Evidence of Suitability from supplier[21] RISF 45 mins (NCC Spec C1.13a DTS)	**Performance** FRL 90/90/90 obtain Evidence of Suitability from supplier RISF 45 mins (NCC Spec C1.13a DTS)

Figure 5.22: Typical fire-rated lightweight timber-framed wall systems for Class 2 mid-rise buildings.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers



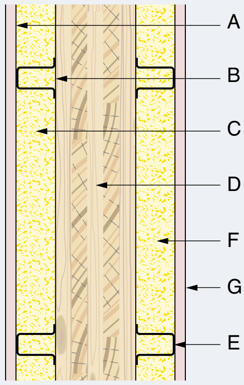
Construction

A – 2 x 13 mm fire-protective grade plasterboard
B – 95 mm CLT
C – 60 sound absorbing insulation
D – 95 mm CLT
E – 2 x 13 mm plasterboard

Performance

Fire resistance – 90/90/90
MRISF – 45 minutes

Note: Performance stated is based on 1 x 13 mm thick fire-protective grade plasterboard lining but 2 x 13 mm thick fire-protective grade plasterboard linings are required due to the 'cavity' between CLT panels, therefore the NCC General Requirements apply.



Construction

A – 16 mm fire-protective grade plasterboard
B – 60 metal batten (nominal)
C – 60 mm glasswool insulation
D – 95 mm CLT
E – 60 metal batten (nominal)
F – 60 mm glasswool insulation
G –16 mm fire-protective grade plasterboard

Performance

Fire resistance – 90/90/90
MRISF – 30 minutes

Note: Performance stated is based on 1 x 13 mm thick fire-protective grade plasterboard lining but 1 x 16 mm thick fire-protective grade plasterboard lining is required to achieve a MRISF of 30 minutes.

Figure 5.23: Typical fire-rated massive timber proprietary wall systems for mid-rise Class 2 buildings.
Note: Confirm the system performance with CLT supplier

The design of the massive timber walls tend to be largely influenced by acoustic considerations and there may be opportunities to refine the above design to optimise both acoustic and fire performance.

5.6.2 Sound Performance of Walls Bounding SOUs

Using the lightweight timber frame wall systems and massive timber wall systems satisfying the NCC Deemed-to-Satisfy fire requirements for bounding construction in the example Class 2 building, the typical acoustic performance of timber-framed systems is shown in Figure 5.24.

The double stud arrangement is typically used as it provides good acoustic separation as internal wall systems separating SOUs with the central cavity protected by cavity barriers to limit the risk of fire spread through concealed spaces. Acoustic separation is not as good with a single stud system but if 90 x 45 mm studs or larger are used the timber framing members can act as in situ cavity barriers without the need for additional dedicated cavity barriers to be fitted within the wall system.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers

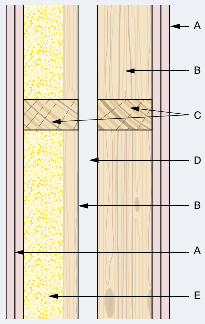
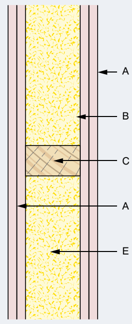
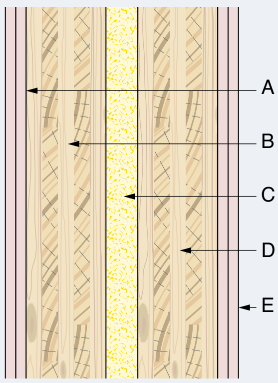
Double Stud	Single Stud
 	
Construction **A** Linings 2 x 13 mm fire-protective grade plasterboard **B** Timber studs typically 90 x 45 **C** Noggings at mid-height **D** Cavity min 20 mm **E** 75 mm mineral wool insulation	**Construction** **A** Linings 2 x 13 mm fire-protective grade plasterboard **B** Timber studs typically 90 x 45 **C** Noggings at mid-height **D** Cavity 15 mm (max.) **E** 75 mm mineral wool insulation
Performance FRL 90/90/90 obtain Evidence of Suitability from supplier RISF 45 mins (NCC Spec C1.13a DTS) **Acoustics:** $R_w + C_{tr} \geq 50$	**Performance** FRL 90/90/90 obtain Evidence of Suitability from supplier RISF 45 mins (NCC Spec C1.13a DTS) **Acoustics**: $R_w < 50$ ($R_w + C_{tr} \geq 50$) can be achieved when using 120 x 35 timber studs suitable for use as bounding wall construction between SOUs and public corridors, stairways, etc – refer Section 2.2)

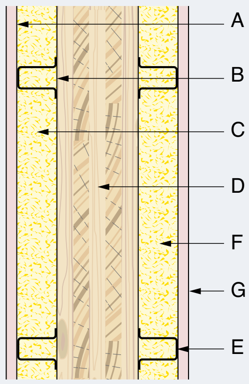
Figure 5.24: Typical acoustic-rated lightweight timber framed wall systems for Class 2 mid-rise buildings.

The design of the massive timber walls tend to be largely influenced by acoustic considerations and performance many vary depending on the manufacturing of the product. The acoustic performance of various tested CLT wall configurations can be found in the WoodSolutions Technical Design Guide *#44 CLT Acoustic Performance.*



Construction
A – 2 x 13 mm fire-protective grade plasterboard
B – 95 mm CLT
C – 60 sound absorbing insulation
D – 95 mm CLT
E – 2 x 13 mm plasterboard
Performance
Fire resistance – 90/90/90
MRISF – 45 minutes
Acoustic - Rw ≥ 50

Note: Performance stated is based on 1 x 13 mm thick fire-protective grade plasterboard lining but 2 x 13 mm thick fire-protective grade plasterboard linings are required due to the 'cavity' between CLT panels, therefore the NCC General Requirements apply.



Construction
A – 16 mm fire-protective grade plasterboard
B – 60 metal batten (nominal)
C – 60 mm glasswool insulation
D – 95 mm CLT
E – 60 metal batten (nominal)
F – 60 mm glasswool insulation
G –16 mm fire-protective grade plasterboard
Performance
Fire resistance – 90/90/90
MRISF – 30 minutes
Acoustic - Rw ≥ 50

Note: Performance stated is based on 1 x 13 mm thick fire-protective grade plasterboard lining but 1 x 16 mm thick fire-protective grade plasterboard lining is required to achieve a MRISF of 30 minutes.

Figure 5.25: Typical acoustic-rated massive timber proprietary wall systems for mid-rise Class 2 buildings.
Note: Confirm the system performance with CLT supplier

5.7 Fire-protected Timber Floors

EWFA RIR 37401400 can be found on the WoodSolutions website that assesses the FRL and RISF of commonly available floor joist products

Floor systems must be designed to satisfy a range of criteria including:

- structural performance (for safety and serviceability)
- fire performance
- acoustic separation
- durability.

Common structural elements used for timber floors include:

- solid timber beams
- LVL beams
- I-section beams with OSB or plywood webs
- parallel chord steel web trusses
- parallel chord timber web truss
- I-section with Steel Web
- massive timber panel systems (e.g. CLT or LVL).

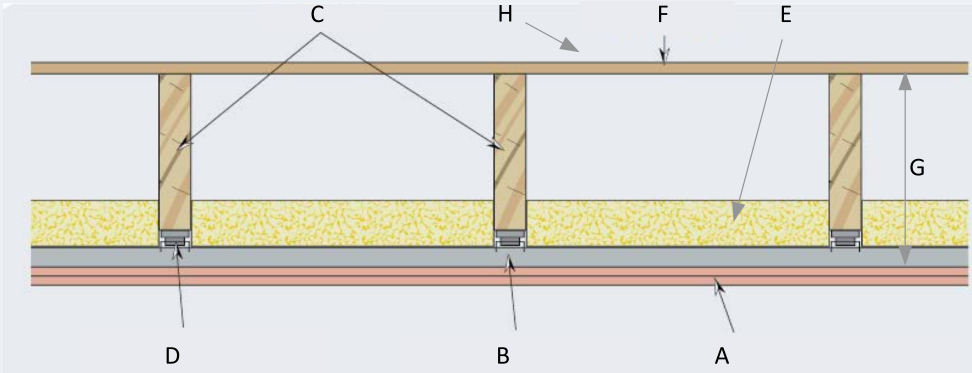
These structural members can be used with a range of flooring systems, internal insulation systems and soffit/ceiling lining systems in keeping with the building finishes and to achieve the required fire and acoustic performance.

5.7.1 Fire Performance of Flooring Systems Protected by Typical Ceiling Systems

Typical floor systems that may satisfy the fire-related NCC Deemed-to-Satisfy fire requirements for fire-protected timber in the example Class 2 building are shown in Figures 5.26 and 5.27.

These particular systems incorporate a ceiling system comprising two layers of 16 mm thick fire-protective grade plasterboard secured to steel furring channels supported from the structural element. Since the ceiling provides the largest contribution to the FRL of the floor/ceiling systems, and the performance will be largely independent of the structural members prior to structural failure, the results can be applied to a large range of combinations of structural element, cavity insulation and flooring systems to which additional finishes may be applied, provided compliance with other all NCC requirements is not compromised.

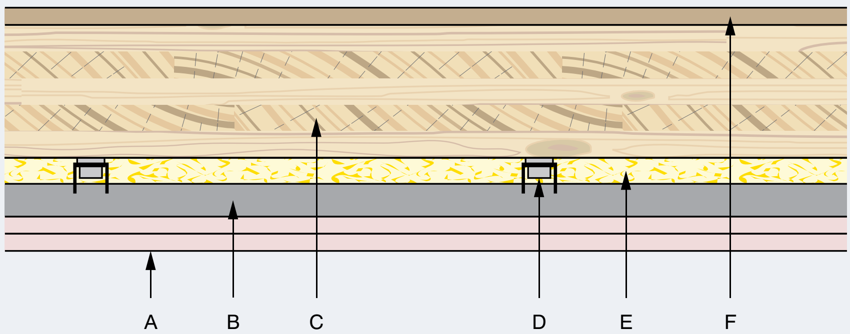
EWFA RIR 37401400 can be found on the WoodSolutions website that assesses the FRL and RISF of the floor system shown in Figure 5.26



Construction
A Linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier and/or EWFA report RIR 37401400)
RISF 60 mins (verify with supplier and/or EWFA report RIR 37401400) or RISF 45 mins (NCC Spec C1.13a DTS)

Figure 5.26: Typical timber-framed floor.



Construction
A Ceiling linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Massive Timber Panel (min 75 mm thick)
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)
E Cavity minimum 40 mm (must be filled with non combustible insulation if massive timber 'concession' applied otherwise may be unfilled)
F Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier). Note higher FRLs may be achieved depending upon massive timber element design
MRISF/RISF 60 mins (verify with supplier) or MRISF/RISF> 45 mins (NCC Spec C1.13a DTS)

Figure 5.27: Typical massive timber panel floor.

For timber-framed floor systems, in particular, the required thickness of the fire-protective coverings tend to be dominated by the FRL criteria (90/90/90 or above) rather than the RISF criteria of 45 minutes.

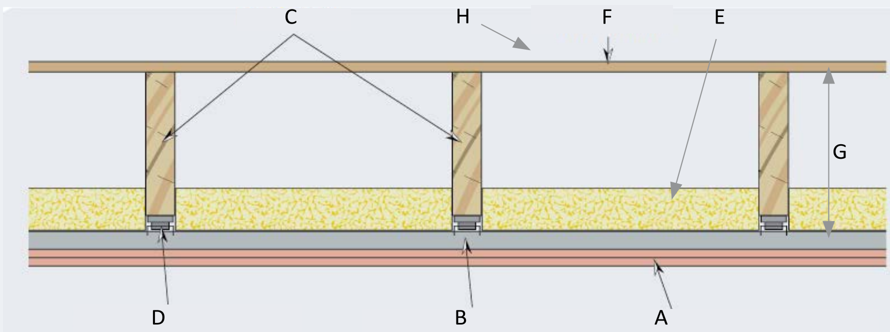
For massive timber panel floor systems, where thicknesses of 150 mm or more may be required to achieve adequate structural performance, the required thickness of fire-protective coverings will tend to be dominated by the MRISF or RISF criteria or acoustic considerations rather than FRL criteria because of the high inherent fire resistance of the massive timber panels.

In many situations, fire-protective coverings based on a single layer of 16 mm fire-protective grade plasterboard may be suitable for massive timber panel floor systems. However, the fire performance of these systems depends on many factors, including adhesives, number of layers and thicknesses of lamella, manufacturing process, timber species and grade, and applied loads. These factors are not fully standardised and vary between manufacturers therefore Evidence of Suitability in the form of fire-resistance test reports from Accredited Testing Laboratories should be sought from the suppliers of the specific CLT system to confirm the FRL of fire-protected timber members.

5.7.2 Sound

The sound performance of a floor/ceiling system depends on a number of elements including: the density of the floor covering (tile, timber, carpet), isolation from the structure (acoustic underlay), ceiling insulation (density), ceiling installation (acoustic mounts) and layers and thicknesses of ceiling plasterboard. The objective is to minimise both airborne (R_w+C_{tr}) and impact sound ($L_{n,w}$) transmission through the floor/ceiling system and the performance of the floor/ceiling system should be verified with the plasterboard supplier for DTS Solutions.

There are a range of flooring products (e.g. timber overlay, carpet) that can be used and achieve the minimum NCC acoustic requirements. The use of a hard flooring surface will influence the impact performance ($L_{n,w}$) of the floor/ceiling system. The following is provided for guidance.



Construction
A Linings 2 x 16 mm fire-protective grade plasterboard
B Steel furring channels at 600 mm max centres
C Timber Beams
Solid timber joists
LVL beams
I-section beams with OSB or LVL webs
I-section with steel web
Parallel chord steel web trusses
Parallel chord timber web truss
D Furring Channel Fixings
Direct fixing clips
Acoustic (resilient) mounts
Suspended ceiling (hanger support)
Suspended ceiling (acoustic hanger)

E Non-combustible cavity insulation may be added
F Flooring (minimum thickness)
Plywood – 15 mm thick
Cement sheet – 15 mm
Particle board – 18 mm
T& G flooring – thickness 15 mm
Engineered flooring – 15 mm
Concrete applied over plywood
AAC – 50 mm
Calcium silicate sheets – 15 mm
G Minimum Cavity Depth – 288 mm from fire protective grade plasterboard.
H Floor finishes such as carpet, acoustic treatments may be added

Performance
FRL 90/90/90 (verify with supplier)[xxv]
RISF 60 mins (verify with supplier)
or RISF 45 mins (NCC Spec C1.13a DTS)
Acoustics (Verify with supplier):
Above system incorporates: 10 mm overlay solid strip flooring, 4.5 mm acoustic underlay on top of flooring products (F), R2.5 ceiling insulation (E), Acoustic (resilient) mounts (D).
$R_w+C_{tr} \geq 50$
$L_{n,w} \leq 62$ (bare floor)
$L_{n,w} \leq 50$ (carpet and underlay)

Figure 5.28: Sound performance of typical timber-framed floor/ceiling system.

The acoustic performance of various tested CLT floor/ceiling system configurations can be found in the *WoodSolutions Technical Design Guide #44 CLT Acoustic Performance.*

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers.

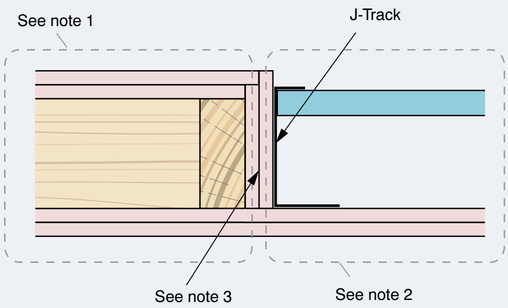
5.8 Service Shafts

While service shafts can be constructed from fire-protected timber walls, in many instances there are substantial advantages in using either steel stud shaft wall or laminated shaft wall construction particularly if the shafts are in locations where sound transmission is not a significant consideration.

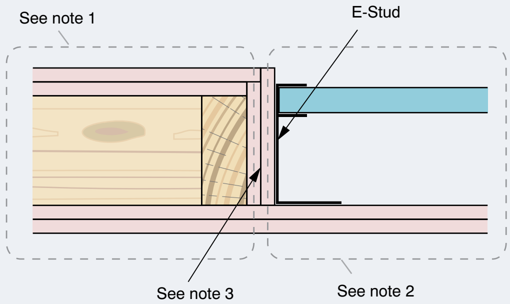
The advantages include:

- ease of construction
- smaller footprint (more usable space)
- simplification of treatment of service penetrations
- greater selection of proprietary fire protection systems for service penetrations that already have Evidence of Suitability to demonstrate the FRLs of the systems.

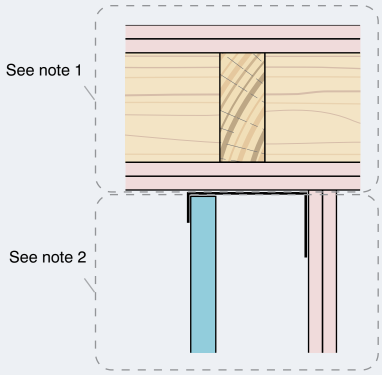
EWFA report RIR 37401400 available form the WoodSolutions website assesses the impact of the interface details in Figure 5.29 on the FRL, RISF and MRISF of the systems



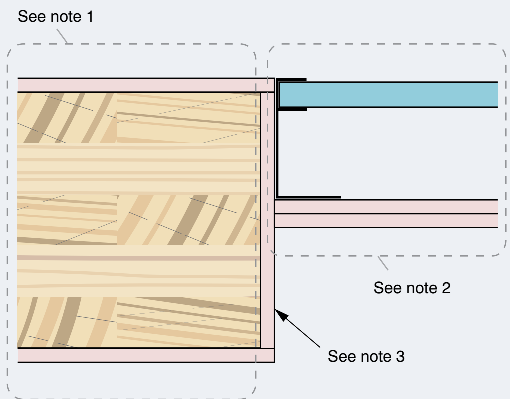
Steel stud shaft to timber stud with J-Track



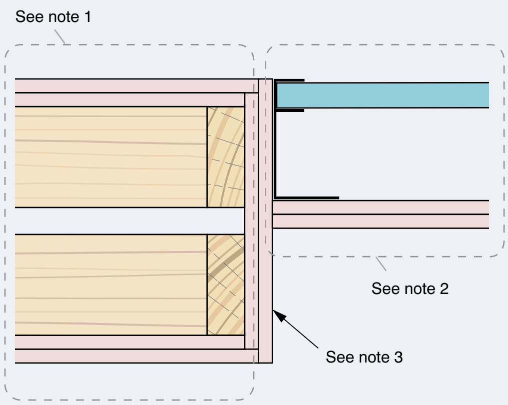
Steel stud shaft to timber stud with E-Stud



Steel stud shaft to timber stud perpendicular



Steel stud shaft to massive timber stud



Steel stud shaft to massive timber panel

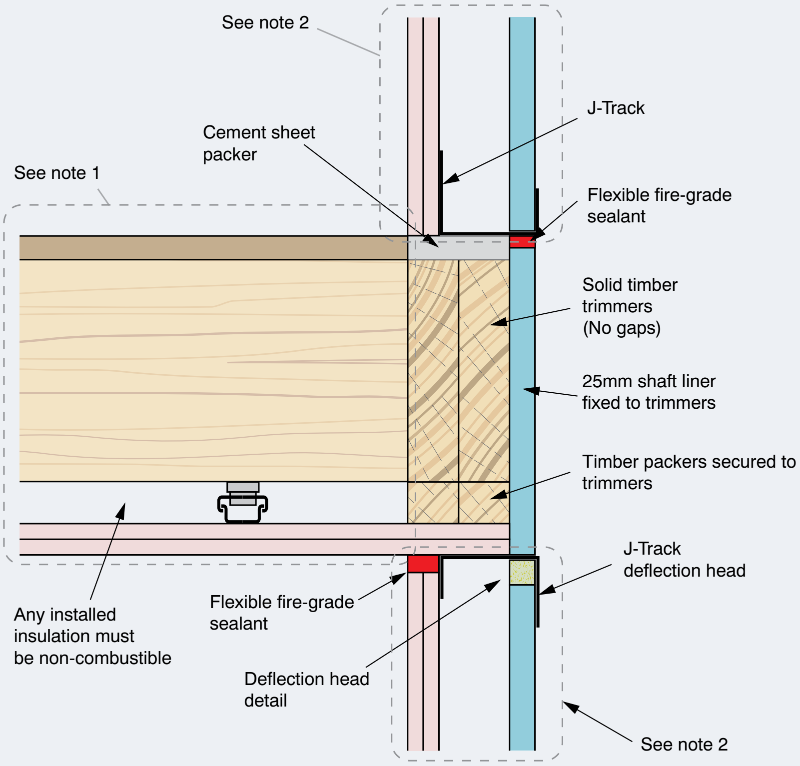
*Figure 5.29: Interfaces between fire-protected timber and steel stud shafts **(continued next page)***

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

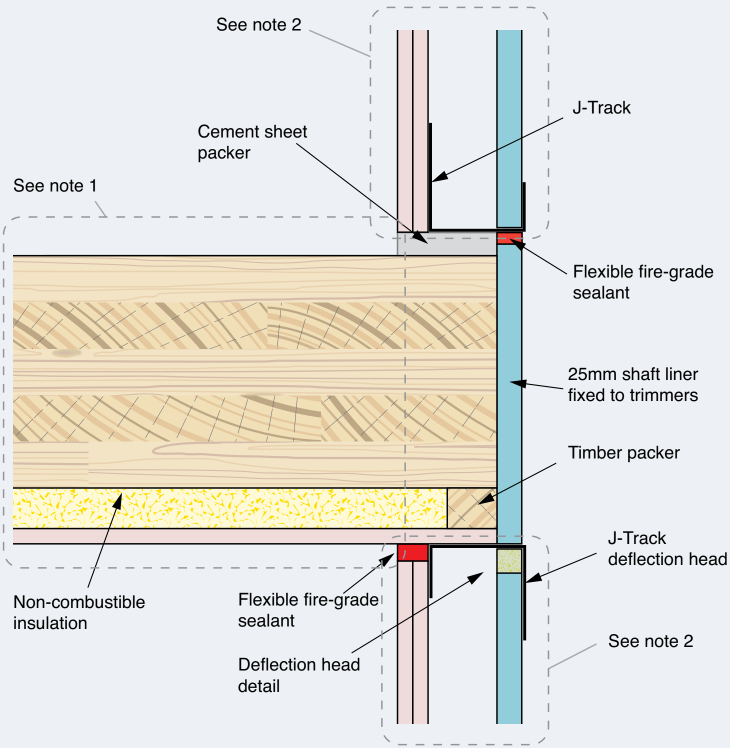
Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws.

EWFA RIR 37401400 available form the WoodSolutions website assesses the impact of the interface details in Figure 5.30 on the FRL RISF and MRISF of the systems



Shaft continuity at timber framed floor



Shaft continuity at massive timber panel floor

Figure 5.30: Interfaces between fire-protected timber and steel stud shafts (continued from previous page)

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

It is important that the fire performance is not compromised at the interfaces between the shaft and fire-protected timber walls and floors. Figure 5.30 shows typical interface details for steel framed shaft construction and Figure 5.31 shows typical interface details for laminated shaft construction. These interface details have been assessed by an Accredited Testing Laboratory (EWFA report reference RIR 37401400) and found not to compromise the performance of the wall or shaft systems.

EWFA RIR 37401400 assesses the impact of the interface details on the FRL of the systems

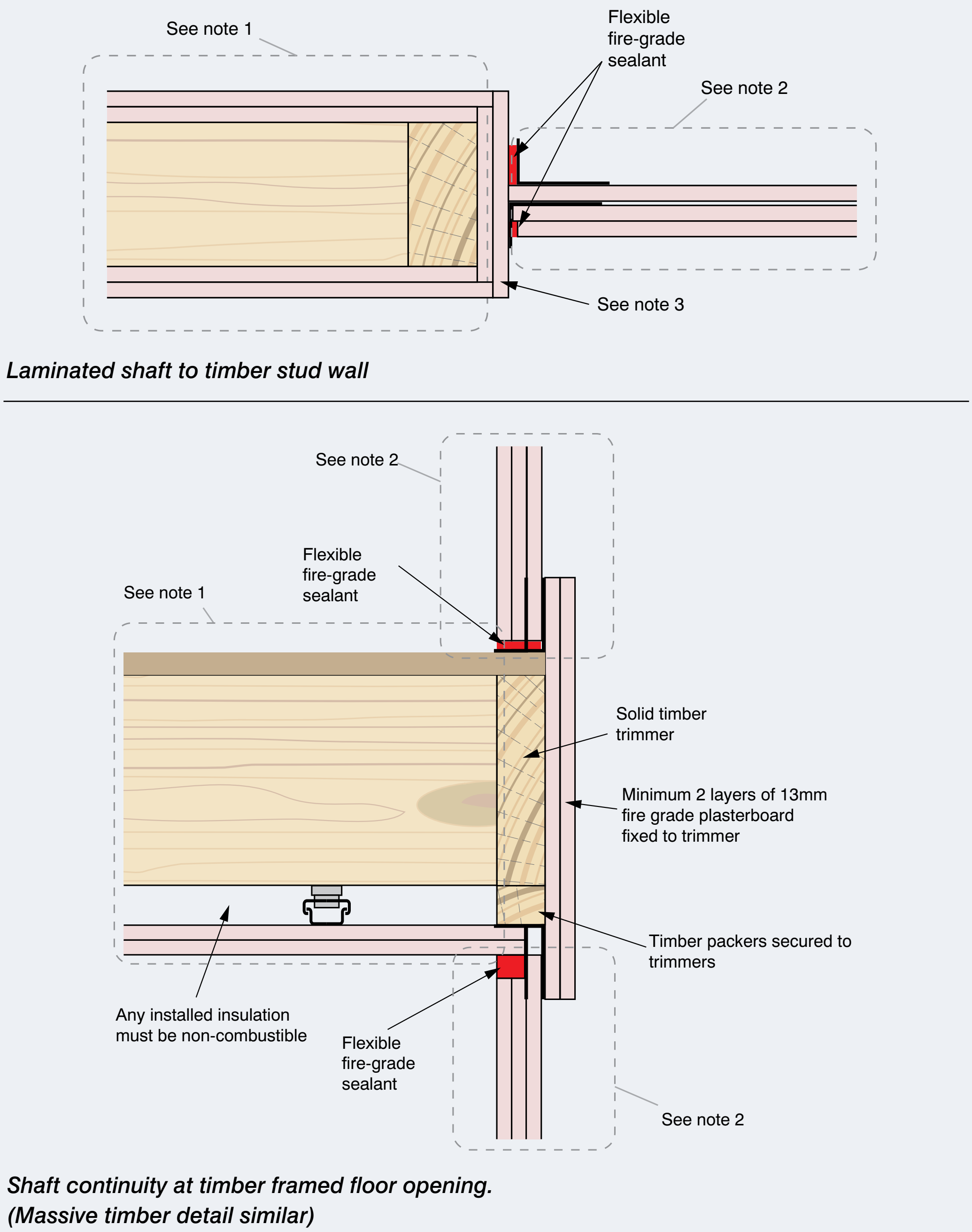


Figure 5.31: Interfaces between fire-protected timber and laminated board shafts.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face. Shaft wall tracks are to be screw fixed to timber elements at 300 mm maximum centres with 62 mm long screws.

5.9 Fire Doors in Fire-protected Timber Walls

Fire door assemblies are required to comply with AS 1905.1 as appropriate in addition to achieving the required FRL. Generally, fire doors are required to be tested when mounted in a wall of representative construction. Evidence of Suitability should therefore be provided from the supplier that relates to the performance of their fire doors when mounted in representative timber elements of construction.

In addition, the fire doors must not compromise the RISF or MRISF performance of the wall. The frame fixing details shown in Figure 5.32 have been assessed by an Accredited Testing Laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber-frame systems and 30 minutes for the massive timber panel systems (Refer EWFA report reference RIR 37401400). Other details may be adopted if appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements for fire doors and fire-protected timber elements is provided.

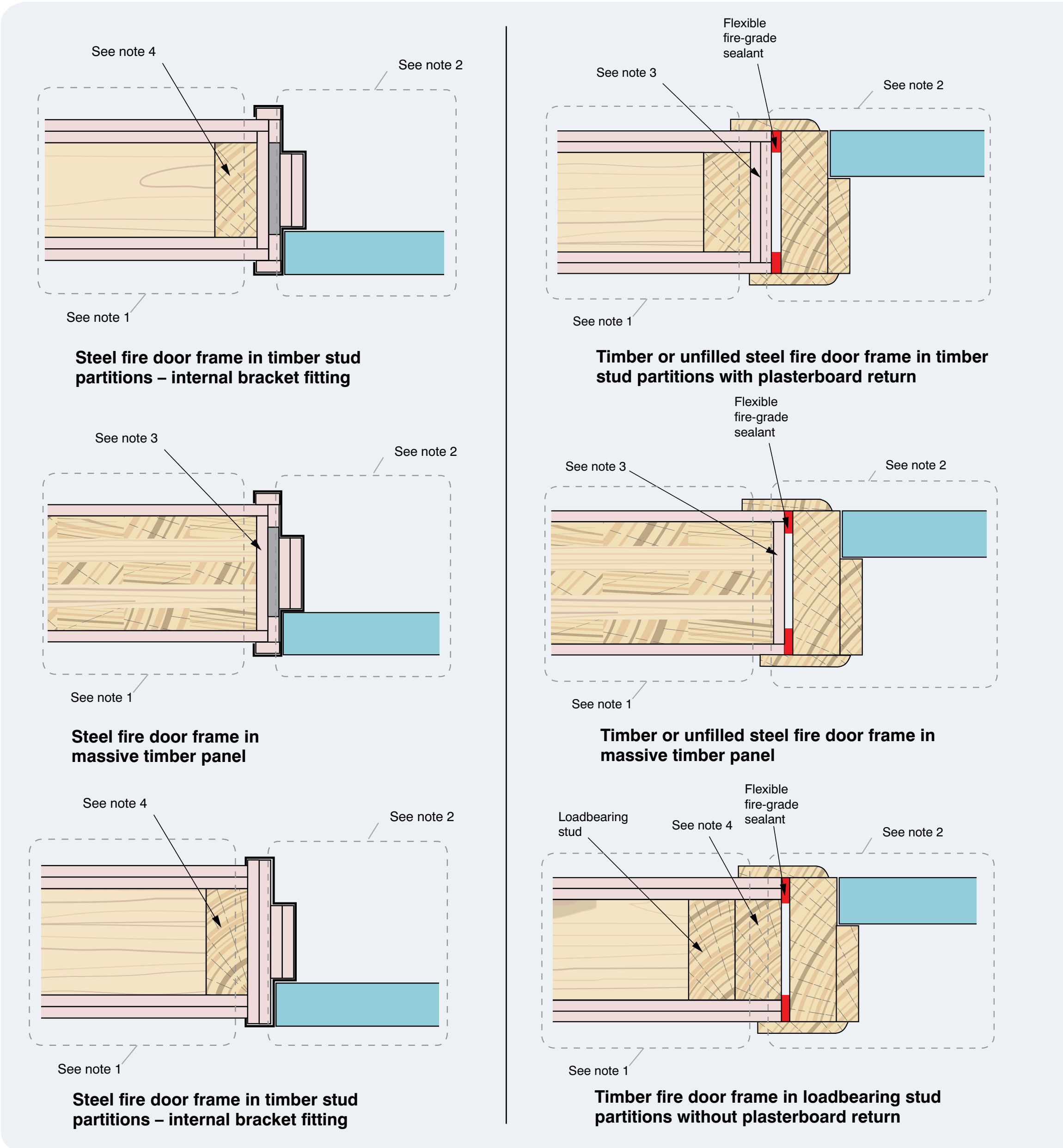


Figure 5.32: Fire door interface details.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Fire Door Assembly with the required FRL determined in accordance with AS 1530.4 and AS 1905.1 as appropriate.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Minimum of 45 mm thick non-loadbearing solid timber cavity barrier framing the cavity opening around the door.

5.10 Construction for Fire-Isolated Stair Shafts

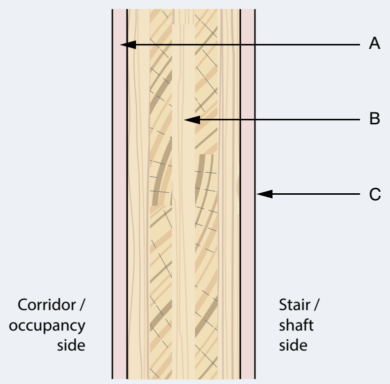
Fire-isolated stair shafts can be constructed from fire-protected timber, concrete masonry and other non-combustible non-loadbearing materials or a hybrid construction may be adopted. The selection will depend on the structural design of the building, construction programming, and other factors.

Where concrete or masonry shafts are used the design will need to account for differential movement between the shaft and timber structure.

For timber-framed construction, walls similar to those shown in Figure 5.22 could be adopted for the example building.

A further concession is provided for massive timber panels in that the fire-protective covering for the internal face of the shaft is permitted to achieve a MRISF of 20 minutes compared to the 30 minutes required for the outer face as shown in Figure 5.33 and Figure 5.34. The fire protective coverings around the face of openings for doors and access panels should be the greater of that required for the external facing or to achieve the required FRL.

Evidence of Suitability in accordance with NCC requirements should be obtained from the product suppliers



Construction
A Linings min 1 x 16 mm fire-protective grade plasterboard (corridor/occupancy side)
B Massive timber min 75 mm thick ***(note greater thickness may be required to achieve required FRL unless additional fire protective coverings applied)***
C Linings min 1 x 13 mm fire-protective grade plasterboard (internal shaft side)

Performance
FRL 90/90/90 obtain Evidence of Suitability from supplier
MRISF 30 mins (1 x 16 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS
MRISF 20 mins (1 x 13 mm fire-protective grade plasterboard) – NCC Spec C1.13a DTS

Figure 5.33: Typical stair and lift shaft construction for single skin massive timber panel construction.

Although a minimum panel thickness of 75 mm is permitted, in most instances substantially greater thicknesses will be required as part of the structural design and/or to achieve the required FRLs. The FRL should be checked to ensure the load levels during the test were comparable to the loads that will be applied under fire conditions.

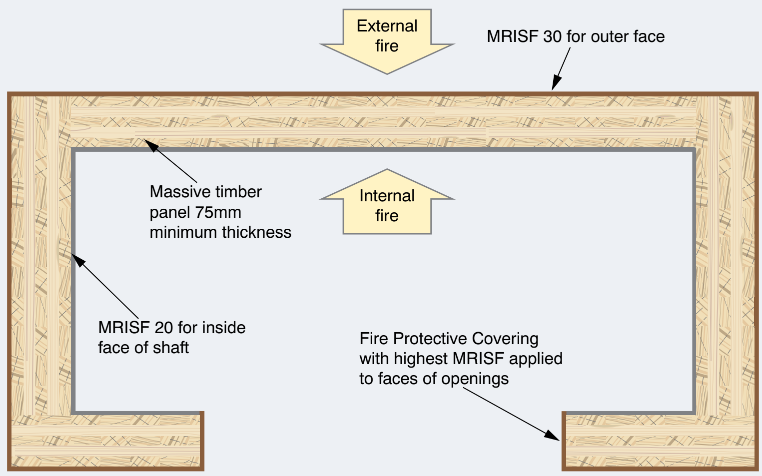


Figure 5.34: MRISF requirements for typical stair and lift shaft construction for single skin massive timber panel construction.

5.11 Construction for Stairways within Fire-isolated Stairs

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway provided the timber used:

- has a finished thickness of not less than 44 mm with an average timber density of not less than 800 kg/m^3 (at 12% moisture content).
- the building is protected throughout by a sprinkler system complying with Specification E1.5 (other than a FPAA101D or FPAA101H system) that is extended to provide coverage within the fire-isolated enclosure
- the underside of flights of stairs directly above landings providing access to ground level or car parking levels being protected by a single layer of 13 mm fire-protective grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres (Figure 5.35).

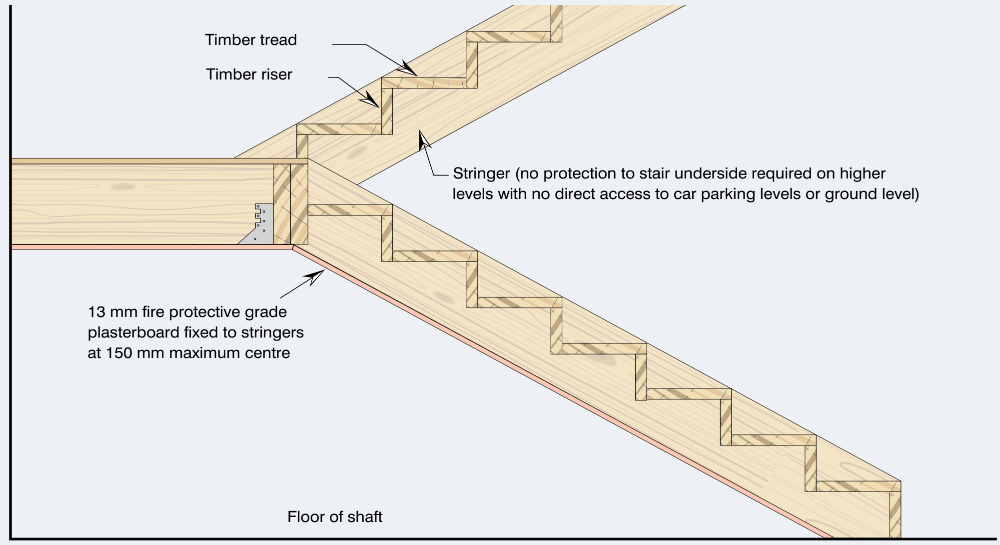


Figure 5.35: Stairway fire protection.

Refer to Section 5.15 for further information about sprinkler installations.

Impact sound from stair usage may vibrate the stair shaft walls, creating a pathway for sound transmission. A practical way to prevent this is by isolating the support for the stair structure by using stringers to support the stairs (top and bottom) rather than the wall adjoining areas requiring sound isolation (Figure 5.36). In some instances, newel posts to support the stringers may be necessary.

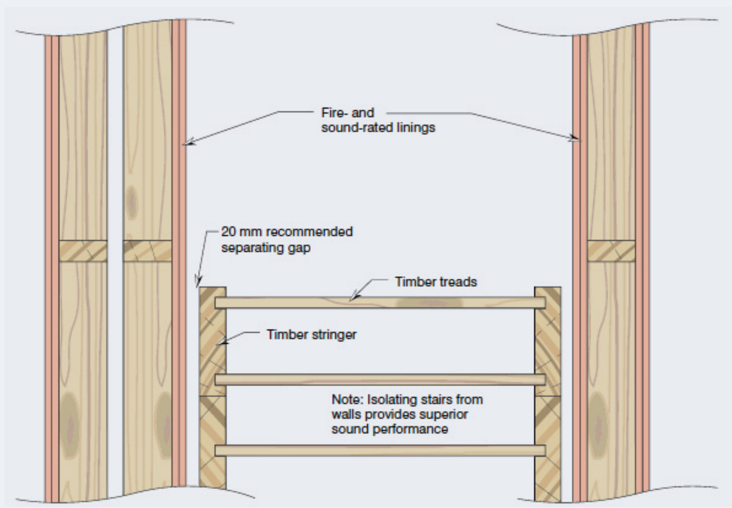


Figure 5.36: Sound isolation of stairway.

If a non-combustible stair (e.g. steel) is installed within the fire-protective timber stair shaft, the sprinkler system does not require extending to provide coverage in a fire-isolated stair.

5.12 Construction for Lift shafts

Lift shafts can be constructed in a similar manner to stair shafts as described in Section 5.10.

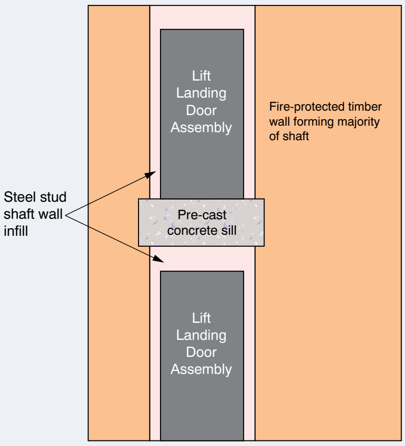
Care is needed to ensure that the lift shaft is compatible with the selected lift system. Compatibility issues should be resolved early in the design process and early liaison with the lift supplier is strongly recommended.

In the short term, most lift landing door assemblies will have been fire tested in masonry/concrete or steel stud shaft wall systems. The following details provide an interface between fire-protected timber and a pre-cast concrete sill and steel shaft wall systems. This can enable lift doors to be installed within sections of the wall of steel stud /plasterboard shaft wall and concrete construction to which existing lift landing door fire-resistance test results can be applied.

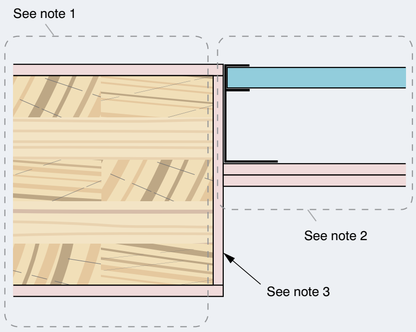
In the longer term, a larger range of lift landing doors is expected to be fire tested in fire-protected timber construction, providing simpler installation details.

The interface details in Figure 5.37 have been assessed by an Accredited Testing Laboratory (refer EWFA report RIR 37401400). The applicability of the Evidence of Suitability to a particular application should be checked with the authority having jurisdiction.

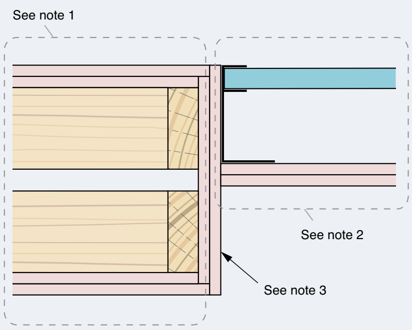
Impact sound from lift use may vibrate the lift shaft walls, creating a pathway for sound transmission. While this can be addressed to some extent using double stud wall assemblies or twin-skin massive timber panel construction utilising two layers of 13 mm plasterboard, there are other options, such as the construction of a framework within the lift shaft that supports the lift assembly independently of the shaft walls.



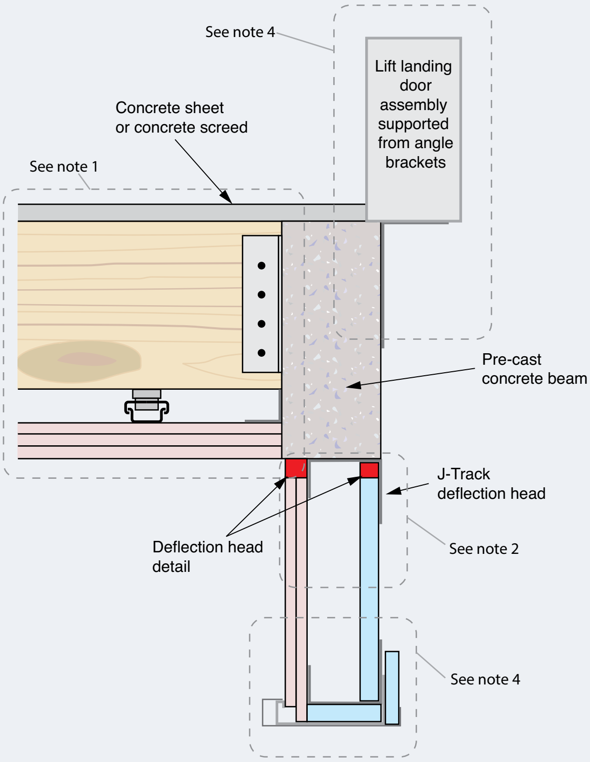
Elevation of lift doors in shaft



Side interface between shaft-wall and double timber stud shaft



Side interface between shaft-wall and massive timber panel



Head and sill detail for interfaces with shaft wall and concrete sill (timber-frame)

Figure 5.37: Typical details for shaft wall conversion for lift-landing door installation.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Lift landing door assembly having the required FRL installed in accordance with the lift door and shaft wall supplier instructions and evidence of compliance confirming the FRL.

5.13 Cavity Barriers and Junction Details

5.13.1 Typical Junction Details at Intersection of Fire-Protected Timber Walls and Floors

Cavity barriers are required at the junctions between fire-protected timber floor assemblies and fire-protected timber walls in framed construction. In many instances the Deemed-to-Satisfy solutions permitting the use of solid timber and/or mineral fibre enable integration of cavity barriers with typical wall and floor junction details.

The key design parameters are to achieve, as a minimum, the required seal thickness in the direction of potential fire spread through the cavity and ensure the seals are continuous.

Typical details for double stud walls and external walls for the example Class 2 building are shown in Figures 5.38-5.41. These details are based on a 'ring beam' design concept which can be useful in the management of the risk of disproportionate collapse. This form of construction is also compatible with the prefabrication of floor cassettes. Prefabrication can provide a number of advantages including:

- acceleration of the construction program
- improved quality control
- improved safety.

The timber blocking can act as cavity barriers between the floor/ceiling cavity and wall cavity but it is still necessary to include horizontal cavity barriers to prevent spread via the wall cavities at each floor level and at 5 metre centres if the floor to floor height is greater than 5 metres. A practical solution is to provide mineral fibre cavity barriers as shown in Figures 5.38-5.40.

Although the mineral fibre cavity barrier is only required to be 45 mm thick in the potential direction of fire spread, where practical, installation of cavity barriers the full floor depth provides a more robust solution since any joins in the ring beam/blocks will also be backed by the mineral fibre.

For single stud internal walls the detail is simplified because the top and bottom plates of the wall frame close off the cavities within the wall as shown in Figure 5.41.

Massive timber panel designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Figures 5.38 to 5.41 include typical examples of joint seals to allow for movement and maintain acoustic and fire separations. The joint sealing details may vary depending upon the installation order of wall and ceiling fire-protective grade coverings amongst other things. Reference should be made to the plasterboard and / or sealant suppliers for Evidence of Suitability if alternative configurations are adopted.

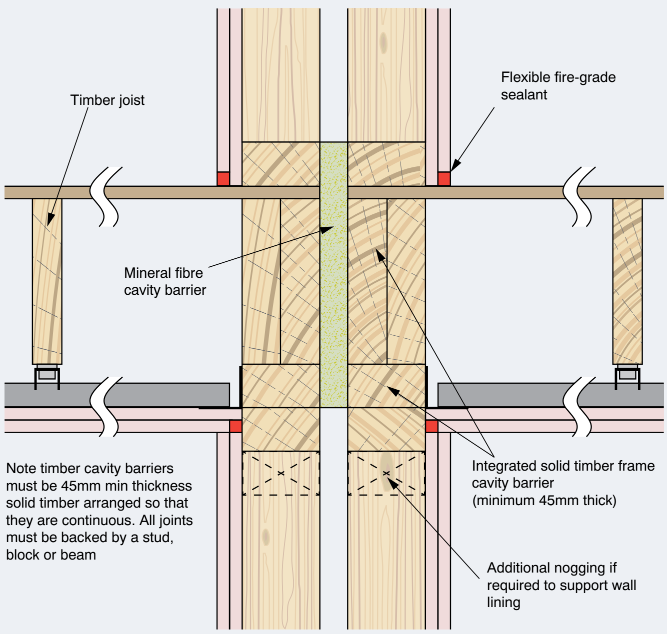


Figure 5.38: Fire-protected timber frame wall/floor junction with integral cavity barriers – beams parallel to wall Class 2 building.

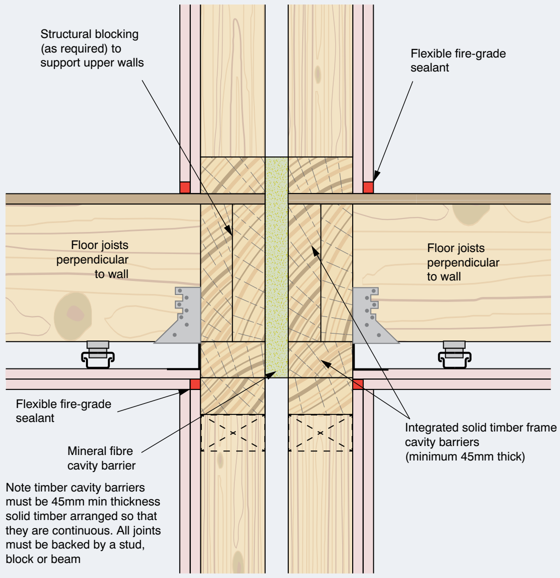


Figure 5.39: Fire-protected timber frame wall /floor junction with integral cavity barriers – beams perpendicular to wall Class 2 building.

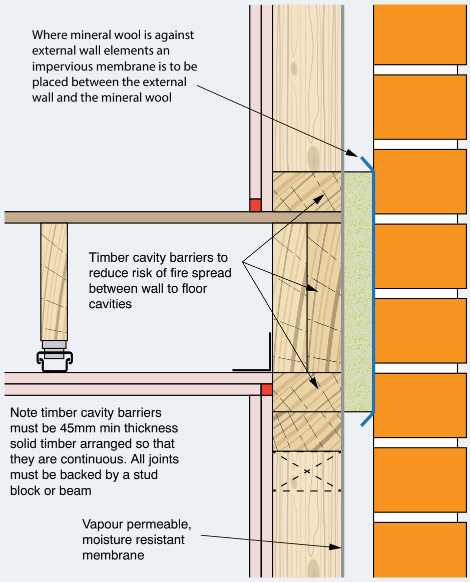


Figure 5.40: Fire-protected timber frame wall /floor junction with integral cavity barriers – beams parallel to wall Class 2 building.

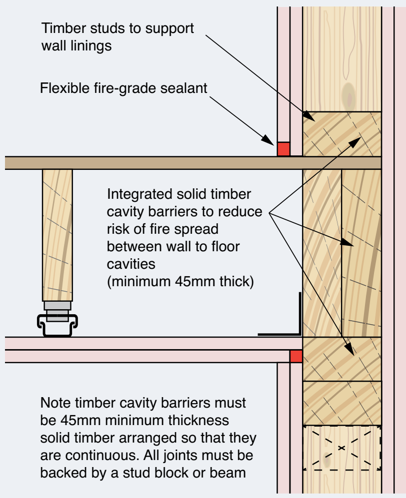


Figure 5.41: Fire-protected single stud timber frame wall /floor junction with integral cavity barriers – beams parallel to wall Class 2 building.

5.13.2 Vertical Cavity Barriers

Vertical cavity barriers are required at the intersection of walls and at 10 metres maximum horizontal centres. Typical details for double stud walls and external walls for the example Class 2 building are shown in Figures 5.42-5.44. Single stud details adopt a similar approach.

For double stud walls separate cavity barriers can be provided for each skin as shown in Figure 5.43 but in most instances a more practical solution is to fit a wider section spanning the full width of the intersecting wall, as shown in Figures 5.42 and 5.44.

Massive timber panel designs are required to avoid cavities and therefore the main consideration with the design of junctions is to maintain continuity of the fire-protective coverings.

Where external cladding or veneer systems form part of the fire-protective coverings (e.g. brick veneer) at cavity barrier positions an impervious membrane must be placed between the mineral fibre and cladding or veneer surface to control moisture transfer from the cladding or veneer.

An alternative approach for external walls that may avoid the risk of bridging at cavity barrier positions is to apply the fire-protective coverings to the outer face of the timber elements as well as the inner face and then fit a non-combustible external cladding system that satisfies the NCC DTS requirements. Typical examples are shown in Figures 5.19 and 5.20.

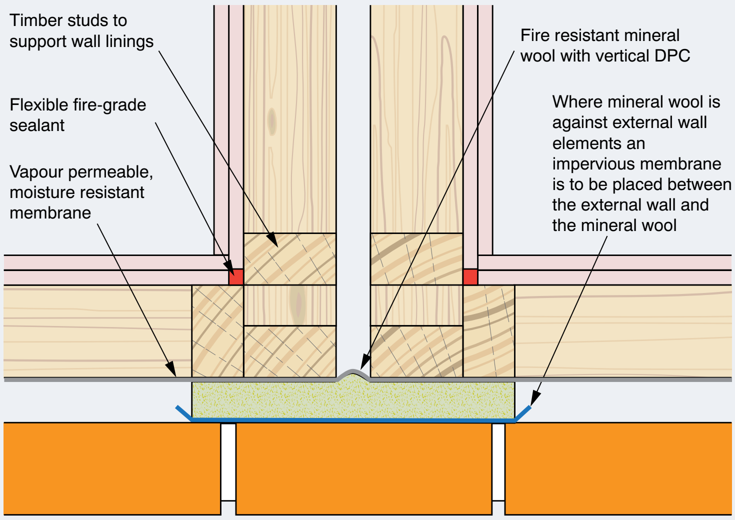


Figure 5.42: Double stud fire-protected timber internal wall intersecting a brick veneer wall.

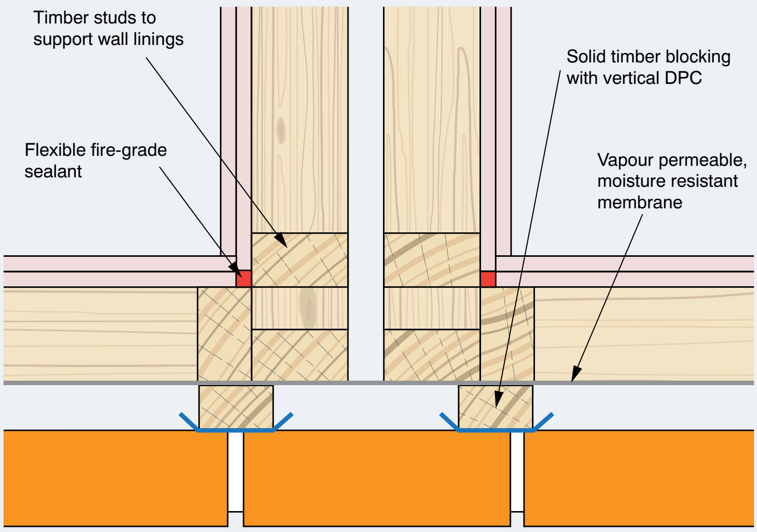


Figure 5.43: Double stud fire-protected timber internal wall intersecting a brick veneer wall with split cavity barrier system.

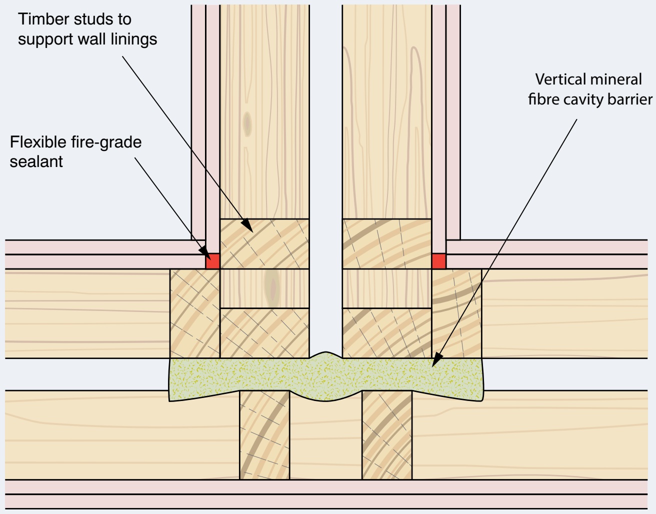


Figure 5.44: Double stud fire-protected timber internal wall intersection.

Provided the timber studs are a minimum of 45 mm thick, intermediate cavity barriers (at maximum 10 metres centres) can be fitted at a stud position as shown in Figure 5.45.

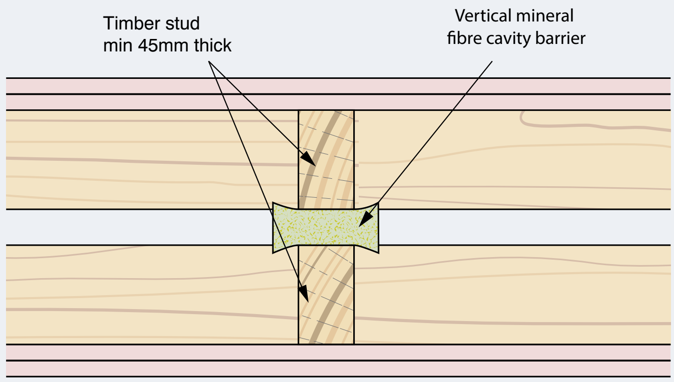


Figure 5.45: Intermediate vertical cavity barrier in double stud wall.

5.13.3 Unprotected Openings in External Walls

Cavity barriers are required around the perimeter of openings, such as unprotected windows in external walls, to prevent premature entry into the fire-protected timber cavities at these positions. A typical example is shown in Figure 5.46.

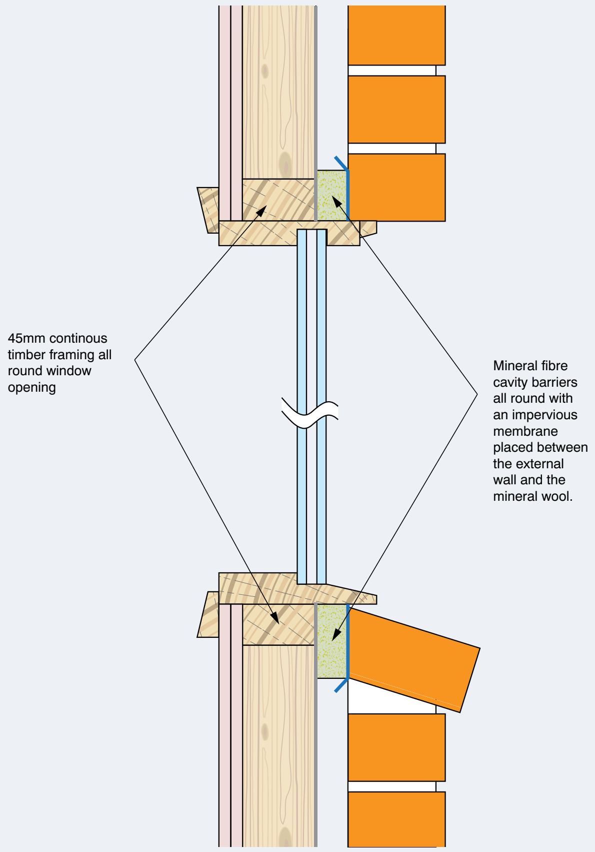


Figure 5.46: Cavity barrier around window in external wall.

5.13.4 Intersection of Non-Fire-Resisting Walls with Fire-Protected Timber Elements

Fire-protective coverings of fire-protected timber elements should not be interrupted at the point of intersection with non-fire-resisting walls to ensure the FRLs and RISF or MRISF are not compromised. Typical examples are shown in Figures 5.47 and 5.48.

Where the non-fire-resisting element is fixed to the fire-protected element additional framing may be required to avoid the risk of failure of the non-fire-resisting element compromising that of the fire-protected element. A typical detail for additional framing is shown in Figure 5.48.

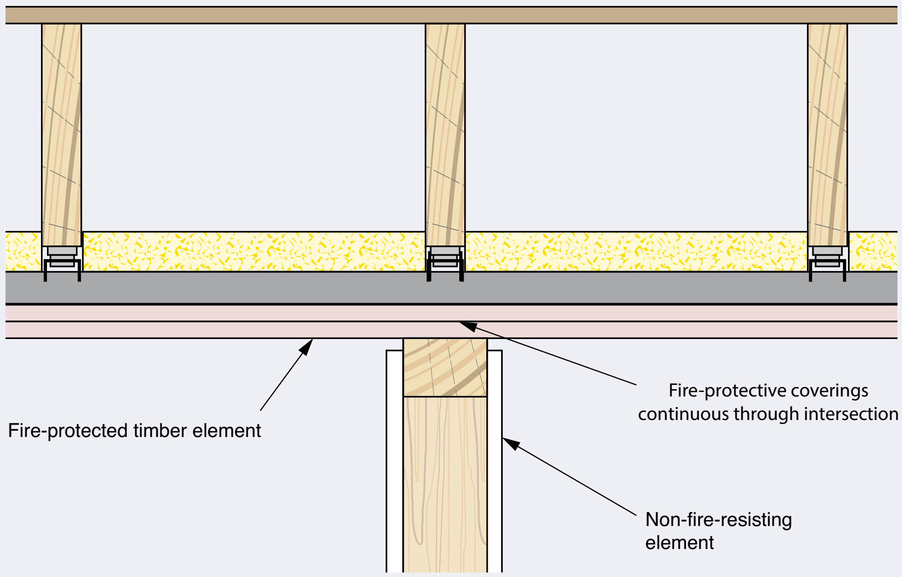


Figure 5.47: Junction of non-fire-resistant wall and fire-protected timber floor.

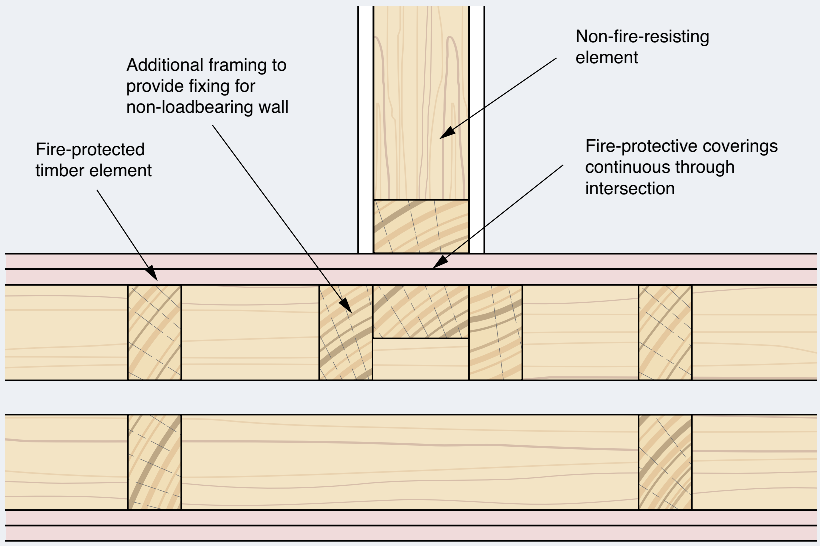


Figure 5.48: Junction of non-fire-resistant wall with fire-protected timber wall including additional framing detail.

With massive timber, the fixing point is less likely to require additional stiffening.

5.13.5 Roof Space Cavity Barriers or Fire-protected Timber Wall Extension

Special attention needs to be given to the design of roof spaces to address the risk of uncontrolled fire spread. There are generally two approaches that can be adopted:

Option 1: Extend SOU bounding fire-protected timber walls to roof level

The bounding construction around SOUs is continued to roof level (Figure 5.49). This has the advantage that the ceilings to the top floor do not need to be fire-resisting because the wall extension can provide the necessary fire and sound separation.

It is critical that the seal against the underside of the roof is capable of achieving the required FRL, RISF or MRISF and that the fire separation is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect extension of the SOU boundary walls.

If this option is adopted a horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 5.49.

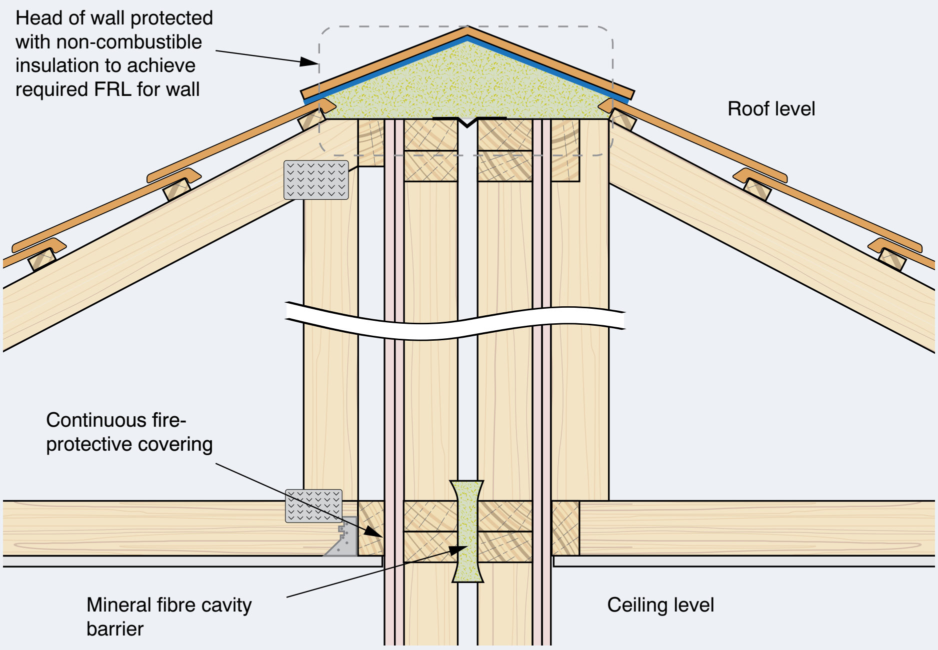


Figure 5.49: Roof space option 1 extending SOU bounding fire-protected timber walls to roof level.

Option 2: Provide fire-protected timber ceiling and cavity barriers within roof space

If Option 2 is applied to the Class 2 example building, assuming timber-framed construction, the ceiling would require an FRL of 90/90/90 and a RISF of 45 minutes and the roof spaces would need to be divided by cavity barriers above each of the SOU bounding walls. Where the roof void is relatively deep it may be impractical to apply the Deemed-to-Satisfy solutions of solid timber or mineral fibre and a plasterboard partition achieving the required FRL of 45 minutes for a cavity barrier may provide a more practical solution as shown in Figure 5.50.

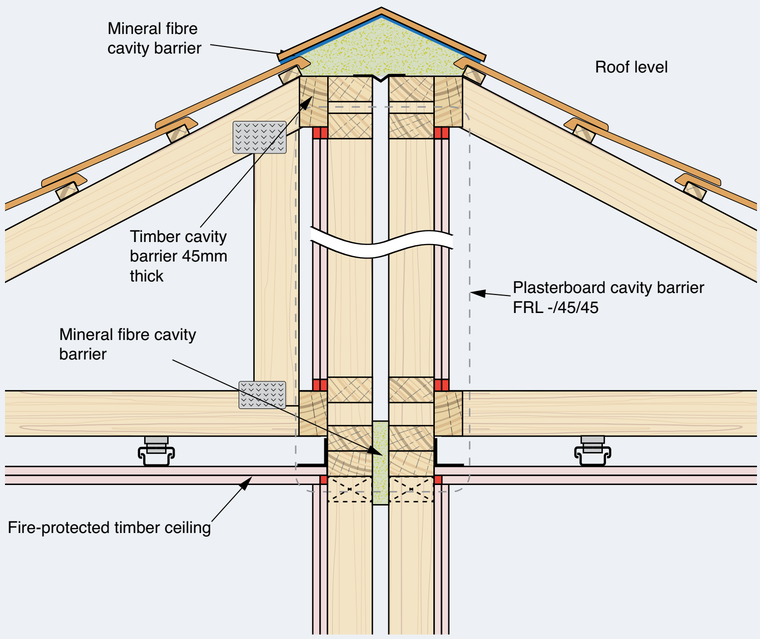


Figure 5.50: Roof space Option 2 fire-protected timber ceiling and cavity barriers within roof space.

If this option is selected it is critical that the seal against the underside of the roof is capable of achieving the required performance and the fire separation provided by the cavity barrier is not interrupted or bypassed at vulnerable positions such as the eaves or where framing members intersect at the extension of the SOU boundary walls.

A horizontal cavity barrier should be provided for timber-framed construction at ceiling level as shown in Figure 5.50.

Depending on the roof design, the roof cavity height can vary from nominally 150 mm to several metres and careful consideration should be given to detailing and checking installations to ensure the design objectives are achieved.

Good practice principles for service penetrations:

1 If practicable avoid service penetrations through fire-protected timber elements.

2 If fire-protected timber elements have to be penetrated by services, group the services and run them through lined openings protected by multi-penetration systems.

3 Ensure the FRLs and the RISF or MRISF levels are maintained at service penetrations.

5.14 Service Penetration Treatments

Careful detailing of services and service penetration systems during the design stages and subsequent correct installation during construction can simplify construction details and stream line the construction process as described in early chapters of this Guide.

The general design approach can be expressed as three fundamental principles

- Select services, service locations and service runs to avoid, as far as practical, the need for service penetrations through fire-protected timber elements (e.g. the use of false walls and ceilings can substantially reduce the number of penetrations that require protecting).
- If service penetrations cannot be avoided, where practical they should be grouped and penetrate lined openings or non-combustible shaft walls, which minimises the risk of exposing the cavity during maintenance operations. This approach also simplifies the installation of new services.
- If service penetrations are required to pass through fire-protected timber elements, ensure the FRL and RISF or MRISF as appropriate at service penetration positions.

The following Sections provide typical generic examples. Over time, it is expected that proprietary systems will become available simplifying the installation process. Refer to Section 5.8 Service Shafts for typical interface details between non-combustible shaft construction and fire-protected timber.

5.14.1 Multi-penetration Systems with Lined Openings

Typical multi-penetration systems with lined openings are shown in Figure 5.51.

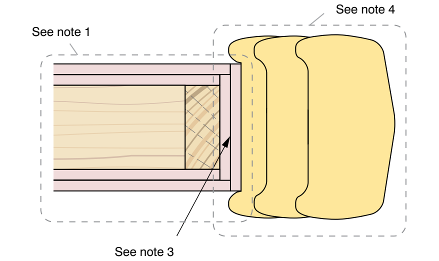
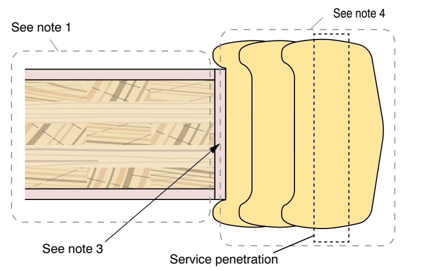
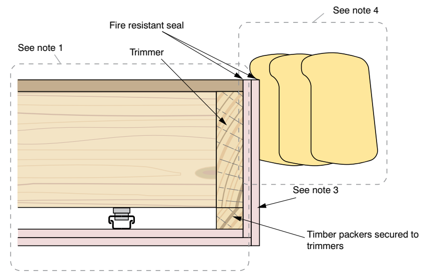
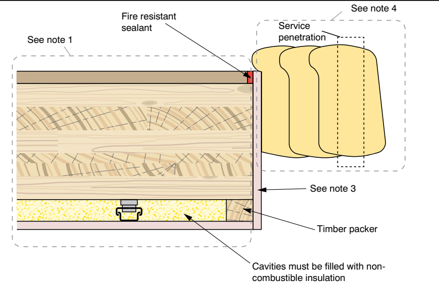
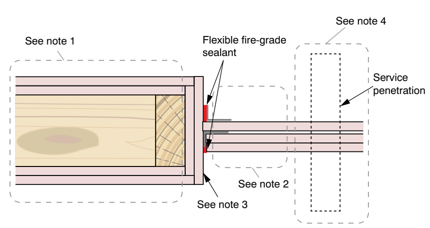
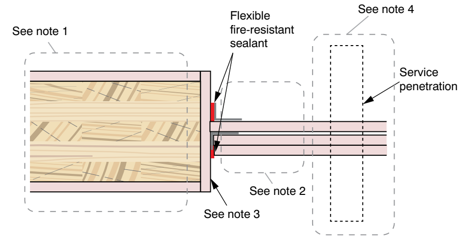
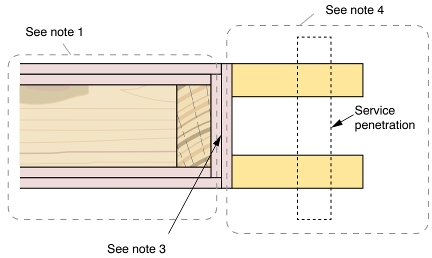
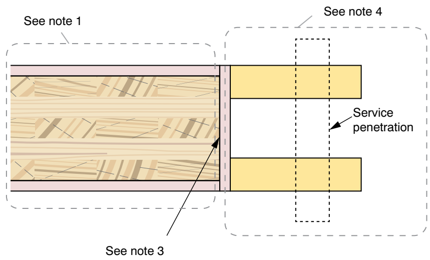
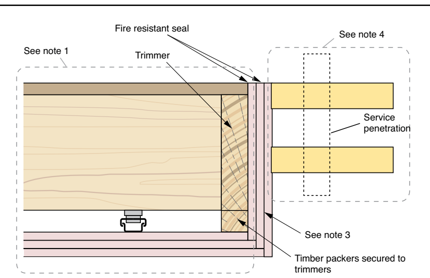
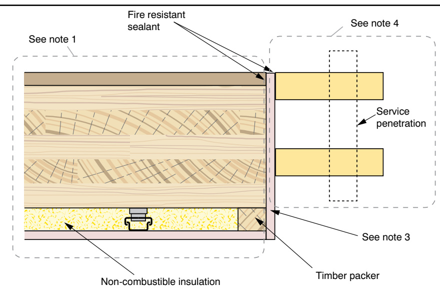
<table>
<tr><th>System</th><th>Timber Frame</th><th>Massive Timber Panel</th></tr>
<tr><td>Pillow in wall opening</td><td>
</td><td>
</td></tr>
<tr><td>Pillow in floor opening</td><td>
</td><td>
</td></tr>
<tr><td>Shaft-wall infill in wall opening</td><td>
</td><td>
</td></tr>
<tr><td>Mineral fibre batt wall opening</td><td>
</td><td>
</td></tr>
<tr><td>Mineral fibre batt floor opening</td><td>
</td><td>
</td></tr>
</table>

Figure 5.51: Typical multi-penetration systems with lined openings.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Shaft wall construction having the required FRL.

Note 3: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 4: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of interface details shown in Figure 5.51

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of interface details shown in Figure 5.52

Interface details shown in Figure 5.51 have been assessed by a registered test laboratory to determine that the details will not reduce the RISF or MRISF to below 45 minutes for the timber stud systems and 30 minutes for the massive timber panel systems (Refer EWFA report RIR 37401400). Other details may be adopted provided appropriate Evidence of Suitability to demonstrate compliance with the NCC requirements is provided.

5.14.2 Fire Damper and Duct Penetrations

The lined opening approach can also be applied to duct and damper penetrations (Figure 5.52).

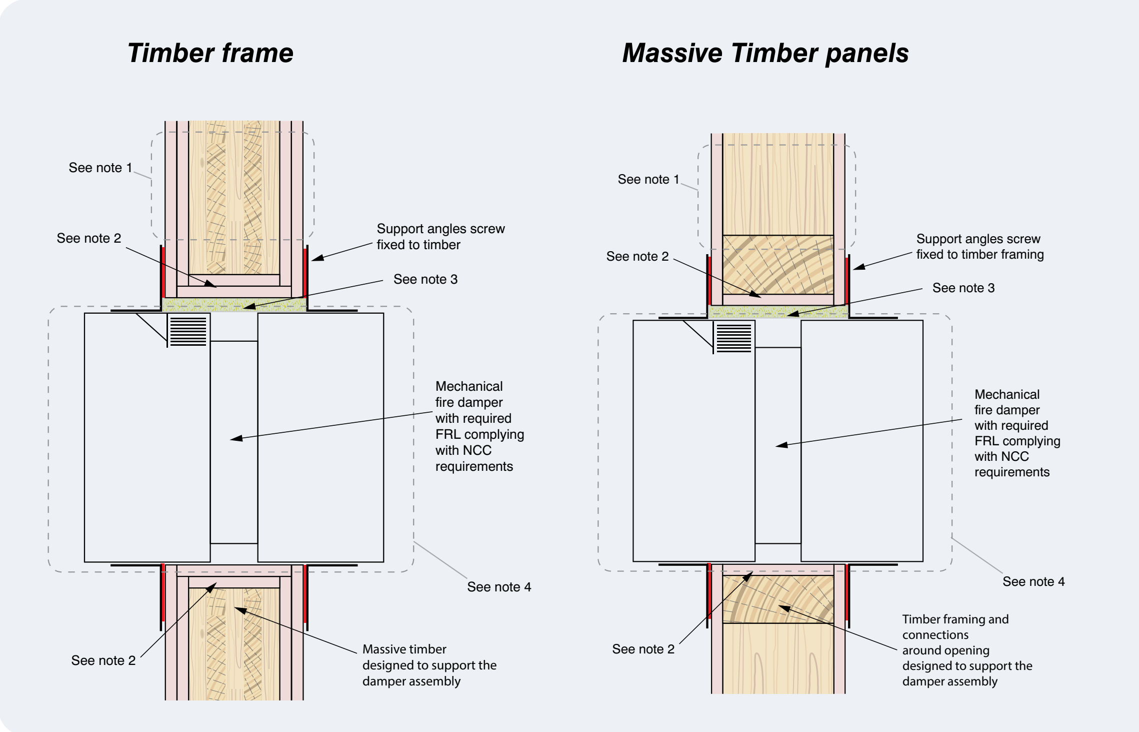


Figure 5.52: Typical details for fire damper and duct penetrations.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire protective coverings that are applied to the fire-protected timber element face.

Note 3: Non-combustible mineral fibre packing may be used for fire damper penetration seal or proprietary fire damper penetration seals that achieve the required FRL with evidence of performance in the form of a report from an Accredited Testing Laboratory to be in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 4: Mechanical fire damper having the required FRL when tested in accordance with AS 1530.4 and complying with AS 1682 Parts 1 and 2 as appropriate.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of wall lining detail shown in Figure 5.53

5.14.3 GPO Outlets and Switches

Where practical, the need to protect GPO outlets, switches and similar penetrations should be avoided by mounting them within internal (non-fire-resisting walls) or false (decorative) linings fitted in front of fire-protected timber elements as shown in Figure 5.53.

Methods of attaching non-fire-resisting decorative linings that will not compromise the FRL, RISF or MRISF performance of wall and floor systems, such as shown in Figure 5.53, have been assessed in a report from a Accredited Testing Laboratory (refer EWFA RIR 37401400).

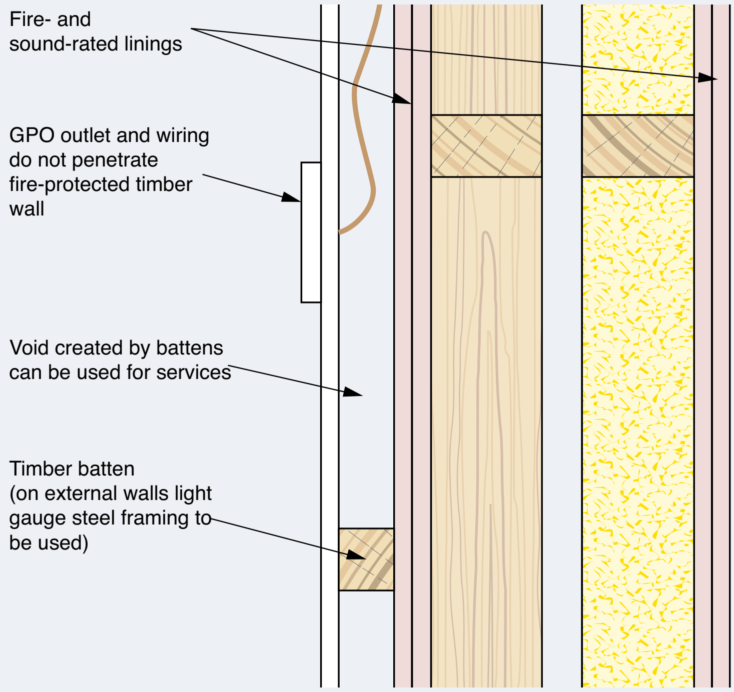


Figure 5.53: False wall system.

If it is impractical to apply an additional lining, a proprietary GPO protection system may be adopted, if it has Evidence of Suitability, demonstrating that the required FRL and RISF or MRISF for the element will not been compromised.

Alternatively, the generic systems shown in Figures 5.54 and 5.55 may be adopted.

New products (e.g. skirting service ducts) also enable services to be run within SOUs without penetrating fire-protective grade linings.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of the GPO interface details shown in Figures 5.54 and 5.55

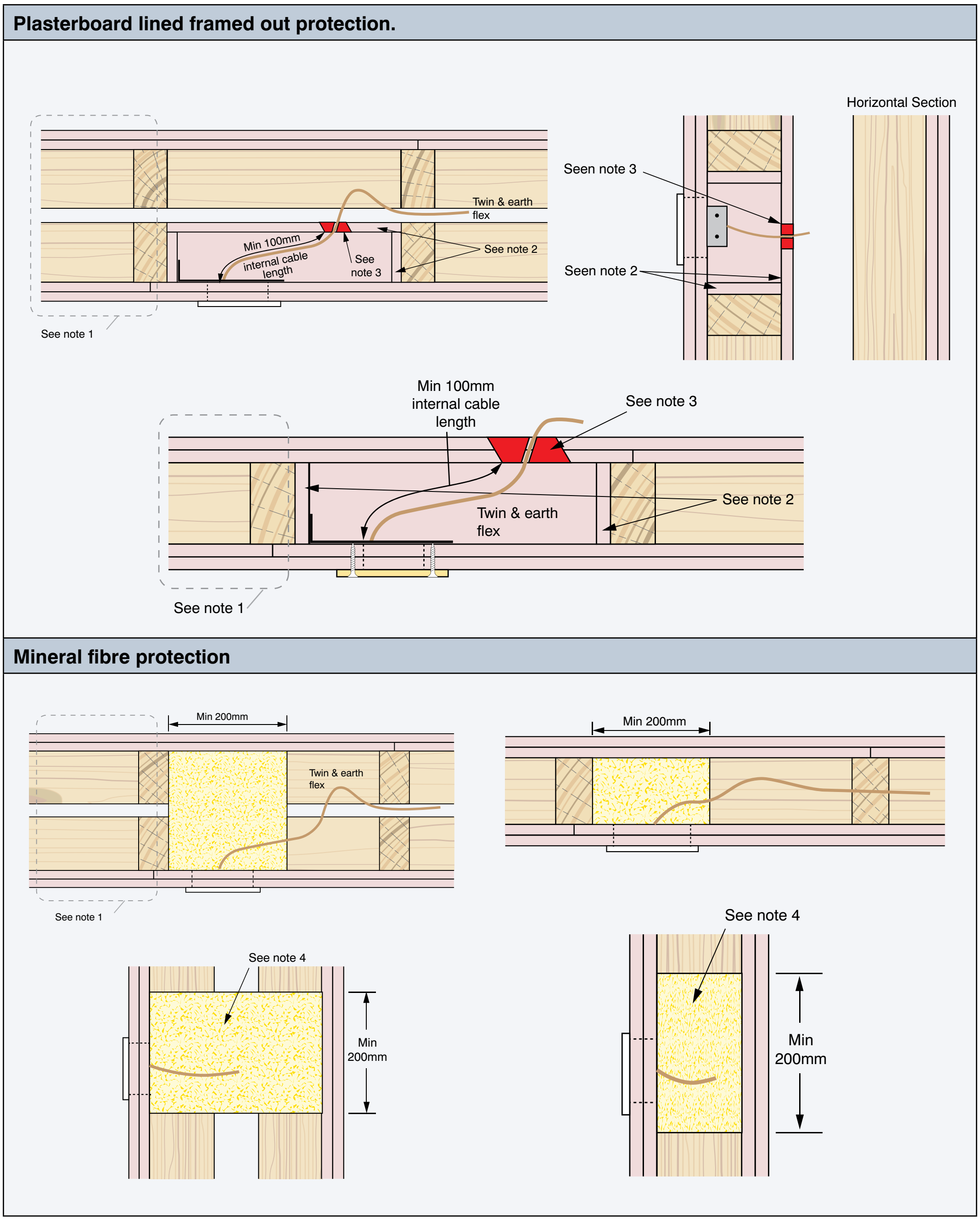


Figure 5.54: Generic GPO protection systems in timber-framed construction.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic' The mastic should have evidence of performance in the form of a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve an FRL of –/60/–.

Note 4: Cavity filled full depth with mineral fibre of minimum density 60 kg/m³ for at least 100 mm to the sides and above and below the centreline of the GPO.

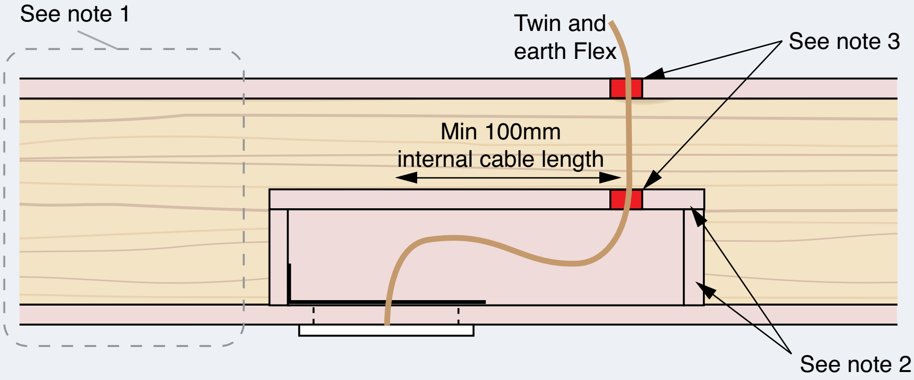


Figure 5.55: Generic GPO protection systems in massive timber construction.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Aperture lined with a minimum of one layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Note 3: Linings must be sealed full depth where penetrated by a service with a 'fire-resistant mastic'. The mastic should have a test report from an Accredited Testing Laboratory demonstrating that when protecting pipe or cable service penetrations through plasterboard elements the system can achieve an FRL of –/60/–.

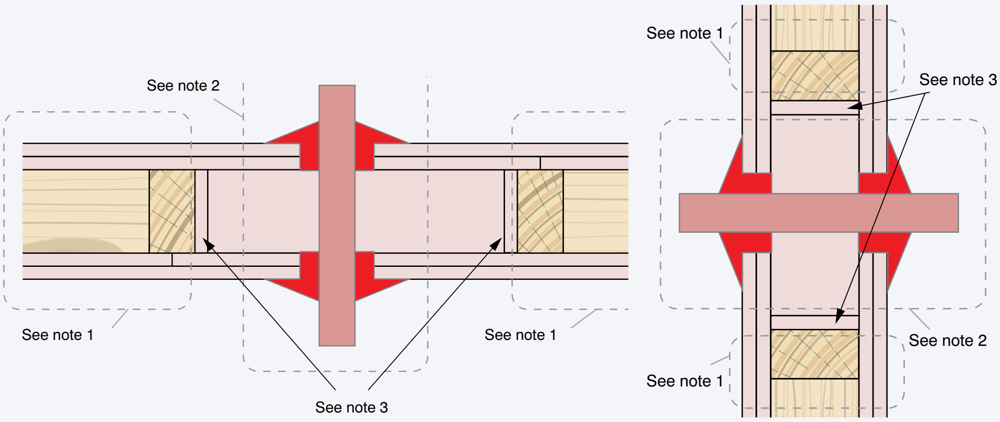
5.14.4 Single Cable and Metal Pipe Penetrations

Where single cable and pipe penetrations through fire-protected timber members cannot be avoided, existing proprietary protection systems that have achieved the required FRL in plasterboard systems can be used in conjunction with internal plasterboard linings or mineral fibre insulation packing as shown in Figure 5.56 to satisfy a RISF of 45 minutes or MRISF of 30 minutes as appropriate.

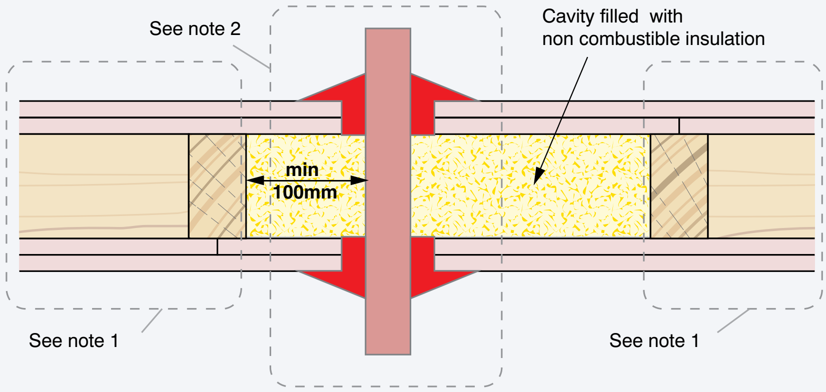
Fire tested proprietary systems may provide more practical options, subject to adequate Evidence of Suitability being available.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of the systems shown in Figure 5.56

Plasterboard lined timber frame. RISF 45 minutes



Mineral fibre fill timber frame. RISF 45 minutes.



Mineral fibre fill massive timber. MRISF 30 minutes.

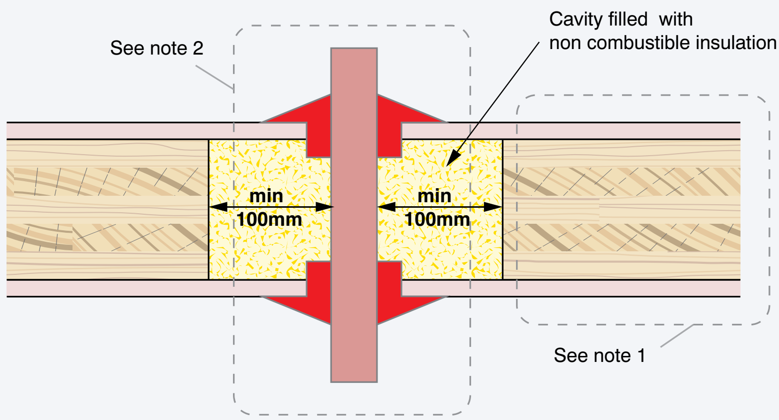


Figure 5.56: Pipe and cable penetrations through fire-protected timber.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of performance to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall system since it forms part of the wall system.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of ceiling lining detail shown in Figure 5.57.

The preferred option for lighting cables, sprinkler pipe penetrations and the like is to run them through the cavity above a false ceiling. A typical false ceiling detail is shown in Figure 5.57. Larger cavities can be provided above false ceilings by using suspended ceiling fixings to accommodate down lights and larger services.

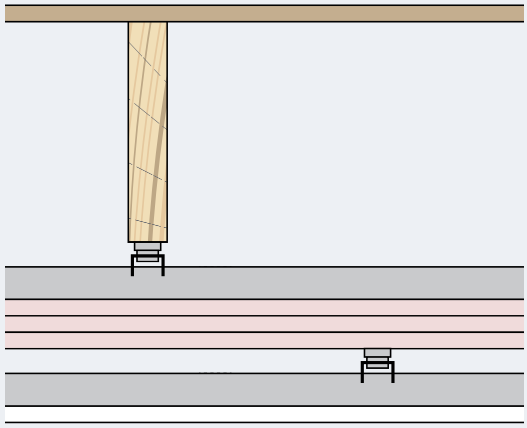


Figure 5.57: False ceiling detail for minimising service penetrations through ceiling systems.

If it is impractical to provide a false ceiling a solution for lighting cable penetrations through fire-protected timber ceilings is to use cover blocks as shown in Figure 5.58. Proprietary systems may be available to protect down-light penetrations and sprinkler pipe penetrations but access for the long-term service and maintenance of these systems and options for reconfiguration would be very limited.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of back blocking system shown in Figure 5.58

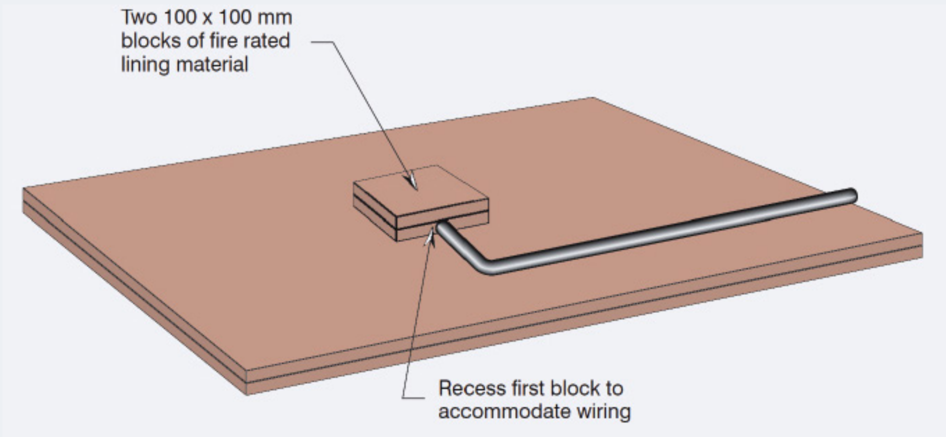


Figure 5.58: Recess block protection system for lighting cables penetrating fire-protected timber floors.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of rebated ceiling system shown in Figure 5.59

5.14.5 Rebated Ceiling Details for Housing Services

Another alternative for ceiling systems is to create a rebate to house services without penetrating a fire-protected element such as a fire-protected timber floor/ceiling system as shown in Figure 5.59. This detail has been assessed by an Accredited Testing Laboratory as achieving an FRL of 90/90/90 and a RISF of 60 minutes. Care should be taken not to attach the rebate framing members to the floor structure to avoid short-circuiting the sound separation.

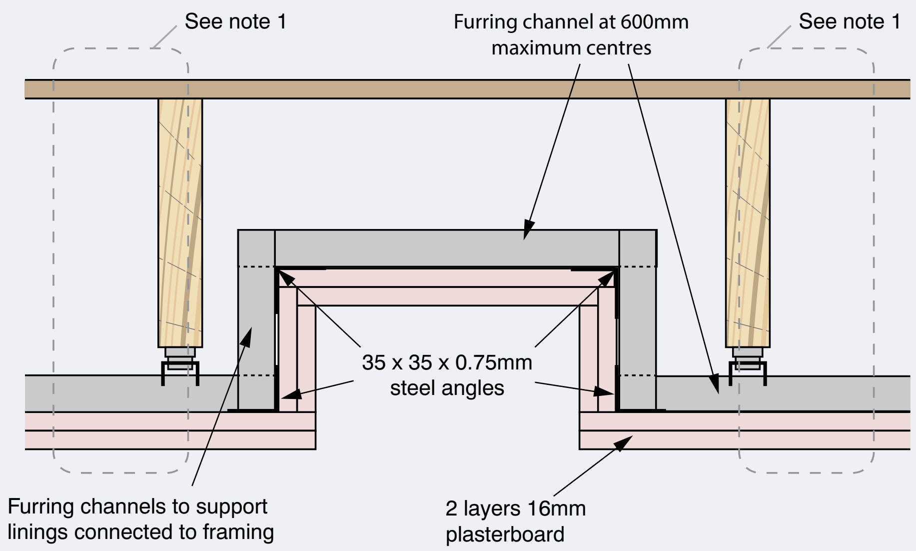


Figure 5.59: Rebated ceiling system.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

The rebate may be fitted with a grill, a section of false ceiling or may be sized to mount individual items of equipment.

5.14.6 Plastic Pipe Penetrations

Where it is impractical to adopt false wall and ceiling linings or utilise non-combustible shaft construction or lined opening multi-penetration systems, the following details, shown in Figures 5.60, 5.61 and 5.62, have been developed to maintain a RISF of 45 minutes or a MRISF of 30 minutes. The systems must have achieved an FRL of at least -/90/90 in plasterboard partitions when used to protect individual plastic pipe penetrations. The following notes apply:

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity Insulation must be non-combustible.

Note 2: Service penetration protected to achieve the required FRL. Evidence of Suitability to be in the form of a report from an Accredited Testing Laboratory in accordance with AS 1530.4 and AS 4072.1 as appropriate.

Note 3: Aperture lined with a minimum of 1 layer 16 mm plasterboard. Greater thicknesses/number of layers may be required for the faces of the wall since it forms part of the wall system.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of interface details to maintain the RISF and MRISF performance of elements pentetrated by plastic pipes as shown in Figures 5.60 to 5.62

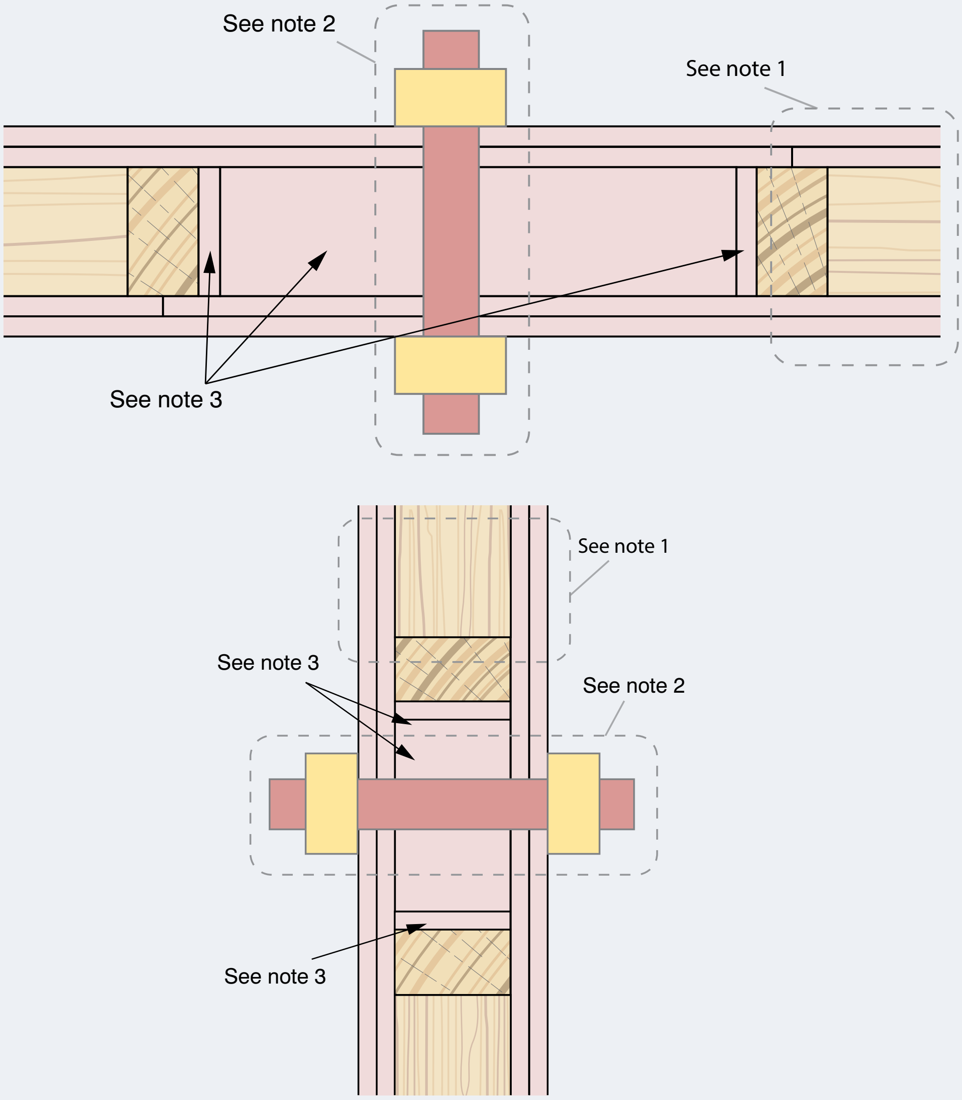


Figure 5.60: Plastic pipe penetration through fire-protected timber-framed walls with internal linings.

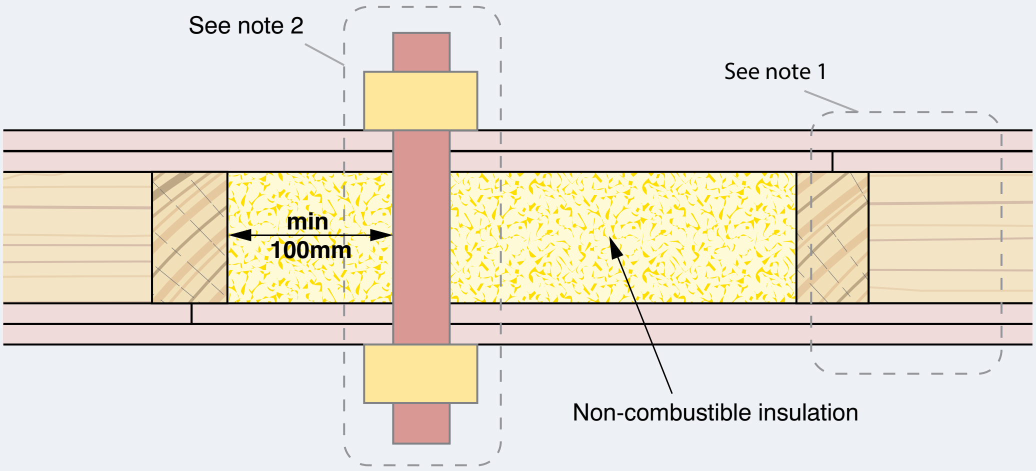


Figure 5.61: Plastic pipe penetration through fire-protected timber-framed walls with non-combustible mineral fibre insulation.

Evidence of Suitability required from supplier to confirm required RISF and MRISF performance of penetrated elements is maintained in addition to the FRL for the system shown in Figure 5.63

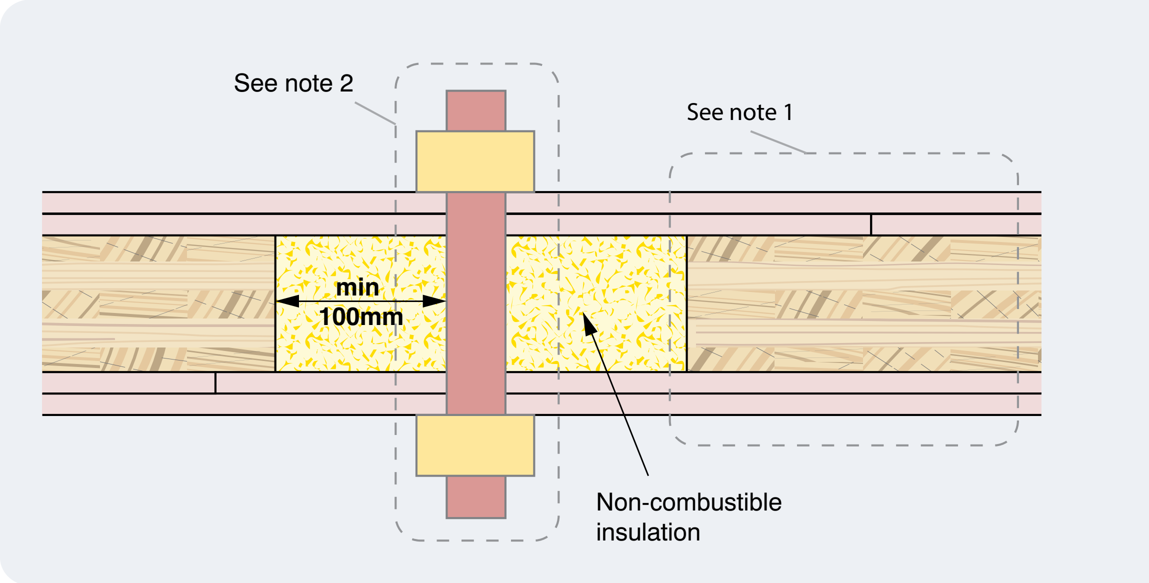


Figure 5.62: Plastic pipe penetration through fire-protected massive timber walls with non-combustible mineral fibre insulation.

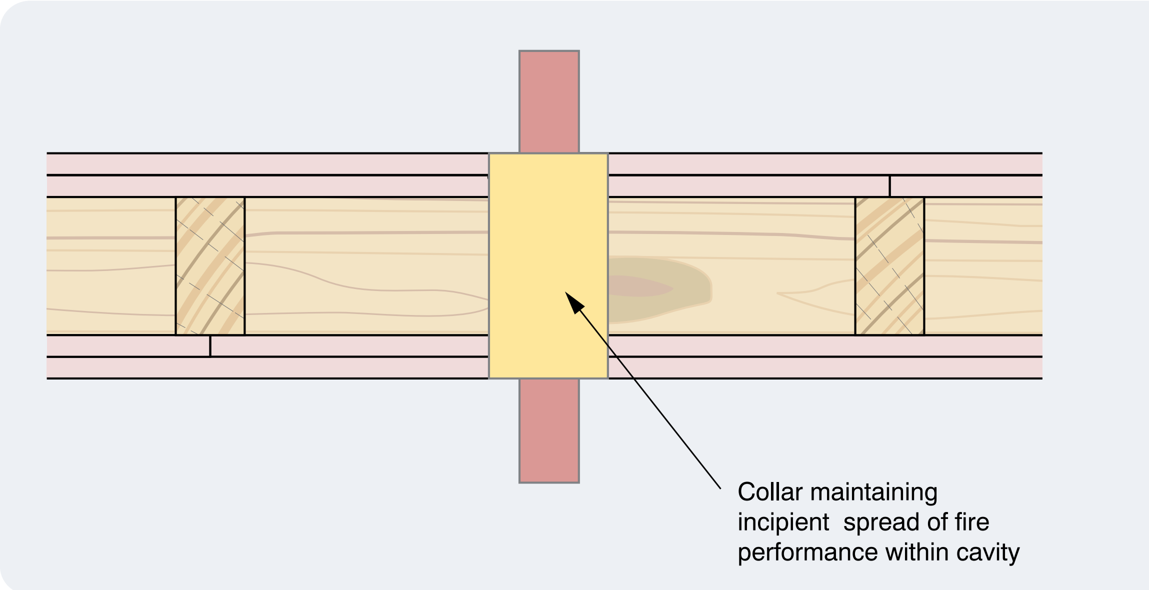


Figure 5.63: Option for a proprietary system with integral insulation protecting a plastic pipe penetration.

5.14.7 Access Panels

Access panels may be used to protect openings providing access to a floor/ceiling cavity as shown in Figure 5.64 or to shafts through fire-protected timber walls as shown in Figures 5.65 and 5.66.

Providing access panels will tend to compromise the sound separation and therefore they should normally be located in areas that are not sound 'sensitive'.

The following notes apply to the typical details shown in Figure 5.64 through Figure 5.66.

Note 1: Fire-protected timber element having the required FRL and RISF or MRISF as appropriate. Any cavity insulation must be non-combustible.

Note 2: Interface protected with the same fire-protective coverings that are applied to the fire-protected timber element face.

Note 3: Proprietary access panel system with the required FRL. For access panels providing access to ceiling cavities an RISF rating of 45 minutes or a MRISF rating of 30 minutes as appropriate is also required to be satisfied.

Refer EWFA report RIR 37401400, available from the WoodSolutions website, for assessment of interface details to maintain the RISF and MRISF performance of elements pentetrated by access panels as shown in Figures 5.64 to 5.66

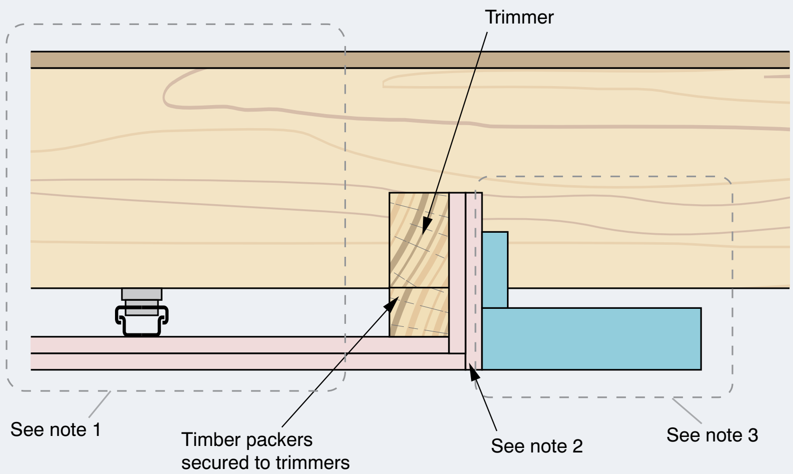


Figure 5.64: Access panel in a fire-protected floor/ceiling system.

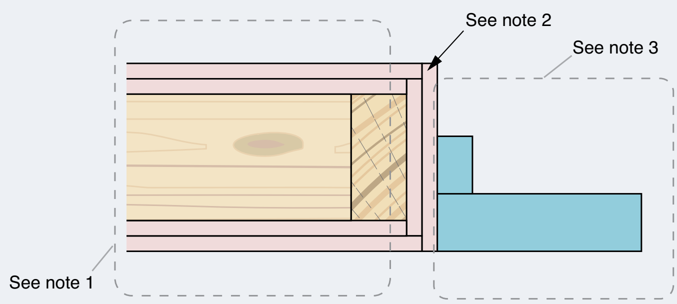


Figure 5.65: Access panel in a fire-protected timber-framed wall.

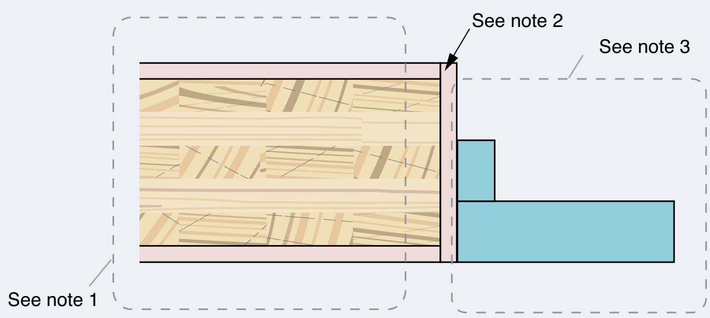


Figure 5.66: Access panel in a fire-protected massive timber wall.

5.15 Automatic Fire Sprinkler Systems

The provision of an automatic fire sprinkler system is accordance with NCC Specification E1.5 (other than a FPAA101D or FPAA101H system) is a mandatory requirement for mid-rise timber buildings if the DTS solution pathway is adopted.

The automatic fire sprinkler system is a critical component of the fire safety design and must be designed and installed by organisations and/or individuals with appropriate competency. Detailed information about the design of automatic fire sprinkler systems is outside the scope of this Guide, however in common with all services there is a need for the design to be integrated with the architectural, structural and passive fire protection systems. The following sub-sections highlight some key considerations but it is not an extensive summary.

It is important that the design documentation clearly specifies the requirements for the sprinkler systems such as locations of pipe runs, types of materials and components to be used, treatment of penetrations, types of sprinkler head and positions.

5.15.1 Piping Materials and Connections

Materials for piping and connection details for fire sprinkler systems should be carefully selected to:

- comply with the NCC Specification E1.5 requirements (other than a FPAA101D or FPAA101H system)
- suit the environment
- minimise the time the system is unavailable after maintenance/repair
- minimise hot works on site such as cutting and welding metal pipes
- facilitate the reinstatement of the performance of fire-protected timber at the points of penetration by sprinkler pipes.

While plastic pipes (e.g. cPVC) can largely negate the need for hot works, if alterations are made to plastic pipes, the sprinkler system could be unavailable overnight while the adhesive sets. This is an important consideration for residential buildings. The reinstatement of the performance of fire-protected timber when penetrated by plastic pipes can be more complex than metal pipe penetrations

Metal pipes may be more appropriate for some applications but they should be pre-prepared so that, as far as practical, all on-site connections can be made without hot works. Many suppliers can providing fittings that can be adjusted on site, such as flexible sprinkler fittings.

Once the materials and components have been selected the pipe runs should be clearly defined to minimise the number of penetrations through fire-protected timber and that if they cannot be avoided they occur where the performance of the fire-protected timber can be readily reinstated.

5.15.2 Sprinkler Head Selection

Although not mandatory in AS 2118.1, residential heads should be used in the residential parts of Class 2 and 3 buildings in line with the sprinkler head listing. They respond faster, reducing the risk to occupants close to the fire.

Sprinkler head options include concealed and semi-recessed. Concealed heads are a common choice for residential settings because they are unobtrusive. However, the following issues should be considered during the selection process and adoption of appropriate mitigation measures:

- Larger cut outs in ceilings are required which can be addressed by use of a non-rated false ceiling. The false ceiling depth should be designed to allow for the fitting of the concealed heads and related pipework.
- The response time will tend to be slower - this should be checked with the manufacturer.
- Overpainting and use of sealants to retain covers can compromise the performance of a head - this should be addressed through regular inspections.

Concealed sprinkler heads are also useful in public areas because they can reduce the risk of vandalism and accidental impacts.

Another useful type of head to consider is the sidewall sprinkler. For some rooms, adequate coverage can be achieved by fitting a sidewall sprinkler, which avoids the need for ceiling mounting and in some applications the sprinkler can be fitted to a non-fire-resisting wall within an apartment minimising the need for penetrations through fire-resisting construction.

5.15.3 Monitored Isolation Valves

The reliability of an automatic fire sprinkler system can be enhanced by specifying monitored isolation valves incorporating a check valve and flow switch at each level that is permanently connected to a fire alarm monitoring service provider by a direct data link.

This approach allows the water supply to the sprinkler system on individual floors to be isolated for maintenance or reconfiguration of the system without the need to isolate the whole building. Since the valves are monitored, the risk of the water supply not being reinstated is also significantly reduced.

This arrangement is compatible with the progressive commissioning of automatic fire sprinkler systems during construction, allowing protection of the lower levels while work progresses on the upper levels and individual floors to be easily isolated for adjustments to systems. This may be adopted as part of the fire safety strategy to address fire safety during construction.

5.15.4 Fire-isolated Stairs and Passageways with Timber Stairways

The NCC allows the use of timber stairways in fire-isolated stairs and passageways subject to the automatic fire sprinkler system coverage being extended to cover the fire-isolated stair in addition to other precautions (refer Section 4.8.2).

In the absence of other specifications, sprinkler heads should be provided in the following locations:

- at the top of the shaft
- under the landings at each floor level
- under intermediate landings
- providing coverage to other positions where there is a significant risk of accumulation of combustible materials.

Spec E1.5a 2(b) requirements are consistent with Sections 5.15.2 and 15.5.3 of this Guide and reflect good practice. Refer NCC Specification E1.5a for further details

5.15.5 NCC Specification E1.5a Additional Sprinkler System Enhancements

If Specification E1.5a concessions are to be adopted a number of enhancements to the basic Specification E1.5 sprinkler system designs are required to be implemented including:

- connection of the sprinkler system to a permanent fire alarm monitoring system connected to a fire station / dispatch centre in accordance with NCC Specification E2.2d
- the automatic fire sprinkler system is fitted with residential sprinkler heads complying with Clauses 4.4, 4.5 and 5.5.2 of AS 2118.4 in bedrooms.

5.16 Other NCC Requirements

This is a guide to the use of fire-protected timber for mid-rise timber buildings as a DTS solution in the NCC. It does not address all NCC requirements that apply to mid-rise buildings nor does it address all NCC fire-related requirements (e.g. fire hazard properties of linings).

Advice should be sought from appropriately qualified practitioners and relevant regulatory authorities regarding compliance with the NCC for specific projects.

Appendix A – Determination of Compliance of Fire-protected Timber

There are three components to the performance of fire-protected timber that need to be satisfied:

- the protected element must achieve the required Fire Resistance Level (FRL)
- the protected element must achieve the required Resistance to the Incipient Spread of Fire (RISF).
- fire-protective coverings must be non-combustible.

Refer NCC A5.4 and Schedule 5 for FRL

Refer NCC A5.6 for RISF

Refer NCC A5.2 for non- combustibility

A1 Non-Combustible Fire-Protective Covering

Unless the NCC deems a material or element of construction to be non-combustible, non-combustible means:

- Applied to a material – not deemed combustible as determined by AS 1530.1 – Combustibility Tests for Materials.
- Applied to construction or part of a building – constructed wholly of materials that are not deemed combustible.

If the fire-protective covering is a composite or multi-layer system, each layer must be non-combustible. It is not acceptable to undertake a single combustibility test on the composite or just the facing materials and claim the fire-protective covering is non-combustible.

Typical examples of multi-layer systems are shown in Figure A1.

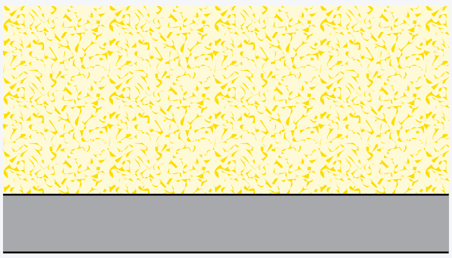
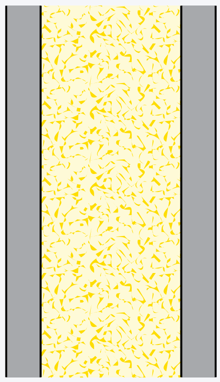
	
Multi-layer system – each layer must be non-combustible	Composite panels – each layer of the composite must be non-combustible
Commonly fire-resistant board supporting non-combustible lightweight insulation used in ceilings protecting floors/beams	Commonly non-combustible lightweight insulating core between non-combustible durable facings used for external claddings

Figure A1: Example of multi-layered fire-protective coverings (all layers).

Clause C1.9(e) of the NCC allows (deems) the following materials, though combustible or containing combustible fibres, to be used wherever a non-combustible material is required:

- plasterboard
- perforated gypsum lath with a normal paper finish
- fibrous-plaster sheet
- fibre-reinforced cement sheeting
- pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm thickness and where the Spread-of-Flame Index of the product is not greater than 0
- sarking-type materials that do not exceed 1 mm in thickness and have a Flammability Index not greater than 5
- bonded laminated materials where:
  - each laminate is non-combustible
  - each adhesive layer does not exceed 1 mm in thickness
  - the total thickness of the adhesive layers does not exceed 2 mm
  - the Spread-of-Flame Index and the Smoke-Developed Index of the laminated material as a whole does not exceed 0 and 3 respectively.

All materials forming the fire-protective covering are either permitted to be used in accordance with NCC Clause C1.9(e) or determined to be non-combustible by testing to AS1530.1.

A2 Fire Resistance Level

A fire-protected timber element must achieve the required FRL specified in the NCC for the particular application. The fire resistance of a fire-protected timber element has to be determined in accordance with Schedule 5.2(b) and (c) of the NCC.

Generally, Schedule A5.2(b) requires a prototype to be submitted to the Standard Fire Test (AS1530.4), or an equivalent or more severe test, and the FRL achieved by the prototype, without the assistance of an active fire suppression system, is confirmed in a report from an Accredited Testing Laboratory which:

- describes the method and conditions of the test and the form of construction of the tested prototype in full
- certifies that the application of restraint to the prototype complied with the Standard Fire Test;

or differs in only a minor degree from a prototype tested under Schedule 5.2(b) and the FRL attributed to the building element is confirmed in a report from an Accredited Testing Laboratory which:

- certifies that the building element is capable of achieving the FRL despite the minor departures from the tested prototype; and
- describes the materials, construction and conditions of restraint which are necessary to achieve the FRL.

The option to use AS 1720.4 char-based calculation methods to determine the fire resistance is not permitted for fire-protected timber. This is because concerns were expressed with respect to the suitability of the AS 1720.4 approach for certain types of adhesives and connections forming parts of engineered timber products. The proprietary nature of massive timber panel products and lack of standardisation of adhesives and other critical materials used in their construction meant that there was insufficient data available at the time to demonstrate the suitability or otherwise of AS 1720.4.

A3 Resistance to the Incipient Spread of Fire

A3.1 Determine Applicable Resistance to the Incipient Spread of Fire Requirements

The Resistance to the Incipient Spread of Fire (RISF) in relation to a fire-protective covering means the ability of the covering to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

The general requirement for fire-protected timber is an RISF of 45 minutes.

The NCC permits a relaxation to the RISF requirements for fire-protected timber providing both the following additional criteria are satisfied.

- the minimum timber panel thickness is not less than 75 mm
- there are no cavities between the surface of the timber and the fire protective covering or between timber members.

The 75 mm dimension relates to the inherent fire resistance achieved when using a timber panel member. If the relaxation conditions are satisfied, the Modified Resistance to the Incipient Spread of Fire (MRISF) criteria are applicable. Typical examples of massive timber installations satisfying these conditions are shown in Figure 4.3 in the body of this Guide.

Figure A2 shows the process for determining the applicable Resistance to the Incipient Spread of Fire requirements. The general requirement for fire-protected timber is a RISF of 45 minutes.

The relaxed requirements for massive timber construction without voids and cavities is a MRISF that applies a higher interface temperature limit and the time periods for which the temperature limit applies varies according to the application in accordance with Table A1.

Table A1: Modified Resistance to the Incipient Spread of Fire required performance for applications where criteria are relaxed (massive timber construction without voids and cavities).

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)
Inside a fire-isolated stairway or lift shaft	20 min
External walls within 1 metre of an allotment boundary or 2 metres of a building on the same allotment	45 min
All other applications	30 min

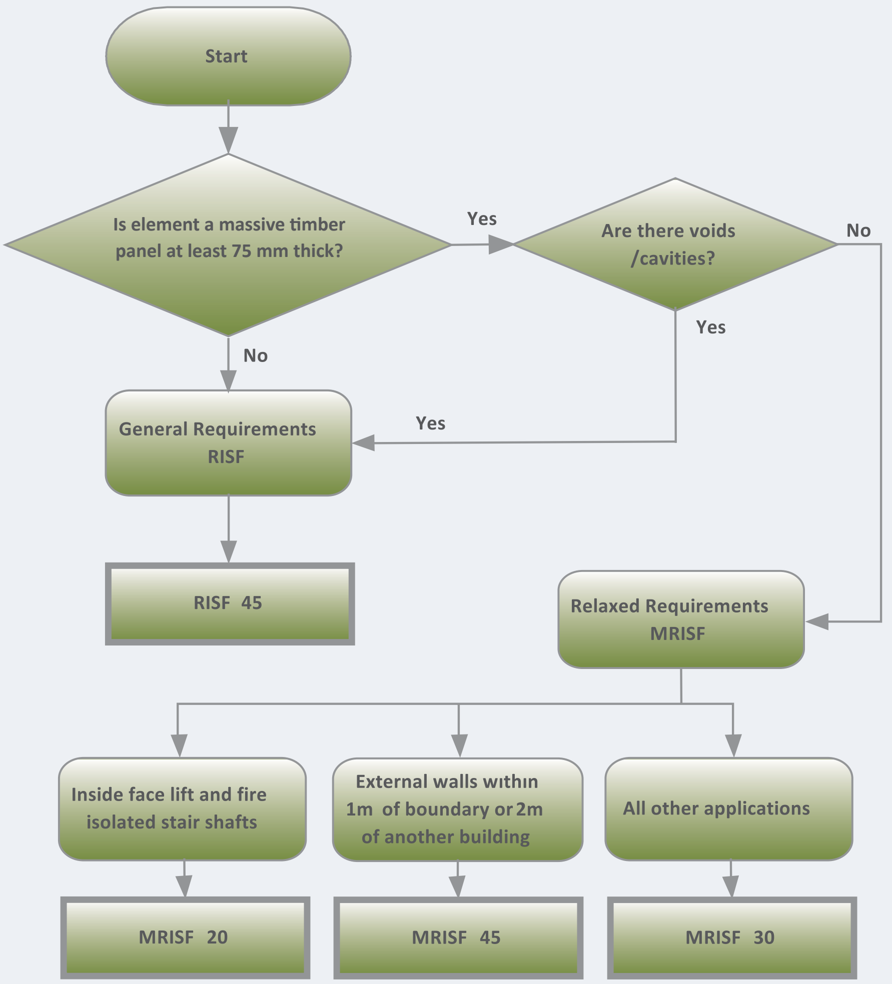


Figure A2: Determination of Resistance to the Incipient Spread of Fire acceptance requirements.

A3.2 Compliance Paths for Resistance to the Incipient Spread of Fire

Three paths are permitted to demonstrate compliance with the RISF requirements;

- simultaneous determination during a full-scale fire resistance test
- smaller-scale fire resistance test (at least 1 metre x 1 metre specimen)
- selection of Deemed-to-Satisfy fire-resisting grade plasterboard coverings.

Simultaneous determination during a full-scale fire resistance test

When a fire resistance test is undertaken to determine the FRL of an element, additional instrumentation can be included in the test to also determine the RISF or MRISF performance, providing a cost-effective approach for new protection systems.

Smaller-scale fire resistance test

There are a large number of systems that have been tested previously to determine their FRLs but in most cases insufficient data will have been recorded to determine the RISF or MRISF performance. Under these circumstances, the use of a smaller specimen (not less than 1 metre x 1 metre) is permitted to obtain supplementary data to determine the RISF or MRISF of the system in a cost effective manner. The fire-protective covering should be fitted in the same manner as that used for the original test that determined the FRL of the system.

Deemed-to-Satisfy Fire-Protective Grade Plasterboard coverings

Specification C1.13 deems fire-protective grade plasterboard facings, if fixed in accordance with the requirements to achieve the required FRL of the element, to also satisfy the requirements for Resistance to the Incipient Spread of Fire (RISF) or Modified Resistance to the Incipient Spread of Fire (MRISF). Table A2 shows the minimum requirements for plasteboard coverings.

Table A2: Fire-protective grade plasterboard coverings Deemed-to-Satisfy RISF requirements.

Requirements	Application	Performance	Minimum Deemed-to-Satisfy fire-protective grade plasterboard
General Requirements	All applications	RISF 45min	2 layers x 13 mm thick
Relaxed requirements for timber panels not less than 75 mm thick without cavities voids or cavities voids filled with non-combustible material	Inside a fire-isolated stairway or lift shaft	MRISF 20 min	1 layer x 13 mm thick
	External walls within 1 metres of an allotment boundary or 2 metres of a building on the same allotment	MRISF 45 min	2 layers x 13 mm thick
	All other applications	MRIFS 30 min	1 layer x 16 mm thick

A3.3 Resistant to the Incipient Spread of Fire (RISF) Test Procedures

The test procedure for determining the Resistance to the Incipient Spread of Fire (RISF) of horizontal elements during a full-scale fire resistance test is provided in Section 4 of AS 1530.4 . Specification C1.13a of the NCC requires the relevant procedures from AS 1530.4 Section 4 to be applied to other elements.

AS 1530.4 requires walls to be full size or not less than 3 m high x 3 m wide and floor/ceiling systems to be full size or not less than 4 m long x 3 m wide. Floor systems are exposed to furnace heating conditions (Figure A3) from the underside and fire-resisting walls are exposed from one side. Asymmetrical walls generally require two tests to evaluate the response to exposure to fire from either side unless the side exposed to fire is specified.

Smaller-scale specimens (not less than 1 m x 1 m) can be used to retrospectively determine the RISF performance of a floor or wall system that has previously achieved the required FRL in a fire resistance test satisfying the minimum size requirements specified in AS 1530.4.

For universal application of results the minimum cavity depth should be fire tested.

To determine the RISF, five thermocouples with insulating pads as prescribed in AS 1530.4 are fixed to the inner face of the fire-protective covering system. They are placed at approximately the centre and the centre of each quarter section as shown in Figure A4.

When testing corrugated specimens, increase the number of thermocouples to six to provide an equal number of thermocouples at the maximum and minimum specimen thickness.

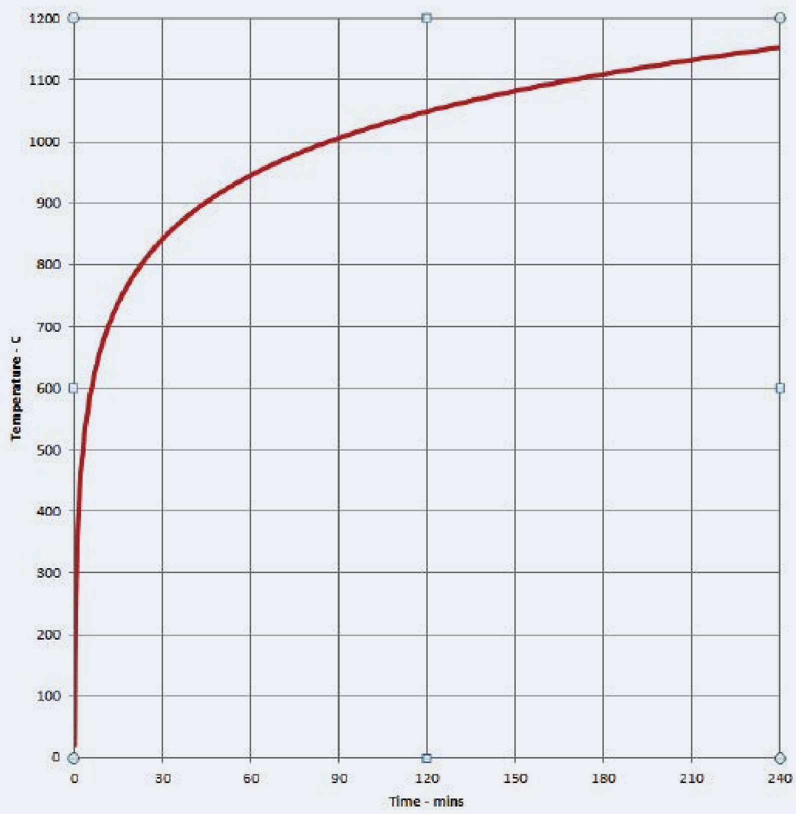


Figure A3: Standard fire resistance test heating regime.

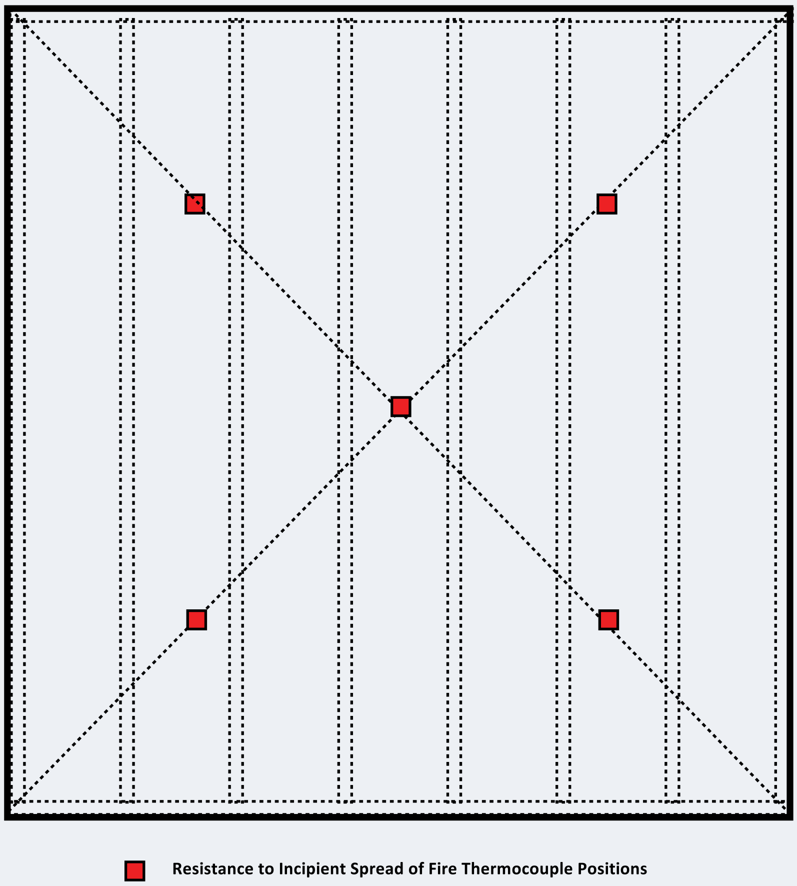


Figure A4: Elevation of a wall showing RISF thermocouple positions.

Sections through typical specimen configurations are shown in Figure A5 to illustrate the correct surfaces to apply thermocouples to determine the RISF. For fire-protected timber, the temperature has to be maintained below the prescribed temperature on the surface of the fire-protective covering facing the void and at the interface with timber elements within the wall or floor. If a wall or ceiling system is protected by a board system, for example, the temperatures are measured on the board surface within the cavity even if non-combustible insulation is applied between the timber studs or beams. However, if the non-combustible insulation forms a continuous layer between the timber elements and the board the thermocouples (t/c) should be applied to the surface of the insulation as shown in Figure A5.

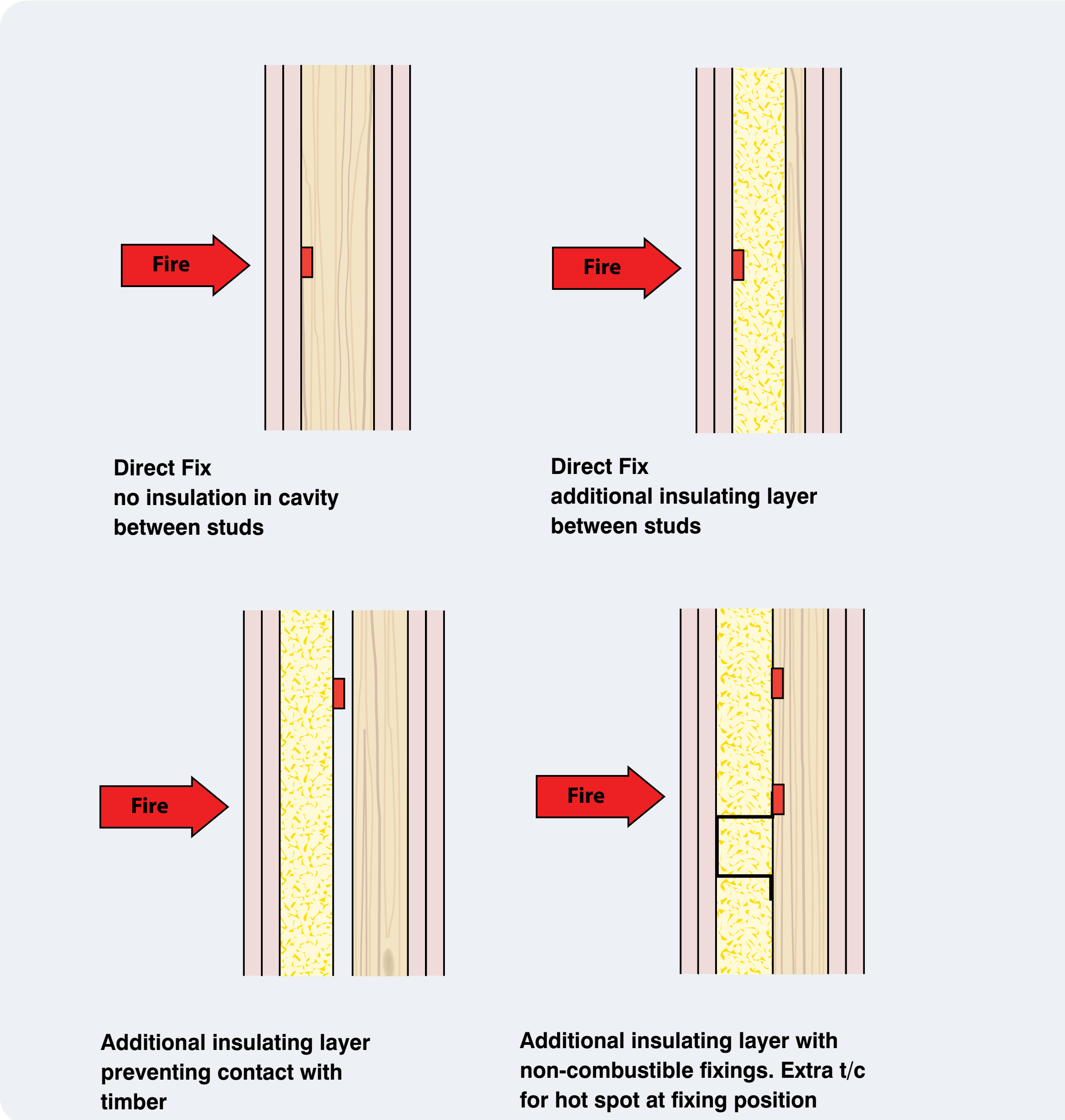


Figure A5: Resistance to the incipient spread of fire thermocouple positions for typical specimen configurations.

Failure in relation to the RISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 250^{0}C.

Smaller scale specimens 1 m x 1 m can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations are shown in Figure A6. Additional thermocouples are shown to allow the simultaneous determination of the FRL of the service penetration system.

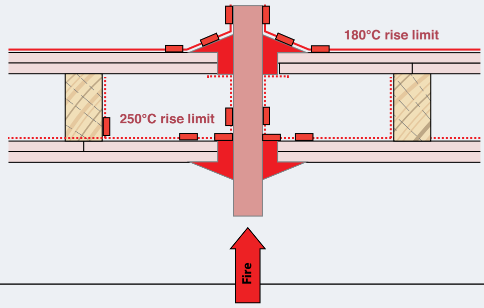
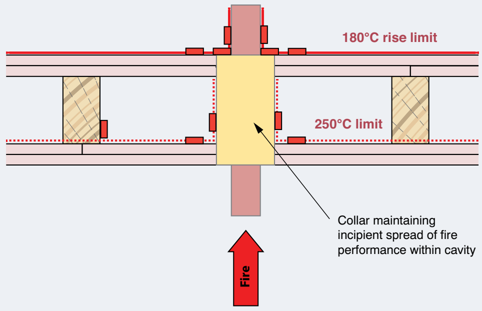
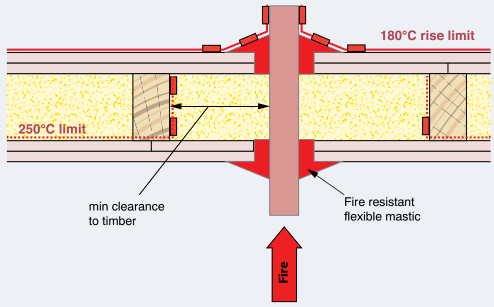
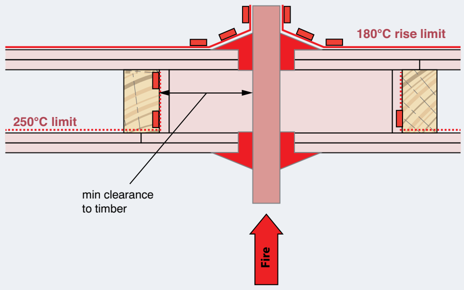
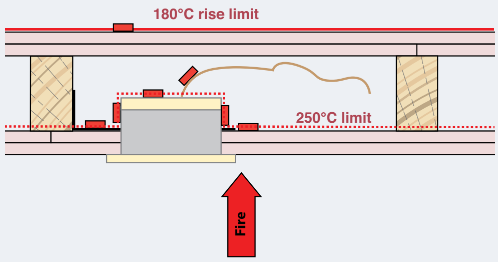
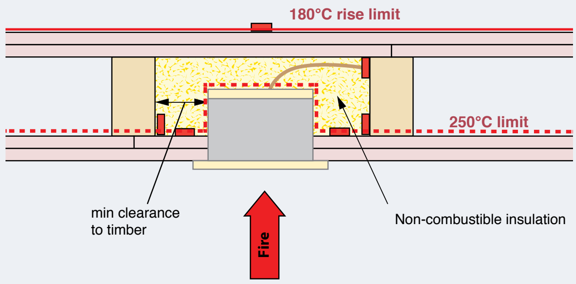
Cable/metal pipe penetration protected with fire-resistant mastic.	
Plastic pipe protected by insulating collar system.	
Cable/metal pipe penetration protected with fire resistant mastic and non-combustible cavity infill. The critical interface for RISF for the service penetration system is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system	
Cable/metal pipe penetration protected with fire-resistant mastic and cavity lined with non-combustible board. The critical interface for RISF for the service penetration is the surface of the lining board where it is in contact with timber elements.	
Proprietary GPO outlet protection system. Note: Thermocouples applied to cable surface connected to the GPO, on fixing bracket and adjacent element.	
GPO outlet with non-combustible cavity infill protection. The critical interface for RISF is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system	

Figure A6: Typical thermocouple positions for determining the RISF of service penetrations.

The thermocouples positions must satisfy the following requirements:

- At not less than two points about 25 mm from the edge of the hole made for the passage of the service.
- Attached to adjacent structural members and those elements that support the penetrating service.
- At points on the surface of the penetrating service or its fire stopping encasement, as follows:
  - at least two thermocouples about 25 mm from the plane of the general surface of the covering and non-combustible insulation
  - where the seal or protection around the service is tapered or stepped, two additional thermocouples beyond the step or the end of any taper if it is expected that the temperatures will be higher at these points.
- Where practicable, at two points on the seal or protection around the service.
- One in the centre of the surface of the penetration nominally parallel to the plane of the fire protective covering if it terminates within the cavity (e.g GPO outlets or down lights).

Failure in relation to the RISF is deemed to occur for the service penetration when the maximum temperature of the thermocouples described above exceeds 250^{0}C.

A3.4 Modified Resistance to the Incipient Spread of fire (MRISF) Test Procedures

The MRISF is applicable to massive timber panels having a thickness not less than 75 mm if there are no voids/cavities through which fire and smoke can spread. The MRISF, amongst other things, relaxes the failure temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures are described in Section 3 of Specification C1.13a of the NCC and are summarised below:

- Tests must be carried out in accordance with AS 1530.4, or an equivalent or more severe test, on the timber element with the proposed non-combustible fire protective coverings fixed in a representative manner.
- Smaller scale specimens (not less than 1m x 1m) can be used to retrospectively determine the MRISF performance of a system that has previously achieved the required fire resistance level in a fire resistance satisfying the minimum size requirements specified in AS 1530.4. If a fire protection system incorporates joints, the test specimens must incorporate representative joints.

To determine the MRISF interface, temperatures must be measured over the following features by a minimum of two thermocouples complying with Appendix C1 and Section 2 of AS 1530.4 as appropriate:

- at joint positions in the protection systems
- at least 200 mm from any joint
- at any other locations where, in the opinion of the Accredited Testing Laboratory, the interface temperature may be higher than the above positions.

Where the fire protective covering is not in contact with the timber (e.g. multi-layer system), the surface of the fire-protective covering is deemed to be the interface.

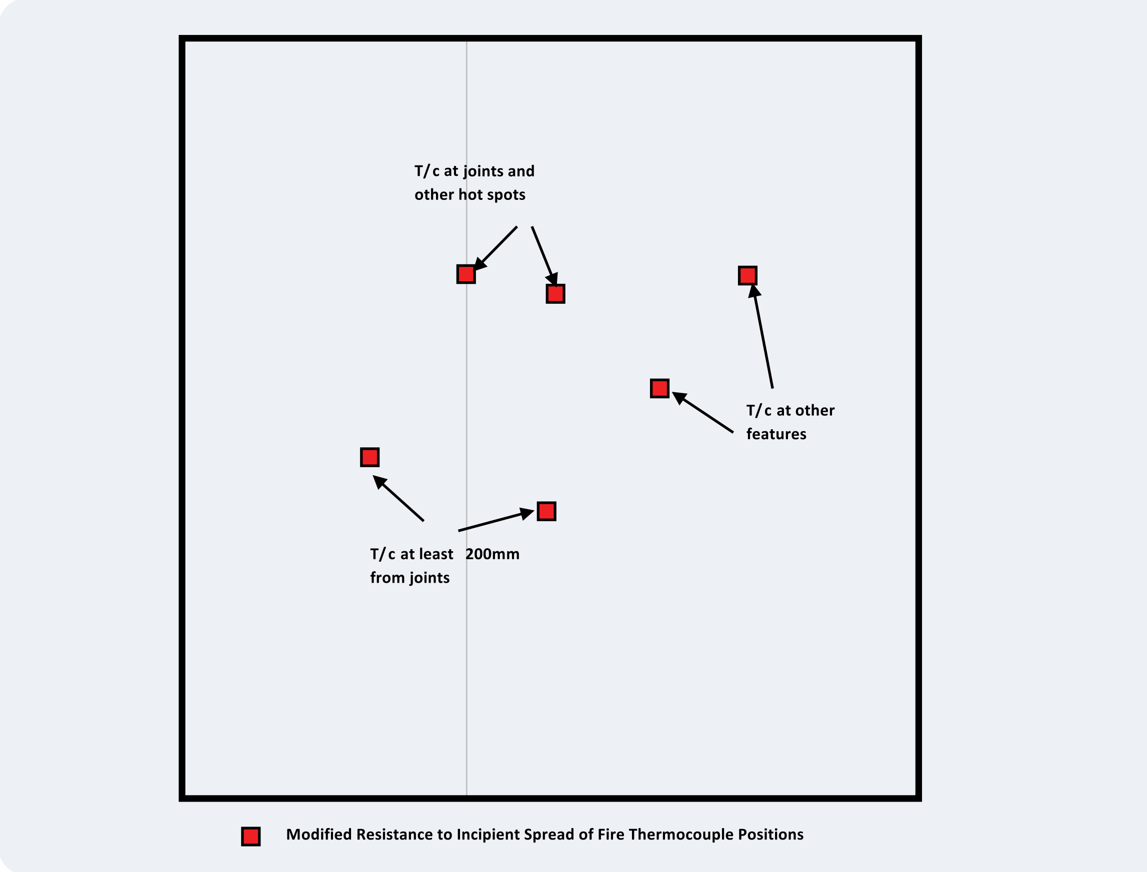


Figure A7: Elevation of a wall showing modified RISF thermocouple positions.

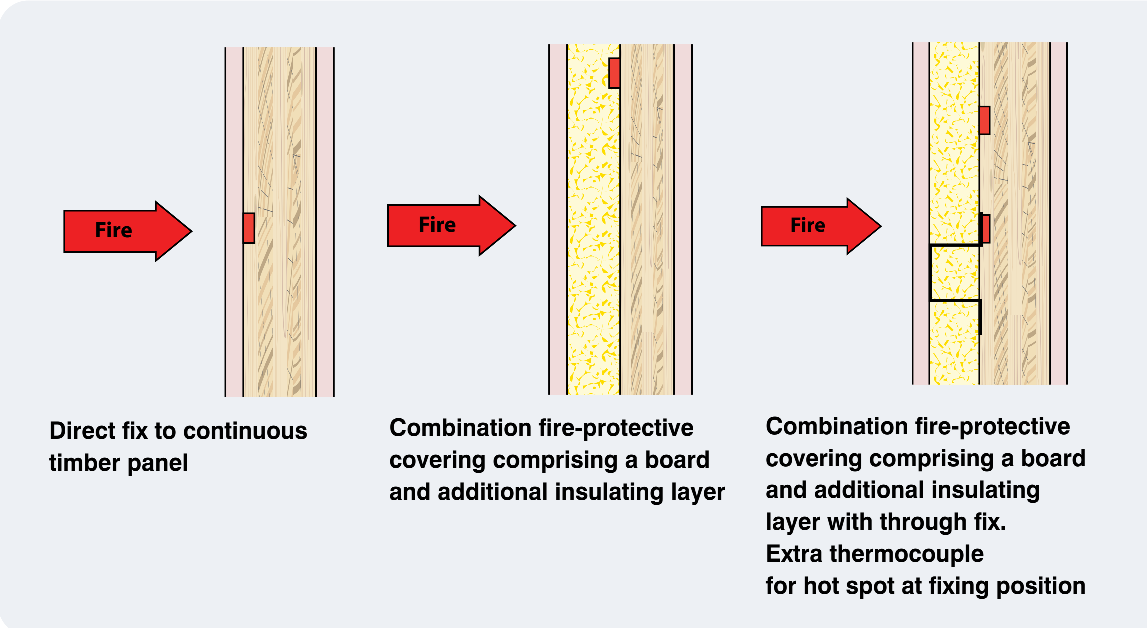


Figure A8: Modified RISF thermocouple positions for typical specimen configurations.

Failure in relation to the MRISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 300°C.

Smaller scale specimens 1 metre x 1 metre can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations to determine both the MRISF and FRLs are shown in Figure A9.

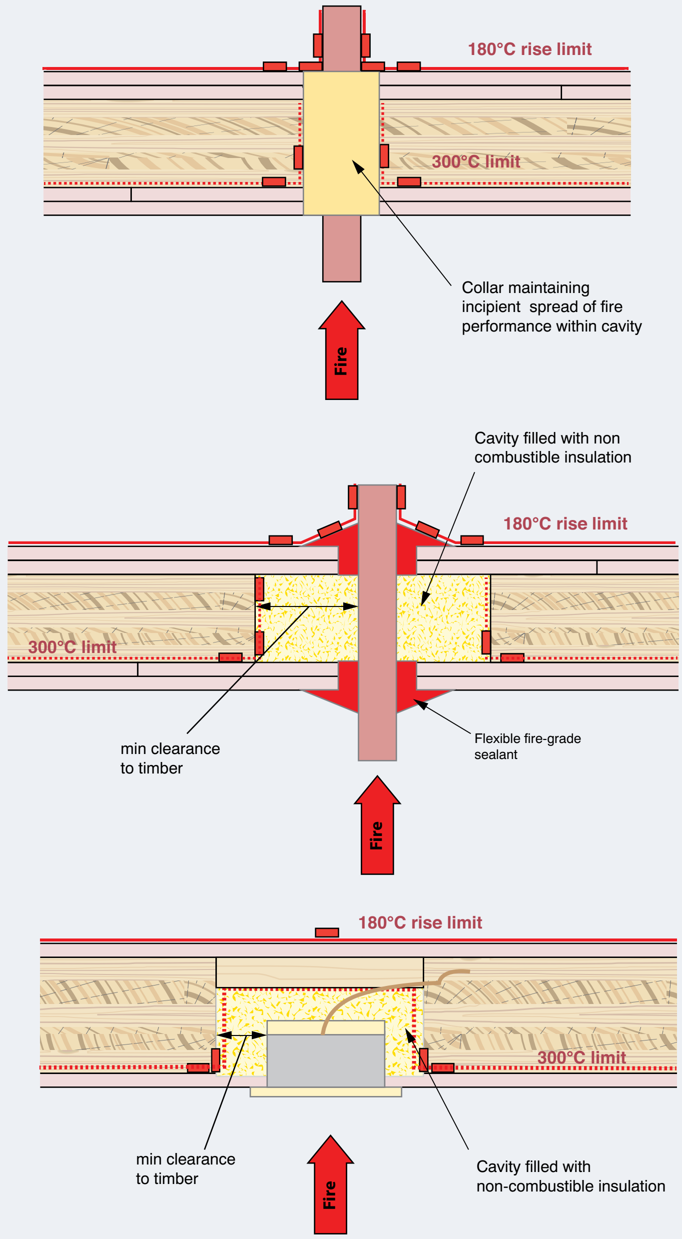


Figure A9: Typical thermocouple positions for determining the MRISF of service penetrations.

B

Appendix B – Determination of the Performance of Cavity Barriers in Fire-Protected Timber Construction

Specification C1.13 of the NCC sets out the requirements for cavity barriers in fire-protected timber construction.

The following compliance options are provided for cavity barriers:

- the cavity barrier system must achieve the FRLs specified in Table B1 when mounted in timber elements having the same or a lower density than the timber members in the proposed application or
- comprise timber of minimum thickness as specified in Table B1 or
- comprise polythene-sleeved mineral wool or non-sleeved mineral wool slabs or strips placed under compression and of minimum thickness as specified in Table B1 or
- another option is that, for cavity barriers around doors and windows, steel frames are also Deemed-to-Satisfy the requirements for cavity barriers provided that the steel frames should be tightly fitted to rigid construction and mechanically fixed. It should, however, be noted that if the windows or doors are of fire-resistant construction, the windows or door system needs to be capable of achieving the required fire resistance when mounted in the wall system, notwithstanding the requirements for cavity barriers.

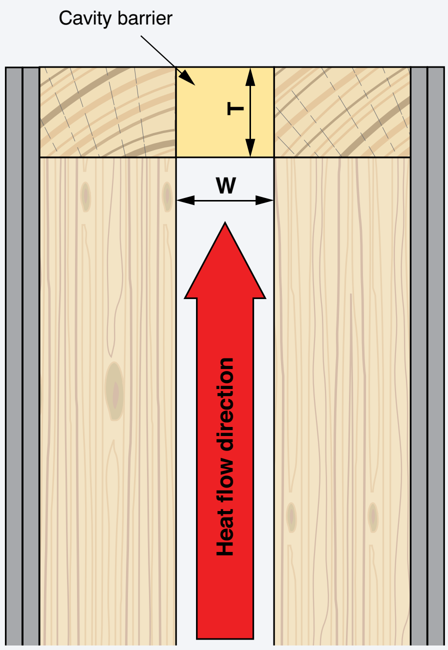
Table B1: Cavity barrier requirements for fire-protected timber.

Cavity Barrier Compliance Options	**FRL required for element cavity barrier is fitted to (minutes)**	
	–/90/90 or less	greater than –/90/90
Cavity Barrier Required FRL – minutes	–/45/45	–/60/60
Timber required minimum thickness	45 mm	60 mm
Mineral wool required minimum thickness	45 mm	60 mm

The minimum thicknesses of protection are required to be measured in the direction of heat flow. The role of a cavity barrier is normally to prevent a fire spreading from the cavity on one side of the cavity barrier to the other. The top plate of a double stud partition (Detail A of Figure B1) is a typical example of this where the direction of heat flow for the cavity barrier would be from the underside to the upper face of the barrier.

The other role for cavity barriers is to reduce the risk of fire spread to cavities occurring around openings for doors and windows within a fire-resisting wall. This configuration is shown as Detail B in Figure B1. For this scenario, the heat flow is from the occupied area of the building through the framing to the cavity. In the Figure, the thickness dimension is identified as 'T'.

Detail A
Cavity barrier at the head of a double stud partition



Detail B
Cavity barrier around door opening

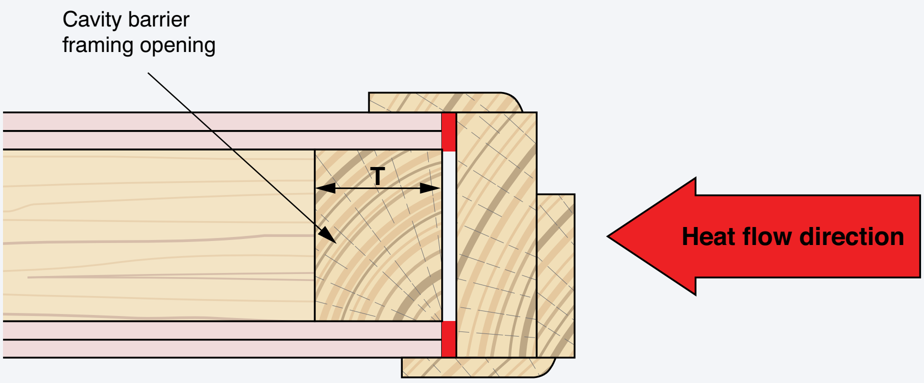


Figure B1: Heat flow direction for cavity barriers.

Proprietary cavity barrier systems may provide more practical options than the Deemed-to-Satisfy solutions for some applications. To encourage the development and use of these systems a compliance path has been provided through the specification of FRLs. For smaller cavity barriers, the performance should be determined by testing the cavity barrier as a control joint system in accordance with Section 10 of AS 1530 using timber members as the separating element. Specification C1.13 permits the results from such a test to be used for applications where the fire-protected timber is constructed from timber with a nominal density at least equal to the tested timber.

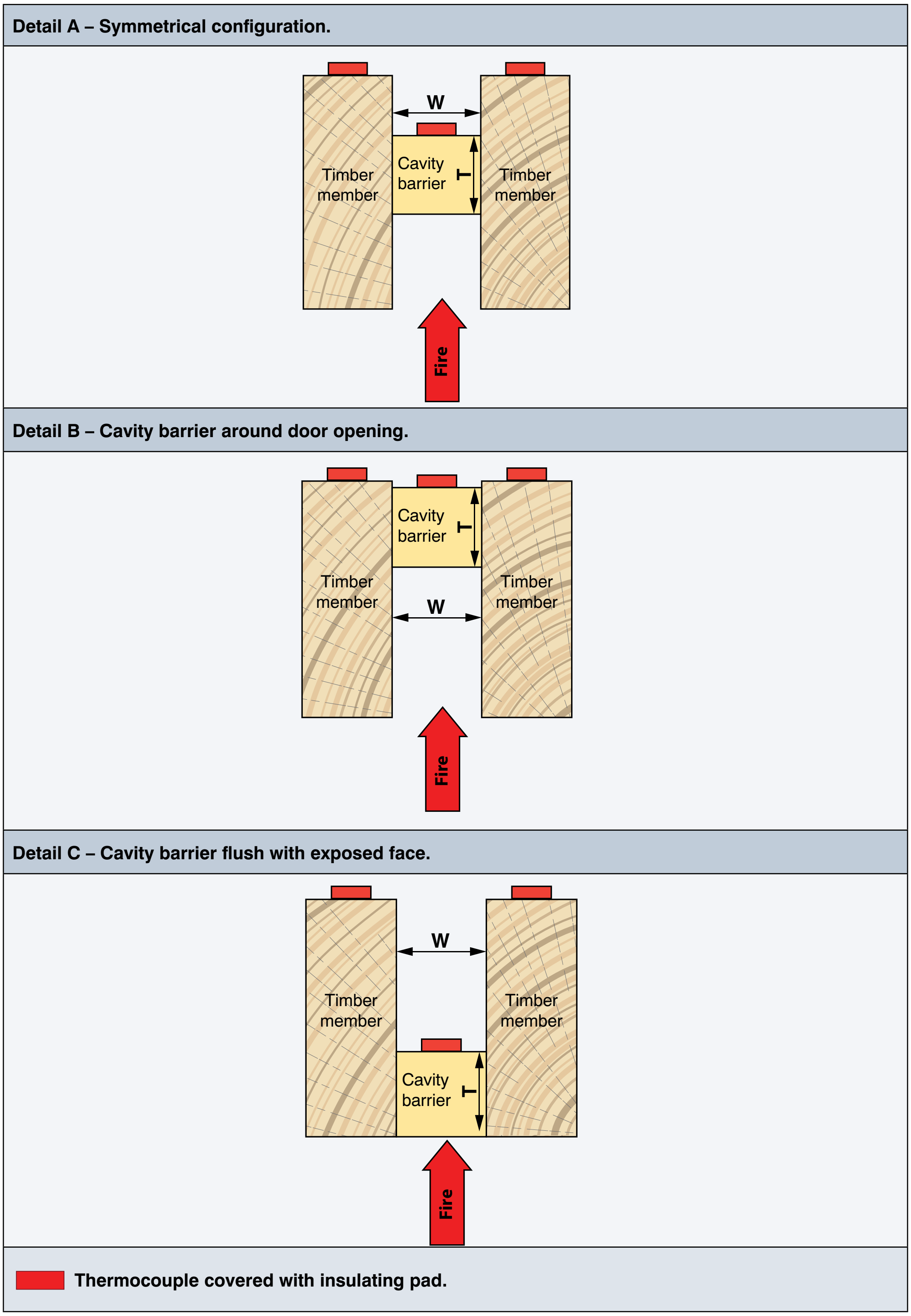


Figure B2: Typical cavity barrier test configurations.

Typical test configurations are shown in Figure B2. The selection of the test configuration(s) depends on how the cavity barrier will be mounted. If it is symmetrical (e.g. fitted at the mid-depth of a timber member), Detail A is appropriate. If the cavity barrier system is not symmetrical both details B and C should be tested unless the most onerous configuration can be determined by the test laboratory or the cavity barrier use is restricted to one configuration. A report from an Accredited Testing Laboratory should state the field of application for the cavity barrier based on the test results.

Cavity barriers can be of combustible construction and therefore a timber framed partition with exposed timber members could be used subject to the wall achieving the required FRL.

In some instances, it may be more practicable to continue the fire-resisting walls up to roof level in lieu of providing a fire-protected timber roof system with cavity barriers. This option is shown in Figure B3.

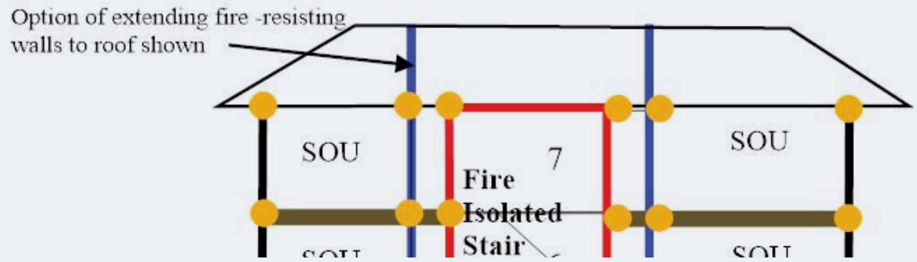


Figure B3: Design option to extend fire walls to roof level in lieu of using large cavity barriers within a fire-protected timber roof system.

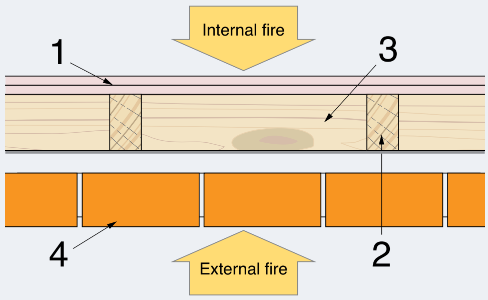
C

Appendix C – Example Data Sheets for an External Wall System

The following data sheet provides an example of the Evidence of Suitability required by the NCC. A brick veneer external wall system has been used because, in addition to fire and sound requirements, thermal resistance, weatherproofing and structural tests apply.

System External Wall 1

External Brick Veneer Timber framed wall system



1 Fire protective grade plasterboard, 2 x 13 mm thick

2.Timber framing in accordance with Evidence of Suitability

3 Cavity. – Cavity insulation may be required to achieve sound ratings and R –value (insulation must be non-combustible)

4 Outer brick veneer 90 mm thick

Typical Performance

Fire-protected timber	FRL90/90/90: RISF45: NC
Sound transmission and insulation	R_w 50: R_w+C_{tr} 50
Thermal resistance	R Value 3.3 m²K/W
Damp and weatherproofing	NCC performance requirement FP1.4
Structural tests	NCC specification C1.8 Clause 3.4

Evidence of Suitability

Fire-protected timber:	
Internal Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 – (e.g.Exova Warringtonfire report 22567A-01) RISF – 45 (NCC Spec C1.13a DTS)
External Fire Exposure	FRL Test or assessment report from an Accredited Testing Laboratory complying with NCC A5.4 or design in accordance with AS 3700 RISF – 45 (AS 3700 design for insulation or test or assessment report from an Accredited Testing Laboratory)
Non-combustibility	Plasterboard NCC C1.9(e)(i) DTS Fire-protected timber NCC C1.13 Concession Cavity Insulation AS 1530.1 test report Brickwork –traditional building material

Sound Transmission and Insulation No NCC requirement for external walls in NCC 2016 but commonly specified for inner city locations. Report from a laboratory or acoustics engineer stating performance achieved.

Thermal Resistance R-Value Report complying with NCC Clause A5.2

Weatherproofing Statement of compliance with relevant requirements of AS 3700 and report confirming applicability of AS 3700 – complying with NCC Clause A5.2.

Structural tests for lightweight construction Report complying with NCC Clause A5.2 expressing results of tests in accordance with NCC specification C1.8.

Notes

Selection of systems that are fit for the purpose and the provision of Evidence of Suitability to the satisfaction of the relevant authority is the responsibility of the designers and product suppliers. Forest and Wood Products Australia Limited (FWPA) and the authors of this Guide make no warranties or assurances with respect to the fitness for purpose of the systems described in this Guide.

Primary Distributors

Various plasterboard distributors

Obtain Evidence of Suitability from product supplier before specifying or installing any product or system

Ensure installation is in accordance with Evidence of Suitability, manufacturer's instructions and design drawings.

D

Appendix D: Glossary

National Construction Code (NCC)
National Construction Code Volume One: Building Code of Australia 2019.

Cavity barrier
A barrier placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements, that complies with Specification C1.13 of the NCC, to limit the spread of fire, smoke and hot gases to other parts of the building.

C_{tr}
refer Spectrum adaption term (airborne noise.)

Discontinuous construction
A wall system typically having a minimum of 20 mm cavity between two separate wall frames (leaves) with no mechanical linkage between the frames except at the periphery intended to reduce sound transmission.

Exit
Includes any of the following if they provide egress to a road or open space:

- an internal or external stairway
- a ramp complying with Section D of the NCC
- a doorway opening to a road or open space
- a fire-isolating passageway
- horizontal exit.

Fire-protected Timber
Fire-resisting timber building elements that comply with Specification C1.13a of the NCC.

Fire-protective grade plasterboard
Plasterboard with glass fibre and mineral additives used to improve strength and control shrinkage under fire conditions. Typically a lightweight loadbearing timber framed wall protected by one layer of 16 mm fire-protective grade plasterboard applied to each face would be expected to achieve an FRL of at least 60/60/60 and if protected by two layers of 13 mm fire-protective grade plasterboard on each face an FRL of at least 90/90/90.

Fire-isolated stair or ramp
A stair or ramp construction of non-combustible materials and within a fire-resisting shaft or enclosure.

Fire-isolated passageway
A corridor or hallway of fire-resisting construction that provides egress to a fire-isolated stairway or ramp or to open space.

Fire Resistance Level (FRL)
The time in minutes, determined in accordance with Clause A5.4 (of the BCA) for the following, in order:

- structural adequacy
- integrity
- insulation

Fire-resisting
As applied to a building element means, having the FRL appropriate for that element

Fire-resisting sealant
Fire-grade material used to fill gaps at joints and intersections in fire-protective linings and around service penetrations to maintain Fire Resistance Levels and Resistance to Incipient Spread of Fire performance of elements of construction. Note: The material should also be flexible to allow for movement and where necessary waterproof.

Fire-source feature

- The far boundary of a road adjoining the allotment; or
- a side or rear boundary of the allotment; or
- an external wall or another building on the allotment which is not of Class 10.

Habitable room
A room for normal domestic activities, e.g. bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room and sunroom. Excludes bathroom, laundry, water closet, pantry, walk-in wardrobe, corridor, hallway, lobby, clothes-drying room, and other spaces of a specialised nature occupied neither frequently nor for extended periods.

Internal walls
Walls within, between or bounding separating walls but excluding walls that make up the exterior fabric of the building. Note: Fire walls or common walls between separate buildings or classifications are NOT internal walls.

$L_{n,w}$
refer Weighted normalised impact sound pressure level.

Lightweight construction
Construction that incorporates or comprises sheet or board material, plaster, render, sprayed application, or other material similarly susceptible to damage by impact, pressure or abrasion.

Massive Timber 'Concession'
A relaxation allowing the Resistance to Incipient Spread of Fire requirements for fire-protected timber to be modified if both the following conditions are satisfied:

- the timber is at least 75 mm thick
- any cavity between the surface of the timber and the fire-protective covering is filled with non-combustible materials.

Massive Timber Panels
Large engineered wood panels of minimum thickness of 75 mm thick. Typical examples include Cross-laminated Timber (CLT), Laminated Veneer Lumber (LVL) and Glulam panels.

Modified Resistance to the Incipient Spread of Fire (MRISF)
The MRISF, amongst other things, relaxes the RISF limiting temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures for MRISF are described in Section 3 of Specification C1.13a of the NCC.

Multi-service penetration system
A service penetration system used to protect a group of services penetrating a single opening in a fire-resisting element such that the FRL, RISF or MRISF of the element is not reduced. Note: Fire protective coverings or other means may be required to be fitted around the opening to ensure that the RISF or MRISF are not reduced.

Non-combustible
Applied to a material not deemed combustible under AS 1530.1 – Combustibility Tests for Materials; and applied to construction or part of a building – constructed wholly of materials that are not deemed combustible.

Performance Requirements
The requirements in the NCC that describe the level of performance expected from the building, building element or material.

Resistance to the Incipient Spread of Fire (RISF)
The ability of a fire-protective covering to insulate voids and the interfaces with timber elements to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

R_w
Refer to Weighted sound reduction index.

Sole-Occupancy Unit (SOU)
A room or other part of a building for occupation by one or joint owner, lessee, tenant, or other occupier to the exclusion of any other owner, lessee, tenant, or other occupier and includes:

- a dwelling (e.g. apartment)
- a room or suite of rooms in a Class 3 building which includes sleeping facilities
- a room or suite of associated rooms in a Class 5, 6, 7, 8 or 9 building
- a room or suite of associated rooms in a Class 9c building, which includes sleeping facilities and any area for the exclusive use of a resident

Spectrum adaption term (C_{tr})
Used to modify the sound performance of wall and floor/ceiling systems to reflect low frequency performance. In combination with Rw (i.e. $R_w + C_{tr}$) is used to account for low frequency noise.

Weighted sound reduction index (R_w)
The rating of the sound isolating properties of building element as described in AS/NZS ISO 717.1.

Weighted normalised impact sound pressure level ($L_{n,w}$)
The measurement of how much sound reaches a receiving room through a building element from a standard tapping machine.

References

WoodSolutions Technical Design Guides

#1 Timber-framed Construction for Townhouse Buildings Class 1a

#2 Timber-framed Construction for Multi-residential Buildings Class 2 and 3

#3 Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b

#4 Building with Timber in Bushfire-prone Areas

#5 Timber service life design – Design guide for durability

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)

#17 Alternative Solution Fire Compliance, Timber Structures

#18 Alternative Solution Compliance Facades Forest and Wood Products Australia Ltd 2015

#20 Fire Precautions during Construction of Large Buildings

#39 Robustness in Structures

#38 Fire Safety Engineering Design of Mid-Rise Buildings

Other:

R-values for Timber-framed Building Elements

Australian Standards

AS 2118.1 Automatic fire sprinkler systems – General requirements

AS 2118.4 Automatic fire sprinkler systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height

AS 2118.6 Automatic fire sprinkler systems – Combined sprinkler and hydrant systems in multi-storey buildings

AS 1170 series – Structural design actions

AS 1720.1 Timber structures – Design methods

AS 5113 Amd 1 Classification of external walls of buildings based on reaction to fire performance

AS 1905.1 Components for the protection of openings in fire-resistant walls – Fire-resistant doorsets

AS 1530.1 Methods for fire tests on building materials, components and structures - Combustibility test for materials

AS 1530.4 Methods for fire tests on building materials, components and structures - Fire-resistance tests for elements of construction

AS 4072.1 Components for the protection of openings in fire-resistant separating elements – Service penetrations and control joints

AS 2444 Portable fire extinguishers and fire blankets – Selection and location

AS 1682.1 Fire, smoke and air dampers – Specification

AS 1682.2 Fire, smoke and air dampers – Installation

Other References

National Construction Code Volume One: Building Code of Australia 2019 – Australian Building Codes Board, Canberra ACT – © Commonwealth of Australia and the States and Territories 2019

International Fire Engineering Guidelines (2005) – Australian Government, Sate and Territories of Australia

Safe Design of Structures – Safe Work Australia

Exova Warringtonfire Australia Pty Ltd (EWFA) Regulatory Information Reports (RIR) issued to Forest & Wood Products Australia:

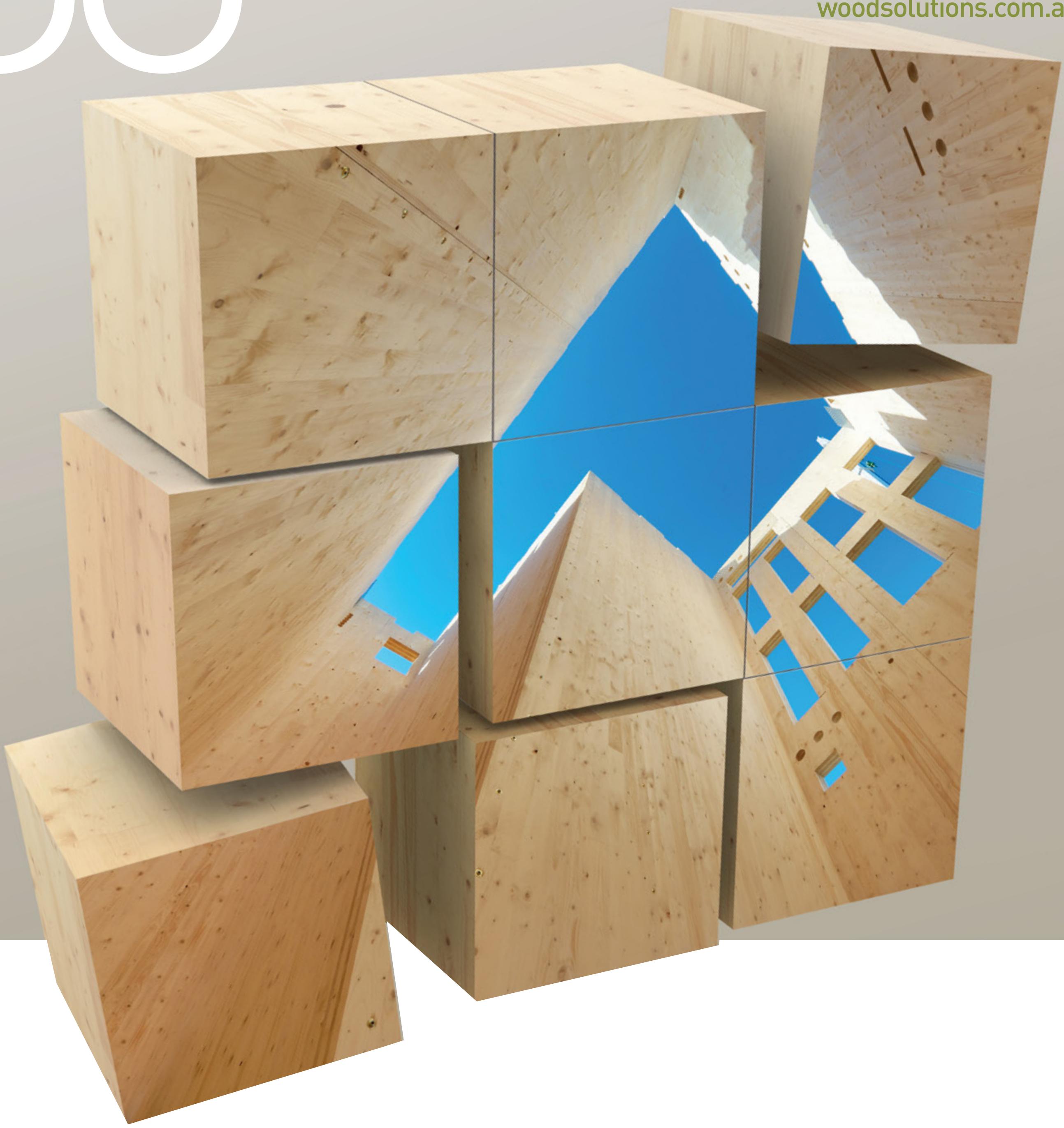
RIR 22567A-04 – The fire resistance performance of timber-framed walls lined with plasterboard if tested in accordance with AS1530.4-2005

RIR 37600400 – The fire resistance level (FRL) of timber-framed floor/ceiling systems incorporating timber and metal web floor trusses or various engineered joists when tested in accordance with AS 1530.1-2014

RIR 37401400 – The fire resistance level (FRL), Resistance to the Incipient Spread of Fire (RISF) and Modified Resistance to the Incipient Spread of Fire (MRISF) performance of various timber-framed and massive timber panel systems


Wood
Solutions
woodsolutions.com.au
Wood
Solutions
Tutorials

38



Fire Safety Design of Mid-rise Timber Buildings

Basis for the 2016 changes to the National Construction Code

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image courtesy of Agrop

WoodSolutions
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-36-2

Researcher:
Paul England, EFT Consulting
paul.england@eftconsult.net

Technical Reviewer:
Dr. Ian Bennetts, SKIP Consulting
consulting@skip.net.au

Published: October 2016
December 2021 (Supplementary Notes included)

2021 Supplementary Notes

Introduction

WoodSolutions Technical Design Guide 38 [1] provides details of the technical supporting data and fire engineering analysis that was undertaken to evaluate the potential impact of the inclusion of a Deemed-to-Satisfy (DtS) pathway for mid-rise Class 2, 3 and 5 fire-protected timber buildings that was subsequently included in the National Construction Code Volume One [2] (NCC 2016).

The body of the guide has not been updated since the analysis was undertaken and uses the terminology and content of the (then) proposed NCC 2016 edition. However, the fire engineering analysis and supporting data still has relevance to potential Performance Solutions.

These supplementary notes identify some relevant changes to the NCC and interpretations of the NCC through the publication of Amendment 1 to NCC 2016 Volume One [3], NCC 2019 Volume One [4] and Amendment 1 to NCC 2019 Volume One [5].

Significant changes to the NCC's structure and terminology have been made since 2016 as part of an initiative to improve its readability. The primary purpose of the restructure was not to materially modify the NCC but to clarify its intent and hence facilitate compliance. The amendments included some technical changes as part of the routine revision cycle for the NCC.

To assist readers of Technical Guide 38, the more relevant changes are summarised below.

General changes to NCC Volume One – structure and terminology

Some of the more relevant changes to the content of Guide 38 are:

- General Provisions have been replaced with a new Section A 'Governing Requirements'
- Substantial amounts of content has been moved to Schedules
- Performance Requirements have been provided with headings
- New Verification Methods have been introduced and changes made to some existing Verification Methods
- The term Registered Testing Laboratory has been replaced with Accredited Testing Laboratory (ATL).

As a result, relevant clause numbers and locations within the NCC of the content relating to fire-protected timber mid-rise building DtS solutions have changed, but the relevant technical provisions remain the same. Figure S1 identifies the current location of the relevant content in the NCC and should be used in lieu of Figure 4.1 in the body of Guide 38 if the NCC Volume One 2019 edition is being referred to instead of the NCC 2016.

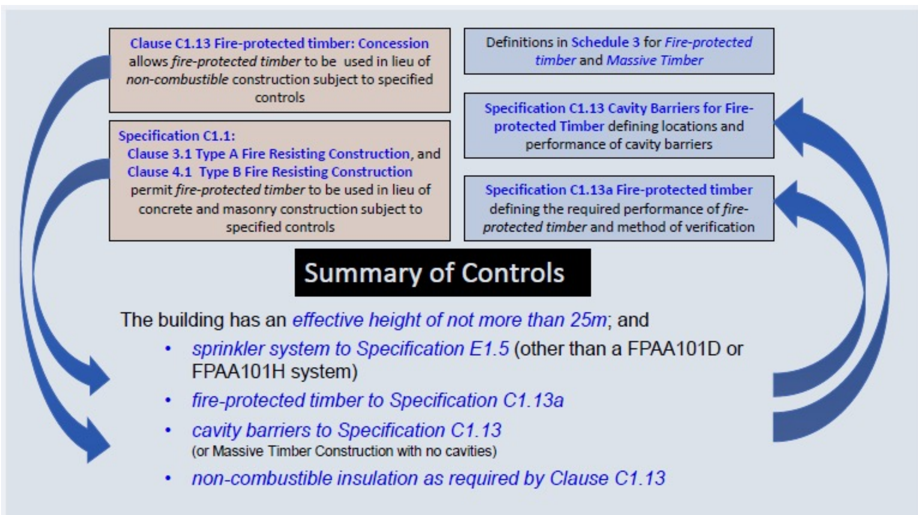


Figure S1: Summary of fire-protected timber controls based on NCC 2019 Amd 1.

Verification methods

BV1 has been revised and provides a means of verifying the reliability of structural components that may have relevance to the robust design of mid-rise timber structures (addressed in more detail in WoodSolutions Technical Design Guide 39). Technical Design Guide 38 includes information on the reliability of fire protection systems, including fire-protective coverings and methods of analysis suited to quantified fire risk assessments.

CV3 provides a verification method for Performance Solutions demonstrating compliance with Performance requirement CP2 to avoid the spread of fire via the external wall of a building when involving combustible external facades. As NCC C1.13 allows fire-protected timber to be used where an element is required to be non-combustible, the addition of CV3 provides another option to determine compliance with CP2 if combustible materials are applied to the external wall, such as a combustible external weather screen. The content of Technical Design Guide 38 does not require adjustment.

CV4, DV4, EV1.1 – The fire safety verification method applies a holistic comparative approach to the assessment of Performance Solutions and nominates a number of scenarios that should be analysed. This approach is consistent with the analysis methods adopted to justify the changes to the NCC allowing the use of fire-protected timber and so Technical Design Guide 38 provides a useful resource to support the introduction of the fire safety verification method in the NCC.

Extension of fire-protected timber concession to all building classes

The concession for the use of fire-protected timber was extended to all classes of buildings generally using similar analysis methods to those used for Class 2, 3 and 5 buildings that were the focus of the 2016 revision of the NCC and content of Technical Design Guide 38.

Non-combustibility

Provisions relating to combustibility within the NCC have been reviewed and revised since the 2016 edition to improve clarity and building compliance, with an emphasis on external walls. These changes do not affect the relevance of the analysis described in Technical Design Guide 38 but still apply to mid-rise fire-protected timber buildings if the DtS pathway is followed. A useful summary of the changes is provided in the ABCB publication 'Fire performance of external walls and cladding – Advisory Note' [6].

FPAA101D and FPAA 101H sprinkler systems

Two automatic fire sprinkler system design codes were introduced for use in some mid-rise residential Class 2 and 3 buildings. These sprinkler systems have reduced coverage and flow rates, among other things, compared to other sprinkler systems prescribed by Specification E1.5 of the NCC and were not evaluated as part of the analysis described in Technical Design Guide 38. The reduced coverage and flow rates would reduce the efficacy and reliability of a sprinkler system, and this would increase the risk to life calculated for the mid-rise timber buildings if the FPAA101D and FPAA101H sprinkler systems were used. ***The NCC 2019 DtS solution for mid-rise fire-protected timber buildings does not permit the use of FPAA101D and FPAA101H sprinkler systems.***

Other Relevant WoodSolutions Technical Design Guides

Technical Design Guides 37R, 37H and 37C [7-9] have been updated to provide guidance on the design of fire-protected timber mid-rise timber buildings using the DtS pathways defined in the NCC 2019 Amd 1 edition.

Technical Design Guides 17[10] and 18[11] have been rewritten to provide additional methods and data to support the design of timber buildings using a Performance Solution pathway and applying holistic approaches consistent with the fire safety verification method and quantification of performance requirements that are likely to be introduced in the NCC 2022.

References

1. England, P., *WoodSolutions Technical Design Guide 38 Fire Safety Design of Mid-rise Timber Buildings. Basis for the 2016 changes to the National Construction Code. 2016*: Australia.
2. ABCB, *National Construction Code 2016 Volume 1, in Building Code of Australia Class 2 to 9 Buildings*. 2016, Australian Building Codes Board: Canberra.
3. ABCB, *National Construction Code 2016 Volume 1 Amendment 1, in Building Code of Australia Class 2 to 9 Buildings*. 2018, Australian Building Codes Board: Canberra.
4. ABCB, *National Construction Code 2019 Volume 1, in Building Code of Australia Class 2 to 9 BuildingsAustralian Building Codes Board: Canberra.* 2019.
5. ABCB, *National Construction Code 2019 Volume One Ammendment 1*. 2020.
6. ABCB, *Fire performance of external walls and cladding* ABCB, Editor. 2020, ABCB: Canberra.
7. England, P. and B. Iskra, *WoodSolutions Technical Design Guide 37R Mid-rise Timber Buildings - Multi-residential Class 2 and 3.* 2019, Melbourne: WoodSolutions.
8. England, P. and B. Iskra, *WoodSolutions Technical Design Guide 37H Mid-rise Timber Buildings -Health-Care Class 9a and Class 9c*. 2020, Melbourne: WoodSolutions.
9. England, P. and B. Iskra, *WoodSolutions Technical Design Guide 37C Mid-rise Timber Buildings -Commercial and Education Class 5,6,7,8 and 9b (including Class 4 parts)*. 2019, Melbourne: WoodSolutions.
10. England, P. and B. Iskra, *WoodSolutions Technical Design Guide 17 Fire Safe Design of Timber Structures-Compliance with the National Construction Code.* 2021, Melbourne: WoodSolutions.
11. England, P., *WoodSolutions Technical Design Guide 18 Fire Safe Design of Timber Structures-Methods of Analyis and Supporting Data*. 2021, Melbourne: WoodSolutions.

Guide Map

The primary purpose of this Guide is to provide details of the fire engineering analysis that was undertaken to compare the changes relating to mid-rise timber buildings included in the 2016 edition of the National Construction Code (NCC) with Deemed-to-Satisfy building solutions for similar non-combustible building configurations in the 2015 edition.

Supplementary information relevant to the fire safety design of mid-rise timber buildings has been included to assist building designers and building approval authorities determining compliance with the NCC.

For clarity the Guide has been broken up into four parts:

Part A provides background to the development of the NCC Deemed-to-Satisfy Provisions for mid-rise buildings and a brief introduction to the NCC for those unfamiliar with the Australian National Construction Code.

Part B includes general information relevant to the fire safety design of mid-rise timber buildings. It includes information relating to demonstrating compliance with the NCC by means of the performance and Deemed-to-Satisfy pathways and responsibilities of practitioners for the safe design of buildings.

Part C provides a record of the technical justification for the 2016 changes to the NCC relating to mid-rise timber buildings – the primary purpose of this publication.

Part D provides supplementary information relevant to the fire engineering design of mid-rise buildings and the technical justification described in Part C.

Contents

A

Part A - Preliminaries

Section 1 provides a general introduction and the background to the development of the National Construction Code (NCC) provisions for mid-rise timber buildings.

Section 2, National Construction Code Basics, provides a brief introduction to some key areas relating to timber for those unfamiliar with the Australian National Construction Code.

1

Introduction

The 2016 edition of the National Construction Code NCC[1] includes, for the first time in Australia, Deemed-to-Satisfy (DTS) design solutions for mid-rise timber buildings. These DTS solutions in the 2016 edition apply to mid-rise Class 2, 3 and 5 (residential and office) buildings, and introduce the concepts of fire-protected timber and the use of cavity barriers to the NCC.

There will be applications where it is desired to vary the DTS requirements by developing a performance solution for different building configurations. For example, a design for a high rise building will need to consider, among other things, increased evacuation times, increased time before search and rescue and fire-fighting commence, and the difficulty of external fire-fighting and rescue from the higher levels.

This Guide provides details of the underlying principles and the fire engineering analysis undertaken to support the 2016 changes to assist those who are designing or determining compliance of performance solutions for timber buildings.

Mid-rise timber buildings are typically 3 to 8 storeys high

1.1 Scope

This Guide will:

- describe the DTS solutions in the NCC 2016 for mid-rise timber buildings and explain the underlying fire safety principles
- provide details of the fire engineering analysis undertaken to compare the mid-rise timber building solution with non-timber DTS solutions
- present useful data and analysis methods relevant to the fire safety design of timber buildings.

1.2 Definition of Mid-rise Timber Buildings

Low-rise timber buildings
are buildings of

- Class 1 construction (1 or 2 storey) or
- Class 2 and 3 buildings up to 3 storeys; 4 storeys if the ground level is a concrete or masonry garage.

Mid-rise timber buildings
have an effective height of not more than 25m

Mid-rise timber buildings are typically 4 to 8 storeys high (the maximum number of storeys depends on the floor to floor height)

High-rise timber buildings
have an effective height greater than 25m.

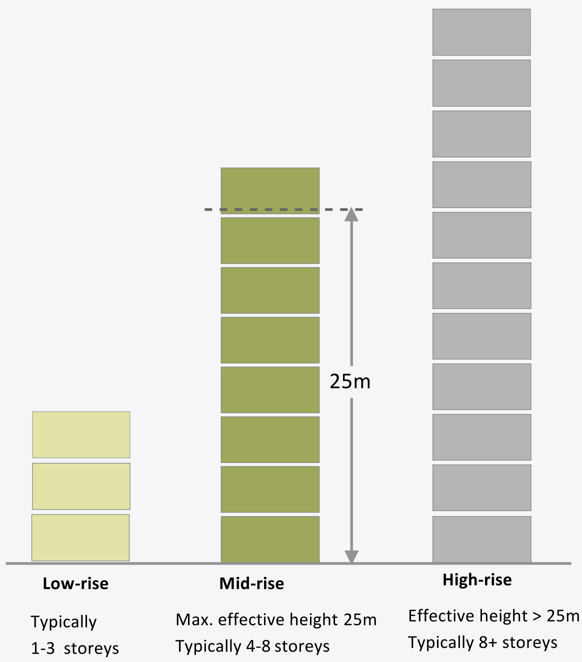


Figure 1.1: Comparison of low-rise, mid-rise and high-rise buildings.

Effective height is defined in the NCC and means the vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units).

1.3 Fire-safe Timber Construction

Traditional timber construction, in common with most other forms of construction, has advantages and disadvantages with respect to fire safety. By developing a fire safety design that takes account of these advantages but mitigates the disadvantages, mid-rise timber buildings can be designed to achieve equivalent or better levels of fire safety than other forms of construction such as the Deemed-to-Satisfy solutions prescribed in the National Construction Code 2015[2].

Some of the most relevant fire-related considerations for timber construction are summarised below, together with potential mitigation methods. Appendix A includes information of the response of timber buildings to fires, providing an introduction to readers unfamiliar with the fire safety design of timber buildings as well as supplementary reference data.

1.3.1 Combustibility

Timber is combustible and this has been the reason for placing substantial restrictions on timber mid-rise and high-rise structures within the National Construction Code (formerly the Building Code of Australia) since its initial release in 1988 and general adoption in the early 1990s. Potential issues raised during the consultation process included:

- If timber members are exposed to fire, the timber members may increase the effective fire load within an enclosure, potentially increasing the fire duration/severity of a fully developed fire. The NCC does not specifically limit the fire load that can be introduced into enclosures (other than requiring certain elements to be non-combustible).
- Timber elements/structures may continue to degrade after exposure to fire conditions. Other materials commonly used for structural elements/structures, including masonry and reinforced concrete structures, also degrade after exposure to fire.

These and other issues are discussed in more detail in the body of the Guide and were addressed primarily by specification of automatic fire sprinklers and fire-protective coverings to the timber elements for the prescribed mid-rise timber building solution in NCC 2016. The application of fire-protective coverings is sometimes referred to as encapsulation.

1.3.2 Fire Spread through Voids and Cavities

The potential for fire and smoke spread through buildings via cavities and voids exists with most types of framed construction, unless measures are taken to address the risk. Fire spread can be accelerated if combustible materials are contained within the voids.

The main mitigation measures to address this risk for the prescribed mid-rise timber building solution in NCC 2016 are:

- automatic fire sprinklers
- fire-protective coverings (to prevent fires entering the void)
- specification of non-combustible insulation
- specification of cavity barriers (to prevent uncontrolled fire spread through cavities if a fire enters or starts within a cavity)
- no unfilled voids or cavities permitted if the massive timber provision is applied (see below).

1.3.3 Inherent Fire Resistance of Timber Members

Most structural members require additional fire protection to be applied to provide an adequate level of fire resistance. For example, structural steel normally requires the application of fire protective boards or coatings and reinforced concrete relies on the concrete cover to protect steel reinforcing bars.

Timber having a large cross-section can achieve fire resistance levels (FRLs) in excess of 60/-/-, because when timber is exposed to fire it forms a protective char layer shielding the inner core of the timber as shown in Figure 1.2

Although national some NCC provisions vary by State. It is vital to know the applicable provisions

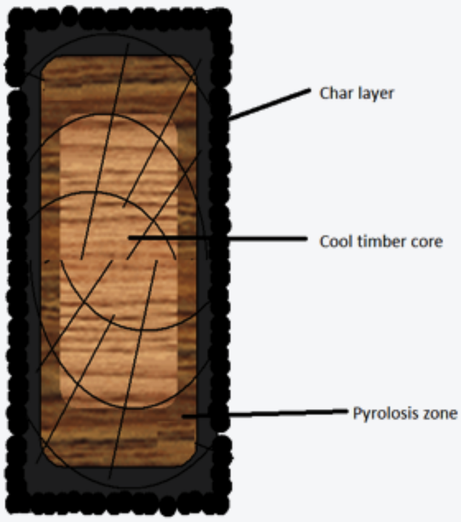


Figure 1.2: Timber member exposed to fully developed fire.

This effect is less pronounced for smaller members and, for engineered products such as lightweight trusses and I-section timber beams, the performance may be dominated by connections or the performance of steel components.

This is recognised in the NCC DTS requirements for mid-rise timber buildings by allowing a 'relaxation' in the performance of the fire-protective coverings for massive timber (without voids and cavities).

In addition, the NCC requires the FRL of a fire-protected timber member to be derived from full-scale tests, rather than solely rely on char calculations based on AS 1720.4, to enable the performance of the adhesives and connections used for engineered products to be verified.

1.3.4 Holistic Fire Safety Approach

A robust fire safety strategy for a building can be achieved by specifying a combination of measures to achieve the objectives that are not overly reliant on any one component.

The prescribed mid-rise timber building solution in NCC 2016 requires the provision of an automatic fire sprinkler system, greatly reducing the frequency of severe fires and hence improving the occupant survivability within the fire compartment of fire origin as well as other occupants within the building.

The combination of automatic fire sprinklers with the above mitigation methods and other requirements within the NCC were shown to provide a significant improvement in life safety for occupants of timber mid-rise buildings compared to equivalent mid-rise buildings of non-combustible construction meeting the Deemed-to-Satisfy requirements of the NCC 2015.

1.3.5 Stakeholder Issues

During the development of the DTS solutions for mid-rise timber buildings, the input of key stakeholders was sought to identify important issues. The main issues are summarised in Appendix B together with an explanation of how the issues were resolved.

2

National Construction Code Basics

The National Construction Code (NCC) is the regulatory framework for determining minimum construction requirements for all types of buildings in Australia.

While most readers will have as a minimum a basic understanding of the NCC, a brief introduction to some key areas is included in this section for those less familiar with the Australian system.

2.1 Building Classes

Refer to NCC Vol One A3.2 for details of all classes of building

The NCC contains mandatory Performance Requirements which apply to 10 primary classes of building. The classes are determined according to the purpose for which the building will be used. The classes considered in the fire engineering analysis described in this Guide were:

- **Class 2** – a building containing two or more sole-occupancy units, each being a separate dwelling, e.g. apartment buildings
- **Class 3** – a residential building which is a common place of long-term or transient living for a number of unrelated persons, including:
  - a boarding-house, guest house, hostel, lodging-house or backpackers accommodation
  - a residential part of a hotel, motel, school, detention centre or health-care building (where accommodating members of staff)
  - accommodation for the aged, children or people with disabilities
- **Class 5** – an office building used for professional or commercial purposes, excluding buildings of Class 6, 7, 8 or 9.

Other major classes defined in the NCC are:

- **Class 1a** – a single dwelling
- **Class 1b** – a boarding house, guest house, hostel or the like with a total area of all floors not exceeding 300 m2 in which not more than 12 persons would ordinarily be resident, which is not located above or below another dwelling or another class of building other than a private garage
- **Class 6** – a shop or other building for the sale of goods by retail or the supply of services direct to the public
- **Class 7a** – a car park
- **Class 7b** – a building used for storage, or display of goods or produce for sale by wholesale.
- **Class 8** – a laboratory, or a building in which a handicraft or process for the production, assembling, altering, repairing, packing, finishing, or cleaning of goods or produce is carried on for trade, sale, or gain.
- **Class 9a** – a health-care building, including those parts of the building set aside as a laboratory
- **Class 9b** – an assembly building, including a trade workshop, laboratory or the like in a primary or secondary school, but excluding any other parts of the building that are of another class
- **Class 9c** – an aged care building
- **Class 10** – a non-habitable building or structure.

Refer to NCC Volume One C1.2

Refer to NCC Volume One C1.1

Refer to NCC Volume One C2.2 for additional area limitations

2.2 Type of Construction

The building class in conjunction with the building height expressed in terms of the rise in storeys is used to determine the type of construction required.

The rise in storeys is the sum of the greatest number of storeys at any part of the external walls of the building and any storeys within the roof space:

- above the finished ground next to that part; or
- if part of the external wall is on the boundary of the allotment, above the natural ground level at the relevant part of the boundary.

Type A construction is the most fire-resisting form of construction and the NCC DTS solutions have in the past imposed severe limitations on the use of timber through the prescription of masonry and concrete construction and non-combustibility for elements required to achieve a prescribed Fire Resistance Level (FRL).

Type B construction does not require FRLs to be as high as those relating to Type A construction, but similar constraints to the use of timber are applied.

Type C construction is applicable to most low-rise buildings. It is the least fire-resisting form of construction and places few fire-related restrictions on the use of structural timber members.

The required Types of construction specified by the NCC are shown in Table 2.1.

Table 2.1: Types of Construction Required by NCC Volume One.

Rise in storeys or effective height	Multi-residential		Office	Retail	Car park/ Storage	Factory/ Laboratory	Hospitals /Public assembly
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
4 or more	**A**	**A**	**A**	**A**	**A**	**A**	**A**
3	**A**	**A**	**B**	**B**	**B**	**B**	**A**
2	**B**	**B**	**C**	**C**	**C**	**C**	**B**
1	**C**	**C**	**C**	**C**	**C**	**C**	**C**

Note: Clause 2.2 of the NCC also applies area and volume limits on fire compartments based on the Type of Construction

2.3 NCC Compliance Pathways

To comply with the NCC, it must be demonstrated that the relevant performance requirements have been satisfied using the assessment methods specified in the NCC. There are two pathways that can be followed (or a combination of the two):

- For a Deemed-to-Satisfy solution, it is necessary to provide evidence of suitability to show that the prescriptive Provisions within the NCC have been met.
- For a performance solution (previously referred to as an alternative solution), specific building solutions are developed for a building which may vary from the Deemed-to-Satisfy Provisions.

Among other things, this Guide provides details of the fire engineering analysis that was undertaken to establish the Deemed-to-Satisfy solutions for mid-rise Class 2, 3 or 5 timber buildings included in the 2016 edition of the NCC. An objective of this Guide is to inform designers and approval authorities of the underlying principles on which the mid-rise timber Provisions in the NCC are based.

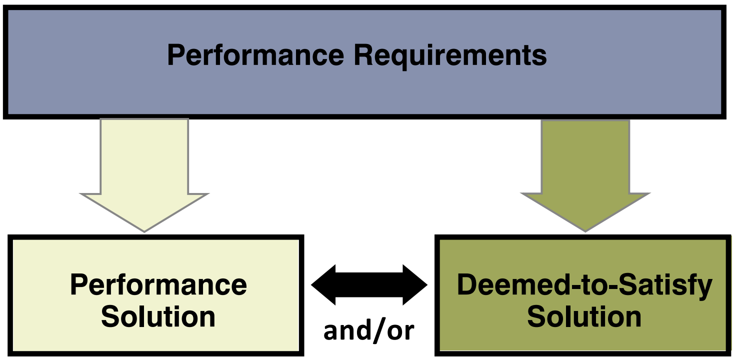


Figure 2.1: Pathways for demonstrating compliance with NCC performance requirements.

2.4 Selection of Compliance Pathways for Timber Buildings

2.4.1 Design Options for Building Classes of Various Heights

In the context of this Guide, timber buildings are defined as buildings where the loadbearing (structural) elements are predominantly timber. It should be noted that there are still opportunities to use timber for some structural and non-structural applications in buildings using other materials for the primary structure of a building.

Table 2.2 summarises options for complying with the NCC performance requirements for Class 2 to 9 buildings with further details provided below. Deemed-to-Satisfy solutions are available for the building configurations shaded in green. All building situations highlighted in blue should be assessed as Performance Solutions using a methodology compatible with that used for justifying the DTS solutions and described in detail in Parts 3 and 4 of this Guide; unless the entire fire safety strategy for the building is derived from first principles.

Table 2.2 Design options for timber buildings.

Rise in storeys or effective height	**Multi-residential**		**Office**	**Retail**	**Car park/ Storage**	**Factory / Laboratory**	**Hospitals / Public assembly**
	Class 2	Class 3	Class 5	Class 6	Class 7	Class 8	Class 9
Effective height greater than 25m	High	High	High	High	High	High	High
Approx. 8	Mid	Mid	Mid	Mid	Mid	Mid	Mid
7	Mid	Mid	Mid	Mid	Mid	Mid	Mid
6	Mid	Mid	Mid	Mid	Mid	Mid	Mid
5	Mid	Mid	Mid	Mid	Mid	Mid	Mid
4	Mid[1]	Mid[1]	Mid	Mid	Mid	Mid	Mid
3	Low[1]	Low[1]	Mid	Mid	Mid	Mid	Mid
2	Low[1]	Low[1]	Low	Low	Low	Low	Mid
1	Low	Low	Low	Low	Low	Low	Low

Note 1: See WoodSolutions Technical Design Guide #2: Timber-framed Construction for Multi-residential Buildings Class 2 & 3 to check if low-rise timber concessions apply.

Low	DTS Solution DG#2 or 3	Mid	DTS Solution –DG#37
Mid	Performance Solution – DG#38	High	Performance Solution

2.4.2 Low-rise Timber Buildings

There are relatively few fire-related restrictions on the use of structural timber members in Buildings of Type C construction irrespective of the Class of Building under the Deemed-to-Satisfy solution pathway and for domestic housing.

The NCC Volume One Deemed-to-Satisfy solution pathway also includes concessions that facilitate the use of timber-framed construction for Class 2 and 3 buildings up to a rise in storeys of 3 and in limited cases up to 4 storeys.

Guidance in relation to construction of these low rise options and Class 1a buildings is provided in the following WoodSolutions Technical Design Guides:

Refer to NCC Spec C1.1 Clauses 3.10 and 4.3 and WoodSolutions Design Guides #1, #2 and #3

#1 Timber-framed Construction for Townhouse Buildings Class 1a – information about complying with the fire safety and sound insulation performance requirements in the NCC for Class 1a attached buildings.

#2 Timber-framed Construction for Multi-residential Buildings Class 2 and 3 – provides information about complying with the fire and sound performance requirements in the NCC for Class 2, 3 low-rise buildings.

#3 Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b – provides information about complying with the fire performance requirements in the NCC for Class 5, 6, 9a and 9b buildings.

These buildings would normally be designed following the Deemed-to-Satisfy solution pathway with performance solutions being used to address minor variations and/or unusual design circumstances.

2.4.3 Mid-rise Timber Buildings

Mid-rise buildings are of Type A or B construction up to an effective height of 25m. The use of timber structural members under the NCC Deemed-to-Satisfy pathway is restricted for mid-rise buildings unless the option to use fire-protected timber in conjunction with automatic fire sprinklers is adopted: as introduced in the 2016 revision of the NCC for Class 2, 3 and 5 buildings. This Guide addresses buildings applying these design principles.

Guidance in relation to construction of these mid-rise options in accordance with the NCC Deemed-to-Satisfy provisions is provided in WoodSolutions Technical Design Guide #37: *Mid-rise Timber Buildings*.

For Class 6 to 9 buildings it will still be necessary to follow the performance solution pathway. Details of the technical derivation of the mid-rise fire-protected timber solution are provided in this Guide: which may assist with the development of a performance solution. Appendix C provides a summary of the Deemed-to-Satisfy clauses in the 2015 edition that were identified as restricting the use of timber and Appendix D identifies the performance requirements that relate to the identified Deemed-to-Satisfy clauses.

Check with the regulatory authority that the building's effective height is not more than 25m if applying the mid-rise fire protected timber solution

The NCC defines effective height as: "the vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units)".

Interpretations of the definition of effective height can vary and if there is any doubt as to whether a building's effective height does not exceed 25 m it is recommended that the interpretation is checked with the relevant authorities.

2.4.4 High-rise Buildings

All high-rise timber buildings will need to follow the performance solution pathway.

2.4.5 Mixed Class Buildings

The NCC Deemed-to-Satisfy solution for Class 2, 3 and 5 mid-rise buildings using fire-protected timber in conjunction with automatic fire sprinklers can also be applied to the Class 2, 3 and 5 parts of mixed-class buildings, provided the different classes are adequately fire separated and the entire building is protected by an automatic fire sprinkler system complying with NCC Volume One Specification E1.5.

This provides added flexibility for the design of new buildings and facilitates the recycling of existing buildings without necessarily relying on performance solutions. For example, fire-protected timber apartments (Class 2) could be constructed above existing concrete-framed retail/car park levels minimising the increase in foundation loads, as shown in Figure 2.1.

Fire-protected timber can be used in conjunction with other forms of construction in mixed class buildings

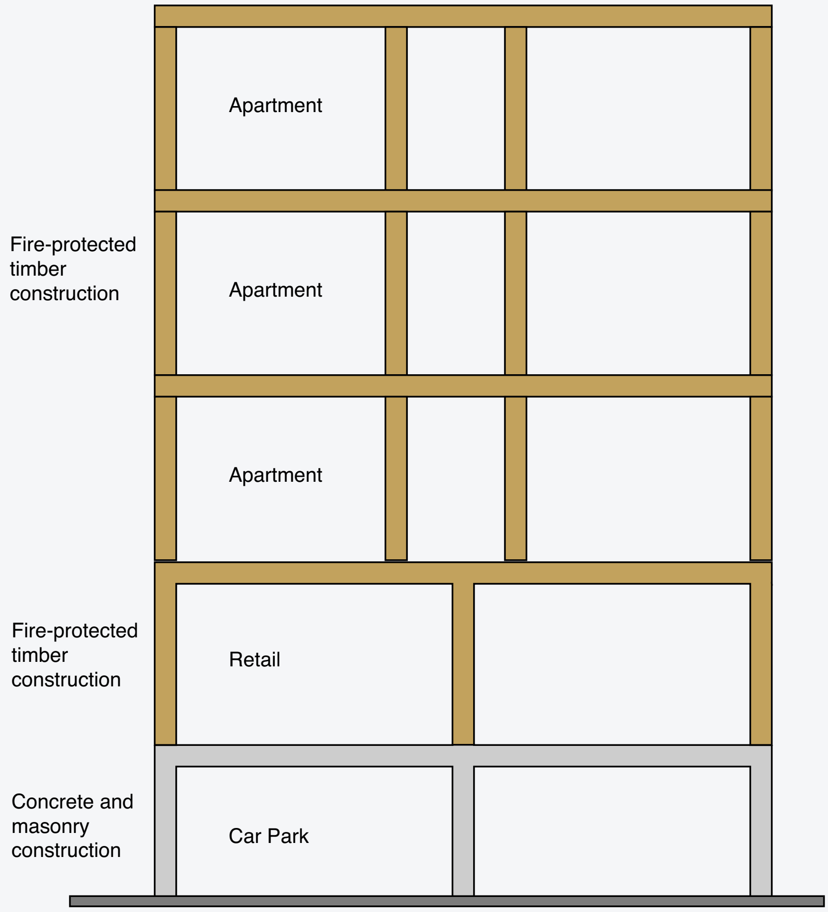


Figure 2.1: Mixed class and mixed forms of construction.

2.5 Resources for Performance Solution Options

In addition to this Guide, the following WoodSolutions Technical Design Guides may assist designers and authorities considering timber performance solutions:

#16 Massive Timber Construction Systems: Cross-laminated Timber (CLT) – introduces the use of CLT in construction, outlining the history, environmental performance and mechanical properties. Also provides an overview of CLT building systems as well as fire, acoustic, seismic and thermal performance.

#17 Alternative Solution Fire Compliance, Timber Structures – provides information about using alternative solutions to allow the use of timber in structural applications not covered by the Deemed-to-Satisfy Provisions of the NCC. It includes a case study of a five storey residential apartment (Class 2) building.

#18 Alternative Solution Fire Compliance, Façades – provides information about using timber façades not covered by the Deem-to-Satisfy Provisions of the NCC. It includes a case study on the use of combustible façades.

#19 Alternative Solution Fire Compliance, Internal Linings – provides information about using timber linings not covered by the Deemed-to-Satisfy Provisions of the NCC. It includes a case study on the use timber linings in a school building corridor.

2.6 Evidence of Suitability

Evidence of suitability for fire resistance and resistance to the incipient spread of fire should be a report from a NATA registered laboratory as prescribed in the NCC

The NCC requires every part of a building to be constructed in appropriate manner to achieve the requirements, using materials and construction methods that are fit for their intended purpose, including the allowance of safe access for maintenance.

The NCC Volume One specifies requirements for Evidence of Suitability in Clause A2.2 but there are the following additional specific requirements that apply to certain aspects of fire safety under NCC Deemed-to-Satisfy requirements:

- NCC Clause A2.3 – Fire-resistance of building elements
- NCC Clause A2.4 – Fire hazard properties
- NCC Clause A2.5 – Resistance to incipient spread of fire.

In most instances, the Evidence of Suitability for the fire resistance or resistance to the incipient spread of fire of an element of construction will be a report from a NATA registered test laboratory presenting the information required by the NCC.

If a performance solution is proposed, a fire safety engineering report should be prepared by a Registered Fire Engineer (note registration requirements vary between the States and Territories). The report should be prepared in accordance with the International Fire Engineering Guidelines and submitted to the relevant regulatory authorities. In many States and Territories additional qualifications/registration is required for Building Surveyors and Certifiers assessing performance solutions.

B

Part B - Fire Safety Design

Part B of the Guide addresses the fire safety design of mid-rise timber buildings.

Section 3 provides an overview of the responsibilities of designers, builders and other practitioners for the safe design throughout a building's life cycle with an emphasis on fire.

Section 4 outlines the NCC Deemed-to-Satisfy Provisions for mid-rise timber buildings and provides some design options for consideration.

Section 5 highlights commonly raised options for performance solutions under the NCC and some of the key issues for consideration when determining compliance with the NCC.

3

Safe Design

3.1 Building Life Cycle

A typical building life cycle is shown in Figure 3.1.

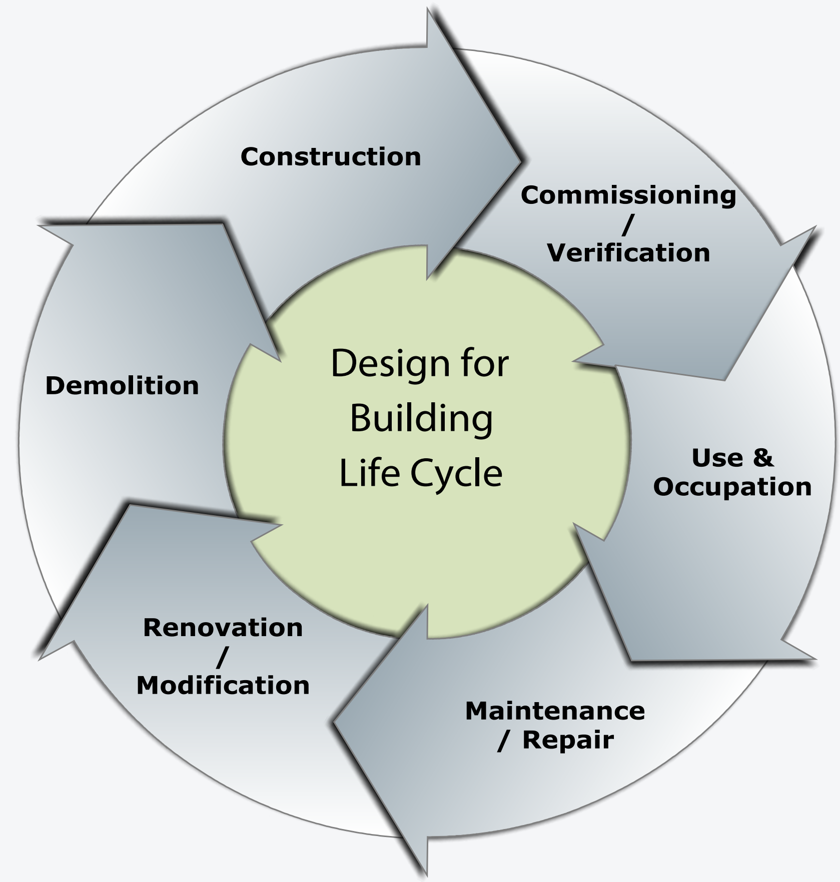


Figure 3.1: Typical building life cycle.

It is important that the impacts of design decisions on all phases of the building's life cycle are considered.

For example, the NCC Deemed-to-Satisfy Provisions may require a particular fire safety feature to be incorporated into a building but, during the design process, it is necessary to determine:

- how the provision can be installed/constructed safely to achieve its required performance
- how the feature will be commissioned and its performance verified
- that the feature will not present a hazard during occupation of a building
- how the feature can be maintained and repaired safely
- measures to be taken to ensure the feature does not present a hazard during renovation/modification or demolition and to ensure that the performance of the feature is not compromised during the renovation/modification process.

Many of these matters lie outside the scope of the NCC but they are addressed through State and Territory Building Acts and Regulations and Workplace Health and Safety Legislation.

3.2 Responsibilities for Safe Design

For further details on how to address WHS requirements refer to Code of Practice; Safe Design and Structures: published by Safe Work Australia

While this Guide focuses on the 2016 changes to the NCC relating to Deemed-to-Satisfy solutions for mid-rise timber buildings, it should be noted that the NCC provides a uniform set of technical provisions for the design and construction of buildings and other structures throughout Australia. The NCC does not regulate matters such as the roles and responsibilities of building practitioners and maintenance of fire safety measures that fall under the jurisdiction of the States and Territories.

State and Territory Building legislation is not consistent in relation to these matters. There are significant variations with respect to:

- registration of practitioners
- mandatory requirements for inspections during construction
- requirements for maintenance of fire safety measures.

In addition to the relevant Building Regulations, Workplace Health and Safety (WHS) Legislation is also applicable, which requires safe design principles to be applied. A Code of Practice on the safe design of structures has been published by Safe Work Australia[3], which provides guidance to persons conducting a business or undertaking who design structures that will be used, or could reasonably be expected to be used, as a workplace. It is prudent to apply these requirements generally to Class 2 buildings as well as Class 3 and 5 buildings, since they represent a workplace for people undertaking building work, maintenance, inspections and the like.

The Code defines Safe Design as: "the integration of control measures early in the design process to eliminate or, if this is not reasonable practicable, minimise risks to health and safety throughout the life of the structure being designed".

It indicates that Safe Design begins at the start of the design process when making decisions about:

- the design and its intended purpose
- materials to be used
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal
- the legislation, codes of practice and standards that need to be considered and complied with.

The Code also provides clear guidance on who has health and safety duties in relation to the design of structures and lists the following practitioners:

- architects, building designers, engineers, building surveyors, interior designers, landscape architects, town planners and all other design practitioners contributing to, or having overall responsibility for, any part of the design
- building service designers, engineering firms or others designing services that are part of the structure such as ventilation, electrical systems and permanent fire extinguisher installations
- contractors carrying out design work as part of their contribution to a project (for example, an engineering contractor providing design, procurement and construction management services)
- temporary works engineers, including those designing formwork, falsework, scaffolding and sheet piling
- persons who specify how structural alteration, demolition or dismantling work is to be carried out.

In addition, WHS legislation places the primary responsibility for safety during the construction phase on the builder.

From the above, it is clear that the design team in conjunction with the owner/operator and builder have a responsibility to document designs, and specify and implement procedures that will minimise risks to health and safety throughout the life of the structure being designed.

WoodSolutions Technical Design Guide # 20: Fire Precautions During Construction of Large Buildings provides further guidance

WoodSolutions Technical Design Guide #37: Mid-rise Timber Buildings provides typical details that can assist in the application of Safe Design principles

Refer to NCC Volume One Cl E1.9 for NCC precautions during construction

3.3 Applying Safe Design Principles

A key element of Safe Design is consultation to identify risks and practical mitigation measures and to assign responsibilities to individuals/organisations for ensuring the mitigation measures are satisfactorily implemented.

This approach should be undertaken whichever NCC compliance pathway is adopted and applies to all forms of construction.

Some matters specific to fire safety are summarised below, but this list is not extensive:

- The NCC and associated referenced documents represent nationally recognised standards for fire safety for new building works.
- The NCC's treatment of fire precautions during construction is limited and focuses on manual fire-fighting, egress provisions and fire brigade fire-fighting facilities. Additional precautions are required to address WHS requirements such as fire prevention and security. See Section 3.4 and WoodSolutions Technical Design Guide #20: Fire Precautions during Construction of Large Buildings for further information.
- Minimise service penetrations through fire-resistant construction. Further information providing design options is provided in Section 4.9 and WoodSolutions Technical Design Guide #37: *Mid-rise Timber Buildings*.
- Group service penetrations through fire-resisting walls with safe access for installation, inspection and maintenance.
- Develop a detailed design of fire safety measures to optimise reliability and facilitate safe installation, maintenance and inspection where practicable. Special attention should be given to protection of service penetrations and cavity barriers.
- Document procedures and allocate responsibilities for determining Evidence of Suitability for fire safety measures.
- Document procedures and allocate responsibilities for the verification and commissioning of all fire safety installations.
- Provide specifications and drawings of all fire safety measures within the building, Evidence of Suitability, commissioning results and requirements for maintenance and inspection to the owner as part of the fire safety manual. (Note: Some State and Territory legislation contains minimum requirements for inspection of fire safety measures.)
- Include information on how to avoid compromising fire safety through the life of a building (e.g. preventing disconnection of smoke detectors or damage to fire-resisting construction) in the fire safety manual.

3.4 Fire Precautions during Construction

Fires may occur on building construction sites due to the nature of the works.

Typical causes include:

- hot works (cutting and welding)
- heating equipment
- smoking materials
- other accidental fires
- arson.

Mid-rise timber buildings complying with the NCC 2016 edition Deemed-to-Satisfy Provisions offer a safe and economical building option. The addition of the fire-protective coverings plays an important role in providing this fire safety and, due to the construction sequencing, there may be a period where the timber is not fully protected and/or automatic fire sprinkler protection is not fully operational. During this period, timber buildings are at their highest risk from construction fires.

The builder and design team need to consider fire precautions during construction. The scope of the NCC is limited to specifying minimum requirements for fire hydrants, hose reels and extinguishers and egress provisions (NCC Clause E1.9).

Consider fire safety during construction throughout the design process

As identified above, it is necessary to address workplace health and safety issues and a broad holistic approach needs to be adopted that considers the building layout and site layout throughout the construction process to minimise the fire risk at a time when the building could be at its most vulnerable.

Typical matters that should be considered include:

- progressive installation of services
- progressive installation of fire-protective grade lining of timber members and compartmentation of the building
- prefabrication and delivery to site with full or partial fire-protective grade lining of timber building elements
- access for fire fighters and egress provisions for staff and visitors on the building site
- selection of materials and work methods that minimise the need for hot works
- security provisions (to address arson)
- access for fire fighters and egress provisions for staff and visitors on the building site
- safe access for maintenance of equipment and minimising the down time of fire safety equipment during maintenance
- detailing service penetration and construction interfaces to minimise the risk of cavity fires during installation.

WoodSolutions Technical Design Guide #20: *Fire Precautions During Construction of Large Buildings* provides additional information that can be applied to the design and planning stages as well as the actual construction phase.

WoodSolutions Technical Design Guide #37: *Mid-rise Timber Buildings* provides additional information relating to good practice design of service penetration systems and other relevant features of mid-rise buildings

4

NCC Deemed-to-Satisfy Solutions

4.1 Overview of the Deemed-to-satisfy Solutions for Mid-rise Timber Buildings

The NCC 2016 introduced Deemed-to-Satisfy Provisions for the construction of mid-rise timber residential and office buildings. An overview of these changes is shown in Figure 4.1.

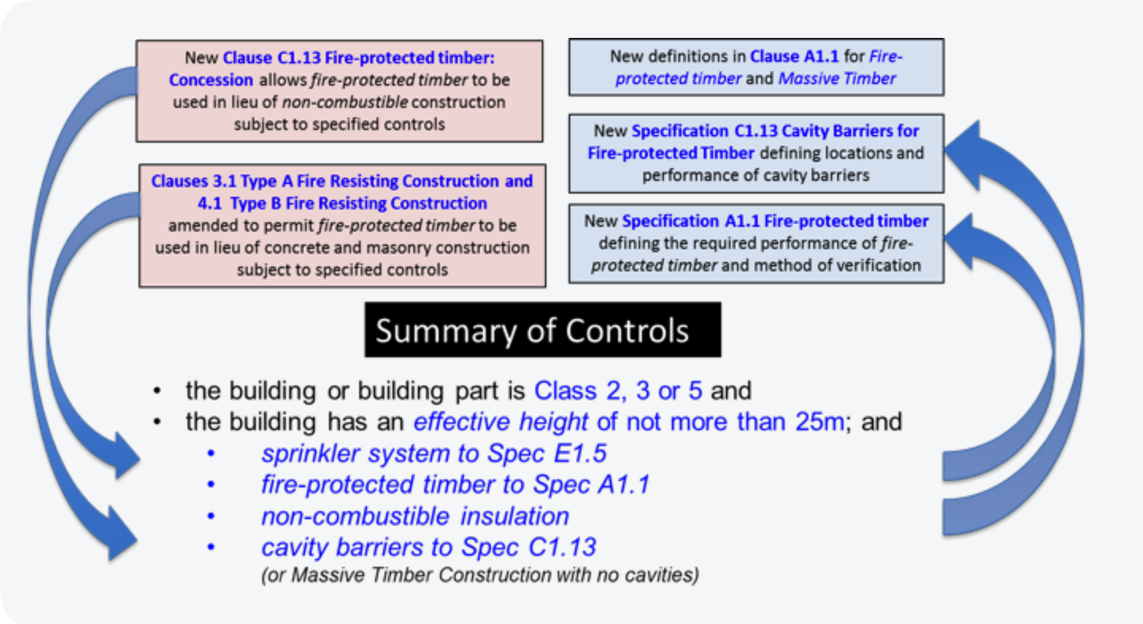


Figure 4.1: Introduction of mid-rise timber provisions to the NCC 2016.

The main features of the mid-rise timber building Deemed-to-Satisfy solutions are:

- The building or building part is of Class 2, 3 or 5.
- Fire-protected timber complying with Specification A1.1 of the NCC is used for loadbearing internal walls, loadbearing fire walls and for elements of construction required to be non-combustible.
- The building has an effective height of not more than 25 m.
- The building has a sprinkler system complying with Specification E1.5 of the NCC throughout.
- Any insulation installed in the cavity of the timber building element required to have an FRL is non-combustible.
- Cavity barriers are provided in accordance with Specification C1.13. of the NCC.

The key objectives of some of the above fire safety precautions that have been adopted to provide a robust building solution are:

Automatic sprinkler suppression system
Objective: To suppress a fire before the structure is threatened and greatly reduce the risk to people and property.

Fire-protected Timber (NCC prescribes FRLs AND non-combustible fire-protective coverings)
Objective: To prevent or delay ignition of the timber structural member so that the response to an enclosure fire will be similar to that for a building constructed on non-combustible elements such as masonry or concrete during the growth period. The fire-protected timber element is still required to achieve the Deemed-to-Satisfy FRLs specified in the NCC.

Cavity Barriers
Objective: To prevent the uncontrolled spread of fire through cavities in the low probability event of either failure of the fire-protective covering or fire start within the cavity.

Further details for the application of the DTS solutions for mid-rise buildings are provided in WoodSolutions Technical Design Guide #37

Non-Combustible Insulation

Objective: To minimise the risk of fire spread through cavities by removing a major fuel source (i.e. combustible insulating materials).

This section provides basic information on the application of the NCC Deemed-to-Satisfy Provisions relating to mid-rise fire-protected timber buildings introduced in the 2016 edition. Further information including typical details are provided in WoodSolutions Technical Design Guide #37 *Mid-rise Timber Buildings* and National Construction Code Volume One 2016.

4.2 Automatic Fire Sprinklers

A key fire safety feature for mid-rise timber buildings is the requirement to provide automatic fire sprinkler systems in accordance with NCC Specification E1.5 throughout the building, including any parts of the building that are not of timber construction. This requirement in conjunction with other fire safety measures is considered to reduce the risk from fires in mid-rise timber buildings below that in other forms of construction complying with the minimum NCC requirements.

4.2.1 Sprinkler Design Standards Permitted by NCC Specification E1.5

Specification E 1.5 allows sprinkler systems to be designed in accordance with

- AS 2118.1:1999 Automatic Fire Sprinkler Systems – General Requirements
- AS 2118.4:2012 Automatic fire sprinkler systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height
- AS 2118.6:2012 Combined sprinkler and hydrant systems in multi-storey buildings.

The scope of AS 2118.4 excludes offices and is limited to accommodation (residential) buildings not exceeding four stories in height. Therefore most mid-rise timber building sprinkler systems will be designed to comply with AS 2118.1 or AS 2118.6.

4.2.2 Designing Fire Sprinkler systems to improve their effectiveness

There are opportunities during the design process to incorporate features that can enhance the effectiveness of an automatic sprinkler system and simplify ongoing maintenance. A few examples of matters for consideration are:

Residential Heads in Residential SOUs and associated corridors

Both AS 2118.1 and 2118.6 allow the use of appropriately listed residential heads in residential building SOUs and associated corridor areas. Residential heads have a more rapid response than standard heads and are more likely to supress rather than control a fire, thus reducing the risk to occupants within the SOU of fire origin. Therefore, residential heads should be specified where appropriate.

Monitored Valves

The reliability of fire sprinkler systems can be enhanced by the provision of monitored components such as main stop valves and subsidiary stop valves. While the NCC provides some requirements for monitored valves, the effectiveness of sprinkler systems can be enhanced by, for example, the specification of monitored stop valves on each floor. This enables sprinkler protection to be maintained throughout the remainder of the building while work is undertaken on part of the sprinkler system and if the valve is left closed upon completion of the work the building owner/operator can be alerted to ensure the error is corrected quickly. Thus the time periods and extent of areas where sprinkler protection is unavailable are minimised. The progressive installation of monitored valves during construction can be used as part of the strategy to address fires during construction by facilitating the progressive commissioning of the sprinkler system.

False Ceilings

If sprinkler pipes are run above a ceiling system that is required to achieve a resistance to the incipient spread of fire (RISF), the ceiling may need to be penetrated to accommodate sprinkler heads, potentially compromising the performance of the ceiling if the sprinkler system fails to operate successfully.

This can be avoided by providing a false ceiling and running the pipes below the RISF ceiling, and the penetrations for the sprinkler heads need only penetrate the non-fire-resisting false ceiling.

This detail also provides flexibility for the installation of lighting systems and other services.

Selection of materials and pipe connections

The use of CPVC piping for sprinkler systems can reduce hot works but, if the pipework needs to be modified, the system may be unavailable; potentially overnight while the adhesive cures. Another option may be the use of mechanical joiners, avoiding the need for hot works and glued connections if components need replacing or modifying.

Protection of voids / concealed spaces

Concealed spaces within fire-protected timber elements greater than 200 mm deep generally require protection in accordance with AS 2118.1 and AS 2118.6. Where these voids include elements such as beams, the void depth is measured from the soffit of the beam.

Where open web beams (trusses) or similar elements are included in the cavity, consideration may be given to providing protection where the distance between a ceiling and the bottom chord is less than 200 mm, since open webs will not obstruct the sprinkler discharge to the same extent as solid beams.

Refer to NCC Spec A1.1 for Fire-protected Timber

4.3 Fire Protected Timber Requirements

The NCC defines fire-protected timber as fire-resisting timber building elements that comply with Specification A1.1.

4.3.1 Fire-Protected Timber – General Requirements

Refer to NCC Spec A2.3 for FRL

Refer to NCC Spec A2.5 for RISF

Refer to NCC Spec A2.2 for non-combustibility

Specification A1.1 applies the following general requirements to fire-protected timber:

- The building element must be protected to achieve the required FRL and have a non-combustible fire-protective covering applied to the timber that achieves a resistance to the incipient spread of fire (RISF) of not less than 45 minutes when tested in accordance with AS1530.4.

Therefore, to adequately specify or check Evidence of Suitability of a fire-protected timber element, three items of information are required:

- Fire resistance level – FRL (determined from AS 1530.4 test or an equivalent or more severe test)
- Resistance to the incipient spread of fire – (RISF) FRL (determined from AS 1530.4 test or an equivalent or more severe test)
- Results from a non-combustibility test in accordance with AS 1530.1 (for materials not deemed non-combustible by the NCC).

Fire Resistance Level (FRL) is the grading period in minutes for the following three criteria expressed in the order listed below separated by forward slashes.

- Structural adequacy – ability of a loadbearing element to support an applied load
- Integrity – ability of an element of construction to resist the passage of flames and hot gases from one space to another
- Insulation – ability of the surface of an element of construction not exposed to the furnace to maintain a temperature below the specified limits.

For example, if an FRL of 90/60/30 is specified, the element would need to satisfy the structural adequacy criteria for 90 minutes, the integrity criteria for 60 minutes and the insulation criteria for 30 minutes. A dash means that there is no requirement for that criterion, i.e. an FRL of 90/-/- means that only the criterion of structural adequacy applies for 90 minutes.

NCC Spec A1.1 includes some Deemed-to-Satisfy fire-protective covering systems based on fire-protective-grade plasterboard

The Resistance to the Incipient Spread of Fire (RISF) in relation to a fire-protective covering means the ability of the covering to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits

A material is classified as non-combustible if flaming is not observed and specified temperature rise limits are not exceeded when a sample of material is exposed to the heating conditions specified in AS 1530.1.

To facilitate a consistent approach to specifying the required performance of fire-protected timber, the following format of notation is recommended: Fire-Protected Timber –

FRL90/90/90: RISF45: NC.

This means that the element must satisfy the structural adequacy, integrity and insulation requirements for 90 minutes; the resistance to the incipient spread of fire criteria for 45 minutes; and the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or be deemed by the NCC to be non-combustible.

While individual test/assessment reports from NATA-registered testing authorities can be used as Evidence of Suitability, it may be more practical for registered testing authorities to provide consolidated reports stating the performance in the above format.

Further information relating to the test procedures to determine the Fire Resistance and Resistance to the Incipient Spread of Fire are provided in Appendix E.

Cavities are permitted within fire-protected timber elements that, without adequate measures in place, can allow fire spread through concealed spaces. The risk of fire spread from enclosure fires to the cavities is substantially reduced by the requirement for an RISF45 applied to the fire-protective covering, among other things, but there is a small residual risk of fire spread to the cavity from an enclosure fire or a fire start within a cavity due to hot works, for example. The risk of fire spread via concealed spaces – should this low probability event occur – is further reduced by the Provisions for cavity barriers and requirements for cavity insulation, if present, to be non-combustible.

Specification A1.1 deems 2 layers of 13 mm fire-protective-grade plasterboard fixed in accordance with the requirements to achieve the required FRL of the element to achieve equivalent performance to an RISF45: NC fire-protective covering.

Thus the timber-framed wall system shown in Figure 4.2 with two layers of 13 mm fire-protective plasterboard either side of a cavity between studs could be classified as Fire-Protected Timber – FRL90/90/90: RISF45: NC; if the loadbearing wall system had achieved an FRL of 90/90/90 under similar or more severe load conditions in an AS 1530.4 fire test, since two layers of 13 mm fire-grade plasterboard are deemed to achieve an RISF45 and plasterboard is deemed NC by the NCC.

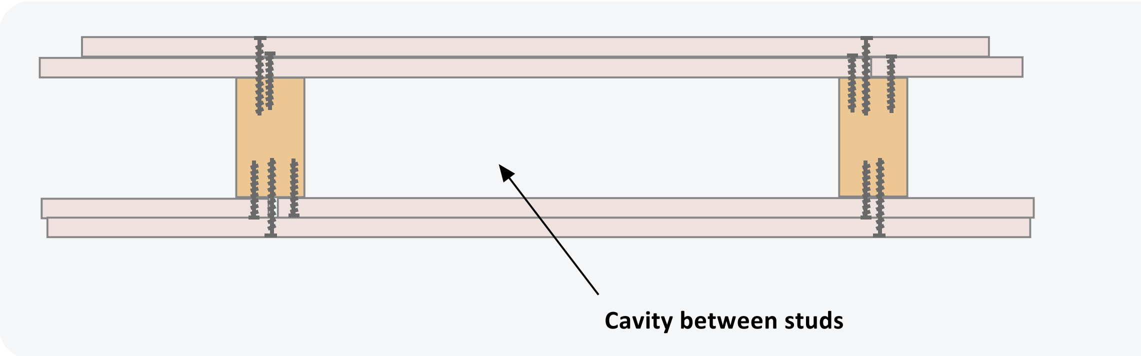


Figure 4.2: Horizontal section through typical FRL90/90/90:RISF45: NC timber stud wall.

The primary objective for the inclusion of the non-combustibility requirement for the fire-protective covering is so that the reaction to fire performance of the fire-protected timber to external and enclosure fires is comparable to elements of construction that are non-combustible; such as reinforced concrete or steel protected with non-combustible materials.

The primary objective for the specification of RISF45 is to reduce the risk of the timber structural elements being ignited prior to burn-out of the contents or fire brigade intervention, in the unlikely event of failure of the automatic fire sprinkler system. To achieve this objective, it is necessary for the RISF performance not to be compromised by the presence of building service penetrations and openings for doors and windows. See Section 4.9.3 and Appendix E for further details on how the RISF performance can be maintained though appropriate penetration fire stopping systems, cavity barriers and lining of openings.

4.3.2 Massive Timber

The NCC permits a 'relaxation' of the general requirements for fire-protected timber, provided both the following additional criteria are satisfied:

- The minimum timber thickness of timber panels is not less than 75 mm.
- There are no cavities between the surface of the timber and the fire-protective covering system.

The 75 mm dimension relates to the minimum dimension of the dressed or finished timber member. In most instances, massive timber elements will have minimum thicknesses much greater than 75 mm to meet the structural adequacy and integrity criteria of AS 1530.4.

Typical examples of massive timber panel installations satisfying the conditions for this provision to apply are shown in Figure 4.3.

The reasons for modifying the fire-protected timber requirements for massive timber are:

1. Timber members having a large cross-section can achieve high fire resistance levels due to the formation of a char that protects the timber core, allowing it to continue to support an imposed load or maintain a fire separating function for significant periods. Therefore if there is an early failure of the fire-protective covering, the timber structure is likely to maintain its loadbearing capacity for a greater period than light-weight construction.
2. By not permitting any concealed spaces between the timber members or between the timber and fire-protective coverings, the risk of fire spread through concealed cavities is addressed.

If the massive timber conditions are satisfied, the following requirements can be adopted for fire-protected timber in lieu of the general requirements:

- The building element must be protected to achieve the required FRL.
- The building element must have a non-combustible fire-protective covering applied to the timber that achieves the modified resistance to the incipient spread of fire (MRISF) of not less than the values stated in Table 4.1, when tested in accordance with AS1530.4.

The modified resistance to spread of fire is determined in accordance with Clause 3 of NCC Specification A1.1. Further information relating to the test procedures to determine the Fire Resistance and Modified Resistance to the Incipient Spread of Fire are provided in Appendix E.

To facilitate a consistent approach to specifying the required performance of fire-protected timber, the following format of notation is recommended:

Fire-Protected Timber - FRL 90/90/90: MRISF 30: NC.

This means that the element must satisfy the structural adequacy, integrity and insulation requirements for 90 minutes; the modified resistance to the incipient spread of fire criteria for 30 minutes; and the fire-protective covering must have been shown to be non-combustible when tested in accordance with AS 1530.1 or be deemed by the NCC to be non-combustible.

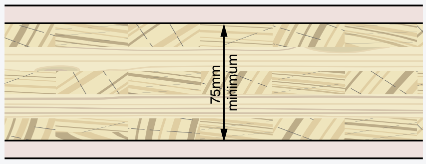
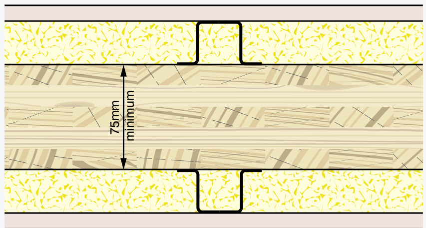
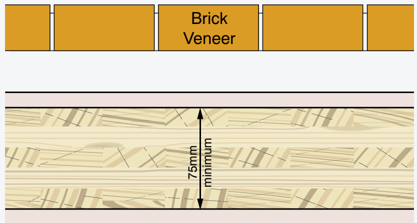
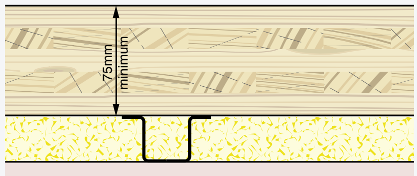
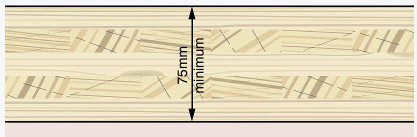
Description	Schematic section
Massive Timber Wall Panels	
Direct fix to massive timber panel.	
Fire-protective covering fixed to solid timber battens with non- combustible insulation - both sides of a wall panel.	
External Brick veneer wall- Note massive timber is faced on both sides with fire protective coverings allowing an unobstructed cavity	
Massive Timber Floor Panels	
Fire-protective covering fixed to furring channels om the underside of a floor panel with non- combustible insulation.	
Fire-protective covering direct fix to massive timber panel.	

Figure 4.3: Massive timber details qualifying under the massive timber provisions.

Table 4.1: Fire-protective covering requirements – massive timber.

Application	Modified Resistance to the Incipient Spread of Fire (MRISF)	Minimum Deemed-to-Satisfy Fire-protective Grade Plasterboard
Inside a fire-isolated stairway or lift shaft	20 min	1 layer x 13mm thick
External walls within 1 m of an allotment boundary or 2 m of a building on the same allotment	45 min	2 layers x 13mm thick
All other applications	30 min	1 layer x 16mm thick

Table 4.1 also includes Deemed-to-Satisfy fire-protective grade plasterboard minimum requirements if fixed in accordance with the requirements to achieve the required FRL of the element for massive timber.

For example, if a non-loadbearing wall system is required to achieve an FRL of -/60/60, an appropriate specification for an element using the massive timber provisions would be:

Fire-Protected Timber FRL -/60/60: MRISF 30: NC

If there is appropriate Evidence of Suitability to show a massive timber element can achieve an FRL of -/60/60 when protected by 16 mm fire-protective plasterboard, then no further evidence is required, since the 16 mm thick plasterboard is Deemed-to-Satisfy the MRISF 30 requirement and the plasterboard is also deemed to be non-combustible.

4.3.3 Fire-protected Timber Element Requirements for Mid-Rise Class 2 or 3 Buildings of Timber Construction (General Requirements)

Mid-rise Class 2 and 3 (residential buildings) are typically more than 3 storeys high and are therefore required to be of Type A construction by NCC Volume One. On this assumption, the fire-protected timber requirements for various wall, floor, ceiling and other building elements are given in Table 4.2 for the typical mid-rise timber apartment building shown schematically in Figure 4.4 and Figure 4.5.

The requirements for external walls are given in Section 4.6.

Refer to NCC Volume One Specification C1.1 for required FRLs and Specification A1.1. for RISF requirements

Table 4.2: FRL and RISF general requirements for timber-framed mid-rise apartment buildings.

Symbol	Description	FRL – Structural Adequacy /Integrity/ Insulation - min		Resistance to the Incipient Spread of Fire (min.)
		Loadbearing	Non–loadbearing	
	Fire stair shaft	90/90/90	-/90/90	45
	Service shaft	90/90/90	-/90/90	45
	Bounding Sole Occupancy Units	90/90/90	-/60/60	45
	Lift shaft walls	90/90/90	-/90/90	45
	Door to fire stair	Not applicable	-/60/30	Not applicable
	Fire door to service shaft	Not applicable	-/60/30	Not applicable
	Door to SOU	Not applicable	-/60/30	Not applicable
	Lift door	Not applicable	-/60/-	Not applicable
	Doors to services risers	Not applicable	-/60/30	Not applicable
	Non-loadbearing walls within an apartment	Not applicable	-/-/-	-
	Floors	90/90/90	Not applicable	45

Refer Specification C1.1 of NCC Volume One Cl 3.5 for the roof concession

Note: Since the roof will have a non-combustible covering and mid-rise timber buildings are required to be sprinkler protected throughout, the roof is not required to achieve an FRL.

In addition to the above requirements, the fire-protective coverings must also be non-combustible.

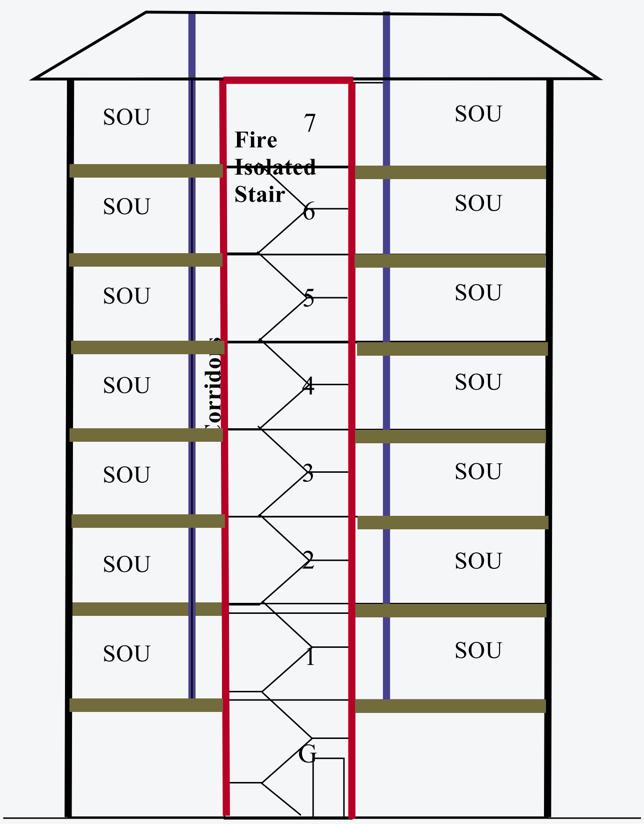


Figure 4.4: Typical section through a mid-rise apartment building.

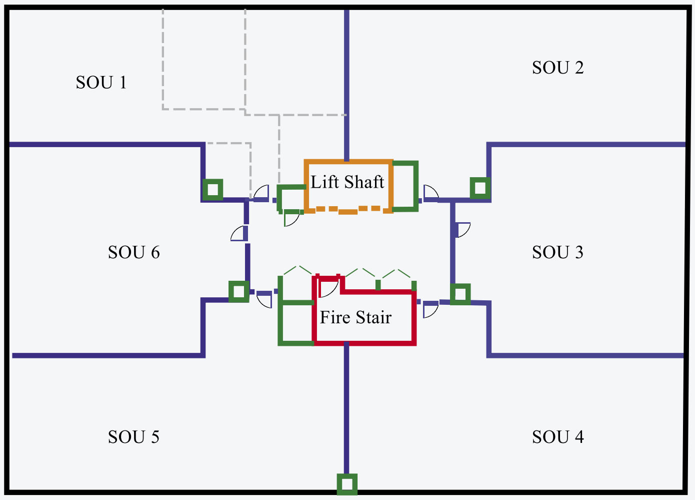


Figure 4.5: Plan of a typical apartment building floor.

4.3.4 Fire-protected Timber Element Requirements for Mid-rise Class 2 or 3 Buildings of Timber Construction for Massive Timber

Refer to NCC Volume One Specification C1.1 for required FRLs and Specification A1.1. for MRISF requirements

The massive timber provisions can only be applied if both the minimum member size and cavity restrictions are satisfied. If these conditions are not fully satisfied for an element then the general requirements must be applied as summarised in Section 4.3.3 above.

A typical mid-rise timber apartment building layout is shown in Figure 4.4 and Figure 4.5 with fire-protected timber elements. The FRLs and MRISF requirements for these elements are summarised in Table 4.3 for applications where the massive timber provisions can be applied. For external walls see Section 4.6.

Table 4.3: FRL and MRISF Requirements for timber mid-rise apartment buildings if the massive timber provision is applicable.

Symbol	Description	FRL – Structural Adequacy /Integrity/ Insulation – min		Modified Resistance to Incipient Fire Spread – min
		Loadbearing	Non-loadbearing	
	Fire stair shaft	90/90/90	-/90/90	30 outside 20 inside
	Service shaft	90/90/90	-/90/90	30
	Bounding Sole Occupancy Units	90/90/90	-/60/60	30
	Lift shaft walls	90/90/90	-/90/90	30
	Door to fire stair	Not applicable	-/60/30	30 outside 20 inside
	Fire door to service shaft	Not applicable	-/60/30	Not applicable
	Door to SOU	Not applicable	-/60/30	Not applicable
	Lift door	Not applicable	-/60/-	Not applicable
	Doors to services risers	Not applicable	-/60/30	Not applicable
	Non-loadbearing walls within an apartment	Not applicable	-/-/-	-
	Floors	90/90/90	Not applicable	30

Note: Since the roof will have a non-combustible covering and mid-rise timber buildings are required to be sprinkler protected throughout, the roof is not required to achieve an FRL.

In addition to the above requirements the fire-protective coverings must also be non-combustible

Refer to Specification C1.1 of NCC Volume One Cl 3.5 for the roof concession

4.3.5 Fire-protected Timber Element Requirements for Mid-rise Class 5 Buildings of Timber Construction (General Requirements)

Mid-rise Class 5 (office buildings) are typically 3 or more storeys high. Three-storey office buildings are generally of Type B construction and those greater than three storeys of Type A construction.

The element requirements for timber-framed mid-rise Class 5 buildings are given in Table 4.4 for Types A and B construction.

The requirements for external walls are given in Section 4.6.

Table 4.4: FRL and RISF general requirements for timber-framed mid-rise office buildings.

Description	FRL – Structural Adequacy /Integrity/Insulation – min				Resistance to the Incipient Spread of Fire (min)
	Type A Construction		Type B Construction		
	Loadbearing	Non-Loadbearing	Loadbearing	Non-Loadbearing	
Common walls and Fire walls	120/120/120	Not applicable	120/120/120	Not applicable	45
Fire stair shaft	120/120/120	-/120/120	120/120/120	-/120/120	45
Service Shaft	120/90/90	-/90/90	Not applicable	Not applicable	45
Bounding walls – SOUs, public corridors etc	120/-/-	-/-/-	120/-/-	-/-/-	45
Lift Shaft walls	120/120/120	-/120/120	120/120/120	-/120/120	45
Door to fire Stair	Not applicable	-/60/30	Not applicable	-/60/30	Not applicable
Fire Door to service shaft	Not applicable	-/60/30	Not applicable	-/60/30	Not applicable
Lift door	Not applicable	-/60/-	Not applicable	-/60/-	Not applicable
Other Loadbearing internal walls, internal beams trusses and columns	120/-/-	Not applicable	120/-/- (Other Loadbearing internal walls and columns only)	Not applicable	45
Floors/Beams	120/120/120	Not applicable	120/-/-[1]	Not applicable	45

Note 1: It has been assumed the floors support loadbearing columns and/or walls and therefore the same FRL, as the part they support, applies.

Refer to NCC Volume One Specification C1.1 for required FRLs and Specification A 1.1. for MRISF requirements

4.3.6 Fire-protected Timber Element Requirements for Mid-rise Class 5 Buildings of Massive Timber Construction

The massive timber provisions can only be applied if both the minimum member size and cavity restrictions are satisfied. If these conditions are not fully satisfied for an element, then the general requirements must be applied as summarised in Section 4.3.5 above.

The FRLs and MRISF requirements applicable to fire-protected timber elements in office buildings are summarised in Table 4.5 for applications where the massive timber provisions can be applied. For external walls see Section 4.6.

Table 4.5: FRL and MRISF requirements for massive timber mid-rise office buildings

Description	FRL – Structural Adequacy /Integrity/Insulation – min				Modified Resistance to the Incipient Spread of Fire
	Type A Construction		Type B Construction		
	Loadbearing	Non-Loadbearing	Loadbearing	Non-Loadbearing	
Common walls and Fire walls	120/120/120	Not applicable	120/120/120	Not applicable	30
Fire stair shaft	120/120/120	-/120/120	120/120/120	-/120/120	30 outside 20 inside
Service Shaft	120/90/90	-/90/90	Not applicable	Not applicable	30
Bounding walls – SOUs, public corridors etc	120/-/-	-/-/-	120/-/-	-/-/-	30
Lift Shaft walls	120/120/120	-/120/120	120/120/120	-/120/120	30 outside 20 inside
Door to fire Stair	Not applicable	-/60/30	Not applicable	-/60/30	Not applicable
Fire Door to service shaft	Not applicable	-/60/30	Not applicable	-/60/30	Not applicable
Lift door	Not applicable	-/60/-	Not applicable	-/60/-	Not applicable
Other Loadbearing internal walls, beams and trusses	120/-/-	Not applicable	120/-/- (Other loadbearing internal walls and columns only)	Not applicable	30
Floors	120/120/120	Not applicable	120/-/-[1]	Not applicable	30

Note 1: It has been assumed the floors support loadbearing columns and/or walls and therefore the same FRL, as the part they support, applies.

4.4 Cavity Insulation Requirements

Refer to NCC Volume One Specification A1.1. Cl 2.3

If cavity insulation is provided within fire-protected timber elements it is required to be non-combustible. Combustible cavity insulation can facilitate ignition of cavity fires and the rapid spread of fire through cavities.

Typical solutions include mineral fibre or glass wool insulation with very low organic binder contents. It is therefore important to check that Evidence of Ssuitability in the form of a current AS 1530.1 report from a NATA-registered testing authority is available for the specific products selected.

4.5 Cavity Barrier Requirements

Cavity barriers are defined in the NCC as a barrier placed in a concealed space, formed within or around the perimeter of fire-protected timber building elements that complies with Specification C1.13; to limit the spread of fire, smoke and hot gases to other parts of the building.

They are required to be provided by the following clauses as part of a prescribed solution:

- Clause C1.13 Fire-protected timber concession
- Clause 3.1 d (iii) of Specification C1.1
- Clause 4.1 e (iii) of Specification C1.1

The use of fire-protected timber in mid-rise buildings is based on the following principles:

(a) reducing the risk of timber structural elements becoming involved in a fire by the use of fire-protective coverings in conjunction with automatic fire sprinklers, and

(b) in the low probability of fire spreading to cavities/voids, or of a fire developing within a cavity, limiting that spread by cavity barriers in conjunction with other measures such as the use of non-combustible cavity insulation.

The risk of fire spread via cavities and voids in designs that use the massive timber provisions is addressed by prohibiting designs that incorporate cavities and voids.

4.5.1 Determining the Positions of Cavity Barriers

Cavity barriers are required at the following positions:

- around the perimeter of fire-protected timber elements
- junctions between fire-resisting floor/ceiling assemblies and fire-resisting walls
- junctions between fire-resisting floor/ceiling assemblies and fire-resisting external walls
- junctions between fire-resisting walls and external walls
- around the perimeters of door and window openings in fire-resisting construction
- horizontal barriers at each floor level with a maximum distance of 5 m between horizontal cavity barriers
- vertical cavities must be provided in walls at maximum of 10 m centres.

Typical positions of cavity barriers are shown for an apartment building in Figure 4.6 and Figure 4.7. A key describing the types of interface being protected is included in Table 4.6.

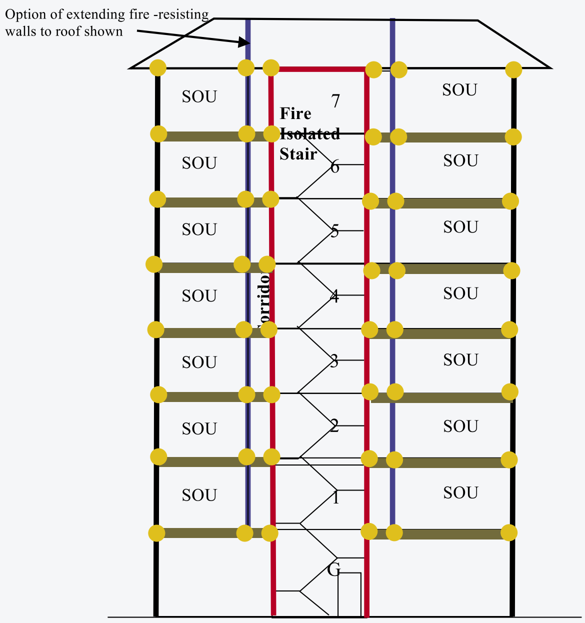


Figure 4.6: Vertical section of an apartment building showing typical cavity barrier positions.

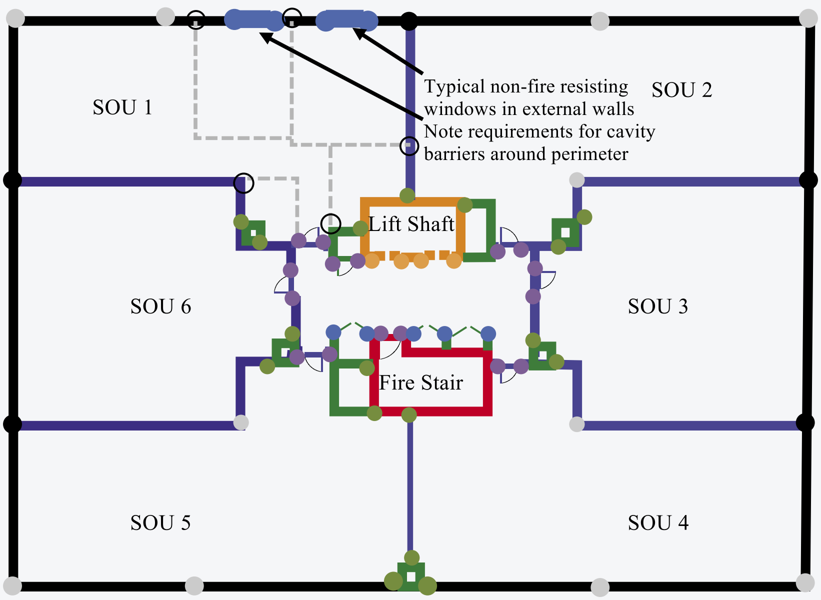


Figure 4.7: Horizontal section of an apartment building showing typical cavity barrier positions.

Table 4.6: Key to cavity barrier positions.

Symbol	Description	Comments
	Horizontal cavity barriers around perimeter of floors	If floor to floor height greater than 5m intermediate horizontal barriers in walls would be required
	Cavity barriers in fire-protected timber walls	Vertical cavity barriers are required at maximum 10m centres
	Cavity barriers around perimeter of non fire resistant doors and windows	Required to prevent entry of fire into cavity when non-fire resistant elements fail
	Interface of fire resistant walls with external walls	Can be incorporated as part of a standard detail
	Interface of shafts with standard walls	Can be incorporated as part of a standard detail
	Interface with fire doors	Normally part of the standard detail for installation since the doorset is required to maintain the fire resistance of the wall
	Interface with lift doors	In some instances it may be more practical to interface with other forms of construction around lift doors
	Interface between non-fire-resisting wall and fire resisting walls	Continuity of the fire-protective coverings should be maintained at the point of penetration

4.5.2 Specifying Cavity Barrier Requirements for Building Elements

Essentially there are two levels of performance required for cavity barriers prescribed by the NCC.

- Cavity barriers with FRLs of -/45/45 for building elements with FRLs up to 90/90/90.
- Cavity barriers with FRLs of -/60/60 for building elements with FRLs greater than 90/90/90 but less than or equal to 120/120/120.

For each of these cases, the NCC prescribes Deemed-to-Satisfy solutions based on minimum thicknesses of timber or mineral fibre in the direction of heat flow as summarised in Table 4.7.

Table 4.7: NCC-prescribed Deemed-to-Satisfy solutions for cavity barriers.

Prescribed solution options	Fire-protected timber FRL	
	90/90/90 or less	>90/90/90 to ≤ 120/120/120
FRL for cavity barrier	-/45/45	-/60/60
Timber – required minimum thickness	45 mm	55 mm
Mineral wool – required minimum thickness	45 mm	60 mm

For fire-protected timber with large cavities, which may occur in floor and roof cavities, for example, it may be more practical to construct cavity barriers from plasterboard supported from timber framing.

Further information relating to the test procedures to determine the Fire Resistance and Modified Resistance to the Incipient Spread of Fire is provided in Appendix E.

4.6 External Walls/Building Façades

In addition to maintaining loadbearing capacity when subjected to fires within a building, the external walls also need to address the risk of fire spread via the building façade under the following scenarios:

- Fire spread from adjacent buildings (or the fire source feature as defined in the NCC) to the subject building. Under the Deemed-to-Satisfy solution pathway for mid-rise timber buildings, this is addressed by means of specification of minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Fire spread from the subject building to adjacent buildings (or the fire source features defined in the NCC). Under the Deemed-to-Satisfy solution pathway for mid-rise timber buildings, this is addressed by means of specification of minimum separation distances, fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and the provision of automatic fire sprinklers.
- Fire spread from an external fire source adjacent to the façade other than adjacent structures including balcony fires. Under the Deemed-to-Satisfy solution pathway for mid-rise timber buildings, this is addressed by means of specification of fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction.
- Vertical fire spread between openings from a fully developed fire within the subject buildings. Under the Deemed-to-Satisfy solution pathway for mid-rise timber buildings, this is addressed by means of specification of fire-resisting construction and the requirement for external walls to be non-combustible or of fire-protected timber construction and the provision of automatic fire sprinklers.

The measures described above are considered in more detail in the following sections.

4.6.1 Fire-Protected Timber Requirements for External Walls

The FRLs required for external walls are nominated in NCC Specification C1.1 and are dependent on the building use (Class of Building), Type of Construction and proximity to the boundary (fire source feature) or other buildings. Mid-rise residential buildings (Class 2 and 3) are required to be of Type A construction and, while most mid-rise office buildings are also required to be of Type A construction, Type B construction is permitted for 3-storey mid-rise office buildings.

The resistance to the incipient spread of fire (RISF) or, if the massive timber provision is applicable – the modified resistance to the incipient spread of fire (MRISF), requirements are nominated in NCC Specification A1.1.

The requirements for Class 2, 3 and 5 buildings of Type A construction are summarised in Table 4.8 and Table 4.9.

Table 4.8: FRL and RISF general requirements for timber-framed mid-rise residential and office building external walls (Type A construction).

Distance from Fire Source Feature	FRL - Structural Adequacy/Integrity/Insulation – min				Resistance to the incipient fire spread – min
	Class 2 and 3 (Residential)		Class 5 (Office)		
	Loadbearing	Non-loadbearing	Loadbearing	Non-loadbearing	
≤1.0 m	90/90/90	-/90/90	120/120/120	-/120/120	45
<1.5 m	90/90/90	-/90/90	120/120/120	-/120/120	45
≥1.5 and <3 m	90/60/60	-/60/60	120/90/90	-/90/90	45
≥3 m	90/60/30	-/-/-	120/60/30	-/-/-	45
External Columns	90/-/-	-/-/-	120/-/-	-/-/-	45

It should be noted that even though non-loadbearing external walls do not require an FRL if more than 3 m from a fire source feature, the fire-protective coverings must be applied and are required to achieve a RISF of 45 minutes, since the external wall is required to be non-combustible to address the risk of external fires on balconies or external areas adjacent to the building and the risk of vertical fire spread through openings if a fully developed fire occurs.

Table 4.9: FRL and MRISF requirements for massive timber mid-rise residential and office building external walls (Type A construction).

Distance from Fire Source Feature	FRL - Structural Adequacy/Integrity/Insulation – min				Modified Resistance to theIncipient Fire Spread - min
	Class 2 and 3 (Residential)		Class 5 (Office)		
	Loadbearing	Non-loadbearing	Loadbearing	Non-loadbearing	
≤1.0 m	90/90/90	-/90/90	120/120/120	-/120/120	45 external 30 internal
<1.5 m	90/90/90	-/90/90	120/120/120	-/120/120	30
≥1.5 and <3 m	90/60/60	-/60/60	120/90/90	-/90/90	30
≥3 m	90/60/30	-/-/-	120/60/30	-/-/-	30
External Columns	90/-/-	-/-/-	120/-/-	-/-/-	30

It should also be noted that, even though non-loadbearing external walls do not require an FRL if more than 3 m from a fire-source feature, the fire-protective coverings must be applied and are required to achieve a MRISF of 30 minutes.

For buildings within 1 m of the boundary (or 2 m of an adjacent building on the same allotment) an MRISF of 45 minutes for the external surfaces is required to minimise the risk of ignition from fires in adjacent buildings but the internal face need only achieve a MRISF of 30 minutes.

The required FRLs for external walls of 3-storey office buildings (Type B) construction are less than the requirements for Type A construction at distances greater than 1.5 m from the fire source feature. The general requirements are summarised in Table 4.10 and the requirements where the massive timber provision applies are summarised in Table 4.11.

Table 4.10: FRL and RISF general requirements for timber-framed mid-rise office building external walls (Type B Construction).

Distance from Fire Source Feature	FRL-Structural Adequacy /Integrity/Insulation – min		Resistance to the Incipient Spread of Fire – min
	Loadbearing	Non-loadbearing	
≤1.0 m	120/120/120	-/120/120	45
<1.5 m	120/120/120	-/120/120	45
≥1.5 and <3 m	120/90/60	-/90/60	45
≥3 m and <9 m	120/30/30	-/-/-	45
≥9 m and <18 m	120/30/-	-/-/-	45
≥18 m	-/-/-	-/-/-	45
External Columns <18 m	120/-/-	-/-/-	45
External Columns ≥18 m	-/-/-	-/-/-	45

Table 4.11: FRL and MRISF requirements for massive timber mid-rise office building external walls (Type B construction).

Distance from Fire Source Feature	FRL - Structural Adequacy /Integrity/Insulation - min		Modified Resistance to the Incipient Fire Spread - min
	Loadbearing	Non–loadbearing	
≤1.0 m	120/120/120	-/120/120	45 external 30 internal
<1.5 m	120/120/120	-/120/120	30
≥1.5 and <3 m	120/90/60	-/90/60	30
≥3 m and <9 m	120/30/30	-/-/-	30
≥9 m and <18 m	120/30/-	-/-/-	30
≥18 m	-/-/-	-/-/-	30
External Columns <18 m	120/-/-	-/-/-	30
External Columns ≥18 m	-/-/-	-/-/-	30

Refer to NCC Volume One Cl C2.6

4.6.2 Vertical Separation of Openings in External Walls

The NCC Deemed-to-Satisfy solution for external walls requires vertical separation of openings to be addressed in buildings of Type A construction to reduce the risk of fire spread between floors if a fully developed fire occurs.

This can be achieved by the provision of spandrel panels or horizontal projections but the NCC waives these requirements if an automatic fire sprinkler system is provided in accordance with NCC Spec E1.5. This recognises that early suppression or control of an internal fire by an automatic fire sprinkler system is an effective means of minimising the risk of fire spread between floors via the façade.

The Deemed-to-Satisfy solution for mid-rise timber buildings requires the building to be provided with a sprinkler system complying with E1.5 installed throughout the building and therefore there is no need to provide additional vertical separation by, for example, the provision of spandrel panels. This simplifies construction and provides greater design flexibility.

4.6.3 External Wall/Façade Systems

External walls form the building façade and are required to serve a number of functions by the NCC in addition to addressing fire safety. These include:

- structural performance – for safety and serviceability
- weather resistance – (resistance to water penetration)
- light and ventilation (including condensation control)
- energy efficiency (thermal insulation)
- durability
- acoustic separation.

Refer to NCC Volume One Spec C1.1 Cl 3.1(b) and Cl4.1(b)

The external face of the wall may form part of the fire-protective covering, e.g. brick veneer construction, or may cover a fire-protective covering to prevent water penetration and serve other non-fire-related functions. In both cases, the NCC requires the external walls to be of non-combustible construction and therefore these coverings must be non-combustible.

If the design brief proposes the use of combustible cladding systems, the performance pathway could be adopted subject to it being able to demonstrate compliance of the wall system with the relevant NCC performance requirements.

4.7 Lift Shafts

Some designs of timber buildings adopt a hybrid approach and incorporate concrete or masonry shafts. Where this approach is adopted, it is important for the potential for differential movement between the timber structure and shaft to be taken into account when detailing connections and interfaces.

When designing lift shafts, it is important to involve the lift supplier at an early stage to ensure the shaft will satisfy their design requirements and applicable regulations.

The remainder of this section will address the fire safety performance of lift shafts of fire-protected timber construction with respect to NCC compliance.

4.7.1 Timber-framed Lift Shaft Construction

Table 4.12 has been derived from Section 4.3 to show the NCC requirements that are applicable to timber-framed lift shafts in mid-rise timber buildings.

Table 4.12: Requirements for fire-protected timber-framed lift shafts.

Criteria	Residential Buildings (Class 2 and 3)	Office Buildings
FRL for loadbearing walls	90/90/90	120/120/120
FRL for non-loadbearing walls	-/90/90	-/120/120
RISF for walls	45	45
Lift landing doors	-/60/-	-/60/-

Further details are provided in WoodSolutions Technical Design Guide #37

The wall FRL and RISF requirements are applicable from both within and outside the shaft.

To minimise sound transmission to adjoining areas, double stud construction may be employed and/or an independent lift support structure provided within the shaft.

The fire resistance of lift landing door assemblies should be determined by undertaking fire tests in a representative wall construction type. At the time of preparation of this Guide, few lift landing doors have been tested in timber-framed wall assemblies.

A practical way to address this is to transition the shaft wall construction around the door opening to a form of non-combustible construction, having FRLs that the performance of the lift door has been already verified in.

An example of transitioning to a steel shaft wall system from a fire-protected timber wall shaft is shown in Figure 4.8 and Figure 4.9.

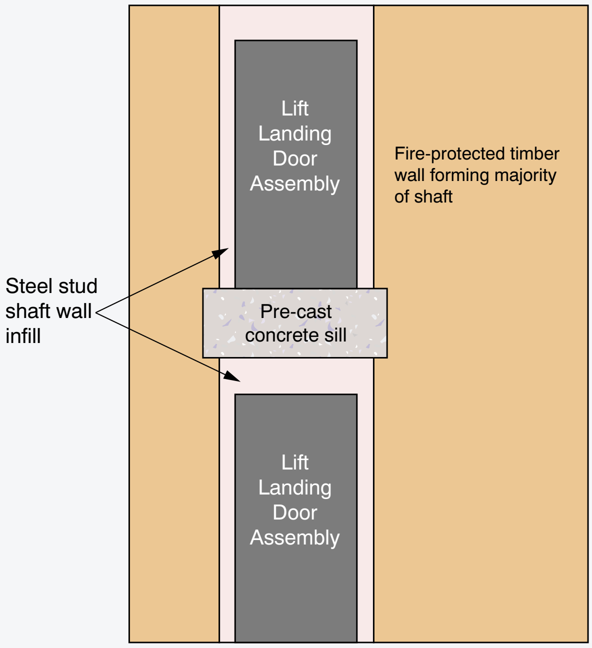


Figure 4.8: Elevation showing wall transition around lift landing doors.

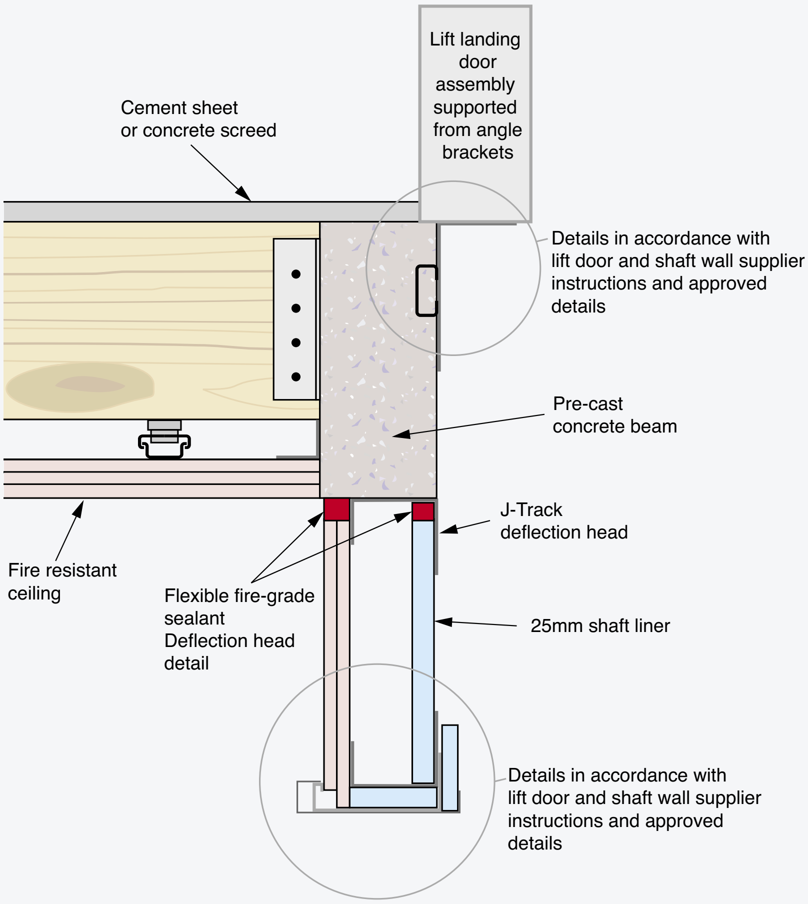


Figure 4.9: Generic detail for sill and head mounting.

4.7.2 Massive Timber Lift Shaft Construction

Table 4.13 has been derived from Section 4.3 to show the NCC requirements that are applicable to timber lift shafts in mid-rise timber buildings if the massive timber provision is applicable.

Table 4.13: Requirements for fire-protected timber lift shafts if the massive timber provision applies.

Criteria	Residential Buildings (Class 2 and 3)	Office Buildings Class 5
FRL for loadbearing walls	90/90/90	120/120/120
FRL for non-loadbearing walls	-/90/90	-/120/120
RISF for walls	30 outside face 20 inner face	30 outside face 20 inner face
Lift landing doors	-/60/-	-/60/-

If the massive timber provision applies, the MRISFs are reduced from 30 minutes to 20 minutes within the lift shaft. This relaxation reflects the lower probabilities of severe fires within these areas but a basic level of protection is retained since, if fires occur within these areas, evacuation paths from the buildings could be quickly compromised due to rapid fire spread in the early stages of a fire. The outer faces still require an MRISF of 30 minutes - refer Figure 4.10.

To minimise sound transmission to adjoining areas, double leaf construction may be employed and/or an independent support structure provided within the shaft. If double leaf construction is employed, the general requirements require the inner and outer faces to achieve a RIFS of 45 minutes. This can be achieved by applying two layers of 13 mm thick fire-protective-grade plasterboard to both the inner and outer faces of the shaft.

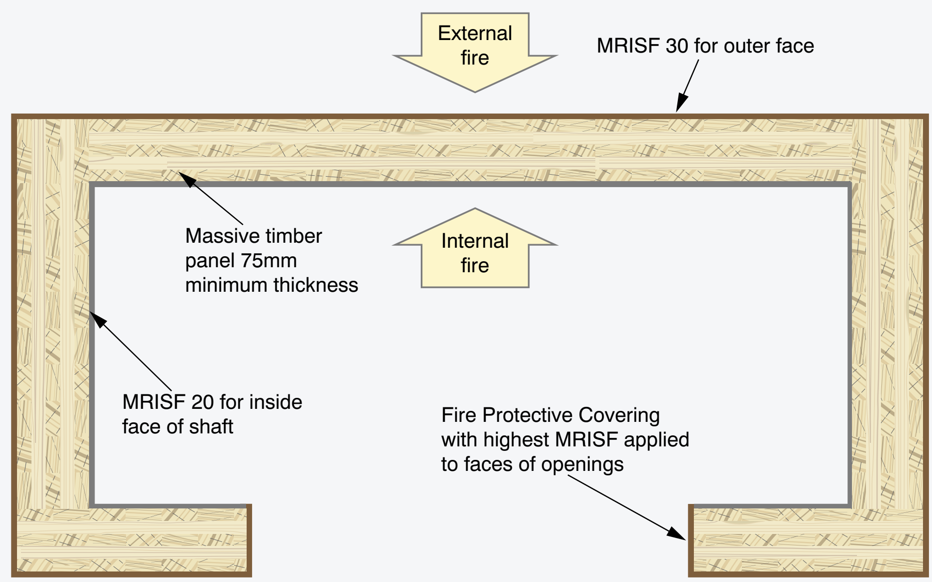


Figure 4.10: Lift shaft MRISF requirements if the massive timber provisions apply.

4.8 Fire-isolated Stairs and Passageways

4.8.1 Fire-isolated Stair and Passageway Construction

The FRLs, RISFs or MRISFs required for fire-isolated stairs and passageways are the same as those required for lift shafts described in Section 4.7.

Fire doors to fire-isolated stairs or passageways are required to achieve an FRL of -/60/30. Several proprietary fire door systems have been tested when mounted in timber construction. Installation details for fire doors capable of achieving FRLs of -/60/30 or above should be obtained from the supplier, since they may vary. Figure 4.11 shows a typical interface detail with a fire-protected timber wall.

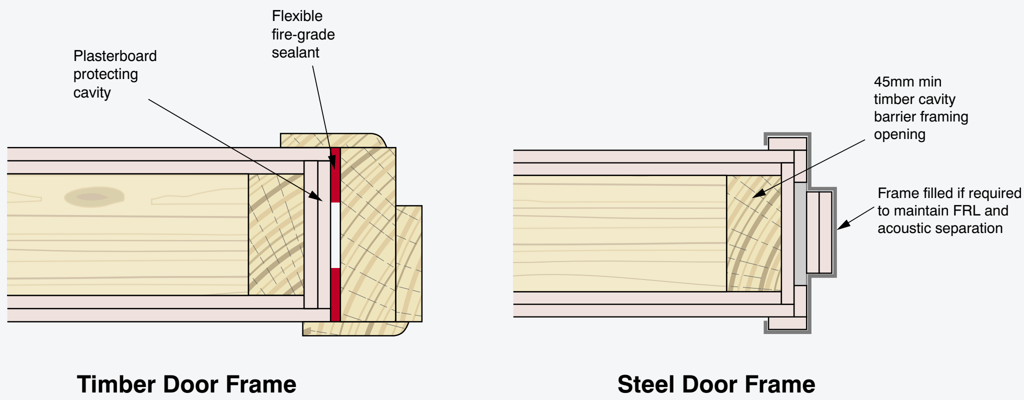


Figure 4.11: Typical fire door installation details.

4.8.2 Timber Stairways Concession

NCC Clause D2.25 provides a concession allowing timber treads, risers, landings and associated supporting framework to be used within a required fire-isolated stairway or fire-isolated passageway subject to:

a) the building being protected throughout by a sprinkler system complying with Specification E1.5 which is extended to provide coverage within the fire-isolated enclosure; and

b) the underside of flights of stairs directly above landings providing access to ground level or car parking levels being protected by a single layer of 13 mm fire-protective-grade plasterboard fixed to the stringers with fixings at not greater than 150 mm centres.

While fire starts in fire-isolated stairs are rare, when they do occur they generally involve stored or introduced materials and often the cause is malicious. While goods are not permitted to be stored in fire-isolated stairs and passageways, areas under the lowest flight of stairs form a convenient dry area for temporary storage. These areas may also not be secured, further increasing the risk of malicious fire starts.

While it could be argued that the extension of the sprinkler system to fire-isolated stairs and passageways addresses this issue, as an additional precaution, the underside of the lower stairs and landing where combustibles could be stored are required to be protected by a fire-protective covering of 13 mm fire-protective-grade plasterboard.

Typical generic details for treating service penetrations are provided in WoodSolutions Technical Design Guide #37

4.9 Building Services

4.9.1 Selection of Building Services and Distribution Paths

The building services and associated cable and pipe runs need to be selected and refined throughout the design process, to ensure the installation of the services and associated fire protection systems is efficient and reliable, and access is provided so the systems can be maintained or be expanded safely, without compromising fire safety systems.

Key points for consideration with respect to fire safety and acoustics are:

The number of service penetrations through fire-protected timber construction and fire-resisting construction generally should be minimised, as far as practicable. This can be achieved by measures such as the use of self-contained air conditioning systems serving each SOU, the use of false ceilings and wall facings allowing services to run behind the non-fire-rated face without penetrating the fire-resisting elements.

Services and connection details that do not require hot works should be selected where practicable to minimise the time fire services such as sprinkler systems will be unavailable. In some instances these requirements may conflict. For example, the use of CPVC piping for sprinkler systems can reduce hot works but the system will be unavailable if the pipework is adjusted – potentially overnight while the adhesive sets. Another option may be the use of mechanical joiners, avoiding the need for hot works or lengthy periods that the sprinkler system is unavailable while adhesives cure.

If service penetrations through fire-resisting construction cannot be avoided, the services should preferably penetrate shaft or service duct walls rather than fire-resisting walls or floors separating occupied areas. This reduces the acoustic impact as well as limiting the consequences if a penetration protection system fails; since smoke and fire spread will initially be limited to the service ducts.

Where practicable, shafts, service risers and service ducts should be readily accessible from public parts of the building to facilitate maintenance and inspection, but access hatches, panels or doors providing access should be secured to prevent unauthorised access.

If service penetrations through fire-protected timber construction cannot be avoided, where practicable, the service penetrations should be grouped together and penetrate framed out openings, which are then fire stopped with proprietary systems such as non-combustible batts, board or pillow systems. This approach substantially reduces the risk of fire spread to cavities at a point of weakness and ignition of fires if hot works are being undertaken on the services.

Different approaches may be required for different classes of buildings.

For example:

- Typical office building layouts comprise one or more cores constructed around lift and stair shafts. It is relatively easy to locate facilities such as toilets and kitchens around the core and provide service shafts such that most services are consolidated around the core. Services such as power, communications and air conditioning systems can be distributed easily, since there is very little fire compartmentation required in the office areas.
- Residential buildings differ from offices in that each SOU is a fire compartment and includes bathrooms and kitchens, and therefore in many instances it is impractical to consolidate services such as Drain, Waste & Vent (DWV) pipes around the central core, and service shafts are therefore needed to be distributed around the floor. For apartment buildings, the use of self- contained HVAC systems tends to be preferred; whereas, centralised HVAC systems may be preferred for hotels and more institutional-style buildings, requiring duct penetration of walls and floors to be addressed.

4.9.2 Service Shaft Construction

The requirements for fire-protected timber service shafts used for ventilation, pipes, garbage or similar purpose are summarised in Table 4.14.

Shafts must also be enclosed at the top and the bottom with a floor/ceiling system of the same Fire Resistance Levels and Resistance to the Incipient Spread of Fire Ratings as the walls; except where the top of the shaft is extended beyond the roof, or the bottom of the shaft is laid on the ground.

The shaft is also required to be sound-rated if it passes through more than one SOU and must have a $R_w + C_{tr} \geq 40$ if the adjacent room is habitable and $R_w + C_{tr} \geq 25$ if it is a kitchen or non-habitable room.

Table 4.14: Requirements for fire-protected service shafts in mid-rise timber buildings.

Criteria	**Residential Buildings (Class 2 & 3)**	**Office Buildings Class 5**	
		Type A Construction	**Type B construction (up to 3 storeys)**
FRL loadbearing elements	90/90/90	120/90/90	120/-/-
FRL non-loadbearing elements	-/90/90	-/90/90	-/-/-
RISF (general)	45	45	45
MRISF (massive timber)	30	30	30

In many instances, it is more practical to construct non-loadbearing shafts from laminated board systems or plasterboard/steel stud shaft wall construction in lieu of fire-protected timber construction. If these forms of construction are adopted and the board is non-combustible, then only the FRLs specified in Table 4.14 apply.

4.9.3 Protection of Service Penetrations

Service penetration systems are required by the NCC to comply with AS 4072.1 and AS 1530.4. For services penetrating fire-protected timber elements, there is an added complication in that the cavity temperatures have to satisfy the resistance to the incipient spread of fire or modified resistance to the incipient spread of fire criteria in addition to the integrity and insulation criteria applied to the non-fire side.

Further explanations of the test procedures are provided in Appendix E.

Typical solutions to address resistance to the incipient spread of fire performance criteria include: boxing out openings with plasterboard, filling the area around the service penetration with mineral fibre insulation or transitioning to a different wall type where service penetrations are required.

4.10 Interfacing With Other Forms of Construction

There can be advantages in adopting hybrid forms of construction in buildings. For example, ground floor and basement areas may be constructed from concrete to minimise the risk of water penetration, minimise potential damage in flood-prone areas or address termite management. In addition, this approach allows Classes other than 2, 3 and 5 to be incorporated in mid-rise buildings that are predominately of timber construction, subject to adequate fire separation between the classes.

The relatively low weight of timber structures also makes timber construction ideally suited to the upward extension of existing buildings facilitating infill developments and recycling existing buildings. For example, it may be possible to add apartments above existing retail buildings without having to undertake extensive reinforcement of the foundations.

Refer to NCC Volume One Cl2.8 for further details

Refer to NCC Volume One Cl2.9 for further details

Refer to NCC Volume One C1.13 for the Fire-protected Timber Concession

4.10.1 Separation of Different Classes

The NCC addresses the separation of different classifications within a building in Clauses C2.8 and C2.9. For the fire-protected timber concession to apply, it is necessary for Classes other than 2, 3 or 5 to be fire separated from the fire-protected timber construction.

For different classifications on the same storey, the parts having different classifications should be separated by a fire wall having the higher FRL for the two occupancies in accordance with Specification C1.1.

For different classifications in different storeys in a building of Type A construction (most mid-rise buildings), the floor between the adjoining parts must have an FRL not less than that prescribed by Specification C1.1 for the lower storey.

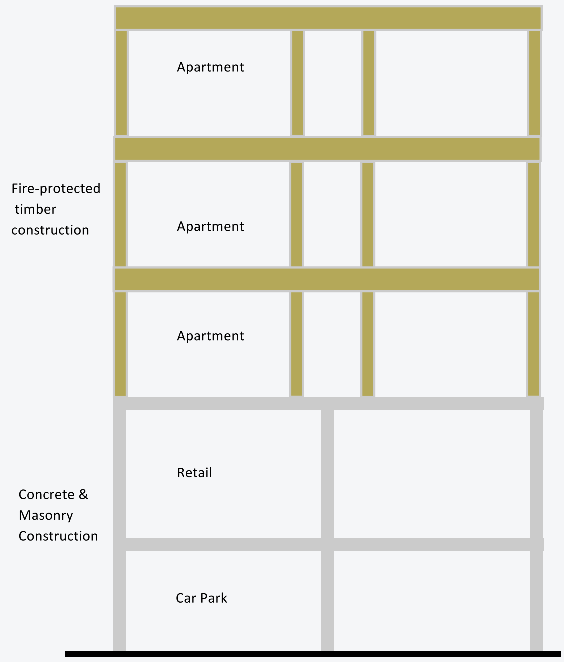


Figure 4.12: Example of multi-class building.

A typical building layout is shown in Figure 4.12 with a retail part of concrete-framed construction below timber-framed apartment levels. For the fire-protected timber concession to apply, the whole building will be sprinkler protected in accordance with NCC Specification 1.1.

Retail use is assigned to Class 6 buildings and therefore from Table 3 of Specification C1.1, the concrete slab separating the retail and apartment levels would require an FRL of 180/180/180. Shafts for lifts, fire stairs and services would be constructed from concrete/masonry on the retail and car park levels, but they may transition to timber-framed construction on the apartment levels. However, in some instances it may be preferred to continue the same form of construction for the entire shaft. Both options are permitted.

4.11 Special Fire Safety Issues

In constructing Class 2, 3 and 5 mid-rise timber buildings, special issues arise as buildings become larger and more complicated. Although this Guide does not attempt to provide information to suit all circumstances, information is provided where there is relevance to timber construction practices.

4.11.1 Smoke-proof Walls

For Class 2 and 3 buildings, the NCC requires that public corridors greater than 40 m long be divided by smoke-proof walls at intervals of not more than 40 m. These walls must be built from non-combustible materials and extend to the floor above, roof covering or Resistant to the Incipient Spread of Fire ceiling.

Smoke-proof walls can be constructed from fire-protected timber provided the RISF of 45 or MRISF of 30 (for massive timber) is achieved. Where the smoke-proof wall is also required to achieve an FRL (e.g. the wall is loadbearing) the fire-protected wall will also need to meet these FRL requirements.

4.11.2 Bushfire-prone Areas

The requirements for Class 2, 3 and 5 buildings to address the risk of bushfires vary between the States and Territories and may fall under different jurisdictions to standard building works. The need to consider bushfire exposures should be determined early in the design processes by the appropriate authority or authorities and addressed accordingly.

It should be noted that the NCC requires external walls to be of non-combustible in mid-rise buildings and the fire-protected timber concession requires timber elements to be protected by non-combustible fire-protective coverings, providing a good basis for the building to resist bushfire attack at the lower to intermediate BAL levels if adequate protection against ember attack is also provided.

4.11.3 Lightweight Construction Requirements

The NCC requires elements that have a Fire Resistance Level, or that form a lift, stair shaft, an external wall bounding a public corridor, non-fire-isolated stairway or ramp, to comply with Specification C1.8 if they are made out of lightweight materials such as timber framing faced with plasterboard.

Specification C1.8 defines a structural test for lightweight construction, and in most parts is directly related to the performance of the linings used. Appropriate evidence of suitability should be obtained from suppliers of lining material to verify compliance during the design phase.

4.11.4 Robust Structural Design

The 2016 revision of the NCC introduced a verification method for Structural Robustness to facilitate compliance with performance requirement BP1.1(a)(iii).

The verification method states:

Compliance with BP1.1(a)(iii) is verified for structural robustness by:

(a) assessment of the structure such that upon the notional removal in isolation of:

- (i) any supporting column; or
- (ii) any beam supporting one or more columns; or
- (iii) any segment of a load bearing wall of length equal to the height of the wall

the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The structural design of mid-rise timber buildings should comply with these requirements and the design guidance is provided in WoodSolutions Technical Design Guide #39: *Robustness in Structures* to ensure the building is adequately robust in the event of localised failure of elements during a fire.

4.11.5 FRL Concessions that are Not Applicable to Fire-protected Timber

The fire-protected timber requirements were based on the FRLs prescribed by Specification C1.1 without reductions in FRLs permitted by the following concessions:

- The Residential aged care building concession specified in Cl2.9 of Specification 1.1.
- Vic H103.1 Fire safety in Class 2 and Class 3 buildings should not be applied.

Therefore the above concessions do not apply to mid-rise timber buildings in the 2016 edition of the NCC.

5

Performance Solutions

5.1 Performance Solutions (Alternative Solutions)

The NCC Deemed-to-Satisfy mid-rise timber building solutions introduced in the 2016 edition are intended to provide additional safe and cost effective options. However, due to the generality of the solutions, it may be desirable or necessary to vary these Deemed-to-Satisfy Provisions for a specific building.

The NCC provides this flexibility by allowing a performance pathway (previously referred to as an alternative solution). One of the primary reasons for publishing this Guide was to provide background information on the underlying principles behind the Deemed-to-Satisfy mid-rise timber building requirements to facilitate the development of performance solutions without compromising the fire safety strategy for the building.

The following sections highlight commonly raised options for performance solutions and some of the key issues for consideration. The options and key issues for consideration should not be considered comprehensive and the processes detailed in the NCC and International Fire Engineering Guidelines should be followed when developing performance solutions and preparing Evidence of Suitability.

5.2 Exposed Timber Elements

There are applications where it is preferred that timber structural elements are exposed rather than being protected by non-combustible coverings with the prescribed Resistance to the Incipient Spread of Fire performance. Typical reasons include aesthetics, practicality and cost; although acoustic and thermal insulation requirements may necessitate the use of linings in many instances.

During consultation, some stakeholders raised the issue that the fire severity may be increased as a result of the additional exposed timber, while others argued that the fire would tend to self-extinguish before the timber would be consumed. The behaviour of timber elements will be very sensitive to a number of variables including: the materials and manufacturing process used in the manufacture of the timber element; orientation; exposed surfaces; presence of re-entrant corners; air flow; and background radiant heat flux.

A review of literature reporting full-scale tests demonstrated that exposed timber could contribute to the fire severity although, in the case of ventilation-controlled fires, the fire duration tended to be extended rather than the peak enclosure temperatures increasing. Further details are provided in Appendix A.

Unless specific data is available, a conservative approach is suggested, assuming that all exposed timber elements contribute their total exposed mass to the fire load and that burning timber elements will eventually fail if there is no fire brigade intervention. However, in most cases, subject to careful detailing of timber of larger cross-sections with the required Deemed-to-Satisfy FRLs, fire brigade intervention would be expected prior to collapse if the building layout facilitates access for fire fighters.

5.3 Extended Travel Distance

A common variation sought for Class 2 and 3 Buildings is an extension of the maximum travel distance from an SOU door to a fire-isolated exit from the 6 m maximum specified under the Deemed-to-Satisfy Provisions.

In addition to increasing travel time and potentially compromising way-finding conditions if smoke spreads to the corridor, this variation can also increase the number of SOUs served by a single stair that are at risk from a fire. The analysis should address this issue.

Due mainly to the provision of automatic fire sprinklers, timber mid-rise buildings were shown to present a substantially lower risk to occupants than a non-combustible construction without fire sprinklers.

The methods used for comparison of the timber options can be modified to address extended travel distances and include the effective increase in the SOUs and associated occupants potentially exposed to the fire risk.

5.4 Addition of Combustible Façades

Non-combustible façades can be added to mid-rise timber buildings over the fire-protected timber external walls. See WoodSolutions Technical Design Guide #37: *Mid-rise Timber Buildings*.

However, a performance solution is required to permit combustible façades to be fitted and it should be noted that for composite panels, each layer of the composite must be non-combustible.

Appendix A.6 provides useful background information on the behaviour of timber relating to external fire exposure.

WoodSolutions Technical Design Guide #18: Alternative Solution Fire Compliance, Façades also provides advice on developing performance solutions for timber façades.

In addition, Australian Standard AS 5113 Fire Propagation Testing and Classification of External Walls of Buildings was published in March 2016. The objective of the Standard is to provide procedures for the fire propagation testing and classification of external walls of buildings according to their tendency to limit the spread of fire via the external wall and between adjacent buildings. This may provide a useful option for deriving Evidence of Suitability for combustible external wall systems.

5.5 Effective Height Greater than 25m

It is viable to construct fire-protected buildings above an effective height of 25 m; particularly if massive timber or hybrid construction forms are adopted from a structural perspective.

All buildings above 25 m effective height require automatic fire sprinkler protection in accordance with the NCC Deemed-to-Satisfy Provisions and therefore, if a comparative study is undertaken, the control building would be a sprinkler-protected non-combustible building.

This effectively means that the fire-protected timber needs to provide equivalent or close to equivalent performance to non-combustible fire-resisting construction.

A possible massive timber option would be to increase the performance of fire-protective coverings throughout the building such that ignition of the timber substrate would be unlikely prior to either burnout of the contents or fire brigade intervention, and the large inherent fire resistance of massive timber elements provides an additional redundancy.

Such an approach is described in the publication Mass Timber Buildings of up to 12 storeys[4], which provides details applicable to massive timber buildings up to 40 m (12 storeys high) within the jurisdiction of the Government of Quebec, Canada. Fire-protective coverings (encapsulation) comprising two layers of 16 mm fire-grade plasterboard are required together with fire resistance levels of 120 minutes for loadbearing elements, among other things.

The analysis methods described in this Guide can be applied to evaluate buildings greater than 25 m high, but fire brigade intervention and evacuation times would be substantially increased at heights above 25 m due to issues such as fire-fighter fatigue and access to higher levels.

C

Part C - Fire Engineering Justification

Part C of the Guide provides details of the analysis undertaken to justify the 2016 changes to the NCC relating to mid-rise timber buildings. It is intended to provide a resource to assist in the interpretation of the Deemed-to-Satisfy solutions and to facilitate the development of performance solutions that are consistent with the NCC 2016 mid-rise timber Provisions. A section listing is provided below:

6

Overview of Fire Engineering Analysis

To evaluate the proposed changes to the 2016 edition of the NCC that provide Deemed-to-Satisfy fire-protected timber solutions for mid-rise buildings, it was necessary to determine the change in risk (probability and consequences) of fire spread as a result of an increase in the mass of combustible materials present and response of timber structures to fire compared to the forms of non-combustible construction permitted in the 2015 edition of the NCC.

A preliminary analysis was undertaken and following discussions with stakeholders:

- changes to the NCC were proposed (see Section 4)
- generic buildings layouts including fire protection systems were defined (see Section 7) and occupancy types identified
- fire scenarios and methods of analysis were defined
- key inputs were agreed as appropriate.

Additional analysis assessing the impact of fires on occupants and property within the SOU of fire origin was not required because the preliminary analysis, based on fire incident data, provided a clear indication of a significant improvement in safety for the timber building options from the addition of automatic fire sprinkler systems. Details of the preliminary analysis are provided in Section 8.

The most critical scenario was identified as potential flashover fires occurring within an SOU, since the majority of fires occur within SOUs and the fire load within apartments is relatively high compared to most other locations, and therefore would be expected to provide the greatest challenge to fire-protected timber systems.

A Monte Carlo (multi-scenario) simulation approach was adopted to compare the outcomes relating to the frequency and consequences of potential fully developed fires starting in an SOU (e.g. apartment) on the remainder of the building and structure. The proportion of potential fully developed fires was estimated based on fire spread data from fire incidents and included all fire scenarios where fire spread beyond the enclosure of fire origin was recorded.

Other scenarios were identified that required further analysis. Table 6.1 summarises all the scenarios considered and refers to the relevant sections of this Guide.

Table 6.1: Fire scenarios considered.

Scenario	Sections
Impact of fires on occupants within the SOU of fire origin	**Section 8**
Impact of fires on occupants outside the SOU of fire origin – non-flashover fires	**Section 9**
Impact of fires on occupants outside the SOU of fire origin – post-flashover fires	**Section 10**
Fires in paths of travel to escape routes	**Section 11**
Fires in fire-isolated stairs	**Section 12**
Fires in lift shafts	**Section 13**
Fires in concealed spaces	**Section 14**
External fire spread – building façade	**Section 15**
Fire spread between buildings	**Section 16**

A supplementary analysis was undertaken for Class 5 Office Buildings. The details are provided in Section 15.

7

Mid-Rise Buildings Chosen for Analysis

7.1 Characterisation Principles

To undertake a comparative analysis, it was necessary to define a generic building and to characterise the building and occupants in sufficient detail that parameters relevant to the comparative study could be identified; but without introducing unnecessary complications. For the study, a generic structural and architectural layout was developed that could reasonably be applied to both Class 2 and 3 buildings with minor changes to occupant profiles and fire safety features. A single fire-isolated stair option was considered likely to be more sensitive to variations in the fire safety design and, since a single fire-isolated stair is permitted in buildings with an effective height of not more than 25 m (the upper limit for mid-rise timber buildings in the 2016 NCC Deemed-to-Satisfy solution), this configuration was selected.

It is common for mid-rise Class 2 and 3 buildings to include basement car parks (Class 7) and other occupancies on the ground floor such as office or retail (Class 5 or Class 6). The height of the ground level was increased to address this potential. Parts of the building having a different classification will be fire separated from the Class 2 or 3 parts in accordance with the NCC mid-rise timber building Deemed-to-Satisfy Provisions and have independent egress and access. It was therefore considered unnecessary to define the ground floor layout and basement levels if provided, other than to define discharge points from the fire-isolated exits serving the upper levels and fire protection measures relevant to fire brigade intervention.

7.2 General Building Layout and FRLs

The benchmark (control) was an apartment building with an effective height of 23.1 m (i.e. slightly below 25m) designed in accordance with the NCC Deemed-to-Satisfy requirements specified in the 2015 edition of the NCC. This control was compared to a building designed in accordance with the mid-rise timber building Deemed-to-Satisfy requirements introduced into the 2016 edition of the NCC and described in Section 4 of this Guide.

Figure 7.1 shows a section through the generic Class 2 or Class 3 building and Figure 7.2 shows a schematic plan of a typical residential floor.

As noted above, the ground floor has a greater floor-to-floor height and may contain a different type of occupancy, but independent access and egress will be provided to and from that level and any basements. Therefore only discharge points from the fire-isolated stair serving the upper levels and fire protection measures relevant to fire brigade intervention (including the entry lobby) are shown in the schematic plan of the ground level in Figure 7.1 and Figure 7.3.

The FRLs prescribed by the NCC for elements of construction are summarised in Table 7.1. Symbols are provided to enable the elements to be identified in Figure 7.1 through to Figure 7.3. The FRLs prescribed for elements of construction, that vary with the distance from the boundary (external walls), are summarised in Table 7.2.

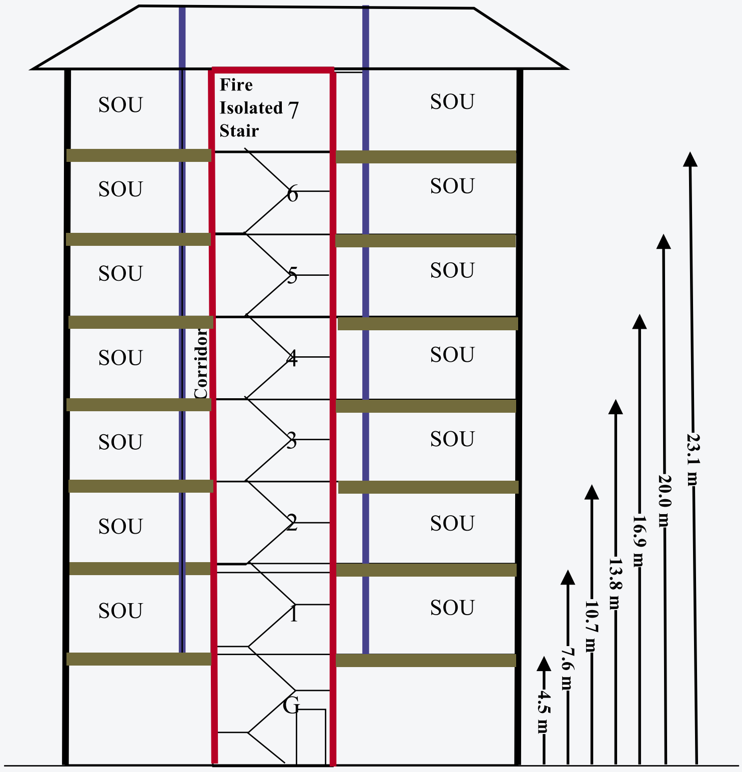


Figure 7.1: Vertical section through generic Class 2/3 building.

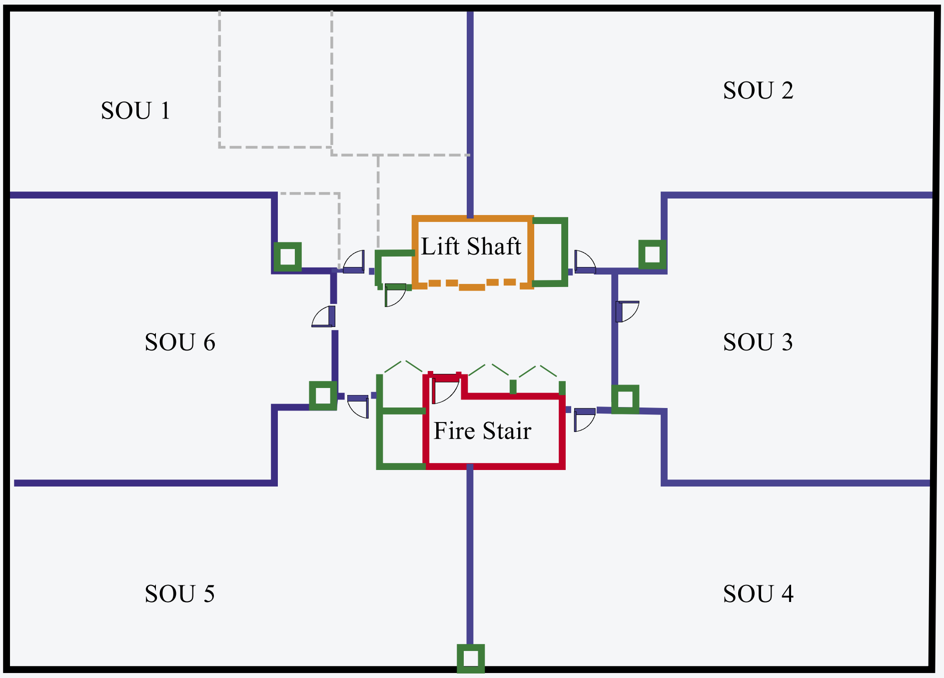


Figure 7.2: Schematic plan of a residential level of a generic Class 2/3 building.

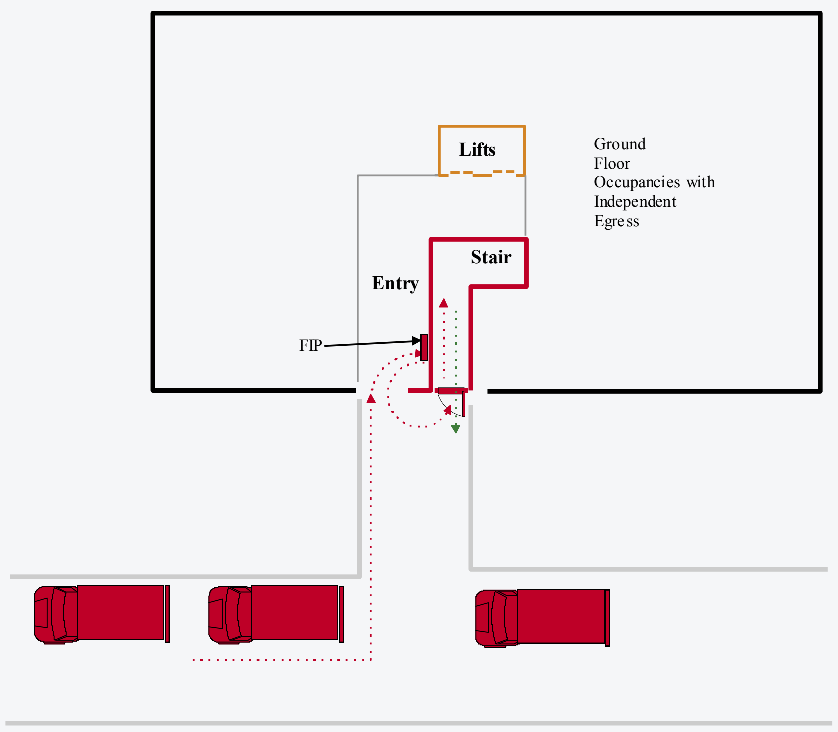


Figure 7.3: Ground floor plan of generic building.

Table 7.1: FRLs for elements of construction – Class 2 and 3 buildings.

Symbol	Description	FRL – Structural Adequacy /Integrity/ Insulation – min		Modified Resistance to the Incipient Spread of Fire (min.)
		Loadbearing	Non-loadbearing	
	Fire stair shaft	90/90/90	-/90/90	30 outside 20 inside
	Service shaft	90/90/90	-/90/90	30
	Bounding Sole Occupancy Units	90/90/90	-/60/60	30
	Lift shaft walls	90/90/90	-/90/90	30
	Door to fire stair	Not applicable	-/60/30	30 outside 20 inside
	Fire door to service shaft	Not applicable	-/60/30	Not applicable
	Door to SOU	Not applicable	-/60/30	Not applicable
	Lift door	Not applicable	-/60/-	Not applicable
	Doors to services risers	Not applicable	-/60/30	Not applicable
	Non-loadbearing walls within an apartment	Not applicable	-/-/-	-
	Floors	90/90/90	Not applicable	30

Table 7.2: FRLs for external walls – Class 2 and 3 buildings.

Symbol	Description	Distance from fire source feature	FRL – Structural Adequacy /Integrity/ Insulation – min	
			Loadbearing	Non-loadbearing
	External walls	<1.5 m	90/90/90	-/90/90
		=15 < 3 m	90/60/90	-/60/60
		= 3 m	90/6030	-/-/-

7.3 Passive Fire Protection Systems

Passive fire protection systems required by the NCC 2015 Deemed-to-Satisfy Provisions are compared to the requirements for mid-rise timber buildings in the 2016 edition in Table 7.3.

Table 7.3: Passive fire protection systems.

System	Deemed-to-Satisfy Provisions for Control Class 2 Steel-Framed Building	Additional/Alternative Measures for Timber Construction
FRLs	Refer Figure 7.1 through Figure 7.3.	No difference
Concrete and masonry construction	Loadbearing internal walls (including shafts and fire walls).	Fire-protected timber in lieu of concrete or masonry
Non-combustible construction	• External walls • Common walls • Flooring and Floor framing to lift pits • Non-loadbearing walls required to be fire-resisting • Non-loadbearing shafts that do not discharge hot products of combustion • Miscellaneous applications	Fire-protected timber in lieu of non-combustible construction
Fire hazard properties	Full compliance with Specification C1.10	No difference
External wall construction, separation distances and openings	Full compliance with Deemed-to-Satisfy Provisions	No difference other than fire-protected timber in lieu of non-combustible construction
Cavity Barriers	No requirements	Required where cavities within fire-resisting structures occur

The steel-frame building with non-loadbearing internal walls was selected for the control building since it is considered to most closely resemble the timber mid-rise buildings. Key elements of construction are summarised in Table 7.4 for the control building and subject buildings

Two subject buildings were required to be compared to the control – a lightweight timber-framed building and a massive timber panel building that used the massive timber provisions which reduces the incipient spread of fire criteria applicable to the general fire-protected timber provisions.

Generally the passive systems are similar for both Class 2 and 3 buildings.

Table 7.4: Passive systems for comparative analysis.

Element	Control Building	Lightweight Timber Frame (Subject Building 1)	Massive Timber (Subject Building 2)
Floor/ceiling assemblies FRL 90/90/90	Concrete slab supported on steel beams. Steel beams protected by sprayed-vermiculite to provide required FRL.	Fire-protected timber floor comprising either solid joists or engineered timber beams spanning between timber-framed walls. Fire-grade plasterboard facings, 2 x 16 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber.	Fire-protected cross-laminate timber horizontal panels spanning between CLT walls. Fire-grade plasterboard facings, 1 x 16 mm used to protect timber.
False non-fire rated standard plasterboard ceiling to allow service runs above for all buildings			
Columns / loadbearing walls 90/90/90	Steel columns protected by sprayed vermiculite and clad with non- fire-grade plasterboard.	Fire-protected timber-frame loadbearing walls. Fire-grade plasterboard facings, 2 x 13 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber.	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 1 x 16 mm.
Non-loadbearing walls	Lightweight steel frame protected by 16 mm fire-grade plasterboard (-/60/60 FRL).	Lightweight timber frame protected by 2 x 13 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers. (Extra plasterboard required to meet incipient spread of fire rating effectively increasing FRL to – /90/90)	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 1 x 16mm.
Lift and stair shafts	Structural steel framework with sprayed-on fire protection in combination with non-loadbearing plasterboard shaft wall (-/90/90).	Fire-protected timber-frame loadbearing walls. Fire-grade plasterboard facings, 2 x 13 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber.	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 1 x 16 mm on outer face of shaft and 1 x 13 mm on interfaces.
Service shafts -/90/90	Solid fire-grade plaster board (multi-layer system).	Solid fire-grade plasterboard (multi-layer system) or fire-grade plasterboard facings, 2 x 13 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber if integrated into apartment wall.	Solid fire-grade plasterboard (multi-layer system) or cross-laminated timber protected by a minimum of 16 mm fire-grade plasterboard.
External wall less than 1.5m from fire source feature FRLs 90/90/90 and -/90/90	Structural steel protected by vermiculite. Lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard	Lightweight timber frame protected by 2 x 13 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers.	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 2 x 13 mm.
External wall 1.5m to less than 3m from fire source feature FRLs 90/60/60 and -/60/60	Structural steel protected by vermiculite lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard.	Lightweight timber frame protected by 2 x 13 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers.	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 1 x 16 mm.
External wall 3m or more from fire source feature FRLs 90/60/30 and -/-/-	Structural steel protected by vermiculite lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard.	Lightweight timber frame protected by 2 x 13 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers.	Fire-protected cross-laminated timber vertical panels. Fire-grade plasterboard facings 1 x 16 mm.
Fire doors -/60/30 modern prototypes with intumescent strips			

7.4 Active Fire Protection Systems

Active fire protection systems required by the NCC Deemed-to-Satisfy provisions are summarised in Table 7.5 for Class 2 buildings.

Table 7.5: Active fire protection systems for Class 2 buildings.

System	Deemed-to-Satisfy Provisions for Control Class 2 Building	Additional/Alternative Measures for Timber Construction
E1.3 Fire hydrants	Internal fire hydrants in accordance with AS 2419.1 provided for each storey	No difference
E1.4 Fire hose reels	Not required for a Class 2 building	No difference
E1.5 Sprinklers	Not provided	System provided in accordance with Specification E1.5 (AS 2118.1/AS 2118.4 as appropriate)
E1.6 Portable fire extinguishers	Provided in accordance with Table E1.6 and AS 2444 as appropriate (impact assumed to be taken into account inherently in estimate of proportion of flashover fires)	No difference (Impact assumed to be taken into account inherently in estimate of proportion of flashover fires)
E1.8 Fire control centre	Not required since building less than 25m effective height	No difference
E2.2 Smoke hazard management **Fire detection/alarm system in accordance with Spec 2.2a. Independent exit from parts of other classes therefore no stair pressurisation required**	Self-contained smoke alarms in SOUs Smoke alarms or detectors in public corridors and other internal public spaces activating a general building alarm	No difference – Self-contained smoke alarms in SOUs Activation of any sprinkler will raise alarm throughout the building.
E2.2 System monitoring	None	Fire sprinkler system monitored with automatic notification of fire brigade

Active fire protection systems required by the NCC Deemed-to-Satisfy Provisions are summarised in Table 7.6 for Class 3 buildings. These are generally similar to those for Class 2 buildings, except for enhancements to the requirements for detection and alarm and monitoring requirements under E2.2 of the NCC.

Table 7.6: Active fire protection systems for Class 3 buildings.

System	Deemed-to-Satisfy Provisions for Control Class 3 Building	Additional/Alternative Measures for Timber Construction
E1.3 Fire hydrants	Internal fire hydrants in accordance with AS 2419.1 provided for each storey	No difference
E1.4 Fire hose reels	Not required for a Class 3 building	No difference
E1.5 Sprinklers	Not provided	System provided in accordance with Specification E1.5 (AS 2118.1/AS 2118.4 as appropriate)
E1.6 Portable fire extinguishers	Provided in accordance with Table E1.6 and AS 2444 as appropriate (Impact assumed to be taken into account inherently in estimate of proportion of flashover fires)	No difference (Impact assumed to be taken into account inherently in estimate of proportion of flashover fires)
E1.8 Fire control centre	Not required since building less than 25 m effective height	No Difference
E2.2 Smoke hazard management **Fire detection / alarm system in accordance with Spec 2.2a. Independent exit from parts of other classes therefore no stair pressurisation required**	Building wide smoke detection system generally in accordance with AS 1670.1 Activation of any smoke or heat detector will raise alarm throughout the building.	No difference to detection system except sprinkler heads can provide coverage in areas prone to false alarms in lieu of heat detectors. Activation of any sprinkler or smoke detector will raise alarm throughout the building.
E2.2 System monitoring	Smoke detection system is monitored with automatic notification to fire brigade	Fire sprinkler system and smoke detection system monitored with automatic notification of fire brigade

7.5 Occupant Characteristics

The occupant characteristics will be identical for the timber (subject) buildings and control (Deemed-to-Satisfy Provisions).

However, the occupant characteristics may vary between Class 2 and the various sub-categories within Class 3 buildings. The definitions for Class 2 and Class 3 buildings from the BCC are summarised below:

Class 2: a building containing 2 or more *sole-occupancy units* each being a separate dwelling.

Class 3: a residential building, other than a building of Class 1 or 2, which is a common place of long term or transient living for a number of unrelated persons, including -

(a) a boarding house, guest house, hostel, lodging house or backpackers accommodation; or
(b) a residential part of a hotel or motel; or
(c) a residential part of a school; or
(d) accommodation for the aged, children or people with disabilities; or
(e) a residential part of a health-care building which accommodates members of staff; or
(f) a residential part of a detention centre.

The occupant characteristics of Class 2 buildings can be considered to be broadly representative of the Australian community and this profile was also applied to Class 3(a), (b), (c), (e) and (f) occupancies which were defined as Type 1 occupants.

Class 3(d) occupancies accommodate larger proportions of people who will require assistance to evacuate and hence present a greater fire risk and were defined as Type 2 occupants. In these instances, some level of staff assistance would be required to facilitate evacuation, which may vary from simply providing direction to providing physical assistance to occupant(s) who may or may not be aware of the emergency.

These two Types were considered to bracket other Class 3 occupants.

7.6 Emergency Exit Provisions

Emergency exit provisions are in accordance with the NCC Deemed-to-Satisfy provisions and were the same for the subject buildings and control. They are shown schematically in Figure 7.1 through Figure 7.3.

8

Impact on Occupants within the SOU of Fire Origin

8.1 General Fire Safety Provisions within a SOU

Under the Deemed-to-Satisfy Provisions of the NCC, each apartment (SOU) has fire-resisting bounding construction forming a fire compartment but there is no requirement for further fire compartmentation within the SOU. For Class 2 buildings, the apartment is normally provided with stand-alone fire alarms, similar in function and location to a typical family dwelling, raising an alarm within the SOU of fire origin only. For Class 3 buildings, a general building alarm system is provided.

It is reasonable to assume that after flashover the probability of any remaining occupants within the SOU of fire origin surviving for a significant period is low.

The following sections consider the two critical variations:

- Where non-combustible construction and masonry or concrete construction is specified in the Deemed-to-Satisfy Provisions of the NCC, fire-protected timber will be used.
- The addition of automatic fire sprinklers in buildings with an effective height of not more than 25 m.

8.2 Fire-protected Timber instead of Non-combustible or Masonry or Concrete Construction for the Walls Bounding an SOU

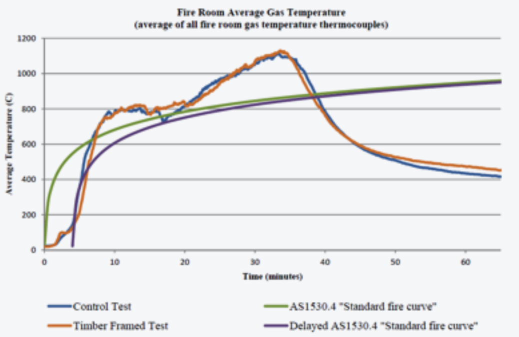
Under the proposed changes, the timber structural members forming the bounding walls of a fire compartment will be protected with non-combustible materials capable of preventing the interface with the timber exceeding 300°C (onset of charring) for massive timber panels, and 250°C for lightweight timber frame construction, for a considerable period – facilitating fire brigade intervention before ignition of the timber substrate. This limit will be exceeded substantially after untenable conditions occur within the SOU of fire origin, and therefore the outcomes would be expected to be similar for the existing Deemed-to-Satisfy Provisions and the use of fire-protected timber for the bounding walls, if the substantial impact of the additional automatic fire sprinkler system is ignored.

This was demonstrated in comparative full-scale fire tests with room enclosures lined with fire-grade plasterboard. One enclosure was of lightweight steel construction (the non-combustible control test) and the other was of lightweight timber-framed construction (timber-framed test). Timber cribs were used as the fire load and an insulated column included in the enclosure to compare the fire severity, as shown in Figure 8.1. Typical results are presented in Figure 8.2 and show that there was no increase in fire severity based on a range of parameters including enclosure temperature, heating rate of an insulated steel column, non-fire side temperatures of the enclosure partitions or cavity temperatures. See England and Eyre[5] for further details.

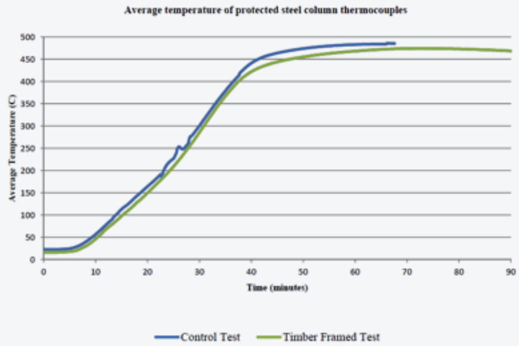
Timber cribs and insulated column before testing

Enclosures during fully developed fire tests

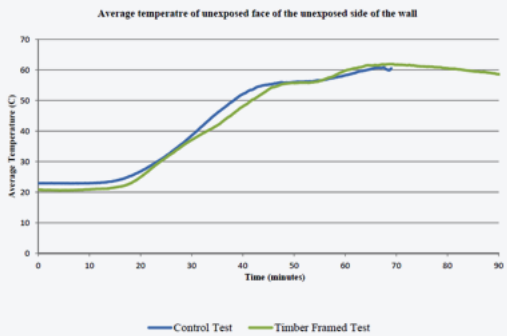
Figure 8.1: Comparative testing of non-combustible and timber-framed construction.



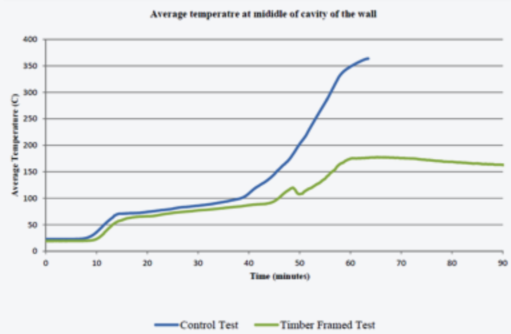
Average Enclosure Temperatures



Average temperature of Protected Steel Beam



Average Unexposed Face Temperatures of Partitions



Average temperature in Centre of Cavity of walls

Figure 8.2: Results of comparative testing of non-combustible and timber-framed construction.

8.3 Additional Provision of an Automatic Fire Sprinkler System

When a fire sprinkler operates successfully, the fire will be suppressed or controlled prior to flashover, greatly enhancing the safety of occupants within the apartment of fire origin.

Due to the relatively small population size and small proportion of residential buildings with automatic fire sprinkler systems in Australia, there is insufficient Australian statistical data to reliably establish the impact of the addition of automatic fire sprinklers to residential buildings. It is therefore reasonable to use data from the US, where the larger population and greater proportion of residential buildings with automatic fire sprinkler protection provides a much larger sample.

A detailed study of the "U.S. Experience with Sprinklers" based predominately on US fire statistics for the period 2006–2010 was undertaken by Hall, 2012[6]. Relevant key findings and data from the report are summarised below:

- Sprinkler systems were present in 22,270 home-reported fires (including apartments) – 6% of all reported home fires.
- The estimated number of home fires large enough to be capable of activating a sprinkler head was 2,520 (11% of reported fires in sprinkler protected homes).
- Percentage of home fires where sprinkler system operated effectively – 92%
- Percentage of all structure fires where sprinkler system operated effectively – 88%
- Percentage of home fires confined to room of origin – 55% without sprinklers, 88% with sprinklers
- Home fire fatalities/1000 reported fires – 7.3 without sprinklers, 1.3 with wet pipe sprinklers (i.e. 83% reduction)
- Average direct property damage/home fire – US$20,000 without sprinklers, US$6,000 with wet pipe sprinkler system (i.e. 69%) reduction.

The report also estimated that the number of false discharges (due to accidental impacts, for example) from fire sprinklers in homes was about 5% of fire incidents where sprinklers operated. This estimate was based on smaller sample sizes using data from 2003–2006.

8.4 Impact of Proposed Changes within an SOU of Fire Origin

On the basis of the above discussion and the analysis included in Appendix F, it was concluded that the net impact of the proposed changes to the NCC compared to existing Deemed-to-Satisfy provisions on the occupants within the SOU of fire origin would be a substantial reduction in fatalities and direct property damage. Due to the magnitude of these changes and the substantial data available that demonstrates the improvement in safety resulting from the introduction of automatic fire sprinkler systems, it was considered unnecessary to undertake further analysis with respect to the risk to occupants within the SOU of fire origin.

9

Impact on Occupants Outside the SOU of Fire Origin – Non-flashover Fires

9.1 Small Flaming and Smouldering Fires

Small flaming fires and smouldering fires of insufficient size to activate a sprinkler head would be unlikely to penetrate the non-combustible insulating layer applied to fire-protected timber and cause ignition of the timber. Therefore no significant difference in outcome would be expected between the subject fire-protected timber buildings and control building meeting NCC 2015 Deemed-to-Satisfy Provisions based on non-combustible construction.

No further analysis of these scenarios was considered necessary and the Deemed-to-Satisfy fire-protected timber solutions within the NCC 2016 and the NCC 2015 Deemed-to-Satisfy solutions described in Section 4 were considered to be equivalent for small flaming and smouldering fires.

Cavity fires have been considered separately in Section 14.

9.2 Large Flaming Non-Flashover Fires

These fires would be of sufficient size to activate an operational fire sprinkler system. If the fire sprinkler system operates effectively, the fire size would be limited or suppressed for the fire-protected timber options and, in addition, a general building alarm would be raised and automatic fire brigade alert would be activated.

In the low probability of sprinkler failure, the timber core would be protected from large flaming non-flashover fires by the fire-protective coverings. Since fire-protected timber is designed to provide protection against fully developed fires, it is considered unlikely that the fire would be of sufficient size to cause ignition of timber, and no significant difference in outcome would be expected between the subject timber building solutions and the control building complying with NCC 2015 Deemed-to-Satisfy Provisions if the sprinkler system failed to operate effectively.

For a large proportion of scenarios, the door to the SOU would be closed, minimising smoke spread to escape routes.

In the low probability event that the door to the SOU or compartment of fire origin was open, smoke spread would be similar for the solutions being considered if the sprinkler system failed to operate; as would the outcomes, as the detection and alarm systems and occupant profiles would be similar. However, in the majority of scenarios where the sprinkler system operates, it would be expected that the risk to life would be significantly less for the sprinkler-protected option.

It was therefore considered that no further analysis of these scenarios was required, and that the NCC 2016 timber solutions would be expected to provide a reduction in the expected risk to life of occupants outside the SOU of fire origin compared to the NCC 2015 Deemed-to-Satisfy Provisions identified in Section 4 for large flaming non-flashover fire scenarios.

10

Impact on Occupants Outside the SOU of Fire Origin – Post-flashover Fires

Fully developed (post-flashover) fires have the greatest potential to challenge the fire-protected timber, and therefore a more detailed multi-scenario quantitative risk assessment was considered necessary. The adopted approach used the EFT Multi-scenario Quantitative Risk Assessment Framework.

10.1 EFT Multi-scenario Quantitative Risk Assessment Framework

The EFT Multi-scenario Quantitative Risk Assessment Framework was developed for the analysis of fully developed fires in multi-storey structures. Details of the framework have been described by England[7] and are described further in this Section. A key feature of the framework is the ability to undertake multi-scenario analyses taking into account the time dependency of factors, such as:

- response of elements of construction to fire incorporating the variability of FRLs and effects of installation faults
- smoke spread
- detection and alarm system activation
- fire brigade intervention
- occupant response.

This removes a major limitation with earlier multi-scenario analysis models and simple event tree analyses, which were unable to consider time dependencies when analysing the impact of changes that effect parameters such as fire brigade intervention, FRLs of elements of construction and occupant response – particularly beyond the floor of fire origin.

The framework comprises three core models and three input models as shown in Figure 10.1. The risk of structural collapse and/or the number of occupants exposed to untenable conditions are the primary outputs, enabling individual and societal risks to be estimated for occupants within the building together with the risk of structural collapse. A major structural collapse could affect people and property outside the building of fire origin and, in many instances, could be an important parameter for comparison of fire safety strategies.

The core models comprise:

- enclosure fire/structural
- fire brigade intervention
- occupant response evacuation and consolidation.

The input models comprise

- sprinkler intervention
- base smoke spread
- detection and alarm.

These core models and input models use various sub-models that can be selected to suit the particular applications, providing the flexibility to allow different sub-models to be incorporated into the framework if they are more suited to a particular application.

The sub-models used in conjunction with the framework to evaluate the mid-rise timber building Provisions included in the 2016 edition of the NCC are described in this Section, together with the adopted inputs. Further information on the models and derivation of inputs is provided in Appendices F and G.

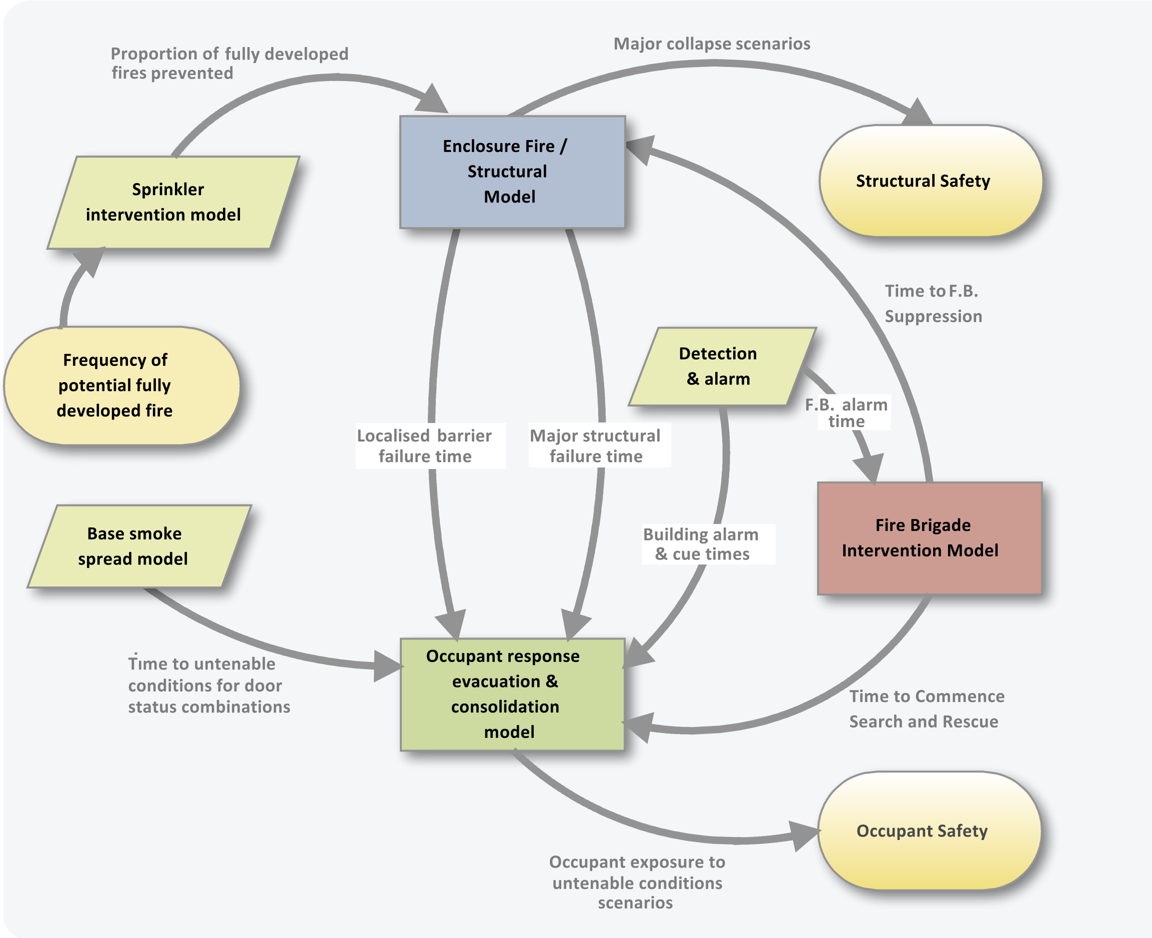


Figure 10.1: EFT Multi-scenario quantitative risk assessment framework.

10.2 Frequency of Potential Fully Developed Fires

The Report on Government Services 2013[8] included a report on accidental residential fires reported to fire services from 2008 to 2012 from which it can be estimated that there are approximately 100 fires reported per 100,000 households per annum, i.e. 10^{-3} fires /household per annum are reported. This is expected to provide a reasonable indication of fire starts in apartments.

The proportion of potential flashover fires was reported in Apte et al.[9] , based on the work of Yung, Benichou, Narrayanan and Whiting. The resulting estimates are summarised in Table 10.1, with unknown fires sizes in the NZ data proportionally distributed.

Table 10.1: Proportions of flashover fires.

Fire type	Australia	US	Canada	NZ
Smouldering fire	24.5%	18.7%	19.1%	27.0%
Non-flashover fire	60.0%	63.0%	62.6%	49.8%
Flashover fire	15.5%	18.3%	18.3%	23.2%

Since most households are single dwellings, it is reasonable to assume that few of these fires occurred in sprinkler-protected buildings, and therefore it is estimated that approximately 18% of fires would progress to flashover if no automatic fire sprinklers were present.

Therefore 1.8×10^{-4} potential flashover fires/household/annum are estimated to occur.

An approximation of the frequency of flashover fires in an apartment block can be obtained by using the following equation:

Frequency of potential flashover fires $= 1.8 \times 10^{-4}$/apartments/annum in the building.

This is potentially a conservative (overestimate of frequency), since it may contain some larger fires that did not reach flashover. However, since the analysis being undertaken is predominately a comparative analysis, this approximation was considered reasonable.

10.3 Sprinkler Intervention Model

For the purposes of the analysis of potentially fully developed fires, it was considered adequate to adopt a simple intervention model based on an analysis of fire data assuming that if the sprinkler operated in accordance with its design objectives it would either control the fire or suppress the fire, preventing transition to a fully developed fire.

Due to the relatively small population size and small proportion of residential buildings with automatic fire sprinkler systems present in Australia, there is insufficient Australian statistical data to reliably establish the impact of the addition of automatic fire sprinklers to residential buildings. It is therefore reasonable to use data from the US, where the larger population and greater proportion of residential buildings with automatic fire sprinkler protection provides a much larger sample.

A detailed study based predominately on US fire statistics for the period 2006–2010, *U.S. Experience with Sprinklers*, was undertaken by Hall, 2012[6]. Relevant key findings and data from the report are summarised below:

- percentage of home fires where sprinkler system operated effectively – 92%
- percentage of all structure fires were sprinkler system operated effectively – 88%.

Further confidence in the applicability of the above data to Australian buildings can be obtained by comparing the above statistics with other studies.

A detailed summary of available sprinkler system component data and reports on the effectiveness for fire sprinkler systems was undertaken by Frank et al.[10]. It found that estimates of effectiveness varied from 70% to 99.5%, depending to a large extent on definitions and selection of data. For probabilistic analysis, it was suggested that a distribution be considered with a peak between 90% and 95%.

Moinuddin, Thomas and Chea[11] estimated failure rates between 3% and 14% for office buildings in Australia, but indicated that by fitting isolation valves on each storey the reliability could be improved by 13%.

Koffe[12] published a paper analysing US sprinkler reliability data from the NFPA, and proposed a reliability of 90%.

A study by BRANZ estimated the reliability of domestic sprinkler systems to be about 95%; however, this study assumed 99.9% reliabilities for operation of the sprinkler head, effectiveness of the sprinkler discharge, design competence and correctness of installation. It also assumed 99% reliability for the operational valves, but this is substantially higher than the estimates of Moinuddin, Thomas and Chea.

A reliability of 92% was adopted for the detailed study, which is consistent with the estimate from the Hall study.

If the sprinkler system operated successfully, it was assumed that the consequences of a fire occurring within an SOU – with respect to the risk to life of occupants – would be limited to the SOU of fire origin and that fire-resisting elements of construction would not fail.

Reference should be made to Section 8 for estimation of the consequences of fires within the SOU of fire origin.

10.4 Base Smoke Spread Model

Prior to failure of fire-resisting elements of construction, smoke spread can occur through various leakage paths but will tend to be dominated by the state of doors such as the door to the apartment of fire origin. If exit paths become smoke logged, it will affect the ability of occupants to self-evacuate and also the speed (and efficacy) with which the fire brigade can undertake search and rescue and suppression activities. Prior to failure of elements of construction, the conditions will be the same for all strategies for scenarios where there is no automatic fire suppression.

The base smoke spread model determines the conditions in various enclosures assuming there are no fire-induced failures of barriers (i.e. smoke spread occurs through existing openings only). The open/closed status of doors is considered and smoke spread is modelled for various combinations of openings states. Experiments on fire doors were used to estimate the performance of doors in the closed state.

Inputs to the occupant response evacuation and consolidation model are:

- the probabilities of the doors being closed during the fire
- the times to low visibility for various combinations of door open states
- the times to untenable conditions in enclosures outside the SOU of fire origin for various combinations of door open states.

10.4.1 Probability of Doors being Open at the Time of a Fire

A report on the fire system effectiveness in major buildings in New Zealand[13] included inspection data from university, hospital, and office/retail buildings relating to more than 5,000 passive fire protection systems, including fire doors. The results shown in Table 10.2 have been extracted from the NZ Study.

Table 10.2: NZ fire and smoke door survey results.

Issue	Fire Doors (%)	Smoke Doors (%)	Riser Hatches (%)
Wedged/blocked	1.9	1.8	
Painted smoke seals	0.5	0	
Missing smoke seals	4.8	10.3	
Excessive clearance	0	1.8	2.5
Carpet under door	1.4		
Excessive force to open	0.5		
Missing closers	1.5	1.5	
Damaged closers	0	0	
Not fully closing	2.9	2.9	
Total	**13.5**	**18.3**	**2.5**

These results are incorporated in Table 10.3, which also includes data provided from other sources including Moinuddin and Thomas[14] and England et al.[15].

Of the 34% of doors with faults in the Kettle study, only 4.5% could not be closed by manual means. It should be noted that regular maintenance/inspection as required in most States and Territories in Australia would have been likely to improve the performance considerably.

The mean of the above results is approximately 80%.

In modern air conditioned apartment buildings, SOU doors would normally be in the closed state, so issues such as chocking doors open would be less likely to occur and the doors would be capable of being closed for security reasons. The self-closing function is, however, important to address, as closing the door may be overlooked during emergency evacuation.

Table 10.3: Summary of fire door survey results.

Source	Estimated-Reliability
Guymer and Parry – US Nuclear Industry 1970-80 data	92.6%
BS DD240 – General fire doors	70%
BS DD240 Self-closing door to protected stairwell	90%
Moinuddin and Thomas, Australia – survey of 16 buildings	79%
Moinuddin and Thomas, Australia – smoke door estimate from 6 buildings	>65%
FM study of 1183 swinging fire doors	86%
NZ study – Fire doors	86%
NZ study – Smoke doors	82%
Kettle UK Study – Single doors	66%

It was therefore considered reasonable to assume that the probability of the door to the SOU of fire origin being closed would be 0.9. The same value was adopted for the fire-isolated stair doors and doors/panels providing access to service shafts. Other SOU doors were assumed to have a probability of being closed of 0.95, since the initial state is likely to be closed and therefore they are not as reliant on a self-closing function. The same values are applicable to both the subject and control buildings and are summarised in Table 10.4.

Table 10.4: Probabilities of fire doors being closed.

Door	Probability of Door Closed
SOU of fire origin	0.9
Other SOUs	0.95
Fire stair doors	0.9
All other fire doors	0.9

10.4.2 Visibility and Tenability Criteria

The study is comparative and, because of the large number of variables considered (including human behaviour), it was considered appropriate to adopt the following relatively crude indicative visibility and tenability criteria:

- A temperature rise of 10°C approximates to poor visibility (occupants are assumed not to evacuate through this level of smoke and fire brigade activities will be slower).
- A temperature rise of 60°C will be assumed to represent untenable conditions for occupants. The interface between the upper and lower layers outside the apartment of fire origin was sufficiently low to assume occupants would be exposed to the upper layer temperatures. This limit is below the short term exposure tenability criteria for temperature suggested by Engineers Australia Society of Fire Safety[16] and will inherently address a 2.5kW/m^2 radiant heat flux tenability limit.
- The fire brigade intervention model[17] nominates a limiting heat flux of 4.5kW/m^2 for search and rescue activities. The Society of Fire Safety Practice Note includes refined limits for Fire Brigade Intervention which are reproduced in Table 10.5, together with the classifications adopted in this study.

Table 10.5: Tenability criteria for fire brigade intervention.

Criteria	Routine Condition	Hazardous Condition	Extreme Condition	Critical Condition
Max Time – min	25	10	1	<1
Max Air Temp – °C	100	120	160	>235
Max Radiation (kW/m^2)	1	3	4-5	>10
Grouping for this project	Reasonable		Challenging	

10.4.3 Performance of Closed Fire Doors

There are two broad categories of fire door currently supplied in Australia. One category is based on prototypes tested since the introduction of the cotton pad test to the standard fire resistance test, to determine performance under the criterion of integrity (modern prototypes). The other is based on prototypes tested prior to the introduction of the cotton pad (old prototypes).

The main difference is that the modern prototypes tend to incorporate intumescent strips to retard the spread of hot gases around the perimeter of the door.

The difference in performance of doors with and without intumescent seals was documented by Young and England[18]. The doors were subjected to the AS 1530.4 Standard heating regime and an instrumented corridor was placed in front of the door. For the door without seals, approximately 100% smoke obscuration coincident with a hot layer temperate increase of approximately 30K was measured in the corridor within 2 minutes of the introduction of a positive pressure differential across the door (i.e. the pressure is higher on the fire [furnace] side than the corridor and increases with height due to the buoyancy of the hot gases). For the door with intumescent seals, 100% of smoke obscuration was measured about 15 minutes after the introduction of a positive pressure, coincident with an approximate 30K temperature rise.

The analysis assumes modern prototype doors with intumescent seals are provided, but it should be noted that old prototypes are still deemed to comply within the NCC. The performance of old prototypes will be bracketed between the door open and door closed conditions.

Full-scale enclosure tests were undertaken to compare the performance of non-combustible construction and timber-framed construction in 2011[15]. These tests also incorporated a corridor at the rear of the enclosure separated by the bounding partition and a fire door. Additional data relating to the door/corridor test is reported separately[19]. The doors were modern prototype -/60/30 fire doors fitted with intumescent seals. Typical temperatures measured in the corridor by four trees are shown in Figure 10.2.

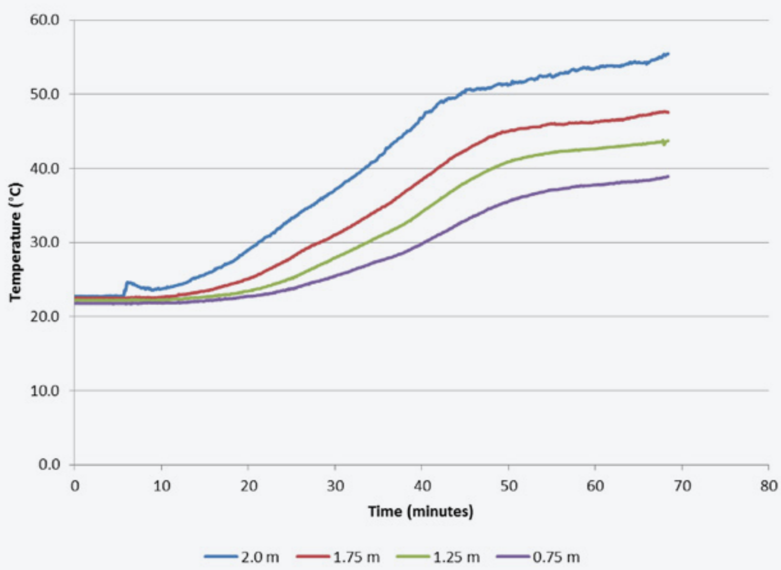


Figure 10.2: Corridor temperatures from comparative natural fire tests (fire doors fitted with intumescent seals).

The mean temperature at a height of 1.75 m did not exceed a 30K rise for 66 minutes when the control test was terminated, and exceeded 30K after about 80 minutes during the timber-framed room test. While this temperature rise correlates with zero visibility (see above) the fire brigade would be able to operate at these modest rises in temperature. It is therefore considered reasonable to assume that the fire brigade would be able to operate safely within the corridor if the door to the apartment of fire origin is closed until failure of the fire doors.

Reasonable levels of visibility will be assumed to be maintained for a scenario time of 30 minutes based on a temperature rise of about 10°C at a height of 1.75 m within the corridors (see Figure 10.2). Until this temperature is exceeded it will be assumed that occupants are capable of navigating the corridor and that fire brigade operations are not hindered by smoke production. Beyond this limit it will be assumed that occupants cannot evacuate through the smoke unassisted.

On the same basis, old prototype doors would be expected to maintain tenable conditions in the corridor for about 2 minutes after flashover but, as noted above, it is assumed modern prototype doors will be used. The use of modern prototype doors will reduce the advantages of the provision of an automatic fire sprinkler system and therefore will yield conservative results in the context of this assessment (lesser performance for timber construction).

As the fire door will tend to have lesser or similar performance than the corridor wall, the combined corridor wall/door performance with respect to fire and smoke spread will be based on the door performance.

10.4.4 Smoke Modelling

To provide approximate estimates of the extent of smoke spread, typical scenarios were modelled using the CFAST Version 6[20]. It was assumed that the fire floor was two levels from the top of the building and, where appropriate, smoke spread via the shafts was modelled to the upper level corridor and subsequently to an upper level of apartments. For the door open scenario, the door to the apartment of fire origin was opened after 3 minutes, which approximated to flashover, simulating a last minute evacuation and failure of the automatic door closing device that is required to be fitted to the door in accordance with the NCC Deemed-to-Satisfy Provisions.

Smoke spread estimates through closed doors were based on the experimental data described in Section 10.4.3. Table 10.6 summarises the results for baseline critical times obtained for various enclosures.

Table 10.6: Baseline critical times for visibility and tenability.

Time to Exceed Limits – min			
Fire Floor Corridor Visibility	**Fire Floor Non-Fire (NF) SOU Tenability**	**Stair Visibility**	**Upper Level SOU Tenability**
SOU Fire Origin Door Open – Visibility 0.5	NF SOU Door Open Tenability 1	Stair Door Open Visibility more than 2.5	SOU Door Open Tenability more than 120
	NF SOU Door Closed Tenability more than 60	Stair Door Closed Visibility more than 10	SOU door Closed Tenability more than 120
SOU Fire Origin Door Closed – Visibility 30	NF SOU Door Open Tenability more than 60	Stair Door Open Visibility more than 60	SOU Door open Tenability more than 12
	SOU Door Closed Tenability 120	Stair Door Closed Visibility more than 69	SOU door closed Tenability more than 120

Fire Brigade conditions were considered reasonable for all areas except on the floor of fire origin if the SOU of fire origin door is open or has failed, in which case the conditions were considered challenging. Since the study was comparative and the base smoke spread would be the same for all the generic buildings, it was considered unnecessary to undertake more detailed smoke modelling.

10.5 Detection and Alarm Model

10.5.1 Overview of Derivation of Inputs

Since the focus of the detailed analysis was fully developed fires, the treatment of the pre-flashover phase and determination of alarm times could therefore be relatively simple, provided reasonable estimates of the fire brigade alarm times and commencement of evacuation could be made relative to the occurrence of flashover.

Large variations in the rate of initial fire growth and detection and alarm times occur with the impact of human behaviour, further increasing the variability particularly in occupancies such as Class 2, where emergency management structures are limited. To address this variability, probabilistic distributions of alarm and response times were derived and used in the analysis, rather than allocating a specific time to an event.

The alarm time for occupants was consolidated into the occupant response and evacuation model for the purposes of this study to allow for reinforcement from secondary cues. Reference should be made to Section 10.8.

The derivation of the distributions for fire brigade alarm times relative to the start of the parametric heating regime (rapid growth phase) is detailed in Section 10.5.2 and Section 10.5.3.

Rectangular alarm time distributions were assumed for the fire brigade intervention model as shown in Figure 10.3 and Table 10.7. The inputs vary between Class 2 and some Class 3 buildings, due to the presence of monitored detection/alarm systems.

Table 10.7: Fire brigade alarm time distributions.

Fire Detection/Alarm System Monitoring Status	Time relative to start of parametric heating regime	
	Minimum – minutes	**Maximum – minutes**
Not monitored	0	4
Monitored	-10	2

In the majority of scenarios, the transition to a fully developed fire (flashover) occurs within two minutes of the start of the parametric heating regime, if it is assumed an enclosure temperature of 600°C corresponds to flashover.

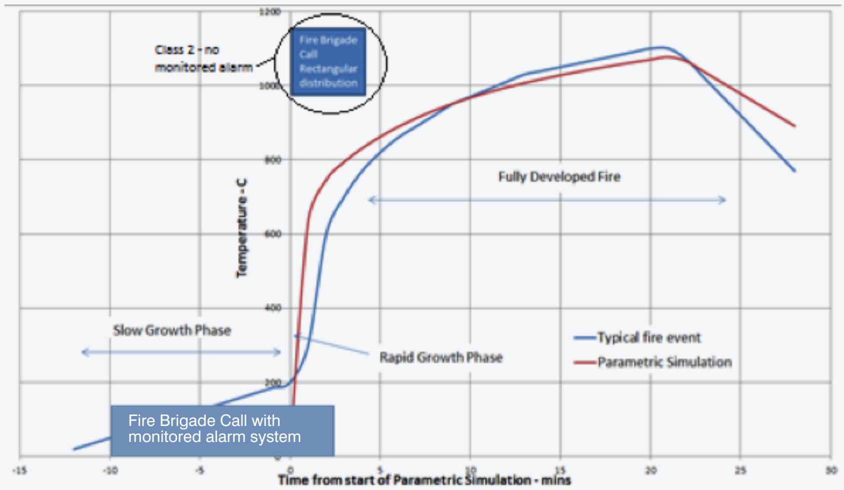


Figure 10.3: Distributions for fire brigade alarm times

10.5.2 Derivation of Class 2 Fire Brigade Alarm

The control Deemed-to-Satisfy Class 2 building did not have a direct connection to a monitoring service and therefore fire brigade notification depended on a call from an occupant or member of the general public.

While the subject timber buildings were provided with automatic fire sprinkler systems connected to a monitoring service, the sprinkler systems would be likely to have failed if a flashover fire occurred. In many instances, if the sprinkler system fails to control a fire due to isolation of part or all of the system, it is likely that the sprinkler system would not initiate an alarm. Therefore, for the purposes of considering fire brigade intervention for fully developed fire scenarios, it was assumed that the fire brigade would be alerted by means of a call from an occupant or member of the general public.

Considering a typical fire scenario after ignition, in many instances there is a slow growth phase (sometimes referred to as the incipient phase) while the fire becomes established. This is followed by more rapid growth, often simulated by a t-squared fire, until flashover – when the fire transitions to a fully developed fire, as shown in Figure 10.3.

The fire brigade could be alerted at any stage after ignition depending upon (among other things):

- the presence of occupants in close proximity to the fire (e.g. within the SOU of fire origin)
- the status of the occupants close to the fire
- response to alarms and fire cues
- the perception of risk associated with the fire.

As the fire approaches flashover, the cues would become stronger and evident to other occupants and passers-by, significantly increasing the probability of an emergency call being made.

Due to the variability of the fire during the establishment phase and the variability of human behaviour, it was conservatively assumed that no alarm calls would be made during this stage and a rectangular distribution of alarm calls over a four-minute period from the start of the parametric simulation was assumed. During this period, large numbers of occupants, as well as potential passers-by, could receive clear unambiguous cues such as flames exiting windows, glass breaking, or large volumes of smoke being produced, and therefore the probability of at least one person making a call to the fire brigade would be high.

10.5.3 Derivation of Class 3 Fire Brigade Alarm

For Class 3 buildings, the detection and alarm system is required to be monitored and therefore if the detection and alarm system operates successfully the fire brigade will be alerted prior to flashover. How much earlier depends on a large range of variables. In extreme circumstances, flashover could occur within 2–3 minutes of the alarm or it could take several hours.

A rectangular distribution commencing 10 minutes prior to the parametric heating curve and ending two minutes after the start of the parametric curve was adopted for the time at which the alarm will be received by the fire brigade, as shown in Figure 10.3. The 2-minute section after the start of the parametric curve has been included to allow for failures of the detection/alarm system, where reliance is on the occupants to raise the alarm (i.e. assumed failure rate of approximately 17%).

With an operational detection, alarm and monitoring system and very long slow growth periods, intervention is likely to occur before flashover and, since the analysis is focused on potential flashover fires, these scenarios are not critical to the study.

10.6 Enclosure/Structural Model

The enclosure/structural multi-scenario model implemented for the mid-rise building comparative analysis is shown in Figure 10.4 and included the following sub-models:

- Fully developed enclosure fire sub-model
- FRL (distribution) sub-model
- FRL conversion sub-model
- Simple structural sub-model

The fire brigade suppression time is obtained from the fire brigade intervention model which is described in Section 10.7.

Outputs from the model include localised failure times and major (global) structural failure times, which are used as inputs to the Occupant Response Evacuation and Consolidation Model.

The major structural failure time can also be used in isolation for comparison of strategies in some applications.

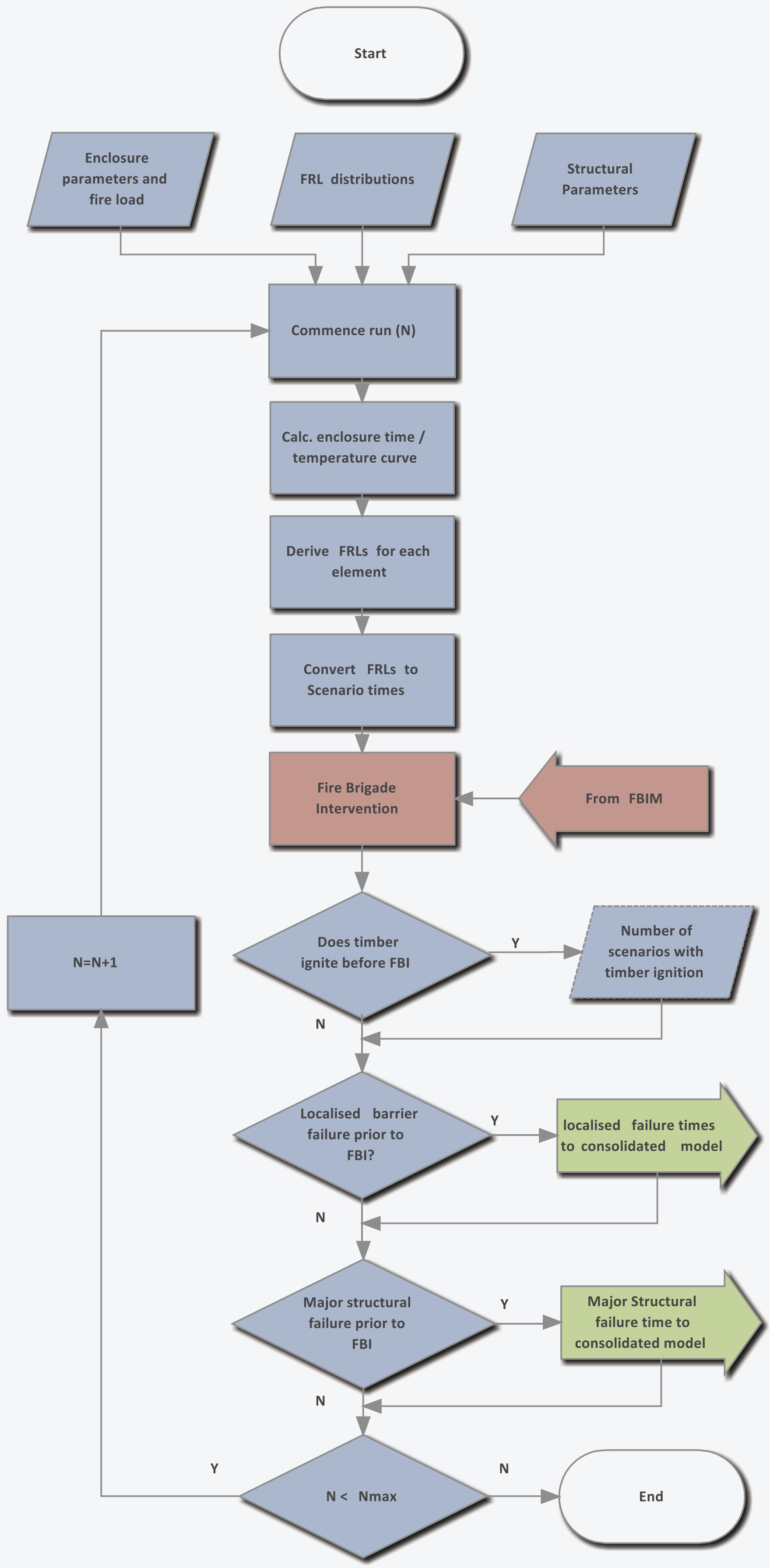
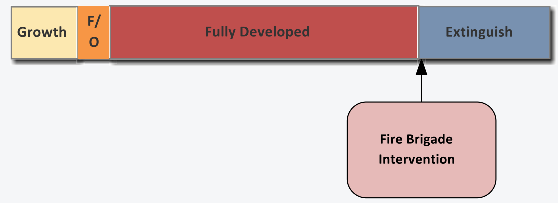


Figure 10.4: Enclosure/structural model.

Typical examples of scenarios are shown on the timelines in Figure 10.5. The outcomes depend on the relative fire severity/duration, timing of fire brigade intervention and the efficacy of the fire-protective coverings/inherent fire resistance of the elements of construction.

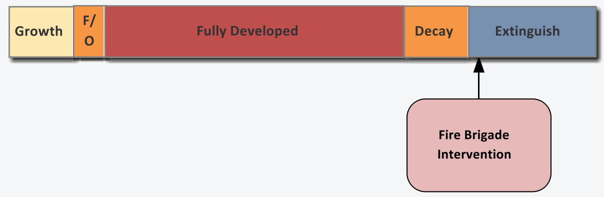
Example 1

Prompt Fire Brigade Intervention before ignition of timber or barrier failure



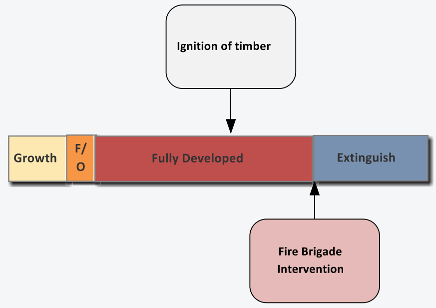
Example 2

Low duration / severity fire relative to protection of elements. Fire decays prior to fire brigade intervention



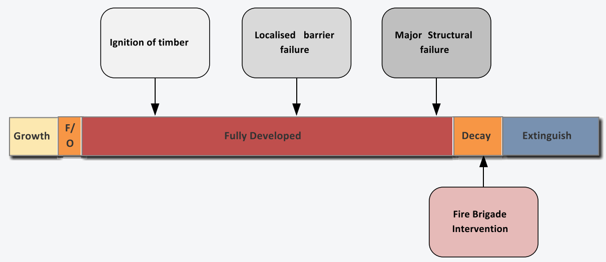
Example 3

Fire Brigade Intervention after ignition of timber element but before barrier failure.



Example 4

No fire Brigade Intervention prior to major structural collapse usually associated with two elements having gross defects causing premature failure.



Note: F/O - Flashover

Figure 10.5: Example abridged timelines for enclosure/structural model.

10.6.1 Fully Developed Enclosure Fire Sub-model

There are numerous closed form models that can be used to generate time/temperature regimes for post-flashover (fully developed) compartment fires based on fuel load, ventilation and thermal properties of boundaries; many of which have been reviewed by Hurley[21].

The method presented in Annex A of EN 1991-1-2:2002[22] was selected because it has also been codified and used extensively.

The method adopts the following equation to define a heating regime based on variables such as thermal properties of the boundary, ventilation conditions, enclosure dimensions and fire load.

$$\theta_g = 20 + 1{,}325\,(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*})$$

A linear relationship is assumed for the cooling phase.

Refer to EN 1991-1-2:2002 for further details of the calculation method.

This model can be easily incorporated in a spreadsheet to run a multi-scenario analysis. Typical results are shown in Figure 10.8, with the standard fire resistance test and hydrocarbon heating regimes from AS 1530.4. The derivation of inputs is summarised below.

Fire load

A study was undertaken into fire loads and design fires for mid-rise buildings by Ocran[23] in 2012, providing the most relevant input data for this study since it supersedes an earlier study by Apte et al.[9] and it relates directly to mid-rise buildings.

Table 10.8 is a summary of Ocran's findings in relation to residential buildings

Table 10.8: Typical fire loads for residential buildings from Ocran[23].

	Fire Load Density (MJ/m²)		
Description	**Mean**	**95th Percentile**	**Maximum**
Residential buildings	370-550 (per room)	-	-
Living room	288-600	450-790	633-1700
Bedroom	534-944	712-846	738-1000
Dining room	393	576	901
Kitchens	807	940	1244

A range of fire load densities for residential occupancies is specified in guides/verification methods typically varying from 400 MJ/m² in the Verification Method: Framework for Fire Safety Design for New Zealand Building Code[24] to 780 MJ/m² in Eurocode 1 Parts 1-2[22].

To address the variability of fire load data, the analysis was undertaken using a fire load of 500 MJ/m² with a standard deviation of 150 MJ/m² supplemented by sensitivity analysis with distributions around mean values of 300 MJ/m² and 780 MJ/m², as shown in Table 10.9.

Table 10.9: Fire load distributions for apartment buildings.

Fire Loads	Fire Load MJ/m²	Standard Deviation MJ/m²	95 percentile MJ/m²	Min MJ/m²	Max MJ/m²
Low sensitivity	300	90	448	100	unlimited
Design value	500	150	747	200	unlimited
High sensitivity	780	115	970	200	unlimited

Note: If ignition of fire-protected timber was predicted, the fire load was increased to allow for a contribution to the fire load from the timber elements as detailed below:

- timber-framed construction – design fire load increased by 500 MJ/m²
- massive timber – total fire load of 2500 MJ/m² assumed.

Further information on the derivation of these values in included in Appendix G1 Contribution of timber elements to fire load.

Floor area

Ocran also reviewed literature to characterise typical room sizes, from which Figure 10.6 has been extracted. For the analysis, room floor areas were generated from the following uniform distributions for room width and length:

- width: 2.5 m to 5 m
- length 2.5 m to 8 m.

A typical distribution generated from these inputs is shown in Figure 10.7. It has the same general form as the results reported by Ocran (shown in Figure 10.6), but with a slight offset to larger room sizes. As the distributions for this analysis were selected to reflect modern building layouts – where kitchen lounge and study areas are often combined – this offset is expected and therefore the derived distribution is considered appropriate.

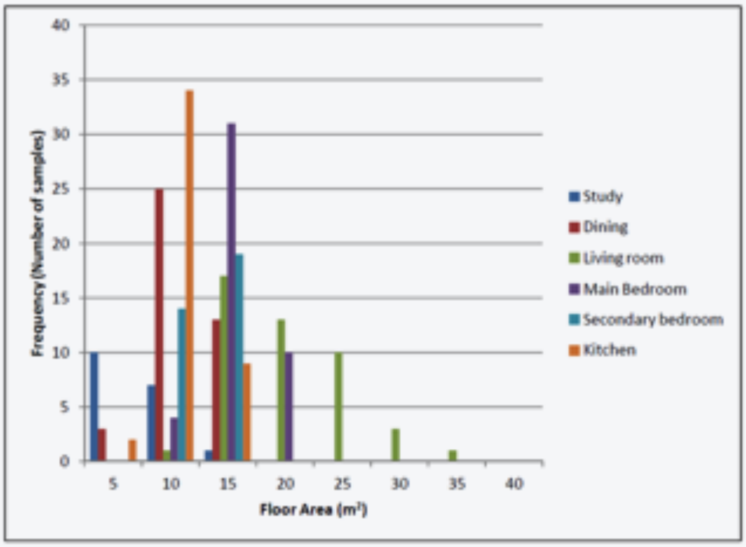


Figure 10.6: Frequency of floor areas in mid-rise residential buildings from Ocran[23].

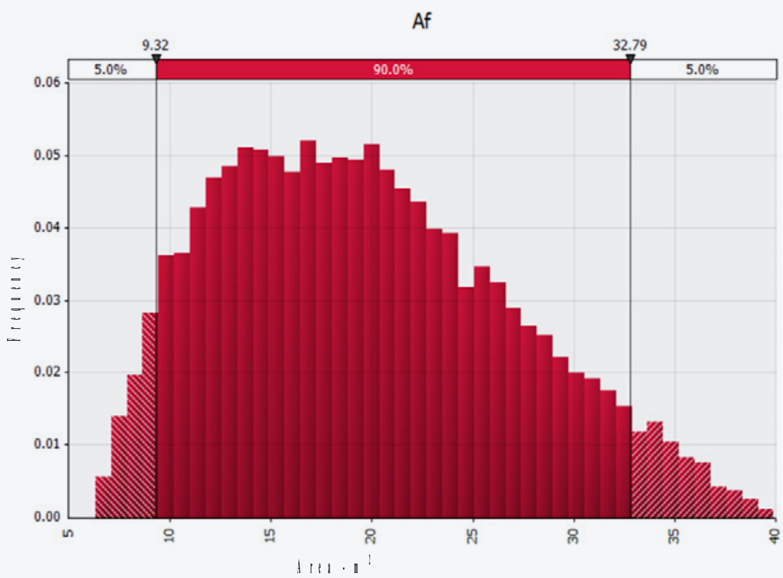


Figure 10.7: Floor area distribution derived for analysis of apartment buildings from uniform distributions for linear dimensions.

There are two approaches to defining the enclosure size to calculate the severity of a fully developed fire within an SOU. One approach is to assume no interior compartmentation and model the SOU as a single space; the other is to assume the interior walls within the SOU (some of which may not be required to be fire-resisting) remain intact and model the SOU as a series of rooms. Initially, and for a significant period post-flashover, the interior walls within the SOU would effectively subdivide the SOU; and therefore the latter approach was adopted and enclosure dimensions were based on a distribution of typical room sizes varying in area from 6.25 m^2 to 40 m^2. Since the study was comparative and a wide distribution of room geometries were considered, this assumption was considered reasonable.

Room height

A typical room height of 2.4 m was assumed to be representative for Class 2 and 3 buildings where floor to floor heights are optimised.

Opening area

The NCC Deemed-to-Satisfy requirements specify a minimum area of natural light of $0.1xA_f$ where A_f is the floor area[25]. This would therefore be a reasonable lower bound ventilation area for an apartment room. Energy Efficiency requirements are introducing practical limits to window areas in many jurisdictions in Australia. Maximum window sizes of 41% of the floor area were derived in an analysis of the impact of increasing the regulatory requirements relating to energy efficiency of buildings in 2009[25]. Based on this information, a uniform (rectangular) distribution of openings areas from 10% to 41% of the floor area was assumed.

Ocran reported the measurement of window sizes taken during a survey of multifamily dwellings yielded an average of 3.1 m^2, a minimum of 1.1 m^2, and a maximum of 8.0 m^2 for living rooms. If it is assumed that the maximum window size relates to the largest room size, the maximum opening area would be about 23% of the floor area. If this is applied to a more frequent room area (20 m^2) (refer Figure 10.7), the maximum opening area would be about 40% of the floor area. The minimum window size (1.1 m^2) applied to a small room area (10 m2) represents 11% of the floor area. These results are therefore considered reasonably consistent with an assumed uniform (rectangular) distribution of opening areas from 10% to 41% of the floor area.

Opening Height

A uniform distribution varying from 0.3 m to 2.1 m was assumed for opening heights to address the range used in contemporary buildings.

Lining properties

The following lining properties were used for the enclosure boundaries:

- thickness – 26mm
- thermal conductivity – 0.27 W/m.K
- density 900kg/m^3
- heat capacity 2000 J/kg.K (allows for combined water).

Pre-flashover growth rate

A fast pre-flashover fire growth rate was assumed to account for the impact of contents such as upholstered furniture, mattresses, etc; however, as the fires tended to be ventilation controlled this assumption was not critical.

Typical outputs

Typical outputs from the fully developed fire enclosure model are shown in Figure 10.8, compared to the standard and hydrocarbon AS 1530.4 heating regimes.

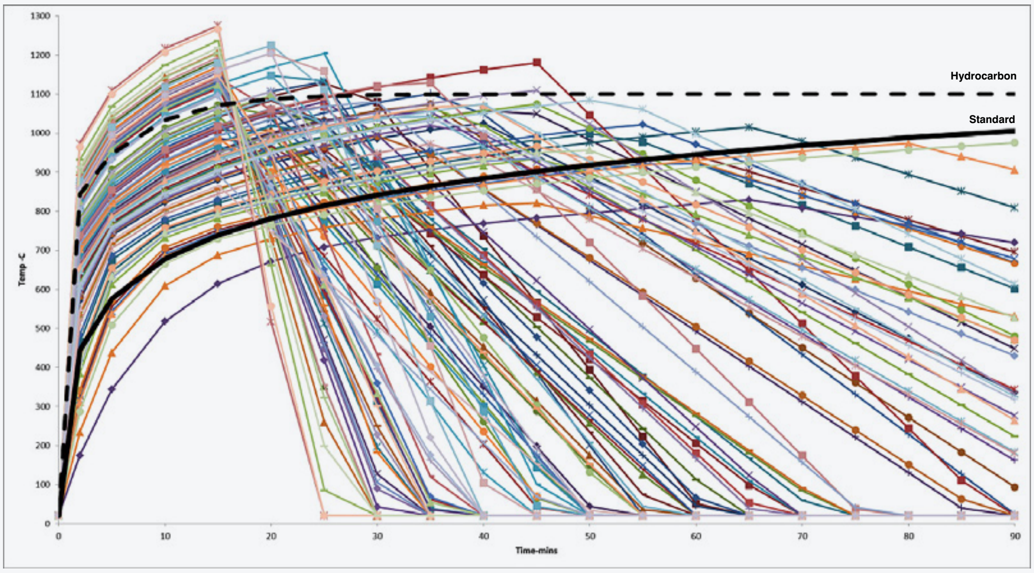


Figure 10.8: Typical enclosure temperatures for design value case (500 MJ/m²) compared to standard and hydrocarbon heating regimes.

The methods described below were used to equate the exposure to the calculated enclosure temperatures to the standard heating regime. For the 500 MJ/m² case, the exposure from about 97% of scenarios was less than or equal to the equivalent of a 90-minute standard fire resistance test, and the exposure from 75% of the scenarios was less than or equal to the equivalent of a 60-minute standard fire resistance test.

These results can be considered to be generally consistent with the current NCC Deemed-to-Satisfy Provisions for mid-rise Class 2 and 3 buildings, which generally require 90/90/90 FRLs for loadbearing elements and -/60/60 for non-loadbearing elements.

Modelling contribution of fire-protected timber to the fire load

If the relevant RISF or MRISF temperature criteria of 250°C and 300°C, respectively, were not exceeded, it was assumed that there would be no increase in fire severity resulting from the use of fire-protected timber. The validity of this assumption was demonstrated in the test described in Appendix A4.

An initial Monte Carlo simulation run was undertaken to determine the proportion of scenarios where the fire is suppressed or burns out prior to failure of a timber-framed element.

For the proportion of scenarios where the timber ignites, the following approaches were adopted:

- For timber-framed construction the fire load will be increased by 500MJ/m².
- For massive timber panels the total fire load was assumed to be 2500 MJ/m².

Details of the basis for the selection of this approach are provided in Appendix G1.

10.6.2 FRL (Distribution) Sub-model

A two-peak distribution was adopted for the FRL of elements of construction.

The primary peak is centred on the nominated/typical FRL for the element with a standard deviation of 10% of the nominated FRL. The secondary peak is centred on the performance, assuming a gross defect is present, with a standard deviation of 10% of the estimated FRL of an element with a gross defect. The probability of a gross defect occurring in a single element was assumed to be 0.005.

The FRLs shown in Table 10.10 were assumed for the structural elements.

Table 10.10: Mean FRLs adopted for elements with gross defects.

Case	Mean FRL – Struct. Ad/Integrity/Insulation – min			
	Loadbearing		Non Loadbearing	
	No defect	Major defect	No defect	Major defect
Control (protected steel)	90/90/90	26/26/26	-/60/60	-/22/22
Timber frame	90/90/90	22/22/22	-/60/60	-/22/22
Massive timber	90/90/90	60/60/60	-/75/75	-/60/60

Due to the high inherent fire resistance of massive timber panels the impact of major defects such as substitution of non-fire-protective coverings has a lesser impact on the fire resistance. Further details of the derivation of the above FRLs is provided in Appendix G2.

10.6.3 Fire Resistance Levels to Scenario Time Conversion Model

In most instances, the time to failure of an element of construction ascertained in a standard fire resistance test will differ from the failure time if the element is exposed to a real or simulated fire scenario (e.g. Annex A of EN 1991-1-2:2002), because the time temperature histories will differ (as shown in Figure 10.8).

A method based on the Equal Steel Temperature concept was developed in which a "target protected steel element" with specified thermal properties was defined and the time temperature history calculated at a critical point based on exposure to the fire scenarios and the standard heating regime. Equivalent exposure was deemed to have occurred when the critical part of the element reaches the same temperature under the different heating regimes.

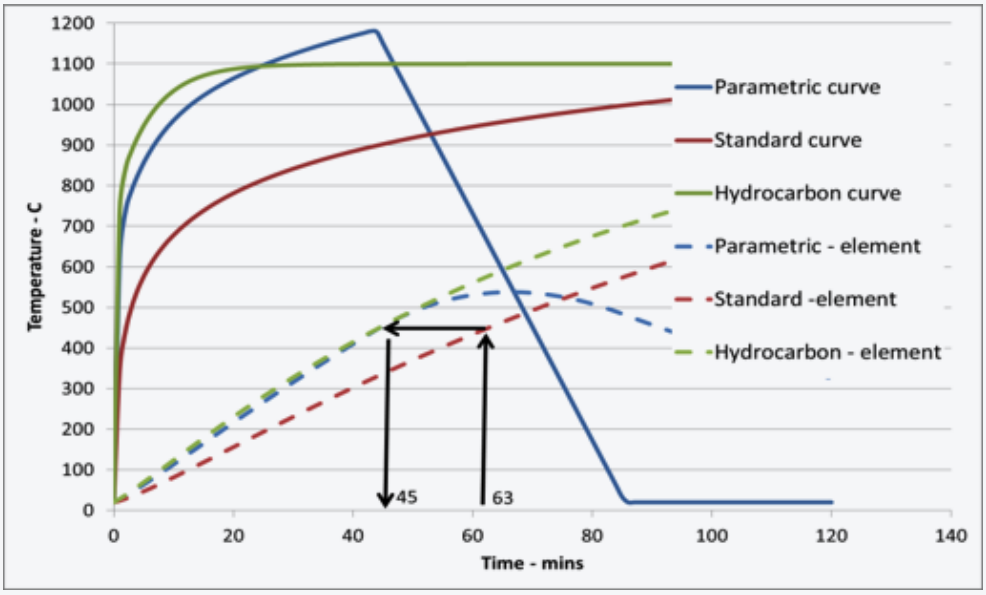


Figure 10.9: Conversion of fire resistance period to fire scenario time.

The process is shown graphically in Figure 10.9. If it is required to determine the time to failure of an element that achieved an FRL of 63/-/- when exposed to the fire scenario (parametric curve) fire, the following approach is adopted:

- The target element attains a temperature of 454°C when exposed to the standard fire resistance test for 63 minutes.
- The target element would need to be exposed to the fire scenario for 45 minutes to attain the same temperature.
- Therefore, the fire scenario failure time would be 45 minutes.

Further details of the method, selection of target element and comparison of predictions with experimental data are provided in Appendix G3.

10.6.4 Simple Structural Model

To evaluate the risk of a major collapse of a structure or part of the structure, it is first necessary to determine which structural elements or combinations of structural elements may initiate a major collapse. For medium and high-rise buildings, many design codes require robust designs to address the risk of disproportionate collapse, which can be achieved by incorporating redundancy in the design such that for collapse to occur more than one key structural element needs to fail.

When considering collapse of a structure exposed to fire, the potential for defects to cause premature failures needs to be considered. However, the probabilities of more than one member having a defect may be sufficiently low that no additional special measures may be required. In other words, the risk associated with defects needs to be evaluated – even if protection to structural members is specified to resist full burnout of a fire.

NCC performance requirement BP1.1 states:

"(a) A building or structure, during construction and use, with appropriate degrees of reliability,

must –

(i) perform adequately under all reasonably expected design actions; and

(ii) withstand extreme or frequently repeated design actions; and

(iii) be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; and

(iv) avoid causing damage to other properties, by resisting the actions to which it may reasonably expect to be subjected."

It was therefore considered reasonable to assume that the structure of the control and subject buildings will be designed in accordance with these provisions and that a level of redundancy will be provided in the design, such that for collapse to occur more than one key structural element needs to fail.

Simple structural layouts for timber and steel versions of the generic apartment building are shown in Figure 10.10 and Figure 10.11.

By considering fire-resisting loadbearing walls as a series of segments and assuming any supporting beams are incorporated in floor elements, a typical timber building apartment structure can be simplified to six structural wall elements and three structural floor elements.

When considering structural adequacy, a typical concrete and steel masonry building apartment can be represented as six columns and three floor units.

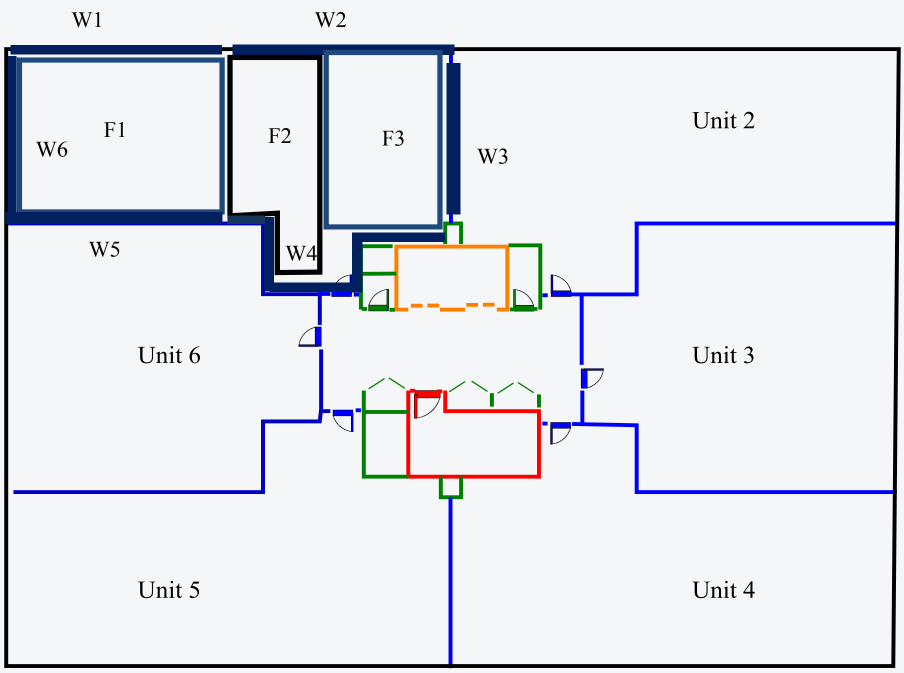


Figure 10.10: Schematic showing simple structural layout for an apartment in a timber building.

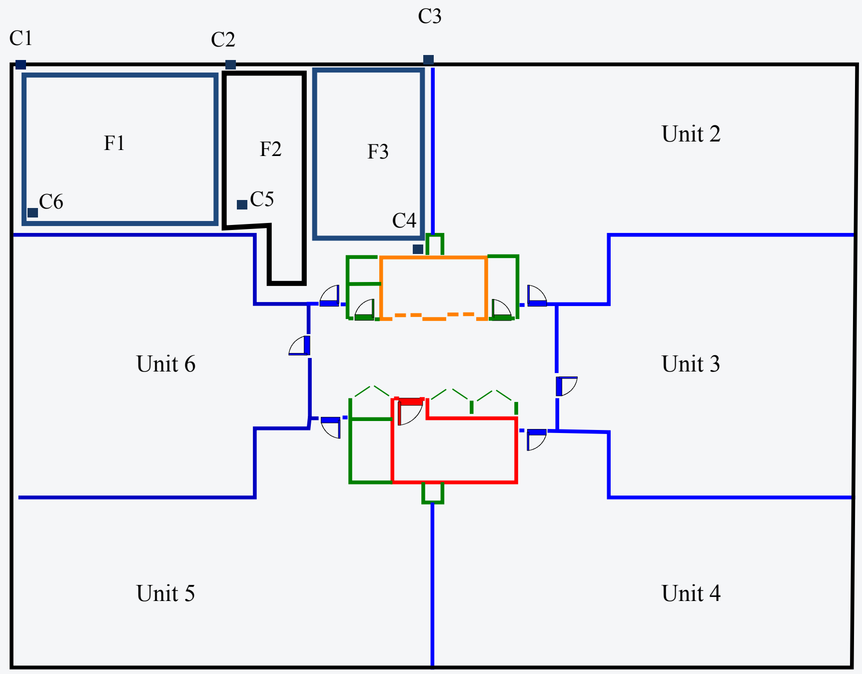


Figure 10.11: Schematic showing simple structural layout of an apartment in a steel building.

For a major structural failure to occur, it was assumed at least two members are required to fail from the nine structural members. This simplification is considered appropriate for a generic comparative analysis but, for specific buildings, a more detailed structural analysis may be appropriate.

10.7 Fire Brigade Intervention Modelling

Fire brigade intervention is an important part of the analysis since it can influence:

- fire duration/fire severity (fire-fighting activity)
- building evacuation time (search and rescue activities).

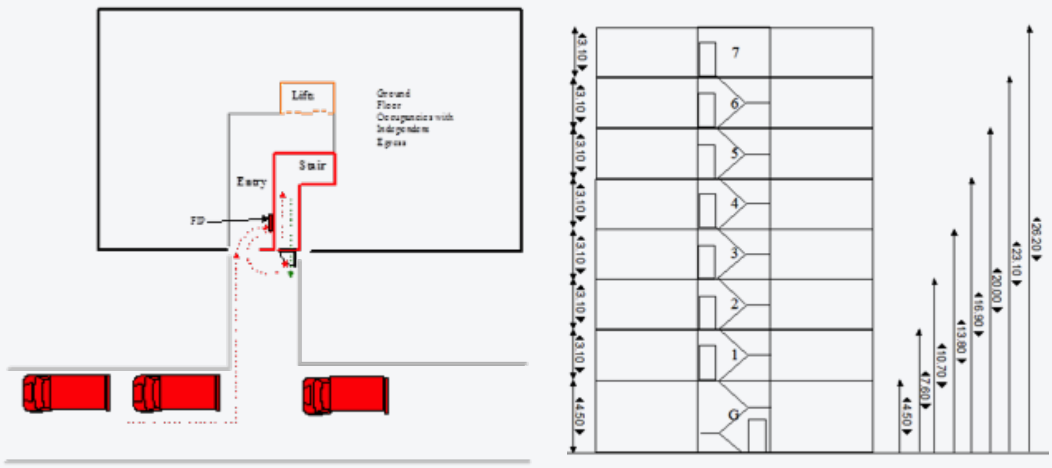
Distributions of the time to commencement of fire-fighting activities were derived as part of the Monte Carlo simulations and input into the enclosure/structural model for each simulation to determine:

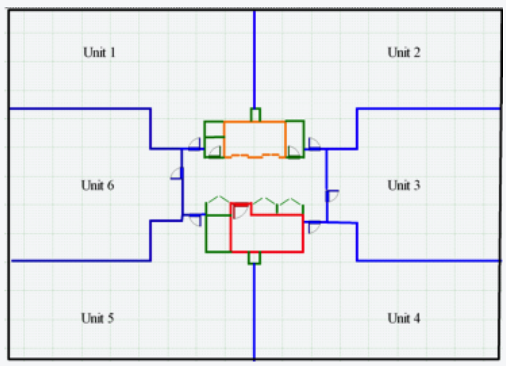
- if fire brigade intervention occurs prior to a major structural failure, if burnout has not already occurred
- if fire brigade intervention occurs prior to failure of the compartmentation, if burnout has not already occurred
- if the fire-protected timber members have ignited beneath the fire-protective coverings, if burnout has not already occurred.

To model search and rescue activities, it was necessary to integrate some aspects of the fire brigade intervention model with the occupant response, evacuation and consolidation model.

10.7.1 Building Layout Features for Fire Brigade Intervention Model

The building layout details that were used for the fire brigade intervention modelling of the generic mid-rise Class 2 or 3 building are shown in Figure 10.12.





Travel Distances

Appliance to main door - **20m**

Main door to FIP - **5m**

FIP to base of stair -**12m**

Steps:

Ground to L1 - **26**

All other Levels **18 steps** plus **4m** horizontal / floor

Travel from stair to Apartment of Fire Origin door - **6m**

Figure 10.12: Building layout and critical dimensions for fire brigade intervention modelling.

10.7.2 Fire Brigade Intervention Model Overview and Inputs

The fire brigade intervention model was adapted from the fire brigade intervention model (FBIM)[17] developed by AFAC but modified to facilitate Monte Carlo simulations as part of a model incorporating fire severity and structural performance. Other modifications were made based on further verification of the FBIM model undertaken by Claridge[27] and to base the response times on data from the 2014 Report of Government Services[28].

Key inputs are summarised in Table 10.11, which also references the source for the input.

Refer to Appendix G4: Verification of Stair Climbing Component within the implementation of the fire brigade intervention model used for comparison of predicted stair climbing times against international studies.

Table 10.11: Summary of fire brigade intervention model inputs.

Description	Input Type	Values	Comments
First alarm to call centre (Class 2)	Rect. Dist.	0 to 240s	Time relative to parametric curve (no alarm monitoring). From Table 10.7
First alarm to call centre (Class 3)	Rect. Dist.	-1200 to 120s	Time relative to parametric curve (detection alarm monitoring) From Table 10.7
Time for receipt of information	Fixed	60s	From FBIM Table C
Time taken to dispatch resources	Fixed	0s	Published data based on time of call used to establish fire brigade response time so already included in time to reach curb side
Time to reach curb side	Truncated Log Normal	Mean 500s SD 230s Min 180s	Derived from the average reported response times for each state in the 2014 Report of Government Services of 7.65mins (50 percentile) and 13.33 mins 90 (percentile) with allowance for note taking time added to South Australian Figures
Time to don BA	Truncated normal dist.	Mean 88s SD 34.1 Min 44	From FBIM Table M
Pick up forced entry tools	Truncated normal dist.	Mean 25s SD 13 Min 13	From FBIM Table P Critical path forced entry tools selected because longer time than high-rise pack. SD Estimated.
Open door with master key	Fixed	10s	From FBIM Table J
Check FIP and to resolve way finding	Fixed	60s	From FBIM Table L and Table K
Initial OIC actions			Coincide with above
Set up additional water supply			Undertaken while other activities underway
Walking speed – horizontal	Truncated Log Normal distribution	Mean 1.39m/s SD 0.57 m/s Min 0.28 m/s Max 3.3m/s	Based on FBIM Graph Q3 full turnout with BA
Walking speed – horizontal through smoke	Truncated Log Normal distribution	Mean 0.7/s SD 0.3 m/s Min 0.14 m/s Max 1.66m/s	Half values without smoke
Stair climbing levels 0 – 10	Normal distribution	Mean 1.3 steps/s SD 0.2 steps/s Min 0.43 steps/s Max 1.68 steps/s	Based on Claridge with some adjustments
Stair climbing levels 10 – 20	Normal Distribution	Mean 500s SD 230s Min 180s	Based on Claridge with some adjustments
Stair climbing levels 10 – 20	Normal Distribution	Mean 500s SD 230s Min 180s	Based on Claridge with some adjustments
Stair climbing levels 20+	Normal Distribution	Mean 1 step/s SD 0.25 steps/s Min 0.5 steps/s Max 1.4 steps/s	Based on Claridge with some adjustments
Rest and recovery period L6 to 10	Rectangular for N>6	Min 0 Max 15(N-6)s	Not applicable below 6 levels N= number of floors
Hindrance factor	Factor	50% increase in travel time to set up position	FBIM Table S Hindrance caused by occupants evacuating to movement of fire fighters. Since the number of occupants/stair is relatively small (limited by travel distance to stairs and building height) the risk of hindrance is low; however, the fire fighter travel time within the stairs has been increased by 50% to account for potential delays
Set up hose 1	Truncated log Normal	Mean 40.9s SD 17.7s Min 14s Max 90s	Level below fire floor FBIM Table v5.2
Force open door to SOU of fire origin	Fixed	30s	FBIM Table I

10.7.3 Fire Brigade Intervention Model Time to Water Application Outputs

Typical times to application of water relative to the start of the parametric fully developed fire scenarios, without and with monitored detection and alarm systems predicted by the modelling, are shown in Figure 10.13 and Figure 10.14, respectively.

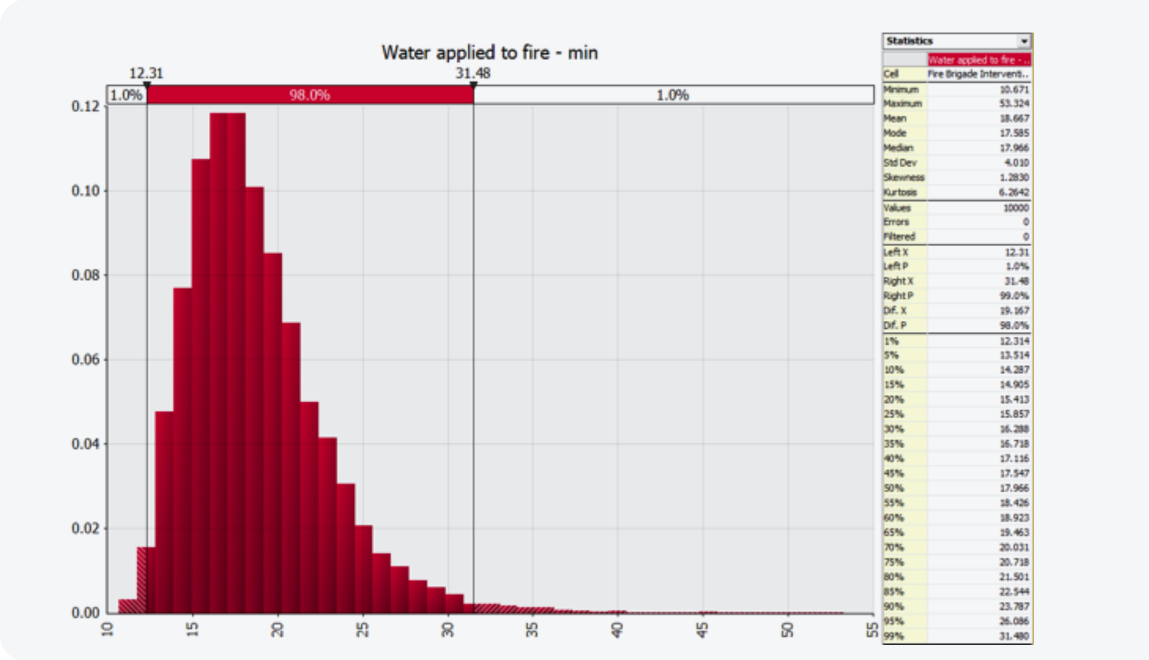


Figure 10.13: Distribution of time to application of water for fire on Level 5 without a monitored detection and alarm system.

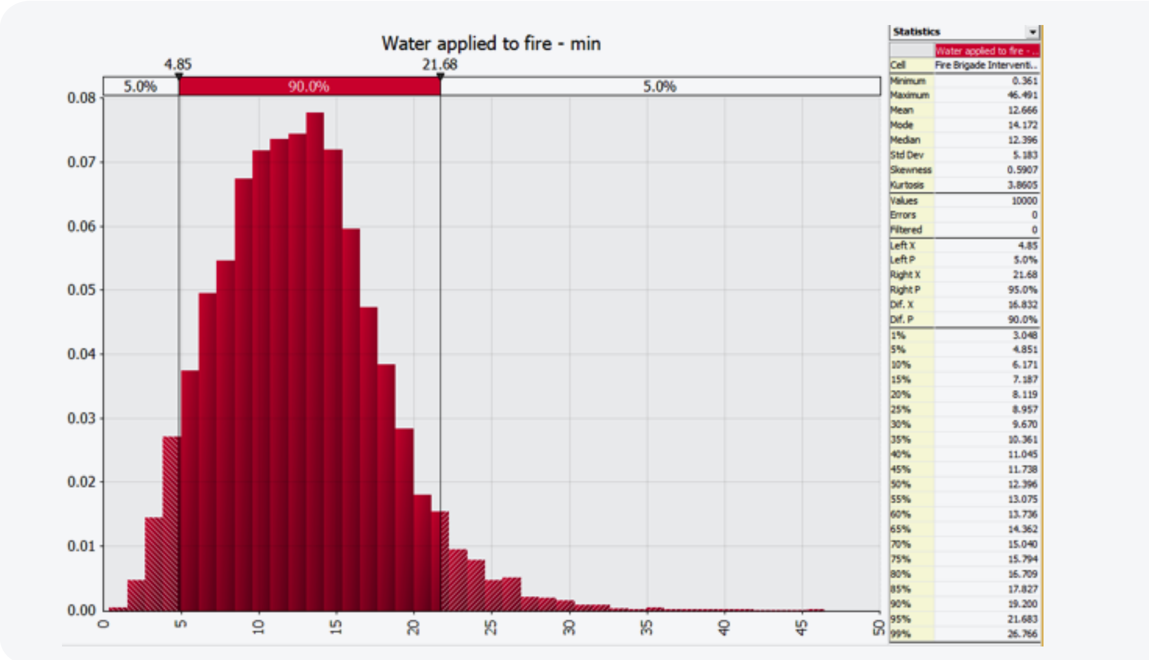


Figure 10.14: distribution of time to application of water for fire on Level 5 with a monitored detection and alarm system.

10.7.4 Fire Brigade Search and Rescue Activities Class 2 and Type 1 Class 3 Occupancies

Coincident with attacking the fire, other fire service teams would be undertaking search and rescue activities. The priorities for these activities would vary, depending on the specific circumstances of a fire event. Therefore a generalised approach has been adopted for this analysis, to enable the activities to be integrated into the occupant behaviour, evacuation and tenability sub-model described below, facilitating the analysis of the building design options as part of a stochastic analysis. The approaches taken and allocated times are shown in Figure 10.15.

The occupant behaviour, evacuation and tenability sub-model establishes tenability conditions in the building for various combinations of door open/closed states, which are then modified based on the behaviour of fire-resistant elements of construction exposed to the fully developed fire. Evacuation of occupants is modelled in a stochastic manner varying with time and a proportion of occupants are assumed to be unable to evacuate without assistance.

Also, it is assumed that if occupants encounter smoke they return to their apartment and wait for assistance.

Occupants waiting for assistance are considered to be evacuated when the search and rescue activities for a specific floor are completed. Fixed times of 800s for the fire floor and 360s for other floors have been assumed, which were calculated on the following basis:

- search of a 90m^2 smoke-filled SOU on the fire floor: $90/0.16 \approx 560$s (using FBIM mean value for searching a smoke-filled room)
- search of the remaining 4 SOUs on the fire floor, which were assumed to be clear of smoke but required doors breaking open to check they were unoccupied (an allowance of 30s for forced entry plus 30s for checking each apartment was made), i.e. 240s for four SOUs
- each of the other floors contains 6 apartments (i.e. approx. 360s).

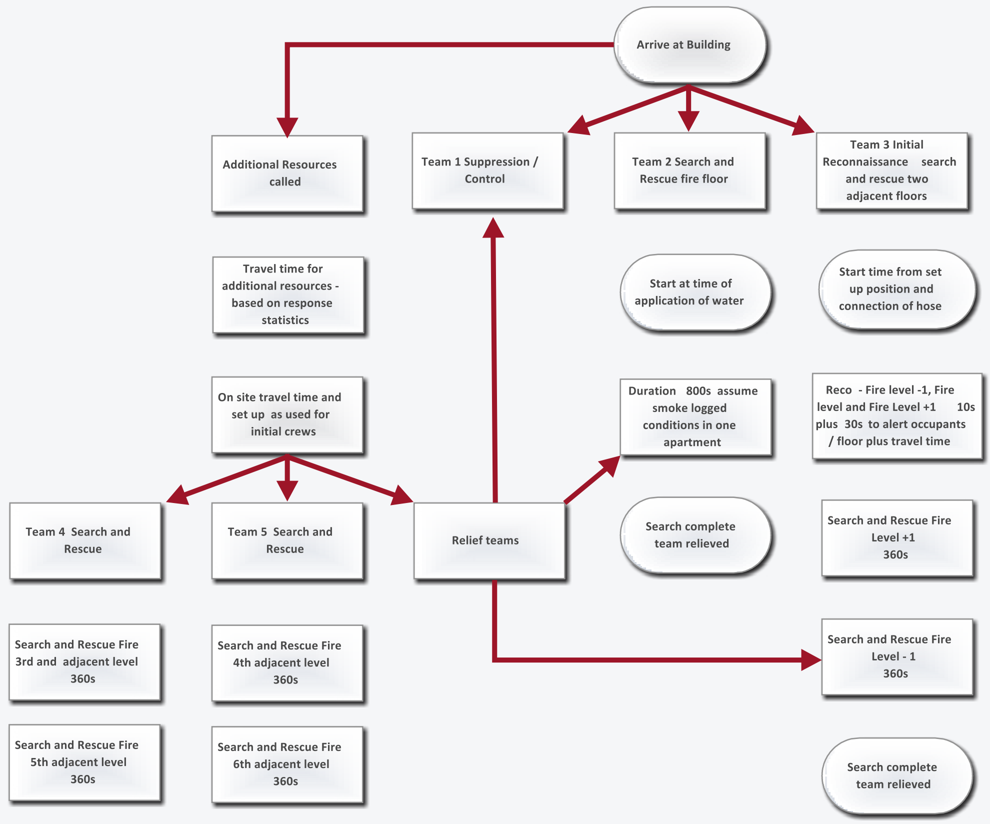


Figure 10.15: Fire brigade search and rescue flow chart Class 2 buildings.

10.7.5 Fire Brigade Search and Rescue Activities Type 2 Class 3 Occupancies

For the Class 3 building with Type 2 occupants, the situation facing fire fighters would be different to a Class 2 building. There would normally be an emergency management structure in place and doors would be unlocked. Staff would be able to inform fire fighters of occupants remaining in the buildings, so activities are concentrated more on rescue than evacuation. It will be assumed that Teams 2 and 3 focus on the floor of fire origin rescuing the four remaining residents of the group requiring assistance on that floor.

FBIM specifies an average speed of 0.05 m/s through smoke, which has been adopted for this analysis. A travel distance of 15 m to the stair is assumed (approx. 6 m between the SOU and stair and 9 m within the SOU) which equates to 300 s. It will be assumed that one fire fighter can assist one occupant at this speed (i.e. a team of three can evacuate two occupants simultaneously with the third fire fighter on a hose).

The allocation of resources is shown in Figure 10.16. The time allocated for rescue on floors without smoke may be considered overly conservative; however, it has been retained to allow for unforeseen events and allow the broadest application of the results. Physical assistance from staff with evacuation of occupants requiring assistance on non-fire floors has also been ignored.

Substantial additional fire brigade resources are required to support the initial response in the model. These additional resources will only be required for no sprinkler/fail sprinkler protection options, and very rare scenarios where global collapse of the structure may be imminent. Such additional resources may not be readily available in isolated areas, but medium-rise care facilities are relatively rare and tend to be located in highly populated areas. Medium-rise construction would not be a preferred model for housing large populations of people requiring high levels of support. Therefore the configuration adopted, and assumed resources to respond, were considered to be reasonable.

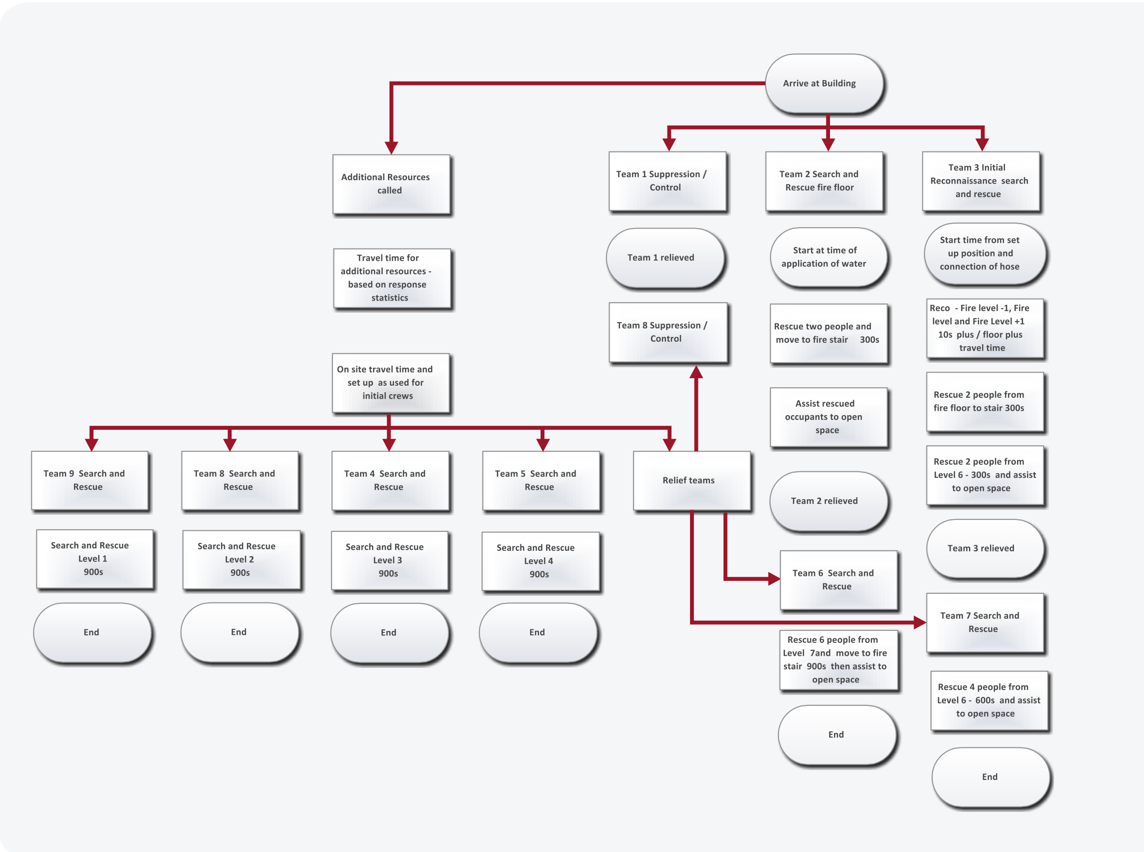


Figure 10.16: Fire brigade search and rescue flow chart Class 3 building with Type 2 occupants.

10.8 Occupant Response Evacuation and Consolidation Model

10.8.1 Occupant Characterisation for Class 2 and 3 Occupancies

The occupant characteristics of Class 2 buildings (e.g. apartments) were considered to be broadly representative of the Australian community.

This profile was also applied to the following Class 3 occupancies which were defined as Class 3 Type 1 occupants:

- Class 3(a) – a boarding house, guest house, hostel, lodging house or backpackers accommodation
- Class 3(b) – a residential part of a hotel or motel
- Class 3(e) –a residential part of a health-care building which accommodates members of staff.

These Class 3 buildings may have improved emergency management systems in place, but the impact of these was conservatively assumed to have no positive effect offsetting the potentially higher fire risks associated with some Class 3(a) occupancies.

The same evacuation model was therefore used for Class 2 and Class 3(a), (b) and (e) occupancies.

Evacuation modelling assumed that the occupants of each SOU would evacuate as a group. For apartments, an average group size of 2.5 was assumed. For some Class 3 occupancies, larger group sizes may apply. An average value of 6.5 per SOU for Class 3 occupancies was generally assumed.

Class 3(d) occupancies (accommodation for the aged, children or people with disabilities) have larger proportions of people that will require assistance to evacuate, and hence present a greater fire risk, and are referred to as Type 2 occupants. In these instances some level of staff assistance will be required to facilitate evacuation, which may vary from simply providing direction to providing physical assistance to occupant(s) who may or may not be aware of the emergency.

These two cases were considered to bracket other Class 3 building types and therefore a separate analysis was undertaken on each of these cases.

10.8.2 Occupant Response and Evacuation Model for Class 2 and Class 3 Type 1 Occupancies

A simple probabilistic model was applied that incorporates distributions for pre-movement times and can be incorporated into a multi-scenario analysis to address the variability of human responses to fire (See Appendix G5: Occupant Behaviour Review).

The response times (times to begin evacuation) were assumed to follow the simple distributions shown in Figure 10.17, which were modelled as discrete distributions with one-minute intervals. The parameters A-D will vary depending on the proximity to the fire, provision of general building alarm system, type of alarm system, etc. The values assumed for the comparative study of mid-rise buildings are summarised in Table 10.12 and are based on a poor/no alarm scenario, since they are applied to occupants outside the SOU of fire origin. A separate analysis based on fire data was undertaken for the apartment of fire origin (See Section 6).

The evacuation was assumed to commence at the start of the parametric fully developed fire scenario, (t=0) at which stage strong unambiguous cues outside the apartment of fire origin would be received reinforcing any building alarm system that had been activated.

This means that the number of outcomes during period B (B–n) would be equal to the period B in minutes plus 1 since the outcome interval is 1 minute and evacuation starts at t=0. For period D the number of outcomes is equal to the time period in minutes.

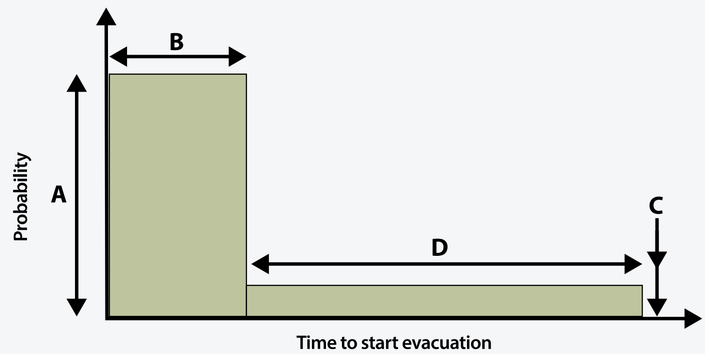


Figure 10.17: Occupant response model.

Table 10.12: Input parameters for occupant response model.

Location	A – prob	B – min	B -n	C – prob	D – min	D -n	No response	Notes
Apartment of fire origin	-	-	-	-	-	-	-	Not applicable
Remainder floor of fire origin	0.132	5	6	0.0079	25	25	0.011	Very strong secondary cues
Two adjacent floors to floor of fire origin	0.072	10	11	0.01	20	20	0.008	Very strong secondary cues
Other floors	0.036	20	21	0.01	20	20	0.044	Strong secondary cues

Due to the relatively low population and lengthy evacuation, the flow of people through stairs would be expected to be unconstrained and therefore the following average travel speeds were assumed:

- horizontal travel clear or light smoke:0.6 m/s to 1.2 m/s (for the subject building it will be assumed that it will take 10 s for occupants to move from their apartment to the stair door and a further 10 s horizontal travel to the exit from the base of the stairs)
- stair travel 20 s/floor.

The model assumed each SOU evacuates as a group.

Figure 10.18 shows the evacuation times (i.e. response plus travel times) with no impact from a fire. The distribution obtained is consistent with observed performance in fire drills and fire events, comprising an initial peak and then low evacuation rates over an extended period (see Appendix G5: Occupant Behaviour Review). The model also incorporates a probability that some occupants will not evacuate, allowing for people who require assistance to evacuate.

The evacuation model assumes that if occupants encounter smoke they would return to their apartment to await assistance. If untenable conditions occur in the apartment or global collapse is predicted prior to fire brigade search and rescue activities, they will be deemed to have been exposed to untenable conditions.

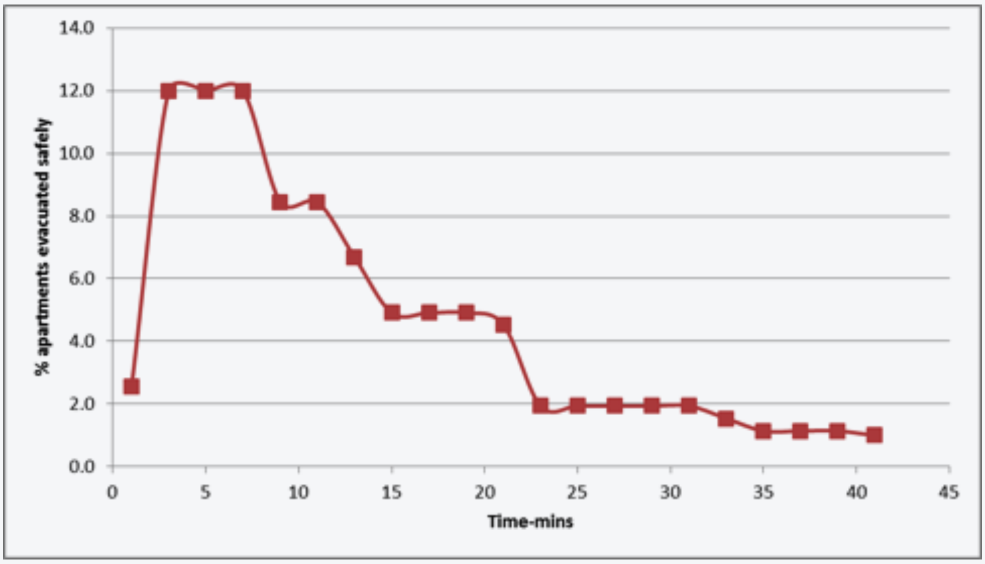


Figure 10.18: Evacuation times with no adjustment for impact of fire (base case).

10.8.3 Occupant Response and Evacuation Model for Class 3 Type 2 Occupancies

Accommodation housing Type 2 occupants will require higher staff levels and an emergency management structure consistent with the number of occupants requiring substantial assistance. Thus the evacuation process will tend to be more controlled, less random and more deterministic than in Class 2 buildings and some Class 3 Type 1 buildings.

In these occupancies, the fire brigade would be notified of the fire emergency by a call from the monitored alarm and detection service shortly after the building alarm was activated. Also at this stage, staff would be alerted, providing additional preparation time during the early stages of a fire in most cases, prior to the rapid growth and flashover phase simulated in the parametric heating regime assumed for fully developed fires.

The assumed staff response after making calls to emergency services, etc, would be to alert occupants, facilitate evacuation of those capable of self-evacuating and start evacuation of those under immediate threat (i.e. on the fire floor).

Based on these activities, it was assumed that the staff would have alerted and prepared occupants such that evacuation of the high dependency group of occupants on the fire floor would commence at t=1 min (i.e. 1 minute after commencement of the parametric heating regime) if safe to do so, leaving four high dependency occupants of the original 6.5 to be evacuated on the fire floor with assistance from the fire brigade.

On all other floors, six high dependency occupants would be assumed to require fire brigade assistance to evacuate (typically one group of occupants).

A managed evacuation process for the remaining occupants who do not require physical assistance to evacuate was modelled adopting the following parameters:

- Travel time to descend stairs is 40 s/level plus 20 s for horizontal travel.
- Fire floor evacuation starts at t=60s.
- Next floor starts evacuation once the previous floor has exited the building.
- Order of evacuation is Level 5,6,7,4,3,2,1 (assuming a fire on Level 5).
- If occupants encounter smoke they would return to SOU and wait for assistance.

Ignoring the impact of smoke, these assumptions yield the evacuation times for the occupants excluding those reliant on fire fighters for evacuation presented in Table 10.13.

Table 10.13: Timing of phased evacuation.

Building level	Time to exit SOU – min	Time to exit building – min
7	9.0	14.0
6	4.7	9.0
5	1.0	4.7
4	14.0	17.0
3	17.0	19.3
2	19.3	21.0
1	21.0	22.0

The outcomes from the evacuation model without the impact of smoke are shown in Figure 10.19. The early evacuation relates to the SOU of fire origin, which is not relevant to the analysis outside the SOU of fire origin. The graph output excludes about 18% of the occupants, who are assumed to require assistance from the fire brigade to evacuate.

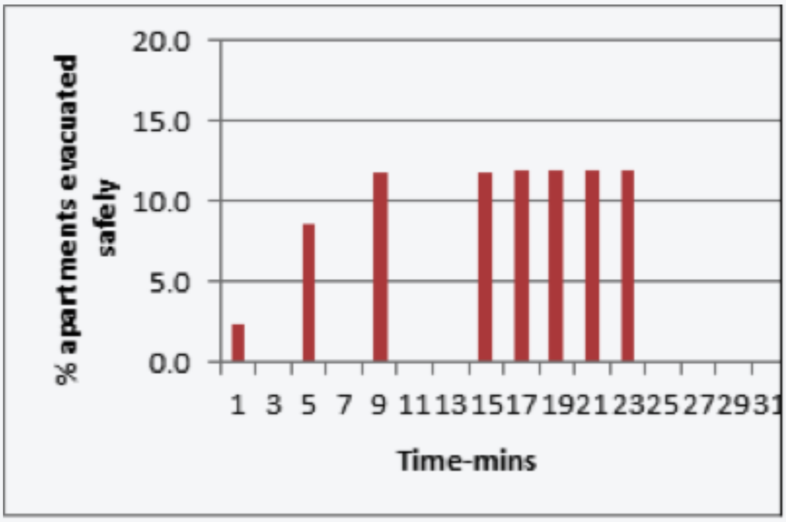


Figure 10.19: Evacuation outcomes ignoring impact of smoke.

10.8.4 Consolidation

The logic behind the consolidation model is shown in the form of a flow chart in Figure 10.20.

For each fully developed fire scenario (chosen at random), the time to untenable conditions and loss of visibility is input from the base smoke-spread model.

Detection/alarm times are then used as input to develop distributions for the response of occupants and fire brigade intervention as described above.

Occupants of each SOU are assumed to evacuate as a group and, if they encounter heavy smoke (loss of visibility), they are assumed to return to their SOU and await fire brigade assistance to evacuate. This is a simplification, since records from fire incidents indicate that under some circumstances occupants will try and evacuate through smoke with varying degrees of success. Since the purpose of the analyses was to compare the safety outcomes with those associated with designs meeting the Deemed-to-Satisfy (DTS) Provisions, a simple approach was considered reasonable.

If untenable conditions occur in an occupied SOU, the occupants are deemed to be exposed to untenable conditions. Also, any occupants in the building at the time of global collapse predicted by the enclosure fire/ structural model are assumed to be exposed to untenable conditions. Failure of two elements was deemed to initiate substantial failure (global collapse) and all remaining occupants are assumed to be exposed to untenable conditions.

The analysis is repeated for each scenario in the multi-scenario analysis and the number of occupancy groups exposed to untenable conditions in each scenario recorded.

Figure 10.20: Class 2 and Class 3 Type 1 evacuation model flow chart.

10.9 Summary of Results from Monte Carlo Analysis

10.9.1 Class 2 Building Occupant Safety Results

Monte Carlo analyses were undertaken for apartment fires occurring on Levels 2, 5 and 7, the results from which were consolidated on the following basis:

- Level 2 fire is representative of fires on Levels 1-3
- Level 5 fire is representative of fires on levels 4-6
- Level 7 is included as the floor with the longest time for fire brigade intervention but represents a special case since no occupants occupy the level above.

Monte Carlo analyses were undertaken for a control steel-framed building, a timber-framed building and a massive timber building. The timber buildings were modelled with and without increased fire loads, with the results being combined based on the estimated proportion of fires that could ignite the timber substrate and may affect fire severity. The impact of defects was incorporated in the fire resistance sub-model.

Finally, the results of the timber buildings were adjusted to take into account the presence of automatic fire sprinklers. The fire sprinklers were assumed to have a reliability of 92% and, if they operated successfully, tenable conditions would be maintained in all adjoining fire compartments and no fire spread would occur.

The results are expressed in terms of the number of occupant groups exposed to potentially untenable conditions where an occupant group represents the occupants of an apartment (assumed to be equivalent to 2.5 people per apartment).

Due to the large number of variables and low frequency of key events, 100,000 scenarios were run for each configuration.

The above results were consolidated by combining the individual Level results using the following relationship:

- Building Consolidated results = ((Level 2 x 3) + (Level 5 x 3) + Level 7)

The frequency of potential flashover fires was estimated to be 1.8 x 10^{-4} fires/annum/apartment and therefore the frequency for the subject building (42 apartments) was assumed to be approximately 7.56 x 10^{-3} fires /annum.

The results were further consolidated by grouping scenarios where one to three occupant groups were exposed to untenable conditions and four or more; which approximates to less than 10 occupants and 10–100 occupants respectively enabling the results to be expressed in a format that can be compared to F-N curves. It should be noted that the occupants of the apartment of fire origin are excluded from this analysis.

Table 10.14: F-N Consolidated results for Class 2 buildings.

No of occupants exposed to potential untenable conditions	Frequency/annum x 10^{-6}		
	Control	Timber Frame	Massive Timber
1-10	164.7	13.2	12.1
10-100	3.62	0.41	0.01

The frequencies of exposure in Table 10.14 are much higher than may be expected from consideration of historic fire losses if they are assumed to represent fire fatalities. This variance can be largely explained by a number of conservative assumptions that have been made in the analysis, namely:

- The assumption that occupants are largely passive if smoke spread occurs to their apartment through, for example, an open door. In most instances they would be likely to take actions to mitigate the risk, such as closing doors, seeking refuge on a balcony or in a room within the apartment.
- The global structural model is simplistic since it is for a generic application, and an assumption has been made that the entire building is lost upon failure of two members, which may not be the case.
- The proportion of open doors was based on the very limited data in literature and for apartment buildings, the probability of SOU doors being closed could be much higher than the 90% assumed for the apartment of fire origin and 95% assumed for other apartments.

However, since the study was comparative and all buildings were treated in a similar manner, it was considered unnecessary to refine the analysis further.

The results for 1 to 10 occupants were dominated by smoke spread through open doors with the differences between the timber buildings and control buildings being largely attributable to the provision of fire sprinklers.

The higher consequence loss scenarios (10–100 occupants) were strongly linked to global collapse, which was more likely to occur with the steel frame and timber frame construction because of the lower inherent fire resistance compared to massive timber. Reliance on the inherent fire resistance is only needed when there is a gross defect with the primary fire protection system(s) and the sprinkler system (if provided) fails.

If defects are ignored, the Deemed-to-Satisfy fire resistance levels within the NCC were found to prevent global collapse if fire brigade intervention was taken into account for the three buildings. This is significant and highlights the importance of design of buildings with reasonable levels of structural redundancy, and appropriate quality controls with respect to passive fire protection system performance verification and installation/maintenance, which apply to all forms of construction. The analysis clearly demonstrated the ability of an additional primary fire protection system such as fire sprinklers to substantially mitigate these risks.

It is sometimes convenient to express outcomes in terms of a comparative risk to life based on the total estimates of occupants exposed to untenable conditions. These results are presented in Table 10.15 and the normalised results in Table 10.16.

Table 10.15: Expected risk to life results for Class 2 buildings.

	Control	Timber Frame	Massive Timber
Occupants exposed/annum – (expected risk to life) x 10^{-5}	54.2	4.9	3.3

Table 10.16: Normalised expected risk to life results for Class 2 buildings.

	Control	Timber Frame	Massive Timber
Normalised expected risk to life	1	0.091	0.061

Irrespective of the method of comparison the results indicate that the mid-rise timber buildings with automatic sprinkler protection provide a substantial improvement in safety of the occupants from potential flashover apartment fires compared to the non-combustible control building without automatic sprinkler fire protection.

10.9.2 Class 2 Building Fire-fighting Activities

An estimate of the impact on fire fighters can also be derived by construction of simple event trees as shown in Figure 10.21 and Figure 10.22.

The key factors are:

- The automatic fire sprinkler system would reduce the proportion of fully developed fires the fire fighters have to deal with.
- With the fire-protected timber systems required in the proposal for change (PFC), the probability of the timber structure becoming involved is reduced. If the timber structure is not involved, there will be no appreciable difference in the conditions facing the fire brigade.
- If the timber structure is ignited, the potential fire duration is increased and therefore there is a greater reliance placed on fire fighter activities.
- If there are cavities, additional fire-fighting measures may be required to locate the fire.

The percentage of potential flashover scenarios that may ignite timber members can be derived from the Monte Carlo analysis. The results for Level 7 of the generic building were 1% and 34% for the timber frame building and the massive timber building, respectively. This occurs because of the lower levels of protection permitted for massive timber compared to general timber construction.

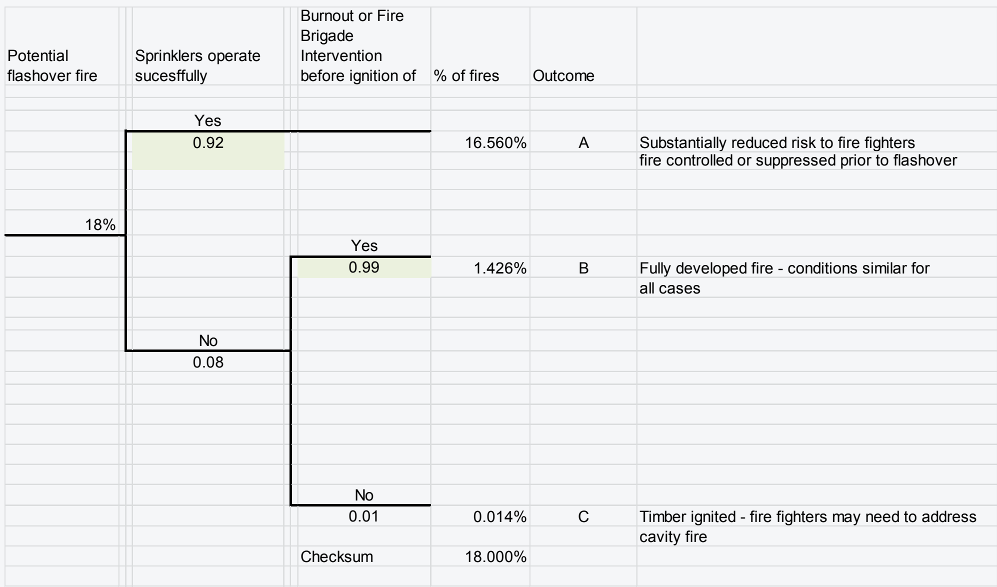


Figure 10.21: Event tree for fire fighter outcomes – timber frame construction.

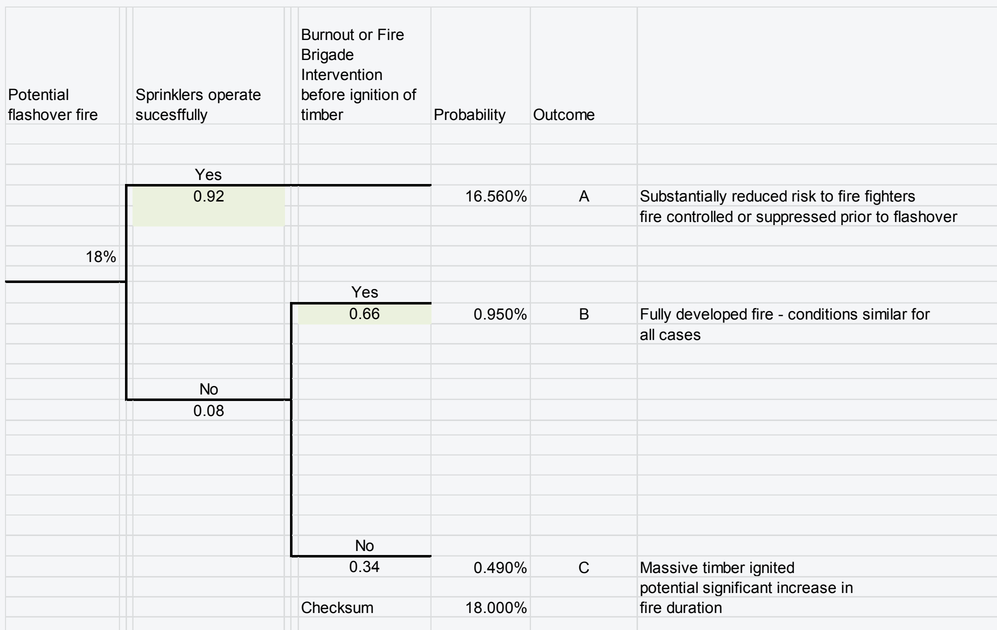


Figure 10.22: Event tree for fire fighter outcomes – massive timber construction.

For the control building, all potential flashover fire scenarios (18% of fires) would lead to outcome B – i.e. fire fighters having to deal with a fully developed fire. For the remaining non-flashover scenarios, the conditions facing fire fighters on arrival will be assumed to be similar for all the buildings (this is considered conservative since larger non-flashover fires may be controlled or suppressed by the automatic fire sprinklers.

Table 10.17: Impact on fire-fighting activities.

Outcome	Control	Timber frame	Massive Timber
A – Low Risk – no flashover	0%	16.56%	16.56%
B – Flashover -standard	18%	1.426%	0.95%
C – Additional actions required	0%	0.014%	0.49%

The additional actions required for timber-framed construction may comprise additional resources to address a potential increase in the fire duration and the need to ensure cavity fires are suppressed. Considering the very low probability of these scenarios (0.014%) and that there is an opportunity to call in additional resources, it is considered that there will be a large net improvement in the conditions faced by fire fighters with the timber-framed option in comparison to the control building.

The additional actions required for massive timber construction may comprise additional resources to address a potential increase in the fire duration. Considering the low probability of these scenarios (0.49%) and that there is an opportunity to call in additional resources, it is considered that there will be a large net improvement in the conditions faced by fire fighters with the massive timber option in comparison to the control building.

It is, however, necessary for fire fighters to develop procedures for fires in these types of buildings; in particular, methods to locate and suppress cavity fires.

10.9.3 Class 3 Building Occupant Safety Results

Based on the analysis of Class 2 buildings, it was determined that the analysis of fires occurring on Level 5 was sufficient to compare the relative performance of the three building types considered. The results expressed in a format that can be compared to F-N curves are shown in Table 10.18 (excluding the occupants of the apartment of fire origin).

For Class 3 buildings, an occupant group would vary between 1 and 6.5 people.

The occupants of Type 1 buildings have similar capabilities for self-evacuation to Class 2 building occupants; whereas, a larger proportion of Type 2 building occupants would require assistance to evacuate.

Table 10.18: Results expressed in an F-N format.

No of occupants exposed to potential untenable conditions	Frequency/annum x 10^{-6}		
	Control	**Timber Frame**	**Massive Timber**
Class 3 Type 1 1–3	21.66	1.74	1.71
Class 3 Type 1 4 or more	0.108	0.015	0.0008
Class 3 Type 2 1–3	27.13	2.16	1.94
Class 3 Type 2 4 or more	0.173	0.016	0.0017

Table 10.19 shows the results normalised to a building complying with current NCC DtS Provisions.

Table 10.19: Normalised expected risk to life results.

Outcome	Control	Timber frame	Massive Timber
Normalised Expected Risk to Life Class 3 Type 1	1	0.084	0.076
Normalised Expected Risk to Life Class 3 Type 2	1	0.082	0.064

The results show a large improvement in life safety, which is to be expected since a range of mitigation measures have been taken to minimise risks associated with timber structural elements, and automatic fire sprinklers have been additionally provided.

10.9.4 Class 3 Building Fire-fighting Activities

The impact on fire-fighting activities was considered and the results for the various buildings are summarised in Table 10.20.

Table 10.20: Estimated impact on fire-fighting activities Class 3.

Outcome	Control	Timber frame	Massive Timber
A – Low Risk – no flashover	0%	16.56%	16.56%
B – Flashover – standard	18%	1.437%	1.32%
C – Additional actions required	0%	0.003%	0.12%

For the control building, all potential flashover fire scenarios (18% of fires) will lead to outcome B – i.e. fire fighters having to deal with a fully developed fire. For the timber buildings, fewer than 1.5% of fires will reach flashover due to the provision of automatic fire sprinklers. Because of the required fire protection coverings, fire fighters are potentially going to be faced with additional actions required for timber-framed construction in only 0.003% of fires and 0.12% of fires for massive timber construction.

11

Fires in Paths of Travel

11.1 Fires in Paths of Travel – Fire Scenarios and Methods of Analysis

Sections 8 through to 10 considered the effect of fires starting in an SOU, which is the dominant location of fire starts. This Section considers fires starting in paths of travel.

From the analysis of fire incidents presented in Appendix F2, 3–4% of fires were estimated to occur in lobbies, entranceways, hallways and corridors in apartment buildings. These areas generally provide access to apartments and lie on the paths of travel to fire-isolated exits. While the frequency of these fires is relatively low, they have the potential to compromise the paths of travel from an apartment to a fire exit and also to cause rapid smoke spread to fire exits and other floors; depending upon the state of doors and other openings. Therefore, this low probability event may lead to high consequences, and further analysis was considered necessary to compare the proposed changes to the NCC 2016 with NCC 2015 Deemed-to-Satisfy (DTS) Provisions.

The most likely fire that has the potential to have a significant impact on fire-resisting elements was considered to be one involving upholstered furniture or similar materials that have a rapid fire growth rate. Such a fire has the potential to block access to an escape path for all occupants on the floor of fire origin if fire growth is not restricted and allows smoke spread to large areas of the remainder of the building, including fire-isolated stairs, before fire brigade Intervention.

Since the fire load in a public corridor or lobby is generally lower than within an SOU and wall and ceiling linings are controlled, the risk of flashover occurring in the corridor is low and, even if flashover occurs, the severity of a fully developed fire within the corridor would be likely to be much less than most fully developed apartment fires.

It therefore follows that the non-combustible fire-protective covering required to be applied to the fire-protected timber would mitigate the risk of the fire-protected timber becoming involved in a fire, and that the rate of fire growth and fire severity would be the same for the proposed timber buildings and a building satisfying the current NCC DTS requirements (if the impact of fire sprinklers is ignored).

The fire scenarios used to compare the building solutions were:

- A rapidly growing fire in a corridor filling the corridor on the floor of origin with smoke, potentially preventing the safe evacuation of occupants from the floor of fire origin.
- If the occupants try to evacuate through the smoke without assistance from the fire brigade, there is a significant probability that they will be exposed to untenable conditions. This probability of exposure is substantially reduced if automatic fire sprinklers operate effectively.
- If the occupants remain in their apartments with the door to the apartment closed, some smoke spread may occur to the apartment but fire and smoke separation will be consistent with the current NCC DTS provisions and will therefore be considered acceptable and to present an acceptable risk.
- If the door to an apartment is open, there is a significant probability that the occupants could be exposed to untenable conditions. This probability of exposure is substantially reduced if automatic fire sprinklers operate effectively. There is also a risk of fire spread to the apartment through the open door leading to flashover within that apartment, which would be prevented if automatic fire sprinklers operate successfully.
- If the door to the stair is open, there is a risk that smoke could spread to the stair potentially generating untenable conditions within or above the level of fire origin. Smoke spread may also occur via the lift shaft to upper level corridors, potentially exposing occupants on upper levels to smoke if they attempt to evacuate. This probability of exposure is substantially lower if automatic fire sprinklers operate effectively.

Although the qualitative discussion above indicated that the risks associated with fires in paths of travel to fire-isolated exits would be reduced for the fire-protected timber buildings due to the provision of automatic fire sprinkler system, a simple event tree analysis was also undertaken to provide some quantification.

Outcomes were estimated assuming rapid onset of untenable conditions in the corridor and in apartments with open doors prior to fire brigade intervention.

11.2 Fires in Paths of Travel – Acceptance Criteria

The fire-protected timber buildings with automatic fire sprinklers were considered to provide an acceptable level of protection against the corridor fire scenario, if the expected risk to occupants was less than the generic version of the building complying with the NCC DTS 2015 Provisions that require non-combustible construction.

11.3 Fires in Paths of Travel – Results and Assessment

The simple event trees used to analyse the potential outcomes from a fire occurring in a corridor are shown in Figure 11.1 and Figure 11.2 for scenarios with and without sprinkler activation, respectively. Details for the inputs for both event trees are summarised in Table 11.1.

Table 11.1: Inputs for event tree analysis.

Event	Sprinklers		Comments
	None	Effective	
Occupants try to evacuate apartments on floor of fire origin	0.1	0.1	Most occupants would see smoke and sense heat and decide not to evacuate
Evacuating occupants return to apartment due to smoke/fire	0.95	0.95	Since conditions are severe most occupants return to apartment
Evacuating occupants exposed to untenable conditions in exit paths	0.5	0.1	Assumed 50% would evacuate past the smoke safely for no sprinkler and 90% if sprinklers activate (less hazardous conditions)
Doors to apartments closed	0.95 (0.9)	0.95 (0.9)	Reduced probability of doors in brackets applied if doors have been used in the emergency and if the automatic closers are not operational may be left open.
Occupants exposed to untenable conditions in apartment if door open	0.25	0	Fire sprinklers would be expected to prevent untenable conditions occurring in an adjacent enclosure

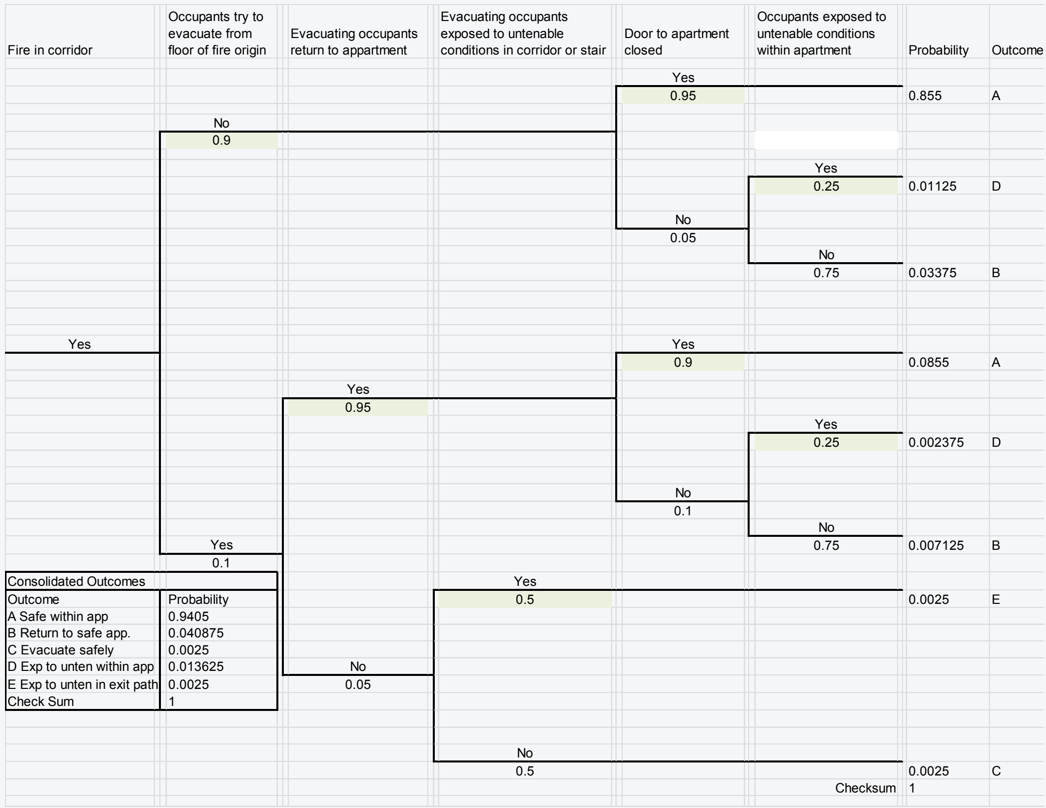


Figure 11.1 Event tree for corridor fire – no automatic suppression.

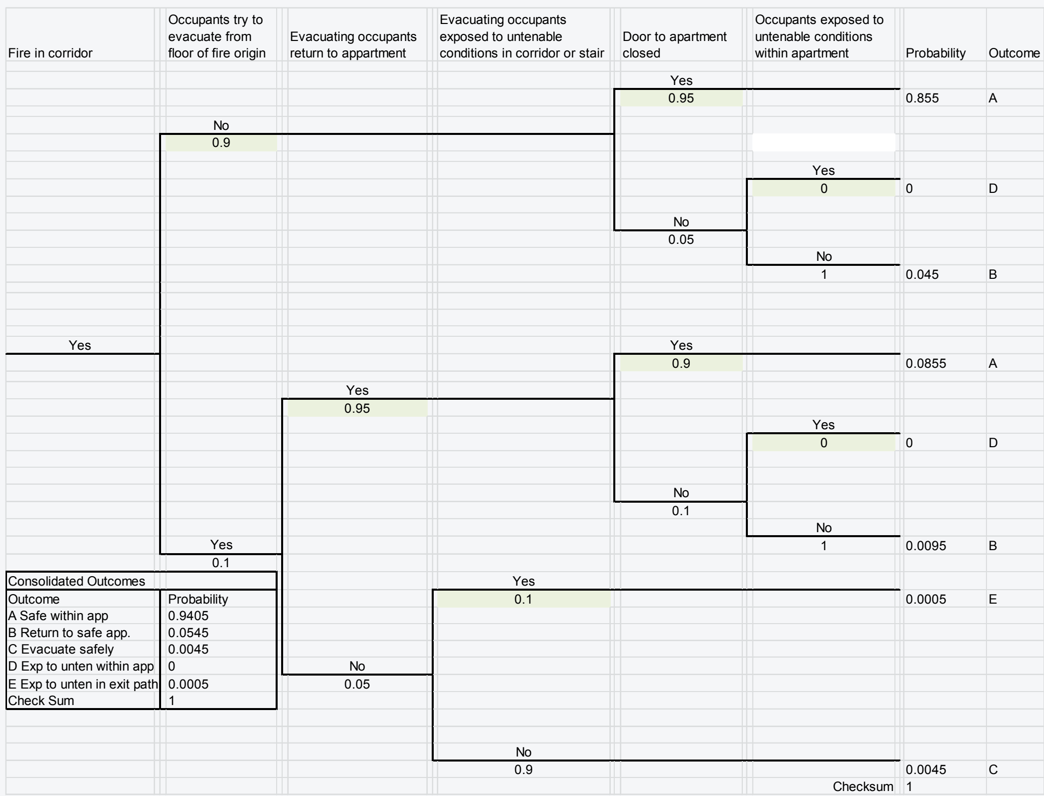


Figure 11.2: Event tree for corridor fire with automatic fire sprinkler suppression.

Allowing for six apartments on the fire floor and an average of 2.5 occupants/apartment (Class 2 and Class 3 Type 1) at the time of fire, the following outcomes were predicted:

- No sprinkler operation: (0.0136 + 0.0025) x 15 = 0.24 people potentially exposed to untenable conditions/fire
- Successful sprinkler operation: (0.0005) x 15 = 0.0075 people potentially exposed to untenable conditions/fire.

If a sprinkler effectiveness of 0.92 is assumed, the outcomes for the fire-protected timber buildings (including automatic fire sprinklers) can be calculated to be:

- (0.08 x 0.24)+(0.92 x 0.0075) = 0.026 people exposed to untenable conditions/fire compared to 0.24 people exposed to untenable conditions/fire for a building meeting the current NCC DTS Provisions.

Normalising the results such that the expected risk to life for this scenario is 1 for the Deemed-to-Satisfy Provisions yields the following:

- Normalised expected risk to life for the control building 1
- Normalised expected risk to life for the fire-protected timber buildings 0.11.

A large improvement in life safety was indicated, confirming the qualitative analysis findings and satisfying the acceptance criteria.

12 Fire in a Fire-isolated Stair

12.1 Fire in a Fire-isolated Stair and Passageway – Data

12.1.1 Proportion of Fires Occurring in Fire-isolated Stairs

From the analysis of fire incidents presented in Appendix F2, 0.1% of fires occur in fire-isolated escape routes and 0.8% occur within interior stairways. Since some fires occurring in fire-isolated escape routes were reported as occurring in interior stairways, it was assumed that between 0.1 and 0.9% of fires occur in fire-isolated stairs of passageways. While the frequency of these fires is low, as is the fire load, it may represent the only exit path for occupants and access path for fire fighters.

12.1.2 TF2000 Stair Fire Test

Overview of TF2000 Stair Fire Test

The analysis was based predominantly on experimental data reported in the DETR Framework Closing Report TF2000 Stair Fire Test[29].

As part of the TF2000 project in the UK, a series of stair fire tests were undertaken to facilitate the extension and harmonization of the UK regulations with respect to fire stairs in mid-rise timber buildings[29]. These tests have direct relevance to the Proposal for Change considered in this report.

The study identified the following fundamental consideration for a stair:

"The stair has to remain useable for fire fighting after initial evacuation of occupants immediately at risk and for subsequent evacuation by the other occupants of the flats who are initially advised to remain in their dwellings."

The above was used to provide guidance in addition to the relevant performance requirements in the NCC.

TF2000 Stair Construction

The TF 2000 fire stair wall/ceiling construction was required to achieve a fire resistance of 60 minutes, which was achieved by applying two 12.5 mm thick layers of standard-grade plasterboard to timber studs.

Within the stair enclosure, the stairway was constructed from fire retardant-treated softwood with fire retardant-treated timber balustrades using a thermosetting adhesive (Urea Formaldehyde).

The stairs were underdrawn with a single layer of standard-grade plasterboard 12.5 mm thick, fixed using clout nails at 150 mm centres to the stringers of the stair only.

Derivation of Acceptance Criteria and Fire Exposure Conditions

The TF2000 project identified the most onerous fire scenario as a fire that starts and grows in the stair due to materials being left or stored in the stairwell that are either accidentally or purposefully ignited.

While materials should not be stored in fire-isolated stairs and passageways, fires involving combustible materials introduced to fire-isolated stairs and passageways do occur (estimated to be between 0.1 and 0.9% of all fires based on the statistics presented in Appendix F2).

Therefore the scenario proposed for the TF2000 project was also considered to represent a credible severe scenario for comparison of the changes for mid-rise timber buildings introduced in the NCC 2016 edition against the NCC 2015 Deemed-to-Satisfy Provisions.

It was acknowledged that a significant fire within a fire-isolated stair could result in the development of untenable conditions irrespective of the form of construction used for the stair enclosure or stairway.

The following acceptance criteria were therefore adopted as the basis for evaluating the changes introduced into the NCC 2016 when exposed to a fire scenario developed by the TF2000 fire group:

- tenable conditions within the stair should be maintained for at least the same duration as the control building
- the stairway should remain serviceable for fire fighters throughout the fire emergency
- fire spread from the stair should be prevented to the same extent as the control building
- the fire-protected timber should not be ignited.

The TF2000 fire scenario included the following items ignited simultaneously:

- a double mattress held in the vertical position by tying to a balustrade on the ground floor
- a 500 mm x 500 mm 16 stick crib with stick size 50 mm x 50 mm mounted underneath the stair
- paraffin-soaked fire strips at the junction between the treads and risers on the first 5 steps.

Summary of TF2000 test results

Key outcomes of the test were:

- The test was practically completed 31 minutes after ignition after complete burnout of the mattress and the timber crib had reduced to glowing embers. The fire was allowed to burnout without suppression.
- Maximum general air temperatures within the stair shaft did not exceed 300°C (timber crib was fitted below the stair and it exposed the underside of the stair to a severe fire but the impact was localised.
- The void between the single 12.5 mm sheet of plasterboard and the underside of the stair directly above the timber crib reached approximately 120°C at the end of the test and the board remained in place indicating that the underside of the stair was adequately protected.
- The void between the single 12.5 mm sheet of plasterboard and the underside of the stair on the first floor above the mattress reached approximately 80°C at the end of the test.
- The stairs could support fire fighters gaining access to the upper levels after the fire.
- Untenable conditions were reached in the stair after approximately 6 minutes with the first fire detectors in the shaft operating after 4 minutes 21 seconds. It was therefore concluded that there was insufficient time for the occupants to evacuate via the stair. Since the contribution from the fire retardant stairs was minimal the same result would occur if the stair and stairway was of non-combustible construction.
- Closed doors to the stair (without smoke seals) prevented untenable conditions being reached on the upper levels.
- The fire did not spread from the stair enclosure.

12.2 Fire in a Fire-isolated Stair and Passageway – Assessment of Performance of Fire-protected Timber Construction

12.2.1 Fire Protection Lining System – General Timber Construction

The Deemed-to-Satisfy requirements for fire-isolated stairs or passageways of timber construction introduced into the NCC 2016 edition require the use of fire-protected timber (i.e. in Class 2 and 3 buildings the walls are required to achieve fire resistance levels of 90/90/90 or -/90/90 as appropriate and the fire protection lining system is required to prevent the interface temperature between the plasterboard and timber reaching 250°C for at least 45 minutes among other things when subjected to a fire resistance test). To achieve this level of performance the fire protection lining system requires a higher level of performance than the 60 minute fire resistance system selected for the TF2000 series.

The NCC 2016 Deemed-to Satisfy requirements for general timber construction relating to fire-isolated stair and passageway shafts were considered to satisfy the acceptance criteria derived in Section 12.1.2 as explained below:

- tenable conditions within the stair will be maintained for at least the same duration as the control building since the protected timber will not be involved in the fire
- the stairway should remain serviceable for fire fighters throughout the fire emergency – same conditions for the control and the fire-protected timber-framed construction

- fire spread from the stair should be prevented to the same extent as the control building – no difference since fire-protected timber would be expected to prevent fire spread (higher FRL required than TF2000 test and additional control assessed as per the following dot point)
- the fire-protected timber should not be ignited – achieved since no ignition occurred in the TF2000 test and fire protection lining system required for fire-protected timber has a higher level of performance.

12.2.2 Fire Protection Lining System – Massive Timber Construction

The Deemed-to-Satisfy requirements for fire-isolated stairs or passageways of massive timber construction that satisfies the requirements permitting the concession to apply, introduced into the NCC 2016 edition also require the use of fire-protected timber. However while an FRL of 90/90/90 or -90/90 is still required, the interface temperature limit between the plasterboard and timber is relaxed to 300°C for at least 20 minutes when subjected to the standard fire resistance test. To achieve this level of performance the wall system will require a higher level of performance than the 60 minute fire resistance system selected for the TF2000 series in terms of fire resistance. The massive timber interface temperature will rise more rapidly than the two layers of 13 mm fire-grade plasterboard and potentially faster than two layers of 12.5 mm thick standard plasterboard used in the TF2000 test depending upon the fixing detail.

However since a single layer of standard plasterboard 12.5 mm thick fitted to the underside of the stair in the TF2000 tests limited cavity temperatures to approximately 120°C directly above the timber crib it is considered unlikely that the fire scenario would generate sufficient heat to penetrate a fire protection lining system having the performance required in NCC 2016 for massive timber fire-protected members (e.g. a single layer of 13 mm fire-grade plasterboard).

The NCC 2016 Deemed-to Satisfy requirements for massive timber construction relating to fire-isolated stair and passageway shafts were considered to satisfy the acceptance criteria as explained below:

- tenable conditions within the stair will be maintained for at least the same duration as the control building since the protected timber will not be involved in the fire
- the stairway should remain serviceable for fire fighters throughout the fire emergency – same conditions for the control and the fire-protected timber construction
- fire spread from the stair should be prevented to the same extent as the control building – no difference since fire-protected timber would be expected to prevent fire spread (higher FRL required than TF2000 test and additional control assessed as per the following dot point)
- the fire-protected timber should not be ignited – achieved since no ignition occurred in the TF2000 test directly above the crib with a timber stair protected by a single layer of non-fire-grade plasterboard.

12.3 Timber Stairway Concession

12.3.1 Timber Stairway Concession – Background

An additional variation from the current NCC Deemed-to-Satisfy Provisions was evaluated to allow the use of timber stairways and ramps subject to the extension of automatic fire sprinkler coverage to include the stairs and passageway and the underside of stairs being protected by a single layer of 13 mm fire-protective-grade plasterboard or equivalent.

The concession is included in Clause D2.25 of the NCC 2016, which is summarised below:

(a) Notwithstanding D2.2(a) timber treads, risers. landings and associated supporting framework

which

(i) has a finished thickness of not less than 44 mm and

(ii) has an average density of not less than 800 kg/m² at a moisture content of 12%

may be used within a required fire-isolated stairway or fire isolated passageway constructed from fire-protected timber in accordance with C1.13 subject to –

(iii) the building being protected throughout by a sprinkler system complying with Specification E1.5 which extends to within the fire isolated enclosure and

(iv) fire protection being provided to the underside of stair flights and landings located immediately above a landing level which –

(A) is at or near the level of egress; or
(B) provides direct access to a carpark

(b) Fire protection required by (a) must be not less than one layer of 13 mm fire-protective-grade plasterboard fixed in accordance with the system requirements for a fire-protective covering.

12.3.2 Assessment of Performance of Timber Stairway

The TF2000 test series focused on the use of fire retardant-treated timber stairways with supplementary materials controls and protection of the underside of the stairways to satisfy the acceptance criteria. A successful fire test was undertaken with this configuration.

The NCC Clause D2.23 Timber Stairway Concession extends coverage of the fire sprinklers to the fire stairs in lieu of requiring fire retardant-treated timber, and additional controls were added requiring the exposed timber to have a finished thickness of not less than 44 mm and an average density of not less than 800 kg/m^2 at a moisture content of 12%.

A number of initial trials were undertaken as part of the TF 2000 series. The first trial was undertaken using untreated stairway components and included three test runs. This data is therefore directly relevant to the changes to the NCC introduced in the 2016 edition.

The first run of trial one was undertaken with a double mattress tied to the balustrades, and a porous fibre board soaked in paraffin was placed on the first step of the lower flight. Complete combustion of the mattress took place. The spindles and handrail of the baluster were involved in the fire causing extensive charring; however, no damage was inflicted on the stair treads and risers and only very limited charring of the vertical face of the lower stringer was observed.

The second run was an attempt to initiate a 'trench' effect by placing paraffin-soaked fibre strips at the junctions between the treads and risers of the first five steps on the lower flight. Ignition of the strips was reported to have led to a short period of sustained burning where the flames were observed 'lying down' and surface charring to a depth of 2–3 mm of the first five steps was the result. The fire died out as the strips and paraffin were consumed and there was no further spread of flame. The structural integrity of the stairs was maintained and verified by personnel walking on the treads.

The third run involved removing the plasterboard from the underside of the stairs and placing a timber crib (16 sticks of 50 x 50 x 500 mm softwood) underneath the lower surface of the first flight. This fire led to a breakthrough of the fire on the stairs and the stringer after 10 minutes with the lower flight becoming fully involved in the fire, and the fire was suppressed.

The second trial was similar to the first trial except that the timber members were treated with fire retardant. The fire retardant treatment reduced the fire spread from that observed in trial one, particularly with the timber crib configuration. However, after the crib test, a number of treads directly above the crib were damaged to the extent that they broke from the stringers when they were stepped on. The use of a PVA adhesive was considered to have contributed to the failure and Urea Formaldehyde adhesives were used in trial 3 and the full test.

In the full test, the plasterboard protection to the underside of the stairs was left in place and prevented the temperature from exceeding 120°C in the cavity, and was intact at the end of the test. In trial 3, the same configuration was tested and the lining fell away at the end of the test but ignition of the underside of the stair did not occur.

If fire-grade plasterboard had been used in lieu of standard plasterboard, the risk of the board falling away would be substantially reduced. Under such circumstances, it is likely that the board would prevent ignition of the underside of the steps, irrespective of whether the timber is fire retardant-treated or not, and the stair assembly from trial one would have been likely to achieve performance consistent with the acceptance criteria.

It therefore follows that for the TF2000 solution, the fire retardant treatment would reduce the extent of fire damage to the exposed timber elements and provide some redundancy if the lining failed.

The NCC 2016 requires a higher specification for the lining protecting the underside of the stair and additional fire sprinkler protections. Since temperatures approaching 300°C were achieved within the main stair, and higher temperatures directly under the stair, these temperatures would be sufficient to activate a sprinkler head reducing the size of the fire and hence reducing reliance on the fire protection lining system. In some instances, tenability may be enhanced, but this will depend upon the nature of the fire source and proximity to the head.

It was therefore considered that the NCC stairway concession satisfies the proposed acceptance criteria as explained below:

- Tenable conditions within the stair will be maintained for a similar duration to the control building since the contribution from exposed timber members will be minimal. While visibility may be temporarily reduced upon activation of the sprinklers, temperatures within the stair will be reduced and the net effect would tend to be neutral or improved tenability conditions for most scenarios.
- The stairway should remain serviceable for fire fighters throughout the fire emergency – generally similar conditions expected.
- Fire spread from the stair should be prevented to the same extent as the control building – an improvement is expected because of the potential for early suppression by the automatic fire sprinkler.
- The fire-protected timber should not be ignited – achieved since no ignition occurred in the TF2000 test directly above the fire with a timber stair protected by a single layer of non-fire-grade plasterboard. The sprinkler system provides an additional redundancy.

13

Fire in Lift Shafts

13.1 Lift Shaft Fire Scenarios and Frequency of Fire Starts

From the analysis of fire incidents presented in Appendix F2, 0.22% of fires occur in lift /dumbwaiter shafts.

Assuming the frequency of fires for the subject building is approximately 42 x 10^{-3} fires /annum, the frequency of fire starts in the lift/dumbwaiters shaft would therefore be approximately 9.2 x 10^{-5} fires / annum.

Since the fire load is small and lining materials within the lift cars are required to achieve either Group 1 or Group 2 performance, it will be assumed that only 5% of these fires grow to be significant fires (i.e. 4.6 x 10^{-6} fires/annum). This is comparable to an estimate of 2.86 x 10^{-6} fires/annum/per lift car made by Bennetts et al.[30]. For the purposes of this analysis, a value of 5 x 10^{-6} fires/annum will be adopted for significant fires, which would be expected to be conservative for buildings with a single lift such as the generic building considered in the comparative analysis of the fire-protected building options.

The following major groups of fire scenarios have been identified:

- fires starting within the shaft
- fires adjacent to the shaft exposing the lift landing doors
- fires occurring within a combustible lift car.

13.1.1 Fire Starts within Lift Shafts

Due to the limited volumes of combustible materials within lift shafts, fires within a lift shaft are expected to be small and typically involve small amounts of debris. It is therefore considered that the fire protection coverings for fire-protected timber verified for fire-isolated stairs can be conservatively applied to this scenario and no further analysis is required.

13.1.2 Fires in Lift Lobby Areas

Bennetts et al[43] undertook fire tests to determine conditions within the lift shaft when a fully developed fire occurs in the adjacent lift lobby. The tests were performed with a plasterboard shaft and temperatures were measured on the plasterboard wall directly opposite the lift landing doors exposed to a fully developed fire. The temperatures peaked below 200°C which would not cause ignition of the protected timber. For the timber buildings, automatic fire sprinkler systems are provided substantially reducing the probability of this scenario occurring. No further analysis is therefore required for this scenario

13.1.3 Fires in the Lift Car

Due to the requirement for Group 1 or Group 2 linings for the lift car, a large ignition source would be required which would be more likely to be malicious rather than accidental. Bennetts et al.[30] identified scenarios where such fires could achieve flashover within the lift car that would burn through the lift car structure and threaten the shaft and structures within it, while acknowledging that such events would be very rare.

Based on full-scale experiments, a design fire exposure of 850°C for 25 minutes for the evaluation of steel structural elements was recommended. This exposure has been adopted for evaluation of the fire-protected timber shafts.

The enclosure/fully developed fire model used for the evaluation of apartment fires was adapted by inputting the above design fire rather than using the apartment dimensions and fire load to generate a series of exposures.

The NCC generally requires the fire-protective linings for fire-protected timber to prevent interface temperatures exceeding 250°C for 45 minutes equivalent fire resistance period. A standard deviation of 10% (4.5 minutes) was assumed for the model.

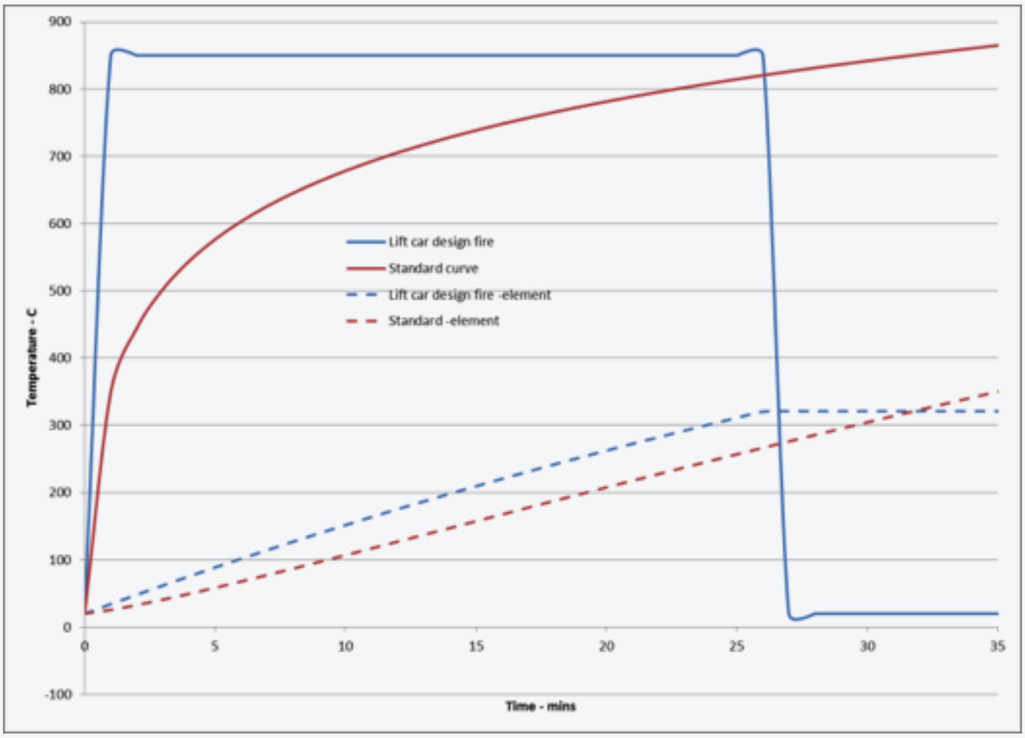


Figure 13.1: Lift car design fire and standard heating regime.

The design fire exposure was equivalent to about 32 minutes fire resistance period as shown in Figure 13.1.

Fire brigade intervention was based on the estimates for a fire on the top floor (slowest response).

The modelling indicated that the fire would burnout before the interface temperature exceeded 250°C without fire brigade intervention in 99.8% of scenarios, and that in the majority of the remaining 0.2% of scenarios fire brigade intervention would be likely to occur before the temperatures of the timber interface exceed 250°C. In an extremely small proportion of scenarios (approx. 2.8×10^{-4}), fire brigade intervention occurs marginally after the 250°C interface temperature is exceeded. As this equates to a frequency of 1.4×10^{-9} fires/annum, and the timber frame shaft wall is required to achieve a fire resistance of 90 minutes, it is considered that the timber frame NCC 2016 requirements provide adequate protection against this scenario with outcomes similar to the control building.

For massive timber, the NCC Deemed-to-Satisfy Provisions require fire protection linings within lift shafts to prevent the interface temperature reaching 300°C for 20 minutes. A standard deviation of 10% (2 minutes) was assumed for the model and fire brigade intervention was based on the estimates for a fire on the top floor (slowest response).

Due to the lesser level of protection, the timber substrate would exceed 300˚C in most cases unless fire brigade intervention occurred before the temperature limit was exceeded. Fire brigade intervention has been estimated to occur prior to the 300°C temperature limit being exceeded in approximately 6.1% of scenarios with ignition expected in the remaining scenarios as shown in Figure 13.2. However, full burnout of the lift car would occur prior to failure of the massive timber walls, which are required to achieve an FRL of 90/90/90 minutes and also have a high inherent fire resistance making them less susceptible to gross defects.

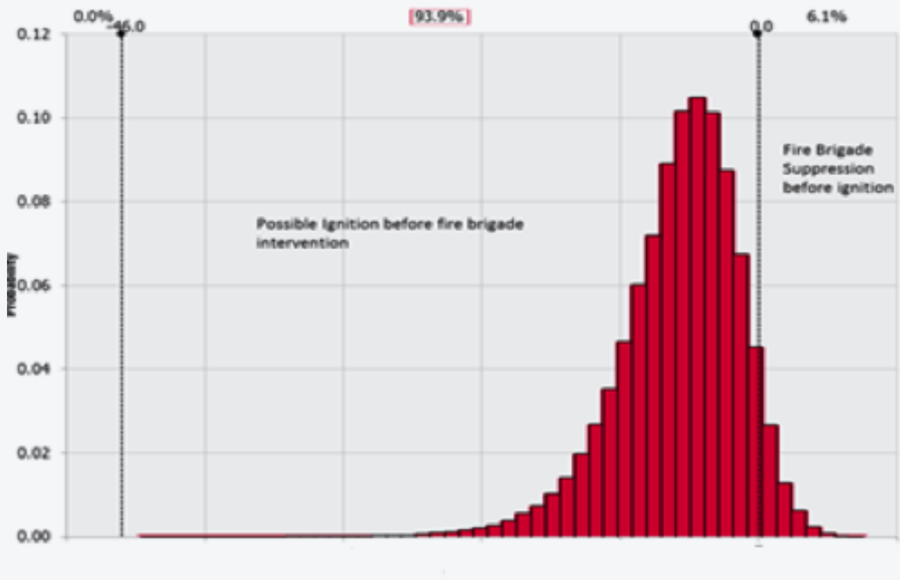


Figure 13.2: Fire brigade intervention outcomes (limit time – fire brigade intervention time) – massive timber.

The consequences of a fully developed fire occurring in the lift car (low frequency – estimated at less than 5 x 10^{-6} fires/annum) would tend to be localised with damage to the fire protection linings and possible ignition of the massive timber substrate being restricted to the area around the lift car. Since the remainder of the linings are likely to remain in place the fire would be unlikely to spread throughout the lift shaft and would be expected to be supressed by the fire brigade with smoke spread to the remainder of the building being similar to an equivalent fire in a building of non-combustible construction. It was therefore considered that the massive timber requirements provide adequate protection against this scenario with outcomes similar to the control building for mid-rise sprinkler protected buildings.

13.2 Lift Shaft Fire Conclusions

The coverings required for fire-protected timber applicable to general timber construction were found to be likely to prevent ignition of the timber substrate in the rare event of a fire in a lift shaft or lift car.

The coverings required for fire-protected timber in massive timber construction were found to be likely to prevent ignition of the timber substrate for most lift shaft fires, except for a fully developed fire in a lift car (which is a very rare event). The level of damage under these circumstances was expected to be localised and the additional risk exposure to occupants is considered low. Having regard for the low frequency of these fires and expected consequences, this level of damage was considered acceptable.

14 Fires in Concealed Spaces

14.1 Concealed Space Fires – Background

Cavities within fire-resisting construction can provide paths for the spread of fire that can bypass the boundaries of a fire-resisting compartment, potentially compromising a fire safety strategy. Spread through cavities can be accelerated by the presence of combustible materials and linings within the cavities.

Prior to the 2016 edition, the NCC addressed this risk, where considered appropriate, by requiring ceiling membranes to be resistant to the incipient spread of fire in accordance with Clause A2.5 of the NCC where there are unprotected cavities connecting fire compartments.

Resistance to the incipient spread of fire is defined as follows in the NCC:

"Resistance to the incipient spread of fire, in relation to a ceiling membrane, means the ability of the membrane to insulate the space between the ceiling and a roof or ceiling and a floor above so as to limit the temperature rise of materials in this space to a level that will not permit the rapid and general spread of fire throughout the space."

Clause A2.5 of the NCC requires the resistance to the incipient spread of fire to be determined by submitting a prototype to the Standard Fire Test (AS 1530.4), which applies a temperature rise criteria of 250°C to the upper face of the ceiling membrane.

In other countries, such as the UK, a different approach is adopted whereby cavity barriers are specified to close off openings that potentially breach fire compartments within concealed cavities.

The NCC 2016 adopted a combination of these approaches to provide a robust solution in addition to requiring automatic fire sprinkler protection.

14.2 Concealed Space – Fire Scenarios

There are two main fire scenarios to consider for fire-protected timber elements:

- Flashover fires with sufficient intensity to penetrate the fire-protective linings and ignite the substrate. Once a timber element is ignited, fire can spread through the cavity – potentially bypassing fire compartment boundaries.
- Fires initiating within the cavities that ignite combustible materials and spread through the cavity – potentially bypassing fire compartment boundaries.

These risks associated with these scenarios are analysed in Appendix I.

14.3 Fire Spread to Cavity from Fully Developed Fire

With the NCC 2016 Deemed-to-Satisfy solution for mid-rise timber buildings in place, the estimated frequency of a potential fully developed fire spreading to cavities (spreading through the cavity past cavity barriers and breaking out into another fire compartment) was estimated to be approximately 1×10^{-6} fires /annum for the building being analysed.

If this occurs, the consequences are not expected to be severe as the onset of untenable conditions and structural damage would be expected to be slow, providing time for search and rescue and evacuation. If a major structural failure was to occur and if the building is designed to resist disproportionate collapse, the failure would expect to be localised.

The risk to occupants was therefore considered acceptably low with the combination of measures specified in the NCC 2016 edition.

14.4 Fires Initiating in Cavities

With the NCC 2016 Deemed-to-Satisfy solution for mid-rise timber buildings in place, the estimated frequency of a fire igniting within the cavity and not being suppressed during the early stages of a fire was estimated to be approximately 2×10^{-5} fires per annum.

The consequences are not expected to be severe if this occurs, due to the requirements for cavity barriers and controls applied to insulating materials. The fires would be expected to be suppressed prior to causing significant damage to adjacent apartments or other fire compartments and, even if the fire progressed unchecked, the onset of untenable conditions and structural damage would be expected to be slow, providing time for search and rescue and evacuation. If a major structural failure was to occur and if the building is designed to resist disproportionate collapse, the failure would be expect to be localised.

The risk to occupants was therefore considered to be acceptably low with the combination of measures specified in the NCC 2016 edition for mid-rise timber buildings.

15 External Fire Spread Building Façade

15.1 Fire Spread from a Fire within the Subject Building

This scenario comprises fire spread due to a flashover fire occurring within the building and spreading to the floor above via windows and other openings. This mode of fire spread can occur with non-combustible construction as well as combustible construction due to flames extending from the fire compartment which tend to adhere to the façade above the opening. If the flame extension is long enough and flame temperature high enough fire spread can occur to the level above, by-passing internal compartmentation. The existing NCC Deemed-to-Satisfy Provisions generally address this to some extent by means of vertical separation of openings in walls. The relevant NCC clause is stated below:

C2.6 Vertical separation of openings in external walls

(a) If in a building of Type A construction, any part of a window or other opening in an external wall is above another opening in the storey next below and its vertical projection falls no further than 450 mm outside the lower opening (measured horizontally), the openings must be separated by –

(i) a spandrel which –

(A) is not less than 900 mm in height; and

(B) extends not less than 600 mm above the upper surface of the intervening floor; and

(C) is of non-combustible material having an FRL of not less than 60/60/60; or

(ii) part of a curtain wall or panel wall that complies with (i); or

(iii) construction that complies with (i) behind a curtain wall or panel wall and has any gaps packed with a non-combustible material that will withstand thermal expansion and structural movement of the walling without the loss of seal against fire and smoke; or

(iv) a slab or other horizontal construction that –

(A) projects outwards from the external face of the wall not less than 1100 mm; and

(B) extends along the wall not less than 450 mm beyond the openings concerned; and

(C) is non-combustible and has an FRL of not less than 60/60/60.

(b) The requirements of (a) do not apply to –

(i) an open-deck car park; or

(ii) an open spectator stand; or

(iii) a building which has a sprinkler system complying with Specification E1.5 installed throughout; or

(iv) openings within the same stairway; or

(v) openings in external walls where the floor separating the storeys does not require an FRL with respect to integrity and insulation.

(c) For the purposes of C2.6, window or other opening means that part of the external wall of a building that does not have an FRL of 60/60/60 or greater.

The mid-rise timber buildings satisfy this clause through the provision of an automatic fire sprinkler system whereas the control building would comply with the options stated in clause C2.6 (a), for example a 900 mm high spandrel panel.

With the increased proportion of plastics making up fire loads in modern apartment buildings there is a trend for a greater proportion of fully developed fires to be highly ventilation controlled during the early post-flashover period until the smaller/less massive components that are easily volatised are consumed after which the fire will move towards stoichiometric conditions and finally fuel controlled conditions before entering a decay phase. Highly ventilation controlled fires can yield longer flame extensions from the fire compartment that can cause the separation distances specified in C2.6 to be ineffective. This is demonstrated in Table 15.1 taken from England and Eyre[31] which shows measurements of heat flux and temperature taken 1.5 m and 3 m above an opening in a full scale façade test. The exposure during the ventilation controlled phases would be expected to cause breakage of windows and subsequent fire spread at distances above the minimum 900 mm separation distance specified in NCC clause C2.6.

Table 15.1: Summary of key results from a Façade test during different stages of a fire taken from England P and Eyre M[44].

Test reference	Test time (minutes)	Burning regime	Enclosure Temp (°C)	Heat flux 1.5 m above opening (kW/m²)	Heat flux 3 m above opening (kW/m²)	Temp 1.5 m above opening (°C)	Temp 3.0 m above opening (°C)
Control	2	Growth (fuel controlled)	50	2	1	46	39
Balcony			67	1	1	53	24
Control	20	Strong vent controlled	813	104	43	1000	741
Balcony			831	67	15	639	461
Control	28	Vent controlled	1018	65	29	777	433
Balcony			1029	41	11	467	386
Control	35	Stoichiometric (approximate)	1090	30	18	636	417
Balcony			1088	13	5	312	313
Control	40	Decay phase (fuel controlled)	785	20	12	467	303
Balcony (removed)			763	17	5	420	262

In addition to the above, the fire-protective coverings required for fire-protected timber were shown in the previous sections to provide adequate protection of the timber elements and the coverings are required to be non-combustible, it was considered that the provision of automatic fire sprinklers in the proposed mid-rise buildings would be more effective than a 900 mm non-combustible spandrel panel. Therefore with respect to this mode of fire spread mid-rise timber buildings are considered to present a lower risk than the control building predominantly due to the provision of automatic fire sprinklers.

15.2 Ignition of the External Façade by Burning Materials/Equipment

Specification C1.1 of the NCC requires external walls to be non-combustible (Clauses 3.1(b) and 4.1(b)) for Type A and B construction and Clause C1.13 "Fire-protected timber: Concession" permits the use of fire-protected timber wherever an element is required to be non-combustible subject to certain conditions. This means that for mid-rise timber buildings, if fire-protected timber is used for external walls, non-combustible covering materials that will prevent ignition of the timber structural members for the equivalent fire resistance test periods of approximately 30 minutes for massive timber and 45 minutes for timber-framed construction must be used, among other things. One layer of 16 mm fire-grade plasterboard and two layers of 13 mm fire-grade plasterboard are Deemed-to-Satisfy these criteria respectively.

These coverings would therefore be expected to resist open fires on balconies and adjacent structures which would be expected to be less severe than an enclosure fire.

Further confidence can be derived by considering the typical fire sources that were developed as described in previous sections for materials introduced into fire-isolated passageways including a mattress and timber crib. Protection against ignition was demonstrated by a single layer of 12.5 mm standard-grade plasterboard located directly above the timber crib.

Ignition of the façade (including the underlying timber) is therefore considered unlikely prior to burnout of small/medium fire sources on balconies or adjacent to the buildings and the probability of such an occurrence would be similar to the control building with unprotected openings providing the greatest weakness.

It should be noted that any additional components such as weather resistant coverings and rain screening must still be non-combustible to comply with the NCC Deemed-to-Satisfy solutions since they form part of the external wall.

Fire spread from large ignition sources (i.e. adjacent structures) is considered in Section 16.

16 Fire Spread Between Buildings

16.1 Fire Spread from Fire-protected Timber Buildings

The risk of fire spread from the subject building to adjacent buildings is considered to be substantially less for the fire-protected timber buildings, compared to an NCC Deemed-to-Satisfy building, because the majority of potential fully developed fires will be suppressed prior to flashover.

For the small proportion of fires involving failure of an automatic fire sprinkler system, the fire severity for fire-protected timber buildings would be expected to be similar to a fire in a building complying with the requirements of the NCC 2015 edition for non-combustible construction, based on the analysis described in earlier sections of this Guide. It was therefore considered that no further analysis of this scenario was required.

16.2 Fire Spread From Adjacent Buildings and Allotments to Fire-protected Timber Buildings

16.2.1 Derivation of Acceptance Criteria for Fire Spread from Adjacent Structures or Allotments

The risk of fire spread to fire-protected timber buildings from adjacent buildings and allotments was assessed based on the radiation exposures nominated in Verification methods CV1 and CV2 of the NCC which state:

CV1

Compliance with CP2(a)(iii) to avoid the spread of fire between buildings on adjoining allotments is verified when it is calculated that—

(a) a building will not cause heat flux in excess of those set out in column 2 of Table CV1 at locations within the boundaries of an adjoining property set out in column 1 of Table CV1 where another building may be constructed; and

(b) when located at the distances from the allotment boundary set out in column 1 of Table CV1, a building is capable of withstanding the heat flux set out in column 2 of Table CV1 without ignition.

Table 16.1: Table CV1 from NCC 2015.

Column 1	Column 2
Location	**Heat Flux (kW/m²)**
On boundary	80
1 m from boundary	40
3 m from boundary	20
6 m from boundary	10

CV2

Compliance with CP2(a)(iii) to avoid the spread of fire between buildings on the same allotment is verified when it is calculated that a building—

(a) is capable of withstanding the heat flux set out in column 2 of Table CV2 without ignition; and

(b) will not cause heat flux in excess of those set out in column 2 of Table CV2, when the distance between the buildings is as set out in column 1 of Table CV2.

Note Refer to Section 16.1 for assessment of fire spread from fire-protected timber buildings

Table 16.2: Table CV2 from NCC 2015.

Column 1	Column 2
Distance between Buildings	**Heat Flux (kW/m²)**
0 m	80
2 m	40
6 m	20
12 m	10

From Table 16.1 and Table 16.2, the maximum radiant heat flux a building is required to resist at any distance from a boundary or adjacent building is 80kW/m² and the maximum radiant heat flux 1 m from a boundary or 2 m from an adjacent building on the same allotment is 40kW/m².

The duration of high intensity burning during a typical enclosure test lasts approximately 20–30 minutes as shown in Figure 16.1, which is taken from England and Eyre[31].

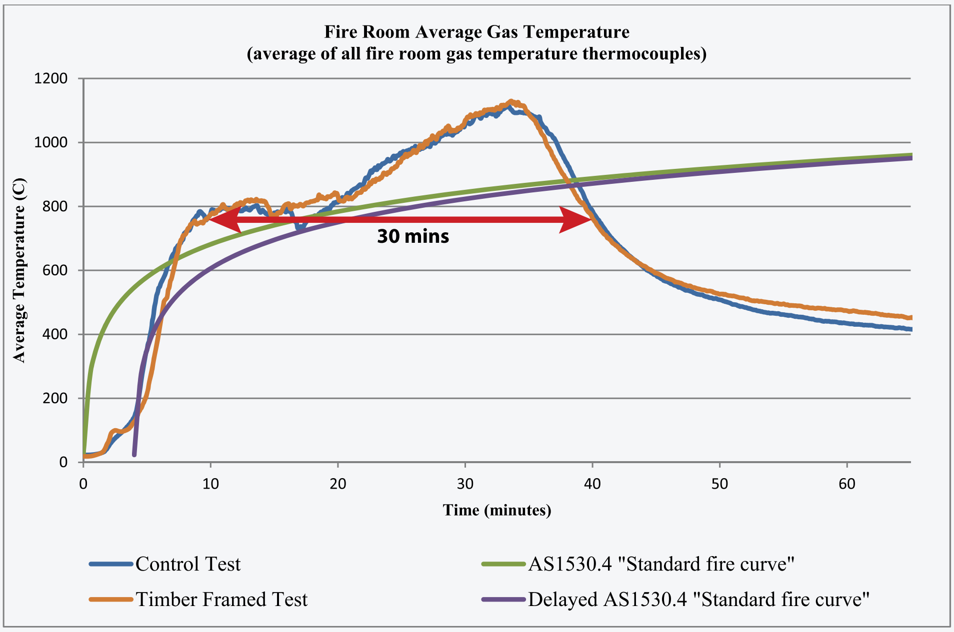


Figure 16.1: Typical fire scenario showing high enclosure temperatures for approximately 30 minutes.

Therefore the following acceptance criteria were adopted:

The temperature of the timber substrate of the Fire-protected Timber shall not exceed 300°C when

a) exposed to an incident radiant heat flux of 80kW/m² for 30 minutes for general application, or

b) exposed to an incident radiant heat flux of 40kW/m² for 30 minutes for buildings that are not less than 1m from the allotment boundary or 2m from an adjacent building on the same allotment.

16.2.2 Analysis of Fire Spread from Adjacent Structures or Allotments

Olsson[32] reported a series of cone calorimeter tests exposing timber specimens protected by plasterboard to radiant heat. Subsequently, Tsantaridis[33] also undertook a large series of cone calorimeter tests that incorporated a larger number of plasterboard-protected timber specimens. In both studies, the majority of the tests were undertaken with radiant heat fluxes of 50kW/m².

Tsantaridis fitted a 2nd order polynomial to the data based on a larger data set than that used for a linear correlation proposed by Olsson. The Tsantaridis correlation has been used for this study which is shown in Equation 1 and Figure 16.2 because of the larger data set and the 2nd order polynomial expression derived being more consistent with theory.

Equation 1 $T_{300} = 0.0796t^2 + 0.7144t$

where T_{300} is the time to 300°C – min, and t is the board thickness – mm

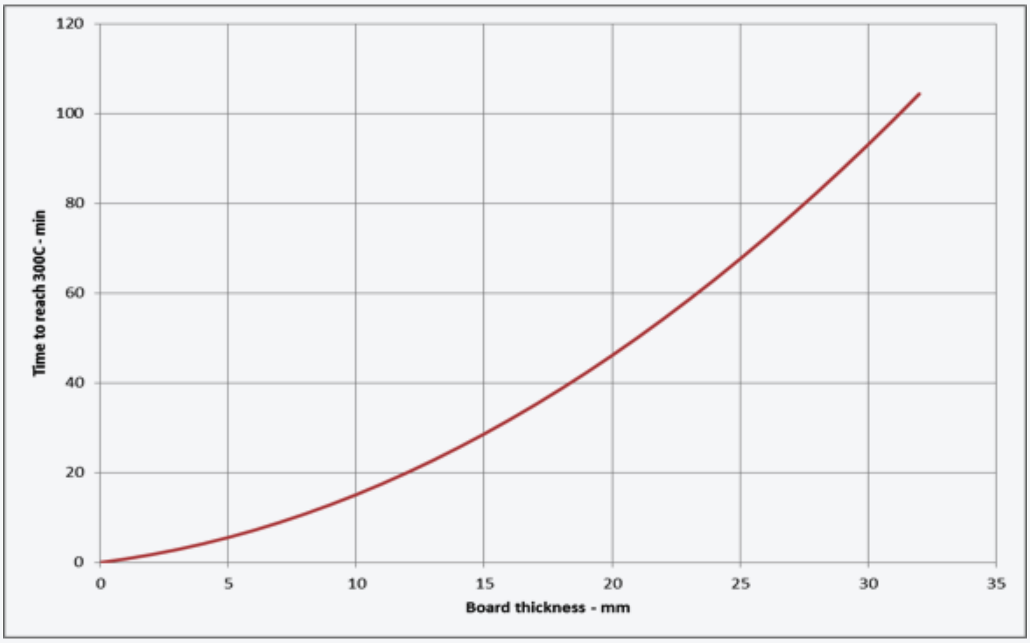


Figure 16.2: Time for timber interface temperature of 300⁰C when protected by differing plasterboard thicknesses and exposed to 50 kW/m² incident radiant heat flux based on Tsantaridis correlation.

It can be seen from Figure 16.2 that both massive timber and timber-framed construction protected by 16 mm and 26 mm plasterboard respectively would be expected to maintain interface temperatures below 300°C for more than 30 minutes, when exposed to a radiant heat flux of 50kW/m², therefore satisfying the criteria of 40kW/m² for buildings sited more than 1 m from the boundary and 2 m from other buildings on the same allotment.

Olsson reported results from timber elements protected by 12.5 mm plasterboard exposed to radiant heat fluxes varying from 25kW/m² to 100kW/m² which are plotted together with the time for the interface to attain 300°C in Figure 16.3.

From Figure 16.3, an incident radiant heat flux of 80kW/m² corresponds to the time for the timber interface to attain 300°C, of approximately 17.5 minutes.

Considering the form of the relationship between board thickness and time to attain an interface temperature, it is conservative to assume (i.e. under-predicts the time to reach a critical interface temperature) that doubling the board thickness will double the time for the interface to achieve a particular critical temperature when exposed to the same incident radiant heat flux, provided the boards remain in place.

Since two layers of 12.5 mm fire-grade plasterboard facings applied to timber studs have demonstrated their ability to remain in place for standard fire resistance tests of the order of 90 minutes, it is reasonable to assume they will remain in place when exposed to 80kW/m² for at least 30 minutes.

Therefore, two layers of 12.5 mm fire-grade plasterboard would be expected to prevent the interface temperature exceeding 300°C for in excess of 35 minutes when exposed to a radiant heat flux of 80kW/m², satisfying the criteria for buildings sited on the boundary and with 0 m clearance from other buildings on the same allotment.

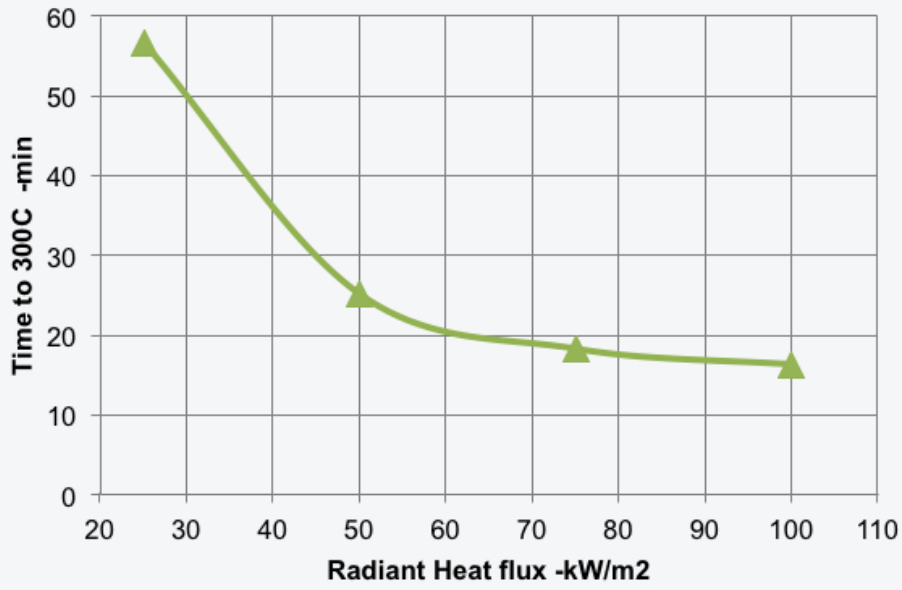


Figure 16.3: Time for timber interface temperature of 300°C when protected by 12.5 mm thick plasterboard and subjected to different radiant heat fluxes – Olsson results.

16.2.3 Results and Assessment of Fire Spread from Adjacent Structures or Allotments

For timber protected by the equivalent of two layers of 13 mm thick fire-grade plasterboard, the timber substrate of the fire-protected timber would not be expected to exceed 300°C when exposed to an incident radiant heat flux of 80kW/m^2 for 30 minutes, satisfying the acceptance criteria for all separation distances.

For timber protected by the equivalent of one layer of 16 mm thick fire-grade plasterboard, the timber substrate of the fire-protected timber would not be expected to exceed 300°C when exposed to an incident radiant heat flux of 40kW/m^2 for 30 minutes, satisfying the acceptance criteria for separation distances greater than 1 m from the boundary and 2 m from adjacent buildings on the same allotment.

The NCC 2016 Deemed-to-Satisfy requirements for fire-protected timber buildings are consistent with these findings, with the same levels of protection for massive timber and general timber construction of external walls being required within 1 m of the boundary or 2 m of adjacent buildings.

17 Application of Class 2 & 3 Findings to Class 5 Buildings

A supplementary analysis of Class 5 (office) buildings was undertaken, drawing heavily on the analysis undertaken for Class 2 and 3 buildings.

Further details of the supplementary analysis undertaken are provided in Appendix K: Class 5 Office Analysis and are summarised below:

A review of fire statistics indicates that Class 5 buildings present a much lower fire risk than Class 2 and 3 buildings, mainly due to the occupancy characteristics.

The analysis of fire incident data indicated that occupant safety would be significantly improved and fire losses reduced within the fire compartment of fire origin as a consequence of the provision of automatic fire sprinkler systems in the timber buildings, compared to the control building without fire sprinklers – as was the case with Class 2 and 3 buildings.

A supplementary analysis was undertaken to analyse the impact of a fully developed fire outside normal working hours. Since the building may unoccupied, the alarm to the fire brigade may not be received until substantially after flashover if the detection systems are unmonitored for the control building or the sprinkler system fails for the timber options.

Since the building would be likely to be unoccupied or have a very low occupancy rate at this time, it was decided that a simple event tree approach with point probabilities was appropriate, instead of the Monte Carlo approach used previously. Details are given in Appendix K4.3. The outcomes are summarised in Table 17.1.

Table 17.1: Event tree analysis of fully developed office fires.

Ref	Outcome	Probability of Outcome		
		Control Building	Massive Timber	Timber-framed
A	Sprinkler controlled	0	0.88	0.88
B	Fire brigade Intervention before ignition of structural element	0	0.1042	0.1130
C	Fire brigade intervention before equivalent FRL period	0.989961	0.012336	0.003504
D	Compartment withstands burnout without FBI	0.00894736	0.001622	0.002672
E	Fire spread without major collapse	0.00107984	0.001880	0.000830
F	Major structural collapse	0.000012	0.000002	0.000002

Outcome A relates to the successful activation of a sprinkler system. Approximately 88% of flashover fires would be expected to prevent in the timber buildings substantially reducing the fire losses and risk to any occupants outside normal working hours.

Outcome B applies only to the timber buildings. If the timber is not ignited the fire-fighting activities and risks would be similar to outcome C for the control building.

Outcome C relates to outcomes where the fire brigade suppress a fully developed fire prior to FRLs being exceeded. In this case the fire-fighting activities may be more complex for the timber buildings because the underlying timber elements may have ignited but the probability of occurrence for timber buildings is low.

Outcome D has the same consequence for all the buildings but the probability is less for the timber buildings, mainly because of the early suppression of most fires by the fire sprinkler systems. The value for massive timber is less than that for timber-framed construction because the fire preventative coverings provide protection for an equivalent of 30 minutes, compared to 45 minutes fire resistance for timber-framed construction.

Outcome E has the same consequence for all buildings but the probabilities vary. Due to the lower performance of the fire-protective coverings in conjunction with a delayed call to the fire brigade, the probability of this outcome is highest for massive timber construction. The lowest probability is for the timber-framed building, because of the higher level of performance required for the fire-protective coverings, which increases the proportion of fires that will burnout if there is no fire brigade intervention compared to the massive timber option.

Outcome F relates to a major structural collapse and the probability of occurrence is higher for the control and the same for the timber options.

Due to the provision of automatic fire sprinklers, fire fighters would face substantially fewer medium rise flashover fires, reducing the risk to fire fighters and minimising fire losses. However, in the rare event of failure of an automatic fire sprinkler system in conjunction with a severe flashover fire occurring and slower than average fire brigade response, there is a risk of the fire involving structural timber members and modified fire-fighting practices may be required.

It was determined that the analysis undertaken for Class 2 and 3 buildings relating to the following fire scenarios was applicable to Class 5 buildings:

- fires in fire-isolated stairs and passageways
- fire spread via the façade
- fire spread between buildings
- fires in lifts
- fire spread via concealed spaces.

Based on the above supplementary analysis, it was determined that:

- the acceptance criteria for the proposed timber mid-rise buildings was that the timber buildings should provide at least an equivalent level of fire safety to a building constructed in accordance with current NCC 2015 Deemed-to-Satisfy requirements. The analysis showed that this could be achieved with the NCC 2016 Deemed-to-Satisfy requirements for mid-rise timber buildings.

The Class 2 analysis indicated that although the risk of global collapse is very low, the frequency was dominated by the presence of gross defects and the inherent fire resistance of the base structural members. The additional analyses confirmed this applies also to Class 5 buildings, but the probability is increased outside normal working hours because fire brigade intervention times could be substantially delayed if there is no automatic alarm sent to the fire brigade, since there may also be no occupant or passer-by to manually call the fire brigade. The consequences with respect to life safety are, however, lower because the buildings are generally either unoccupied or have low levels of occupancy outside normal working hours.

D

Part D - Appendices

Appendix	Title
A	Response of Timber Buildings to Fires
B	Matters Raised by Stakeholders and Other Parties
C	Relevant Deemed-to-Satisfy Clauses NCC 2015
D	Performance Requirement Review
E	Determination of Compliance of Fire-protected Timber
F	Analysis of Fire Data
G	Multi-scenario Quantitative Risk Assessment Supplementary Data
H	Summary of UK Timber Frame 2000 Project – UK
I	Analysis of Fire Spread Via Concealed Spaces (Incipient Spread of Fire)
J	Comment on Structural Design Implications
K	Class 5 Office Analysis
L	Peer Review Letter
M	References

A

Appendix A - Response of Timber Buildings to Fires

A.1 Overview of Enclosure Fires

Figure A1 provides a general overview of the progression of a typical enclosure fire.

After ignition, a fire may spread to involve other items in the enclosure or self-extinguish, depending on many factors such as the ignition source; type of materials/objects first ignited; proximity to other objects and linings; and the material properties of the adjacent objects and linings.

The main focus of the NCC is on control of the fire properties of enclosure linings and elements of construction, rather than control of the building contents for residential and office buildings.

If a fire continues to grow it may be supressed or controlled by automatic systems, such as fire sprinkler systems if they are present, or manually by the occupants.

If these interventions do not occur or are unsuccessful and the fire continues to grow, it will tend to transition from a fire involving one or more items to a fully developed fire involving all exposed combustible materials within an enclosure. This transition is commonly referred to as flashover. Fire brigade intervention may occur prior to flashover or post-flashover, depending, among other things, on the call out time, response time and growth rate of the fire and may prevent flashover occurring in some instances or reduce the severity/duration of a fully developed fire.

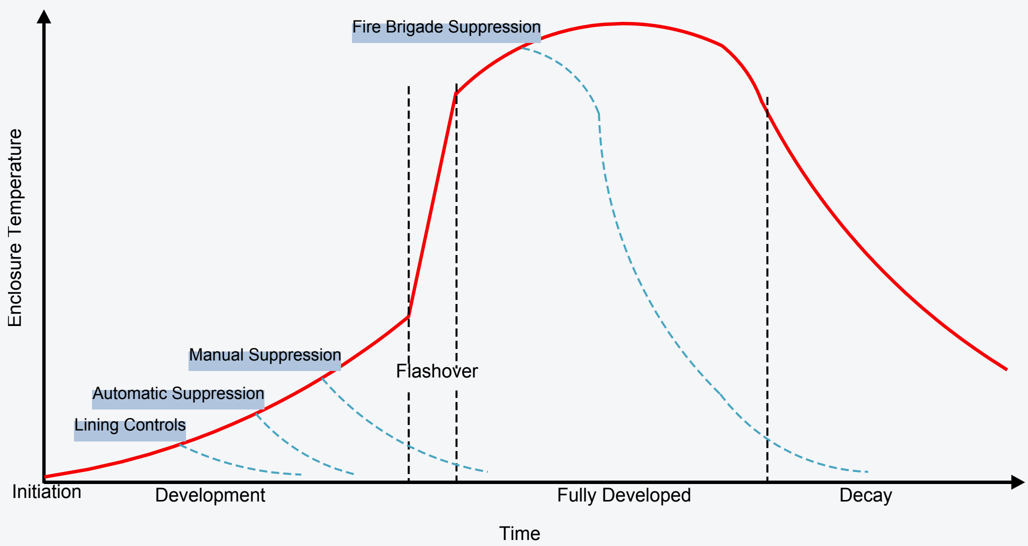


Figure A1: Enclosure fire stages.

Depending upon the ventilation conditions, amount of fuel and fuel characteristics, a fire may be fuel controlled or ventilation controlled. In many instances, a fully developed fire may initially be ventilation controlled immediately after flashover and transition to a fuel-controlled fire as the rate of production of volatiles reduces, as fuel packages with larger exposed surface area to mass ratios and low heats of gasification are consumed. The burning regime may impact on enclosure temperatures and flame extension from openings. A useful reference for further information is *An Introduction to Fire Dynamics* by D. Drysdale[34]. Maximum enclosure temperatures tend to occur when conditions are close to stoichiometric conditions (i.e. all fuel and air is consumed in the combustion process without any excess left over. If there is no intervention, the fire will eventually decay.

A.2 Overview of Degradation of Timber at Elevated Temperatures

When exposed to temperatures above 250°C to 300°C for relatively short periods (e.g. a fire event), timber will decompose – releasing volatiles – but it will also tend to form a char layer. This provides a degree of protection to the underlying timber, such that timber elements having a large cross section can exhibit high levels of inherent fire resistance.

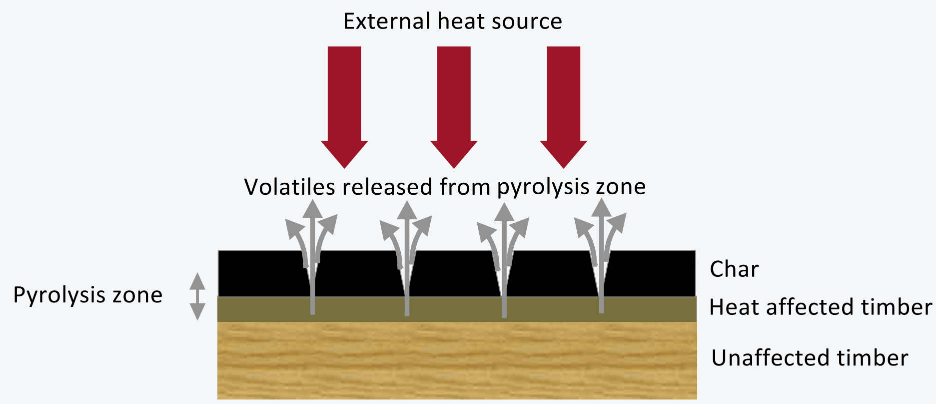


Figure A2: Schematic showing a section through a burning section of timber.

The rates of charring and production of volatiles depend on a number of variables. These include: timber species, cross-section, external heat flux, presence of inorganic impurities and moisture content; but for some engineering applications 'standard' design values are commonly adopted, such as those in AS 1720.4.

Adhesives and other fixings can also influence the response of engineered timber elements to elevated temperatures. For example, some types of adhesive can cause premature loss of the protective char layer, accelerating both the production of volatiles and char rate.

For a more detailed discussion of the burning behaviour of timber, refer to appropriate references such as Drysdale[34].

A.3 Summary of Fire-protected Timber Performance Criteria

Within the context of the NCC 2016, fire-protected timber is a defined term and compliance with Specification A1.1 is required. Specification A1.1 states that fire-protected timber is required to achieve the specified FRL of the building element, and have a non-combustible fire-protective covering applied to the timber which achieves a resistance to the incipient spread of fire (RISF) of not less than 45 minutes when tested in accordance with the relevant requirements of AS 1530.4.

AS 1530.4 applies a maximum temperature limit of 250°C for resistance to the incipient spread of fire. The NCC deems the 45 minute incipient spread of fire criteria to be satisfied if at least two layers of 13 mm thick, fire-protective-grade plasterboard are fixed in accordance with the requirements to achieve the required FRL for the element.

A relaxation is permitted for massive timber panels, provided the timber is at least 75 mm thick and there are no cavities between the surface of the timber and the fire-protective covering, or between timber members. If all these conditions are met, the modified resistance to the incipient spread of fire (MRISF) criteria may be applied, which require the temperature at the interface of the protection system and the timber to be not greater than 300°C during a fire resistance test performed in accordance with AS 1530.4 for the periods listed in Table A1.

Table A1 also includes Deemed-to-Satisfy minimum thickness of fire-protective-grade plasterboard .

Table A1: Massive timber panel – modified resistance to the incipient spread of fire (MRISF) requirements.

Requirements	Application	MRISF -min	Minimum Deemed-to-Satisfy fire-protective-grade plasterboard
Relaxed requirements for timber elements not less than 75 mm x 75 mm without cavities/voids or cavities/voids filled with non-combustible material	Inside a fire-isolated stairway or lift shaft	20	1 layer x 13 mm thick
	External walls within 1 m of an allotment boundary or 2 m of a building on the same allotment	45	2 layers x 13 mm thick
	All other applications	30	1 layer x 16 mm thick

Refer to Appendix E3 of this Guide for a more detailed description of how the RISF and MRISF criteria should be applied to elements such as walls, floors and service penetrations.

A.4 Enclosure Fires with Fire-protected Timber Building Elements

The objective of the fire-protective coverings is to prevent or delay ignition of the timber structural member, so that the response to an enclosure fire will be similar to non-combustible elements and masonry or concrete and so that the enclosure fire severity will not be increased due to the additional fire load presented by timber construction:

- during the growth period and
- prior to fire brigade intervention or burn-out of the contents, in the event of failure of the prescribed automatic fire sprinkler system and progression of the fire to the post-flashover phases.

The application of fire-protective coverings to timber is also known as encapsulation.

The performance of fire-protected timber was demonstrated to fulfill these objectives in comparative full scale fire tests undertaken with room enclosures lined with fire-protective-grade plasterboard[5,31].

The test configuration is shown schematically in Figures A3 and A4. It comprised an enclosure with internal dimensions 4 m x 4 m x 2.4 m high, with an opening of 2 m wide x 1.2 m high located in the centre of the front wall with the sill at a height of 0.5 m above the floor. The façade and corridor related to other research studies and are not discussed further here.

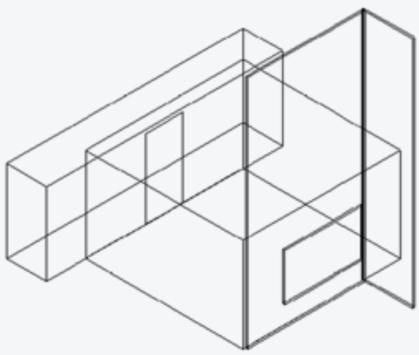


Figure A3: Schematic of test configuration used for comparative testing of fire-protected timber construction and non-combustible construction.

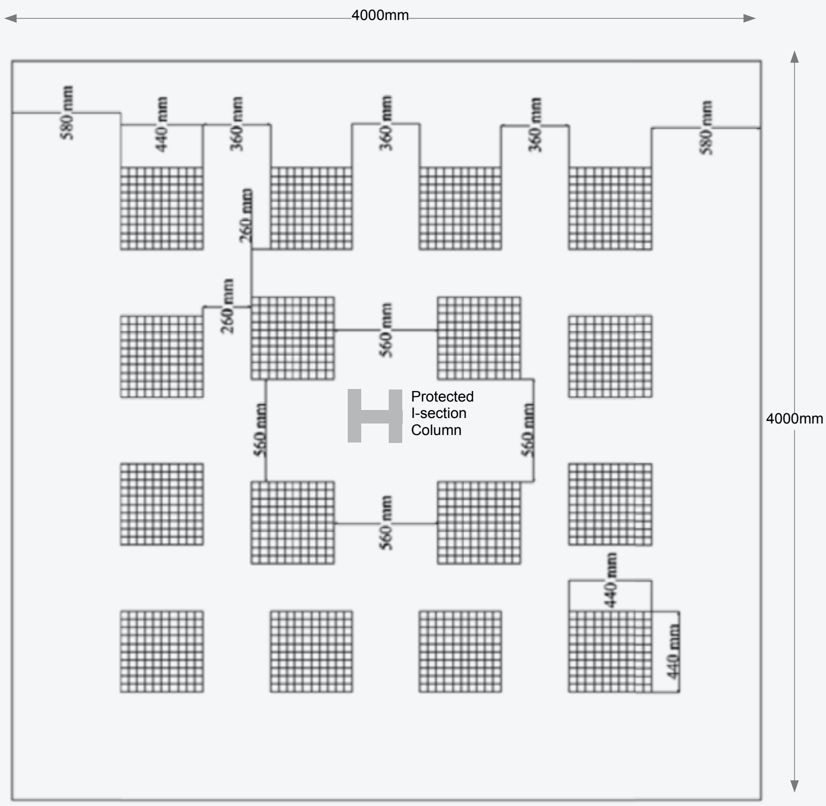


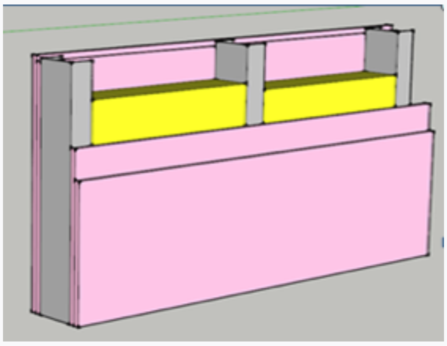
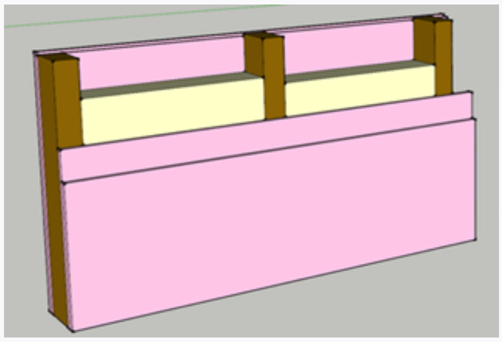
Figure A4: Plan of test enclosure showing crib layout and target fire-protected column.

A steel column protected by ceramic fibre was provided within the enclosure, as shown in Figure A5, to provide an indication of the comparative fire severity of the enclosure fires in addition to thermocouple trees measuring the enclosure temperature.

Figure A5: Test enclosure configuration showing protected steel column, timber cribs and thermocouple trees.

Both enclosures were lined with the same thicknesses of fire-protective-grade plasterboard to achieve the same nominal FRL of 90/90/90. Further details are provided in Table A2.

Table A2: Comparative test configurations.

Steel-framed non-combustible control	Fire-protected timber-framed construction
	
Wall Construction	
Steel studs Fire-protective covering – two layers of 13 mm fire-protective-grade plasterboard Non-combustible insulation	Softwood timber studs Fire-protective covering – two layers of 13 mm fire-protective-grade plasterboard Combustible insulation
Ceiling Construction	
Steel I-Joists Fire-protective covering – two layers of 16 mm fire-protective-grade plasterboard fitted to furring channels Non-combustible insulation Particleboard flooring	Timber I-Joists Fire-protective covering – two layers of 16 mm fire-protective-grade plasterboard fitted to furring channels Combustible insulation Particleboard flooring
Imposed Fire Load 740 MJ/m^2	**Imposed Fire Load 740 MJ/m^2**

The results summarised in Figure A6 show that there was no increase in the severity of the fire based on the following parameters measured during the test:

- enclosure temperature
- temperature of a protected column within the enclosure
- temperature on the non-fire side of the walls
- temperatures within the wall cavities
- temperatures on the unexposed side of the ceiling.

The comparative test fires incorporated initiation and development, and transition to fully developed and subsequent decay stages of a fire. The fully developed stage included periods of strong ventilation-controlled burning just after flashover with transition to a fuel-controlled regime.

The control test had to be terminated after 66 minutes due to failure of the ceiling system, whereas the timber-framed test was terminated after 114 minutes due to ignition of the ceiling insulation and observed burning droplets. It should be noted that the NCC requirements prohibit the use of combustible insulation in fire-protected timber construction, removing the risk of ignition of combustible ceiling insulation.

Temperatures measured within the wall cavities and on the upper surface of the ceiling confirm the earlier degradation of the steel-framed construction compared to timber-framed construction (refer Figure A6), which indicate that the steel-framed wall systems are more susceptible to degradation in performance due to rapid heating rates than timber-framed studs.

The greater susceptibility of steel-framed construction compared to timber-framed construction, when exposed to heating rates greater than the standard fire resistance test heating regime, has been observed in other studies (e.g. Li et al.[35]) and the earlier degradation can be explained to some extent by the higher thermal expansion of steel tending to open up joints and weaken fixings.

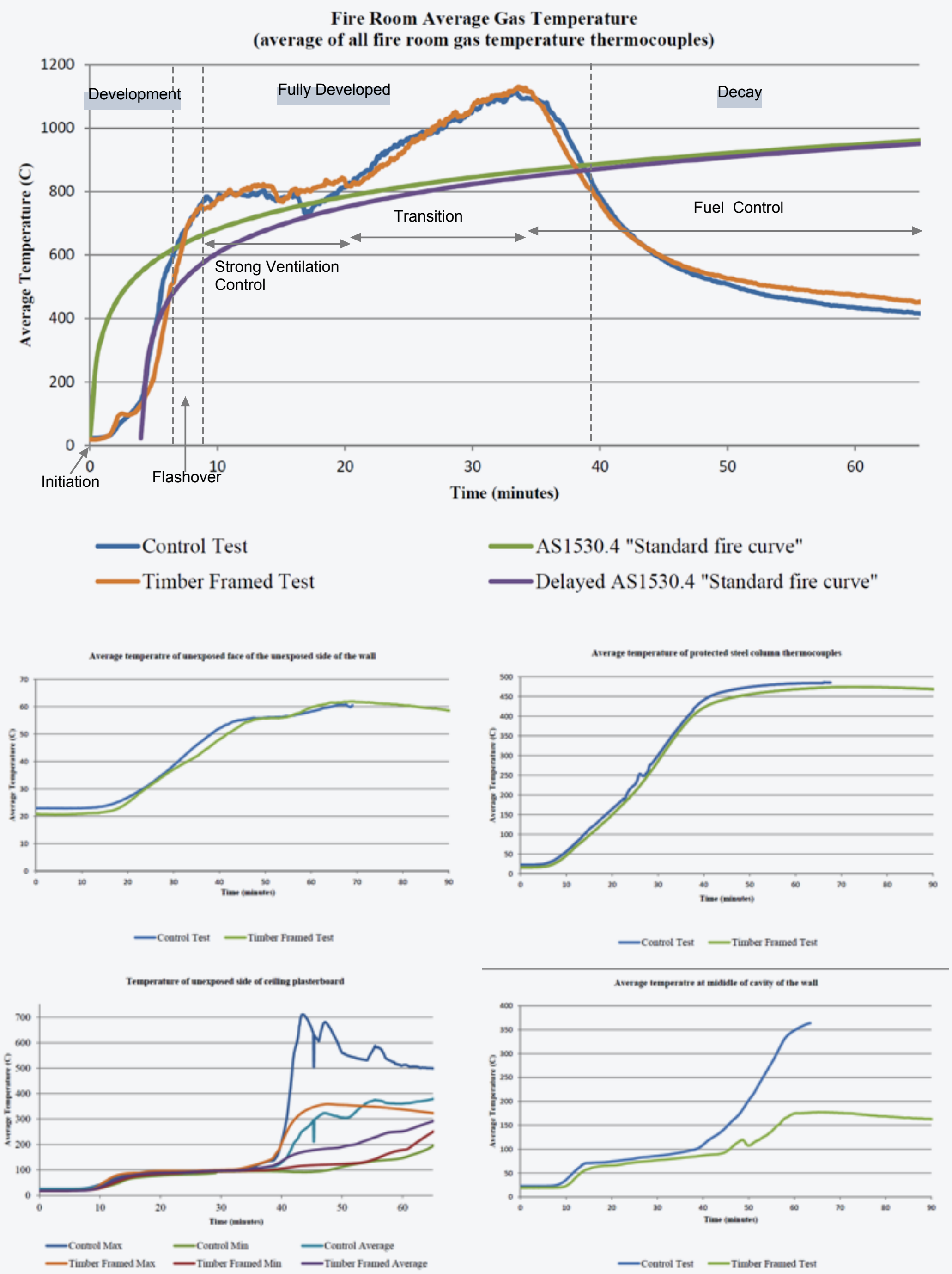


Figure A6: Results of comparative testing of non-combustible and fire-protected timber-framed construction.

The timber frame temperatures within the wall measured during the test peaked just below 300°C (see Figure A7) and the maximum temperature measured on the unexposed face of the ceiling reached a maximum of just above 350°C during the first 66 minutes, for which comparative data is available.

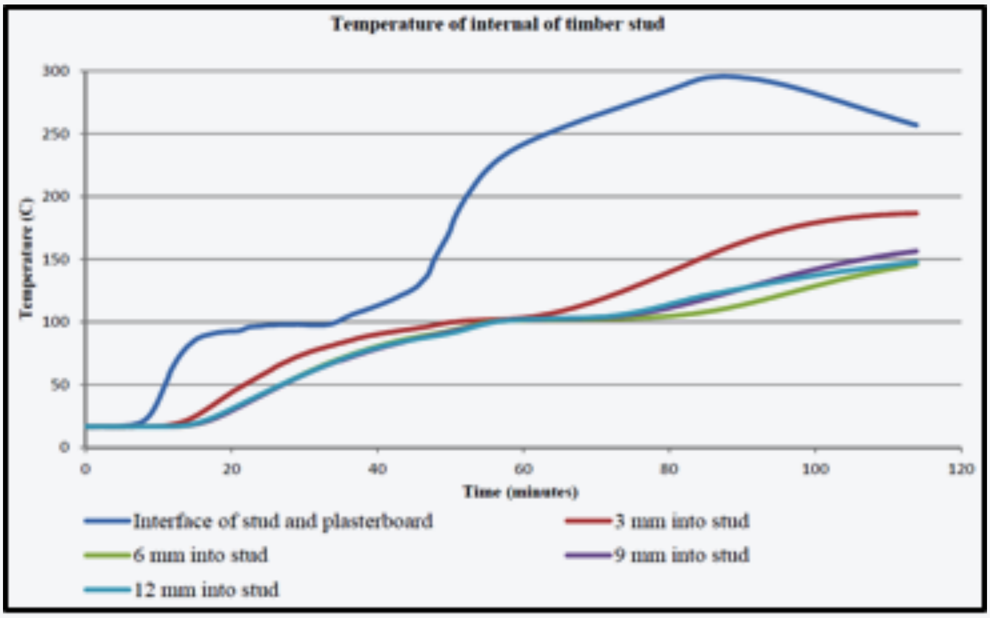


Figure A7: Timber frame wall temperatures.

The above comparative test demonstrated that the behaviour of the enclosure fire was not changed as a consequence of the use of timber construction with fire-protective linings that maintain timber temperatures below 300–350˚C. Therefore the objectives for fire-protected timber construction without fire brigade intervention or the impact of automatic fire sprinklers were satisfied.

Fire severity is a function of the ventilation conditions, fire load and lining properties. As the reliability of all fire protection systems is not 100%, the fire-protective coverings may fail in extreme cases; however, in most scenarios the fire will be suppressed or controlled as a result of automatic suppression by the sprinklers or by fire brigade intervention. This is likely to occur substantially before burnout of the contents for buildings designed in accordance with the Deemed-to-Satisfy requirements for mid-rise buildings. The multi-scenario analysis described in this Guide considered the probability and consequences of all the above scenarios.

A.5 Enclosure Fires with Exposed Timber Elements

Timber elements of construction may be exposed in buildings as part of the architectural design or may become exposed during a fully developed fire as fire-protective coverings degrade and fall away.

The impact of the additional exposed timber on enclosure fires is discussed in the following sub-sections.

A.5.1 Fire Initiation and Development

If timber elements are exposed in normal service, they will need to satisfy the relevant NCC requirements for wall and ceiling linings and floor coverings that seek to reduce the risk of lining materials and floor coverings unduly accelerating the rate of fire growth during the fire initiation and development phase (refer NCC Specification C1.10).

Since the primary focus of this Guide is mid-rise fire-protected timber buildings, which require non-combustible fire-protective coverings to be applied, the impact of combustible linings on the rate of fire growth during the fire initiation and development phase of an enclosure fire will not be considered in detail. Information on the fire performance of exposed timber linings, floor coverings and attachments during the fire initiation and development phase reference can be found through the following link, which provides test results for timber species relevant to the NCC Deemed-to-Satisfy pathway:

http://www.woodsolutions.com.au/Articles/Resources/Fire-Hazard-Properties-Floor-Coverings.

WoodSolutions Technical Design Guide #19: *Alternative Solution Fire Compliance Internal Linings* provides guidance in relation to the NCC performance pathway.

A.5.2 Fully Developed Fires

If additional timber elements are exposed to a fully developed fire, the effective enclosure fire load will be increased. The impact of this increase will primarily depend upon the burning regime at the time of exposure, the surface area of timber and mass of timber exposed, and the fire resistance of the element. The types of adhesive used in the manufacture of engineered timber products can also be significant under some circumstances.

The following cases are used to explain the potential impacts of exposed timber elements:

Case 1: Fully developed ventilation controlled fire prior to exposure of the timber.

If the fully developed fire is close to stoichiometric conditions or ventilation controlled prior to exposure of the additional timber elements, temperatures within the enclosure are unlikely to increase and may decrease – in some instances – if large areas of timber are exposed, because additional energy is consumed, degrading the wood to produce and heat volatiles that are then lost from the enclosure without undergoing combustion. Under these circumstances, flame extension from openings is likely to extend as the unburnt volatiles may undergo combustion as they mix with air outside the enclosure.

This was demonstrated in a test series reported by Hakkarainen[36]. A series of comparative natural fire enclosure tests were undertaken, which included exposed CLT panels, protected CLT panels and protected lightweight timber-frame construction. The same imposed fire load of approximately 720 MJ/m^2 of floor area and ventilation conditions were employed in all tests. The enclosure was 4.5 m x 3.5 m x 2.5 m high, with a 2.3 m wide x 1.2 m high window and also included a simulated façade above the opening.

The specimen configurations are summarised in Table A3 together with the observed performance of the plasterboard linings, test duration and reason for termination.

Table A3: Hakkarainen et al. tested constructions.

Test	Structural Elements	Fire Protection	Retention of Protection	Test Duration – mins (termination reason)
Test 1	CLT walls and floor/ceiling	None – exposed	Not applicable	50 (excessive flaming*)
Test 2	CLT walls and floor/ceiling	12.5 mm standard plasterboard	Fell away from ceiling at tops of walls approx. 18 minutes	46 (malfunction of smoke venting system)
Test 3	CLT walls and floor/ceiling	15.4 mm fire-grade plasterboard over 12.5 mm standard-grade plasterboard	First layer of boards fell away from ceiling after 27 minutes,	46 (malfunction of smoke venting system)
Test 4	Timber frame with mineral fibre insulation	15.4 mm fire-grade plasterboard over 12.5 mm standard-grade plasterboard	First layer of boards fell away from ceiling after 32 minutes	48 (burn through of ceiling)

* It is not clear whether Test 1 was terminated due to limitations of the test facility or failure of the elements of construction.

Figure A8 shows the mean enclosure temperatures as measured by the central thermocouple tree extracted from Hakkarainen[36] for each of the configurations summarised in Table A3. The time/temperature plot for the first six minutes (pre-flashover) was similar for all cases.

The mean temperature of the enclosure with unprotected CLT was similar to the CLT enclosure protected by standard plasterboard. This was probably due to the standard plasterboard degrading and falling away.

The mean temperatures of the CLT and lightweight timber-framed enclosures protected with fire-grade plasterboard were similar to each other.

It is noteworthy that the mean enclosure temperatures for the unprotected or partially protected CLT were substantially below the fully protected CLT and lightweight timber frames for the first 30 minutes after flashover.

Beyond the first 30 minutes after flashover, as the rate of burning of the fire load simulating the contents reduced, the temperatures of the fully protected timber enclosures started to reduce as the fire approached the decay phase but the temperatures of the enclosures with exposed CLT started to increase. The differing behaviours can be explained by considering changes in the burning regime within the enclosure. The combination of the contents and exposed CLT produced a strongly ventilation controlled fire after flashover, reducing the combustion efficiency. After about 30 minutes, the temperatures started to increase as the fire approached stoichiometric conditions because the fire load simulating the contents had been consumed.

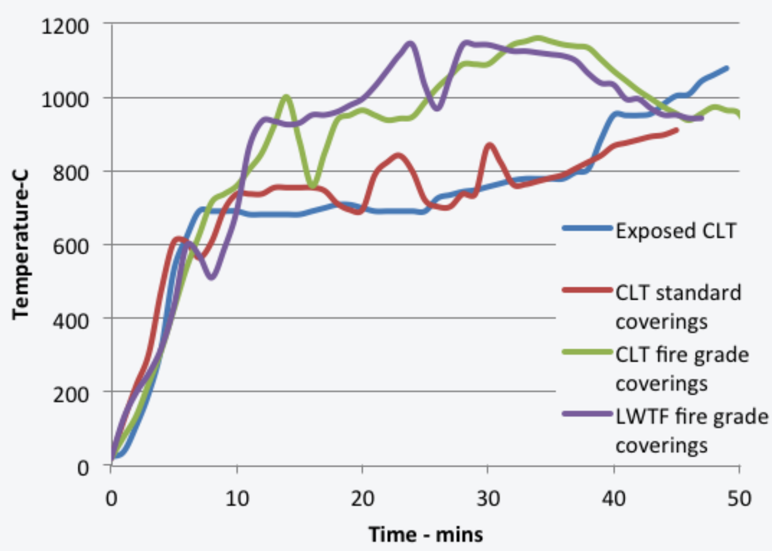


Figure A8: Average enclosure temperatures with and without plasterboard coverings extracted from Hakkarainen[36].

Unfortunately, three of the tests were terminated between 46 and 50 minutes and therefore comparisons of the potential fire duration, flame extension from enclosures and subsequent behaviour of the fire cannot be made. Similar observations were also made in a more recent study by Li, X et al.[35].

Case 2: Fully developed fuel controlled fire prior to exposure of the timber.

If the fully developed fire is fuel controlled prior to exposure of the additional timber elements, temperatures within the enclosure may increase since there may be sufficient excess air available within the enclosure for combustion of the additional volatiles produced by the timber. However, if there are large timber surfaces exposed, the increased rate of production of volatiles may change the burning regime to ventilation control and – depending upon the excess fuel factor – enclosure temperatures may decrease but as noted for Case 1 flame extension from the enclosure may increase.

Case 3: Fully developed fire transition to the decay phase

As fuel is consumed, the fire will decay and will generally be fuel controlled. The presence of additional exposed timber elements will generate additional volatiles that will tend to extend the fully developed fire phase. The magnitude of the extension of the fully developed phase will depend upon the rate of production of volatiles from the remaining contents and exposed timber element surfaces, among other things. If the contents are fully or substantially consumed before the timber elements, it is possible for the fully developed fire to continue – depending upon the enclosure configuration, the area of exposed timber surfaces, etc.

This was demonstrated in a series of tests performed by Carleton University and reported by McGregor[37] to investigate the contribution of CLT panels to room fires. The clearest comparison can be obtained from Tests 4 and 5. These tests were performed in an enclosure constructed of CLT panels with internal dimensions 3.5 m x 4.5 m x 2.5 m high with an opening 2 m high x 1.07 m wide. Furnishings/contents representing bedroom fire loads of 553MJ/m^2 and 529MJ/m^2 for Tests 4 and 5, respectively, were provided.

The CLT panels were exposed in Test 5 and protected with two layers of 13 mm fire-grade plasterboard in Test 4. In Test 4, there was no contribution from the CLT, with the plasterboard providing full protection.

From examination of the average enclosure temperatures, it can be observed that until the fire load (excluding the CLT) had been substantially consumed, the enclosure temperatures were similar – the time lines are offset to exclude the pre-flashover phase as shown in Figure A9.

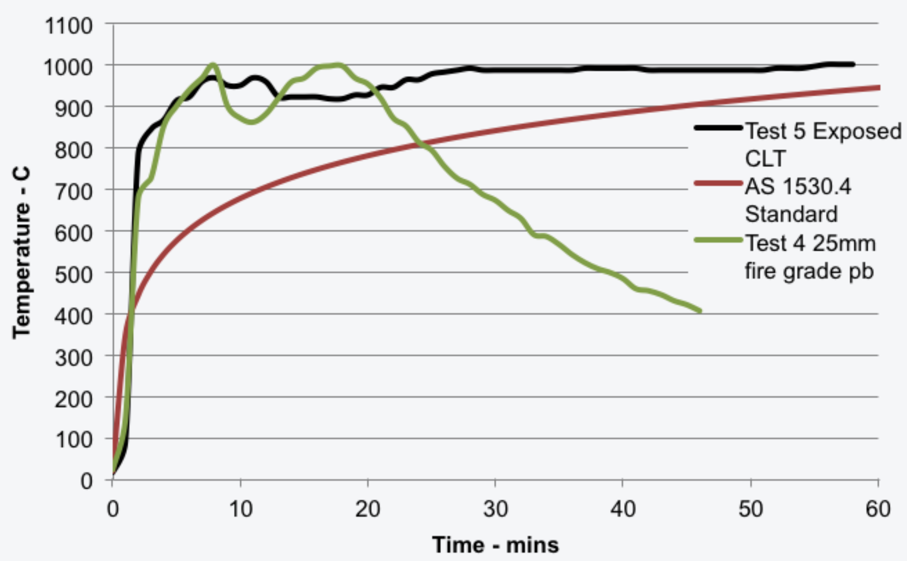


Figure A9: Average enclosure temperatures with and without plasterboard coverings extracted from McGregor[37].

For the protected enclosure, the fire burnt out and decayed; whereas, in Test 5 the CLT continued to burn, extending the duration of the fully developed fire beyond 62 minutes, at which stage the test was terminated.

A.6 External Fire Spread

A.6.1 External Fire Spread Scenarios

There are four common exposures that may initiate external fire spread, which are summarised in Figure A10.

Scenario 1: Fire spread from adjacent buildings to the subject building

Scenario 1 relates to the risk of fire spread from adjacent properties to the subject building and it is addressed under the NCC Deemed-to-Satisfy Provisions by the specification of:

- minimum separation distances for openings in walls
- non-combustible construction for mid-rise construction (Type A or B construction)
- fire-resisting construction – depending on the distance from the potential fire source.

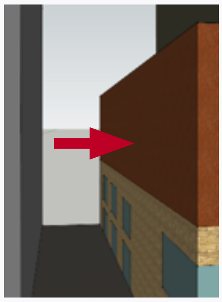
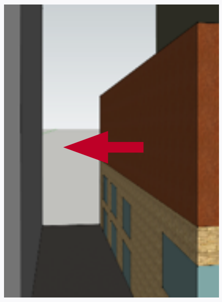
Scenario 1	Fire spread from adjacent buildings to the subject building	
Scenario 2	Fire spread from the subject building to adjacent buildings	
Scenario 3	Fire spread from an external fire source adjacent to the facade other than adjacent buildings, e.g. car fire, waste bin fire, furnishings on balconies etc.	
Scenario 4	Vertical fire spread originating from an internal fire typically from a fully developed fire within the building spreading to the façade / external walls	

Figure A10: External fire spread scenarios.

Scenario 2: Fire spread from the subject building to adjacent buildings

Scenario 2 relates to the risk of fire spread from the subject building and it is addressed under the NCC Deemed-to-Satisfy Provisions by the specification of:

- minimum separation distances for openings in walls
- non-combustible construction for external walls for mid-rise construction (Type A or B construction) to avoid combustion of the façade increasing the risk of fire spread to adjacent structures
- fire-resisting construction to avoid the building collapsing onto adjacent properties, limiting the maximum fire compartment size and preventing the opening up of additional openings
- automatic fire sprinklers in buildings above 25m high reducing the probability of a fully developed fire occurring that may threaten adjacent properties.

A verification method is also provided in the NCC to address fire spread between adjacent properties (Scenarios 1 and 2). The requirements are quantifiable, such that the building itself is required not to impose a heat flux greater than limits specified as shown in Table A4, for various distances from the boundary or an adjoining property or road. The subject building is also required to resist ignition when exposed to the heat flux stated in Table A4, if it is constructed within the nominated distances of another building or boundary.

Table A4: Maximum heat flux for various distances from the boundary and adjacent buildings.

Distance from Boundary	Distance between Buildings (m)	Maximum heat Flux kW/m^2
On the boundary	0	80
1 m from the boundary	2	40
3 m from the boundary	6	20
6 m from the boundary	12	10

Scenario 3: Fire spread from an external fire source adjacent to the façade

The risks associated with Scenario 3 are predominately addressed by the Deemed-to-Satisfy requirement in the NCC for non-combustible construction of external walls.

Scenario 4: Vertical fire spread originating from an internal fire

The risks associated with Scenario 4 are predominately addressed by the Deemed-to-Satisfy requirement in the NCC for non-combustible construction of external walls and the following three options for vertical separation of openings:

Spandrel panels: A section of external wall, curtain wall, or panel above an opening that is 900 mm or higher and extends at least 600 mm above the upper floor surface and is made from non-combustible material with a minimum FRL of 60/60/60, as shown in Figure A11.

Where curtain or panel walls are used, any gaps between the surface and the building's structure must be packed with a non-combustible material that will withstand thermal expansion and structural movement of the walling without the loss of seal against fire and smoke.

Horizontal Projection: Projects outwards from the external face of the wall not less than 1,100 mm; and extends 450 mm beyond the openings and is made from non-combustible material with a minimum FRL of 60/60/60, as shown in Figure A11.

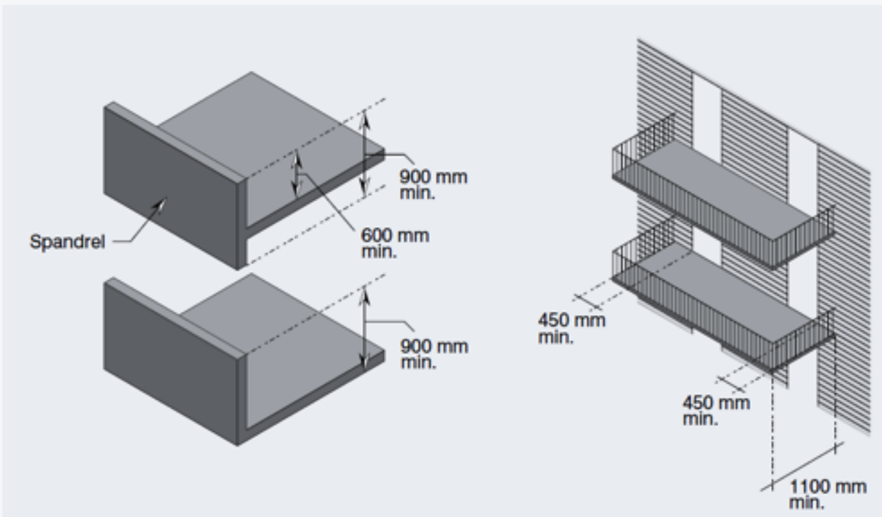


Figure A11: Vertical separation of openings.

Source: WoodSolutions Technical Design Guide #18: Alternative Solution Fire Compliance, Façades

Automatic Sprinkler Fire Protection: The requirements for fire separation are waived if an automatic fire sprinkler system is provided throughout the building.

The efficacy of spandrel panels and horizontal projections is very sensitive to the ventilation conditions of the fire venting from the opening below. This has been demonstrated in numerous studies.

The results summarised below were obtained from the compartment fire tests described in Section A.4: Enclosure Fires with Fire-protected Timber Building Elements. A cement sheet faced façade was constructed above the opening to a height of approximately 6m above ground level and included a wing wall (re-entrant detail). One test included a 600 mm horizontal projection above the opening and the other test had a vertical façade. Within the enclosures, the burning regimes were similar, and varied during the fire from strongly ventilation controlled to fuel controlled, providing a useful comparison between façades with and without horizontal projections over a range of conditions. The opening was 2 m wide x 1.2 m high located in the centre of the front wall with the sill at a height of 0.5 m above floor level.

The images in Figure A12 showing the different burning regimes at various times and the results presented in Table A5 have been extracted from Technical Design Guide #18. Further details can be obtained from England and Eyre[5,31].

2 mins Growth Phase

15 mins Fully developed ventilation controlled phase

35 mins Transition to fuel controlled phase

50 mins Decay

Figure A12: Burning regimes of enclosure tests.

Table A5: Incident heat flux and gas temperatures over façades with different burning regimes.

	Test Time (mins)	Burning Regime	Enclosure Temperature (Co)	Heat flux 1.5 m above opening (kW/m²)	Heat flux 3 m above opening (kW/m²)	Temperature 1.5 m above opening (°C)	Temperature 1.5 m above opening (°C)
Flat Facade	2	Growth (fuel controlled)	50	2	1	46	39
Horizontal Projection			67	1	1	53	24
Flat Facade	20	Strong vent controlled	813	104	43	1000	741
Horizontal Projection			831	67	15	639	461
Flat Facade	28	Vent controlled	1018	65	29	777	433
Horizontal Projection			1029	41	11	467	386
Flat Facade	35	Stoichiometris (approximately)	1090	30	18	636	417
Horizontal Projection			1088	13	5	312	313
Flat Facade	40	Decay phase (fuel controlled)	785	20	12	467	303
Horizontal Projection			763	17	5	420	262

The results show that even modest horizontal projections can significantly reduce the incident heat flux on the façade, but with ventilation-controlled fires the 900 mm vertical separation between openings would be unlikely to prevent fire spread with the ventilation conditions in this test, since incident heat fluxes over 100kW/m^2 were measured 1.5 m above the opening.

Figure A13 also demonstrates the potential impact of re-entrant details increasing flame adhesion and extending the flames further (estimated to peak at about 7 m above the opening).

Figure A13: Maximum flame extension estimated to be more than 7 m above opening.

The fire load of the test compartment was 41 kg/m^2 (kg wood per m^2 floor area), which equates to about 740 MJ/m^2 based on a heat of combustion of 18 MJ/kg compared to the 25 kg/m^2 suggested in ISO 13785-2:2002[38] for evaluation of façades. The selected fire load was intended to be representative of a relatively high fire load for residential occupancies, and therefore produced a strongly ventilation-controlled fire until some of the fuel load and fuel surface area had reduced.

Of the three Deemed-to-Satisfy options, the most effective is therefore to minimise the risk of a fully developed fire occurring in the first place through the provision of automatic fire sprinkler systems.

The additional requirement for non-combustible construction of external walls may limit or retard the spread of fire in some instances if the automatic fire sprinkler system were to fail or in the event of an external fire, but there would be a residual risk of fire spread between floors.

A.6.2 Fire-protected Timber Performance

Scenario 1: Fire spread from adjacent buildings to the subject building

Since the fire-protective coverings applied to fire-protected timber are required to be non-combustible, ignition due to the imposition of heat from an adjacent building will not occur if the fire-protective coverings provide sufficient insulation to prevent the temperature of the underlying timber reaching ignition temperatures (typically in excess of 300°C for common configurations). Under these conditions, the behaviour will be similar to a non-combustible external wall subject to any additional screening provided to protect the fire-protective coverings from weather being non-combustible.

Scenario 2: Fire spread from the subject building to adjacent buildings

If the fire-protective coverings prevent the timber elements being involved in the fire, the severity of a fully developed fire would be similar to that of an equivalent enclosure of non-combustible construction (see Appendix A.4: Enclosure Fires with Fire-protected Timber Building Elements). Therefore, the heat flux imposed on adjacent structures would also be similar.

A series of four large scale apartment encapsulation tests were undertaken by NRC as part of a recent investigation into mid-rise wood construction which has been summarised by Su and Lougheed[39].

Four tests were conducted using a three-storey simulated building with the fire ignited on the mid-level. A brief description of each form of construction tested; peak heat fluxes measured in front of the window openings; and total heat release rate from the fires are summarised in Table A6.

Table A6:Summary of peak heat flux measurements in front of window openings and peak heat release rates from Canadian encapsulation tests.

Test Ref	Description and Details of Internal Linings	Heat Flux kW/m² from openings				Max HRR – MW*
		Bedroom		Living Room		
		2.4m	4.8m	2.4m	4.8m	
LWF1	Timber-framed, protected with 2 layers of type X pb 12.7 mm thick. Ceiling fixed via steel furring channels at 405 mm centres	23	7	21	7	8**
CLT	Walls 105 mm CLT, Floor 175 mm, CLT all protected by 2 layers of type X pb 12.7 mm thick direct fixed	25	9	23	7	8.4
LSF	Walls – Steel studs protected with one layer type X pb 15.9 mm thick. Ceiling – Steel joists protected by 1 layer type X pb 12.7 mm thick fixed via steel furring channels at 610 mm	25	9	33	10	10.5
LWF2	As LWF1 but with only 1 layer of standard pb 12.7 mm thick applied to external wall	28	10	25	10	10.6

** HRR includes both combustion within the structure and in the plumes outside the test building.*
*** This was reported as an estimate.*

Unfortunately, direct comparisons between the test results are difficult because of the differing methods of application of the fire-protective coverings; different encapsulation levels; and inclusion of non-loadbearing internal timber walls with minimal encapsulation (12.7 mm standard plasterboard) for the experiments with timber frames and CLT.

However the following observations are relevant:

- The results from test LWF1 and CLT generally yielded similar results with respect to HRR and radiant heat from openings and the levels of encapsulation were similar.
- The non-combustible steel-framed test LSF yielded higher peak radiation levels and HRR. This was due to the failure of the non-loadbearing external wall which increased the opening size in the living room. This had the effect of increasing the rate of burning within the living room enclosure but also increasing the rate of release of volatiles from the enclosure. The larger opening also increased the size of the radiant heat source increasing radiation levels in front of the opening and reducing the duration of the fire. As a result, a direct comparison of the results from tests LWF1 and CLT cannot be made.
- The external openings in test LWF2 did not appreciably increase due to the early degradation of the standard plasterboard but the internal timber framework was exposed, effectively simulating a gross defect with an encapsulation system (i.e. substitution of Type X board with standard board). This increased the maximum HRR and radiant heat released from the living room and bedroom openings compared to tests LWF1 and CLT.

In summary, the results showed that encapsulation can appreciably reduce fire severity in timber buildings, and hence radiation levels in front of openings, but other factors such as the size of ventilation openings are also important.

Scenario 3: Fire spread from an external fire source adjacent to the façade

The NCC requirements for fire-protected timber in mid-rise buildings for external walls require as a minimum the same level of protection against ignition/incipient spread of fire as the inner face. For buildings within 1 m of a fire source feature (e.g. allotment boundary), or 2 m of an adjacent building, the massive timber provision for fire-protective coverings is not applied and the higher levels of protection required for general timber structures are adopted. The severity and duration of an external fire source is unlikely to exceed that of a fully developed fire and therefore the performance of fire-protected timber would be expected to be similar to that of non-combustible construction. (Note: any additional façade materials such as weather barriers must also be non-combustible).

Scenario 4: Vertical fire spread originating from an internal fire

The test results and discussion in Appendix A.6.1: External Fire Spread Scenarios/Scenario 4 are directly applicable to fire-protected timber, since the peak exposure of the external wall is likely to be the result of combustion of the contents only and the external levels of fire protection are as a minimum similar to the internal protection levels. In addition, mid-rise fire-protected timber buildings are required to have automatic fire sprinkler protection, greatly reducing the probability of a fully developed fire occurring.

A.6.3 Exposed Timber Performance

Scenario 1: Fire spread from adjacent buildings to the subject building.

If the external wall includes unprotected timber elements, ignition and fire spread may potentially occur if the incident radiant heat significantly exceeds the critical heat flux for ignition (typically assumed to be approximately 12kW/m^2). A separation distance between buildings of 12 m (6 m from the boundary) is required from Table A4 to maintain radiation levels below 10kW/m^2.

The selection of timbers with relatively high resistance to ignition may enable modest increases in the radiant heat that can be resisted by exposed timber façades.

For example, Figure A14 shows the time to ignition calculated using the Janssens' Procedure[40,41] applied to cone calorimeter data for Grey Ironbark[42]. The data points used to derive the constants were between radiant heat fluxes of 25 and 60 kW/m^2, but the critical heat flux derived of approximately 12.3 kW/m^2 gives some confidence in the predicted times to ignition below 25kW/m^2.

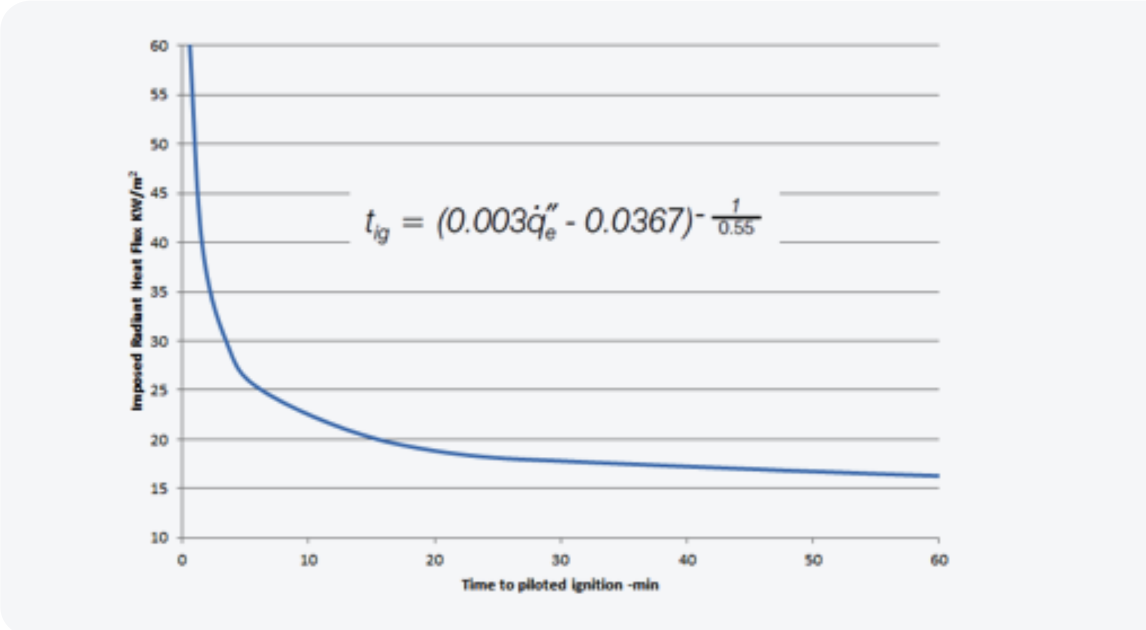


Figure A14: Time to piloted ignition of Grey Ironbark exposed to radiant heat from Richardson and England[42].

For an incident radiant heat flux of 17.5 kW/m^2, the time to piloted ignition can be calculated to be greater than 30 minutes, but this reduces to about 15 minutes for an incident radiant heat of 20kW/m^2, illustrating that the results are very sensitive as heat fluxes approach 20kW/m^2.

In some instances it may therefore be useful to interpolate the limiting heat fluxes for distances between 3 m and 6 m from the boundary. For example, subject to agreement with the authority having jurisdiction, the following equation could be used to interpolate between the specified boundary distances in Table A6 above:

$$Q = 39.6 - 16.88 \log_e(d)$$

Where Q is the heat flux (kW/m^2) and d is the distance from the boundary (m). The correlation is shown in Figure A15.

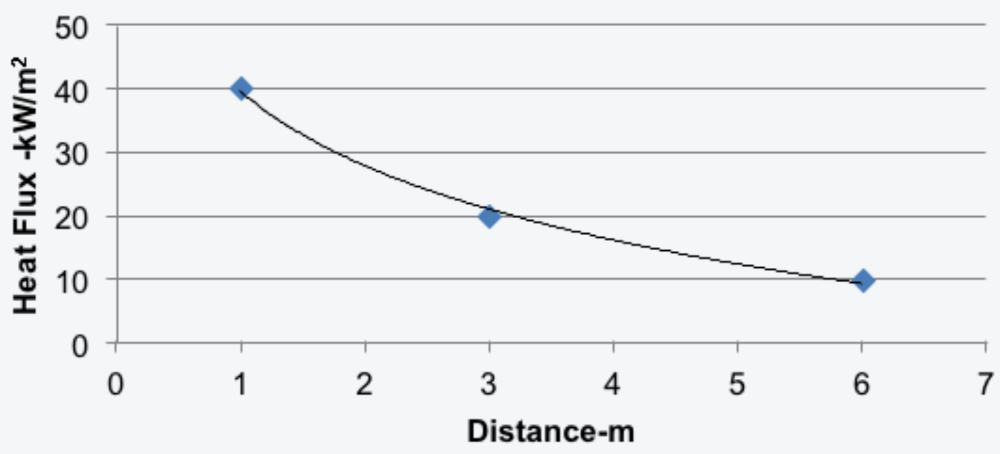


Figure A15: Interpolation of separation distances prescribed in CV1.

For example, an incident radiant heat flux of 17.5 kW/m2 corresponds to a separation distance of 3.7 m. Without interpolation, a separation distance of 6 m would be required.

The performance of timbers can be improved by the use of fire retardants. To be effective, the fire retardants would need to have sufficient durability for external use and be able to retard ignition over lengthy exposure periods to high radiant heat fluxes.

Scenario 2: Fire spread from the subject building to adjacent buildings

This scenario is closely aligned to Scenarios 3 and 4, in that, if a fire ignites the building façade and propagates across the external surfaces of the building, a potential consequence is an increased risk of fire spread to adjacent structures, compared to non-combustible construction and fire-protected timber construction. The probability of fire spread can be reduced by:

- preventing ignition
- limiting fire spread or the consequences if ignition occurs – typical options include one or more of the following measures:
  - controlling material properties (e.g. timber selection or use of fire retardants)
  - controlling timber element configurations
  - limiting the size of 'packages' of exposed timber and separating packages such that fire spread is limited to a single package (e.g. use exposed timber for features on the façade and/or break up the façade with horizontal projections)
  - increased separation distances for combustible areas from adjacent buildings.

Scenario 3: Fire spread from an external fire source adjacent to the façade

This scenario includes fire starts on balconies, and there have been a number of significant fires involving composite panels resulting in rapid fire spread (e.g. the Lacrosse Docklands[43]). Timber does not exhibit some of the burning characteristics that contributed to the severity of these incidents but, if combustible materials are used on the façades of mid-rise buildings, the response to external fire scenarios need to be considered when developing a performance solution. (NCC Deemed-to-Satisfy Provisions generally require non-combustible construction or fire-protected timber construction for external walls.) In some instances, Scenario 3 may be considered less severe than Scenario 4 and therefore Scenario 4 is commonly used for design purposes, but each case should be considered on its merits.

Scenario 4: Vertical fire spread originating from internal fires

As discussed above, the size of plumes projecting from openings during a fully developed fire and associated flame extension is dependent on the ventilation conditions, the opening configurations and façade configurations. For fuel controlled fires, there may be minimal flame extension but for strongly ventilation-controlled fires flame extensions can be substantial, as demonstrated in Figure A13.

If the façade of the building includes exposed combustible components, flame extensions can be further increased as volatiles are released and the risk of other modes of fire spread such as cavity fires may be increased.

The primary method to address this mode of spread is the use of automatic fire sprinklers to minimise the risk of a fully developed fire occurring but, if combustible façade systems are intended to be used, their performance should also be evaluated (ignoring the impact of sprinklers) to ensure a robust building solution is provided.

AS 5113:2016 Fire propagation testing and classification of external walls of buildings identifies appropriate test methods and performance criteria.

Sources of additional information are:

- Fire Hazards of Exterior Wall Assemblies Containing Combustible Components[64]
- Fire Safety Engineering Design of Combustible Façades[51]
- WoodSolutions Technical Design Guide #18: *Alternative Solution Fire Compliance, Façades*.

A.7 Cavity Fires

Cavities within fire-resisting construction can provide paths for the spread of fire that can bypass the boundaries of a fire-resisting compartment, potentially compromising a fire safety strategy. Spread through cavities can be accelerated by the presence of combustible materials and linings within the cavities.

Fire spread through cavities/concealed spaces is relatively infrequent but, when it does occur, fire can spread rapidly and the seat of the fire may be difficult to locate, presenting challenges to fire fighters. While not unique to timber buildings, there have been major incidents involving fire spread through cavities in timber buildings.

Common causes of fire starts within cavities are electrical faults or hot works during maintenance activities but enclosure fires can also spread to cavities and via cavities to other enclosures.

For mid-rise timber buildings designed to the NCC 2016 Deemed-to-Satisfy solutions, a multi-tiered approach has been adopted to provide a robust fire safety strategy that is not solely reliant on any one element, incorporating the following measures to prevent the fire spreading to the cavity in the first place:

- Fire sprinkler system installed in accordance with Specification E1.5 of the NCC reducing the number of severe fires
- Specification of fire-protected timber, which requires fire-protective linings to provide resistance to the incipient spread of fire, in addition to contributing to the fire resistance of the element
- Application of the resistance to the incipient spread of fire criteria to service penetrations.
- Specification of cavity barriers to be fitted around windows and doors to maintain the integrity of the fire-protective linings.

If a fire does spread to a cavity or ignition occurs within the cavity the following measures have been specified in the NCC 2016 to minimise the consequences:

- Any insulation in wall and floor/ceiling cavities must be non-combustible to ensure that if insulation is provided within the cavities, it will tend to limit growth and fire spread and not accelerate it
- Cavity barriers at junctions with other fire-resisting elements of construction and at prescribed maximum centres are specified to restrict spread
- Larger floor cavities are required to have fire sprinklers fitted within the cavity in accordance with the requirements of NCC Specification E1.5, which will limit growth and fire spread within the protected areas
- The structural design should be robust such that progressive collapse is unlikely if a structural member fails to support the applied load.

The NCC Provisions recognise that massive timber panel construction is less susceptible to cavity fires, subject to appropriate detailing, and permits a modified resistance to the incipient spread of fire criteria to be adopted in some circumstances.

B

Appendix B - Matters Raised by Stakeholders and Other Parties

Issue	Discussion	Outcome
1: Will the increased use of timber increase the fire growth rate?	The proposed variation relates predominately to walls/ shafts since the current Deemed-to-Satisfy (DTS) Provisions do not prevent the use of timber floors beams and columns for most applications. Where timber is to be used as part of an assembly required to be non–combustible or of concrete/masonry construction, it will be fire-protected. Therefore there will be no increase in the fire growth rate, which in most circumstances will be dominated by furnishings. The DTS lining controls also apply.	No increase in fire growth rate expected. The addition of automatic fire sprinklers will limit the size of larger fires in most scenarios for timber buildings, therefore the growth rate of larger fires will be reduced after activation of automatic fire sprinklers, providing a net reduction in effective growth rate.
2: Will the time to flashover be reduced by the use of timber?	The timber will not accelerate the time to flashover when used for an element required to be of non-combustible or masonry or concrete construction, because it will be protected with fire-protective coverings that are required to be non-combustible and protect the timber substrate during a flashover fire.	No reduction in the time to flashover and number of flashover fires will be reduced by more than 90%, because of the prescription of automatic fire sprinklers.
3: Will adhesives used in manufactured massive timber products reduce the time to untenable conditions?	General furnishings are expected to dominate the production of toxic gases during a fire because of the high volumes of synthetic upholstered materials, with contributions from the adhesives used in engineering products expected to be minimal. In addition, the proposed changes relate to the use of timber products protected by fire preventative coverings, which will be expected to prevent breakdown of the timber and adhesives until after untenable conditions have occurred in the enclosure, in the low probability event that the automatic fire sprinkler system fails.	The variations being considered in this study require protected timber, and automatic fire sprinklers are provided, which are expected to increase the time to untenable conditions providing a safer building. The Deemed-to-Satisfy smoke production limits in C1.10 of the NCC apply.
4: Are untenable conditions reached earlier or differently to normal residential fires?	Due to the use of preventative coverings, no change in the time to untenable conditions would be expected when compared to non-combustible elements or masonry or concrete construction. The introduction of automatic fire sprinklers will induce greater mixing/cooling of smoke after activation, but a significant net improvement in safety will be provided where automatic fire sprinklers are provided.	Untenable conditions more likely to be reached at a later stage in a fire, but sprinklers will modify how untenable conditions are reached close to the fire and a substantial net improvement in safety is expected.
5: Does plasterboard fall-off occur during a fire? If so, at what stage and what is the impact?	Whether plasterboard falls off and, if so, at what stage of a fire depends on many factors, including fire severity, type of plasterboard, thickness, fixing system, active fire protection measures, etc. For the elements of construction being considered in this study, a performance specification has been developed which requires a preventative covering that prevents the temperature of the interface with timber members exceeding either 250°C (timber-frame) or 300°C (massive timber) when exposed to the standard heating regime, to provide an opportunity for fire-fighting activities to begin – should the automatic fire sprinklers fail.	Plasterboard or other non-combustible fire-protective coverings would not be expected to fall off prior to sprinkler activation or when fire brigade intervention occurs within the most likely time frames. In the rare circumstances of sprinkler failure coincident with a slow fire brigade intervention time, the ability of the preventative coverings to remain in place will depend on a number of factors, including fire severity. The Monte Carlo analysis took all these factors into account and compared a building designed in accordance with the proposed changes with one that satisfies the 2015 NCC Deemed-to-Satisfy Provisions. If coverings fall off, the fire duration is increased in the modelling to evaluate the consequences.

Issue	Discussion	Outcome
6a: What is the impact when the facings fall away from timber-framed construction?	The impact will depend on what stage the fire has reached when the facings fall away. The Proposal-for-Change (PFC) to the NCC requires the facings to remain in place for a substantial period should a flashover fire occur to prevent ignition of the timber for sufficient time to facilitate fire-fighting activities. The boards are likely to remain in place for a substantial period after this stage and in many instances ar elikely to be in place after burnout of the contents. If the facings fall off, the timber frame will be exposed to the enclosure fire. This is most likely to occur while the fire is fully developed or in the decay stage. When the timber is exposed it will release volatiles and form a char layer reducing the rate of production of volatiles. If the timber is exposed whie the fire is ventilation controlled the enclosure temperatures are unlikely to increase due to a lack of oxygen. If the fire is fuel controlled, there may be an increase in enclosure temperature but the impact will be small because of the relatively small exposed surface area of timber.	The Proposal-for-Change (PFC) includes an automatic fire sprinkler system as the primary fire protection system minimising the risk of timber members being exposed in addition to the prescription of fire-protected timber. Notwithstanding this the analysis considered scenarios where the timber may be exposed and showed that the timber buildings provided a substantially greater level of safety than 2015 Deemed-to-Satisfy measures.
6b: What is the impact when the facings fall away from massive timber construction?	Refer to item 6a. Similar behaviour will be expected from massive timber except in some cases with laminated products a lamination may fall away, exposing fresh timber and possibly initiating short term growth, and the exposed timber surface area will be greater. As noted above, the inclusion of an automatic fire sprinkler system and use of fire-protected timber will offset any negative effects.	Refer to item 6a.
7: Could exposed timber systems increase fire severity, causing other fire-resistant elements to be over-run?	In theory, yes, but the likelihood of this happening with the specification of fire-protected timber and sprinklers is very low. The risk associated with these scenarios has, however, been evaluated in the analysis, with the fire duration/severity being increased to account for the contribution from the exposed timber for scenarios where coverings fail (exposing timber) and the impact on other fire-resistant elements has been modelled.	These scenarios were considered in the detailed analysis but, due to the range of mitigation measures including the provision of automatic fire sprinklers, a net improvement in fire safety was demonstrated.
8: Can timber reignite or ignited timber in cavities continue to burn and, if so, are special measures required?	Timber can reignite and continue to burn after the main fire is supressed in some instances and, although the probability is low, ignition of timber in cavities can also occur. Lightweight timber-framed construction is more susceptible because of the smaller cross section of timbers and potential for cavities behind coverings. The following special measures are provided to address this risk: • provision of fire sprinkler systems • specification of fire-protected timber to reduce the risk of ignition prior to fire brigade intervention (higher levels of protection are specified for timber-frame construction) • requirements for cavity barriers • any cavity insulation to be non-combustible. Notwithstanding the above precautions, as is the case with all fires, it will be necessary to monitor a fire to avoid re-ignition and incipient spread in the same way as any structural fire. Australian fire fighters are experienced in dealing with fires in timber-framed construction and may need to adapt some of these procedures for mid-rise timber buildings.	The PFC requires the following additional measures to mitigate the risk of re-ignition and fire spread through cavities: • provision of fire sprinkler systems • specification of fire-protected timber to reduce the risk of ignition prior to fire brigade intervention • requirements for cavity barriers • any cavity insulation to be non-combustible. The analysis showed an improvement in fire safety for a mid-rise timber buildings designed in accordance with the PFC compared to the control building complying with current NCC 2015 Provisions .

Issue	Discussion	Outcome
9: How is the strength and stability of the building addressed during and after a fire event?	This is a disproportionate collapse issue that should be addressed as part of the Structural Design for each building, as it should be for all other types of structures under NCC Provisions, e.g. BP1.1. A workshop was held with structural engineering experts to identify current approaches and determine if there was a need for specific advice relating to timber. The outcome of the meeting was that guidance was needed in relation to all forms of construction – not just timber. A Guide is being developed by WoodSolutions that, among other things, will require at least one level of redundancy to be provided to address the risk of disproportionate collapse and typical details for timber construction will be provided. The Monte Carlo analysis considered levels of redundancy and assumed major collapse if two or more groups of members fail. Fire spread through cavities in timber-framed construction has been identified as a special case and the Guide will require the structural design to additionally withstand the loss of two sections of walls directly above each other, providing increased redundancy to address reliability of cavity barriers.	This will be addressed through structural design process to avoid disproportionate collapse as required by current Provisions in the NCC for all buildings. To facilitate this, a Guide is being developed by WoodSolutions which will provide guidance to structural engineers. It will include specific advice for mid-rise timber buildings.
10: How will connections be protected under fire conditions?	Connections within engineered timber members, such as web connections, glued joints, etc, are required to be evaluated during the fire resistance test. Fire resistance of connections between elements will be maintained using well-established practices for timber elements of construction such as: • redundancy in connectors: example connections on both side of a fire-separating element • applying fire protection over the joint • use of sacrificial timber to protect metal connectors • fire-resisting plasterboard • concrete floor topping. To minimise the risk of poor design detailing, the existing range of WoodSolutions guides will be reviewed and expanded as necessary. The WoodSolutions guidance documents will also provide advice on suitable detailing to prevent disproportionate collapse.	Normal practices will be adopted based on standard details and/or results from fire resistance tests. Clear guidance will be provided in WoodSolutions Guidelines.
11: Is there a risk of outward collapse of massive timber external wall panels?	This question was asked at the structural engineers' workshop and there was a clear consensus that there is no increased risk of outward collapse. This is due to the low thermal expansion of timber, initial shrinkage as water is driven off and insulating properties minimising distortion, and thus induced eccentricity under fire conditions compared to other common building materials such as concrete and masonry. There is still a need for proper detailing of connections and, as noted in item 10, the existing range of WoodSolutions guides will be reviewed and expanded as necessary.	No increased risk of outward collapse. Normal practices for connections will be adopted based on standard details and/or results from fire resistance tests. Clear guidance will be provided in WoodSolutions Guidelines.

Issue	Discussion	Outcome
12: Is there a risk of Fires in Voids spreading rapidly?	Without appropriate mitigation measures, spread through cavities can occur, which in extreme circumstances could be rapid. This is most likely to occur with lightweight timber-framed construction, rather than massive timber construction where careful detailing can avoid cavities. The PFC requires a broad range of measures to address this risk in a robust manner including: • Automatic fire sprinklers to reduce the risk of severe fires spreading to cavities. They are also required to be fitted within larger cavities in accordance with AS 2118.1 providing additional protection. • The specification of fire-protected timber to minimise the risk of ignition of protected timber members (higher levels of protection are specified for timber-framed construction). • Insulation within cavities is to be non-combustible. • Cavity barriers are specified to prevent or retard spread across compartment boundaries via cavities and spread to and from cavities around openings such as windows. The detailed analysis took account of the results of studies into this form of spread and experimental data to assess the risks to compare the risk to occupants in the proposed timber building compared to the control building.	This risk will be managed through the specification of the following measures: • Automatic fire sprinklers to reduce the risk of severe fires spreading to cavities. They are also required to be fitted within larger cavities in accordance with AS 2118.1 providing additional protection. • The specification of fire-protected timber to minimise the risk of ignition of protected timber members (higher levels of protection are specified for timber-framed construction). • Insulation within cavities is to be non-combustible. • Cavity barriers are specified to prevent or retard spread across compartment boundaries via cavities and spread to and from cavities around openings such as windows.
13: Does timber construction expose fire fighters to increased risk?	A key outcome of the analysis is a significant reduction in the risk to fire fighters in medium rise timber buildings, largely resulting from the specification of automatic fire sprinklers and fire-protected timber. The fire sprinklers will reduce the number of high-risk fully developed fires that the fire brigade have to respond to by a factor of approximately 10, and the fire-protected timber in most instances will prevent ignition of the timber substrate before fire brigade intervention occurs, providing similar outcomes for scenarios where the sprinkler system fails to those of the control building.	A significant reduction is expected in the risk fire fighters are exposed to.
14: Is there a need to change fire-fighting procedures?	There is extensive experience fighting fires in low-rise timber-framed construction in Australia and the same general principles can be applied to mid-rise construction. During stakeholder engagement, extensive discussions have been undertaken with the fire authorities to brief them on the proposed forms of construction so that they can adapt fire-fighting procedures as appropriate. Some suggestions have been identified include identifying and carrying appropriate tools to gain access to cavities and the use of thermal cameras to determine concealed hot spots.	Some minor adjustments to fire-fighting procedures may enhance the effectiveness of fire brigade intervention in mid-rise timber buildings and the fire authorities have been involved throughout the development of the PFC. The specification of Deemed-to-Satisfy solutions for mid-rise timber buildings will help standardise construction details and reduce the risk of fire fighters being exposed to ad hoc arrangements, improving safety and effectiveness of fire fighters.
15:What evidence of FRLs will be required?	Suppliers will need to provide evidence of FRLs of their systems in the usual manner prescribed in the NCC, e.g. reports from registered testing authorities. This applies to items such as: • fire-protected timber (including additional information relating to incipient spread of fire and protection of massive timber substrates) • cavity barriers • service penetrations (including incipient spread of fire requirements in 2015 edition of AS 1530.4) • fire doors • lift landing doors.	Suppliers of proprietary engineered timber systems, such as lightweight timber trusses, CLT and coverings to be used for fire-protected timber systems, along with cavity barriers will be required to undertake fire resistance tests demonstrating the performance levels are met by the specific materials and methods of construction adopted for the systems.

Issue	Discussion	Outcome
16: How are the limitations of standard fire resistance test method/FRLs taken into account?	As part of this analysis, a review of lightweight timber and massive timber elements, enclosures and buildings subjected to standard fire resistance tests and natural fire experiments was undertaken. The results were compared using the correlation methods verified against experimental data to adjust the performance for differing heating rates, providing confidence in the application of fire resistance test data. The required use of fire-protected timber requires the use of fire protection systems commonly used to protect structural steel, providing consistency with current Deemed-to-Satisfy Provisions. In addition, methods were derived to model the additional contribution from timber elements if they are exposed directly to fire, for example, by premature failure of a covering.	Correlation methods have been developed to modify fire resistance test results to address issues such as variations in heating rates and adjust model inputs to account for increased fire severity due to exposure of timber members if coverings fail. These methods have been verified against natural fire data.
17: What will be the requirements for fire stairs?	Fire stairs will be required to be constructed of fire-protected timber (i.e. lined with non-combustible fire-protective coverings). The use of timber stairways will be permitted, subject to the extension of automatic fire sprinkler coverage to the fire-isolated stair and the protection of the underside of the stair on the ground level to address the risk of material storage (even though such activities are not permitted).	Fire-protected timber will be used for stair shafts (i.e. non-combustible fire-resistant linings). If timber stairways are to be fitted. the automatic fire sprinkler shall be extended to cover the stair and the underside of the stairs protected on the ground level.
18: How will evacuation of people with disabilities be managed?	The timber mid-rise building as described in the PFC will provide increased levels of safety because of the provision of an automatic fire sprinkler system, which will maintain tenable conditions throughout the fire exits and path of travel to fire exits for considerably longer than a building complying with the current NCC Deemed-to-Satisfy building, in most circumstances. This provides increased opportunity for the safe evacuation of people with disabilities. The modelling undertaken assumes some occupants will require assistance to evacuate, which inherently considers people with disabilities who may require assistance.	While specific approaches may vary between buildings, the requirement for automatic fire sprinklers will increase the time that tenable conditions are maintained in evacuation paths, greatly improving safety for people with disabilities.
19: What façade fire spread precautions will be required?	The external façade surface will be non-combustible. Fire-protected timber will be used, minimising the risk of the timber substrate being ignited and providing a similar level of protection to the current Deemed-to-Satisfy Provisions. In addition, automatic fire sprinklers are specified as are cavity barriers with minimum FRLs of –/45/45, further reducing the risk of fire spread via the façade.	The following combination of systems is specified which reduces the risk of fire spread via the building façade compared to the NCC Deemed-to-Satisfy requirements for the control building: • automatic fire sprinklers • fire-protected timber • cavity barriers.
20: Are any special maintenance requirements necessary for timber construction?	Normal maintenance Provisions should be adequate. These include • requirements to reinstate the fire resistance performance of fire-resistant barriers when new services are fitted • notifications when active fire safety measures are non-operational due to maintenance/service activities and reinstatement at the end of each working day • hot work permit systems, etc.	No changes are required above normal good practice standards. The analysis considered the performance of elements with defects, checking that the design is robust and outcomes are not disproportionate to the failure of elements

Issue	Discussion	Outcome
21: How is the potential increase in the consequences of fires during construction going to be addressed?	The potential increase in consequences from fires during construction is to be addressed through a holistic approach to safety, as outlined in WoodSolutions Technical Design Guide #20.	Publication of WoodSolutions Technical Design Guide #20: *Fire Precautions During Construction of Large Buildings.*
22: What will be the impact of fire-fighting water on a CLT building?	CLT buildings overseas have been subjected to wet weather conditions for significant periods during construction without significant detrimental effects. CLT structures are airtight structures. Glue lines act as barriers to moisture movement. Therefore, CLT construction will perform better than traditional construction methods in some instances.	Impact of fire-fighting water other than staining is expected to be minimal.
23: How is fire spread between buildings addressed?	The external façade surface will be non-combustible and fire-protected timber will be used, minimising the risk of the timber substrate being ignited by radiant heat from an adjacent building. The fire-protected timber also reduces the risk of the external façade being ignited by a fire within the building creating a larger heat source radiating to adjacent structures. In addition, the provision of an automatic fire sprinkler system will substantially reduce the risk of fire spread to adjacent buildings	The following combination of systems is specified which reduces the risk of fire spread via the building façade compared to the NCC Deemed-to-Satisfy requirements for the control building: • automatic fire sprinklers • fire-protected timber.
24: What is the performance of connections used in timber element construction at elevated temperatures and under load? **Literature would suggest that the complex degradation of timber could result in the loss of embedding strength of timber (critical for mechanical fastenings) at relatively low temperatures (approx. 80–120°C); likewise some adhesives can crystallize, losing significant bond strength, at similar temperatures. This could result in failure even with sprinkler activation.**	The requirements for fire-protected timber in the PFC include: • Coverings that prevent interface temperatures reaching 300°C for a minimum of 20 minutes in low fire load areas and 30 minutes in most areas of the building for massive timber construction with the fire-protected timber exposed to the AS 1530.4 standard heating regime. • Coverings that prevent interface temperatures reaching 250°C for a minimum of 45 minutes for timber-framed construction with the fire-protected timber exposed to the AS 1530.4 standard heating regime. • Requirements for fire-protected timber elements to achieve the FRLs specified in the NCC (for Class 2 and 3 typically 90 minutes structural adequacy for loadbearing elements and 60 minutes for non-loadbearing elements). The specification of the standard fire resistance test means that all systems including engineered timber products must be subjected to fire resistance tests and loadbearing members are required to be tested under load. Systems that are vulnerable to premature failure during the early stages of a fire would be screened by these tests. The supplementary specification of minimum levels of fire-protective coverings provides further confidence and elements with high levels of inherent fire resistance are also required to have these coverings applied. The coverings are to be non-combustible and have to achieve the required performance when exposed to a standard fire resistance test. The elements surface would not be expected to be exposed to elevated temperatures prior to activation of an automatic sprinkler, except for fires initiating within timber members and failure of the automatic fire sprinkler system. Both these scenarios are considered in the detailed analysis.	All fire-protected systems are required to have their performance determined by being subjected to the standard fire resistance test which will prevent poor performing systems being approved under the proposed DTS Provisions. Both coverings and automatic fire sprinklers have been specified to provide a robust solution and the analysis has considered failure of fire protection systems to check the proposed changes provide robust building solutions.

Issue	Discussion	Outcome
25: Is AS1530.4 a relevant test for determining fire resistance for timber construction elements? **Does the methodology or assessment criteria need review or amendment?** **The AS1530.4 test requires an external heat flux input to maintain a temperature within a furnace. However, traditional non-combustible elements would receive a controlled heat flux exposure to maintain the temperature profile, whereas combustible elements, which once ignited contribute to the furnace temperature, may require a reduced (and potentially unrealistic) external heat flux to maintain an equivalent temperature profile. The comparability of combustible and non-combustible element test results (FRLs) should be explicitly investigated. Furthermore, what is the failure point and are they comparable? Failure conditions of a block wall assembly will be significantly different to the failure conditions of timber elements.**	AS 1530.4 is the standard method specified in the NCC for determining the fire resistance of elements of construction including timber elements, and is similar to most international standards that serve the same purpose; therefore it has direct relevance. However, this does not mean that the test has no limitations and that any limitations should not be considered in the analysis, development or review of DTS Provisions in the NCC. In the context of the analysis being undertaken, the results of fire resistance tests are applied to enclosure fires which can be characterised as a time temperature relationship that will vary with the size of the enclosure, fire load, ventilation conditions and thermal properties of the bounding construction. With the likely types of fire load, they will be initially ventilation controlled. Peak temperatures will occur close to stoichiometric conditions. The time temperature regime specified in AS 1530.4 will represent some of the fire scenarios, but not all, and with modern plastics and building configurations much more rapid growth rates and high early temperatures can occur. When undertaking fire resistance test, the gas supply is adjusted to achieve the required heating regime and does not just vary if combustible materials are tested; for example, the thermal performance of the element is also critical with materials such as aerated concrete requiring substantially less fuel than normal weight concrete. Therefore, one of the main weaknesses is also a strength of AS 1530.4, in that it applies a standard heating regime by which to determine the performance of a broad range of systems – many of which have different sensitivities to heating rates. To address the above and other limitations the following approach was adopted and applied to all elements considered in the analysis: • Multiple fire scenarios were generated based on varying values for fire load, room size and ventilation conditions. • A procedure was developed to convert standard fire resistance times to scenario times based on a critical element. • This approach was verified against full-scale experiments including natural fire tests including specimens under load to ensure the results obtained would be reasonable. The same procedure was adopted for combustible and non-combustible elements used in the analysis. Mechanisms of failure do vary. For example, some types of masonry wall and steel-framed construction are much more sensitive to P-Δ effects resulting from differential heating than timber-framed construction. The spalling of concrete was ignored in the analysis, improving the potential performance of the control building used in the comparative analysis and hence yielding a conservative comparison.	The limitations of the test method were considered in the detailed analysis and taken account of in converting fire resistance periods to fire scenario times. Results were compared to a large number of natural fire experiments and alternative heating regimes, such as the hydrocarbon heating regime, to provide confidence in the results.

Issue	Discussion	Outcome
26: What are the consequences of involvement of the timber elements in a fire? **The degradation of timber due to elevated temperatures and combustion is an extremely complex process that is understood from a generic basis only (i.e. charring rate analysis, 1-D heat conduction analysis). The behaviour of the timber in a fire, and the subsequent impact on the total structure, is unknown in this size of building and requires further research and/or a very conservative design philosophy. The PFC seems to suggest that the consequences of a spreading fire in a timber building would result in similar conditions for fire fighters to that encountered in a building of non-combustible construction; however this inference is not validated or contextualised against time of localised or total structural failure.**	The detailed analysis has drawn on a number of international research studies in addition to studies undertaken in Australia, including the TF 2000 project undertaken in the UK. The TF 2000 project included a natural fire test on a full-scale 6 storey mid-rise timber-frame building. The outcomes of the project were used to confirm that mid-rise timber buildings can meet the functional requirements of the Building Regulations in the UK. Compared to the UK requirements, the PFC proposal for Australia requires higher fire resistance ratings for loadbearing elements and the provision of automatic fire sprinklers. Therefore, the detailed analysis has drawn on directly relevant research and has taken a very conservative design philosophy. The detailed analysis considers the risk of localised and global collapse.	The matters raised have been addressed as part of the detailed analysis as described in the discussion. The preliminary analysis did not discuss these matters in depth.
27: Are sprinkler systems in these buildings a mandatory component or are they subject to alternative solution? **The PFC suggests that sprinklers are essentially a layer of redundancy; however, the involvement of the timber elements in a fire is directly linked to flashover conditions being achieved. As sprinkler systems can control/suppress a fire to prevent the onset of flashover, sprinkler systems are now a critical aspect for maintaining structural adequacy. As detailed above, significant unknowns exist if the timber elements become involved in the fire, therefore sprinklers are essential to mitigate the risk of consequences associated with these unknowns.**	It is agreed that sprinkler systems are important and they are mandatory in the DTS solution put forward in the PFC. While the use of the term ‘redundancy’ is considered appropriate in a risk context, there have been some modifications in the wording used in the detailed analysis to indicate that sprinkler systems are a critical component.	The provision of automatic fire sprinklers is a critical element of the proposed DTS solution.
28: Is the current sprinkler standard, including reliability and maintenance, suitable given the potential structural consequences of sprinkler failure/ineffectiveness? **The PFC suggests that both lightweight timber-framed construction and massive timber building systems can be “protected” through similar passive fire protection covering. As these are two completely different systems with starkly different issues associated with elevated temperatures and combustion, this further highlights the criticality of sprinklers to ensure that the timber, regardless of construction method, does not become involved in the fire. The design, installation, and maintenance of the sprinkler system should reflect this to provide a very high reliability. However, it should be noted that simply the presence of sprinklers does not address the issues associated with the performance of timber building systems at relatively low elevated temperatures and fire spread in cavities.**	The analysis adopted a value for the reliability for automatic fire sprinklers of 92%, based on a literature review and stochastic analysis that considered the outcomes of scenarios where the automatic fire sprinklers may fail. Due to the conservative approach adopted, the net result of the probabilistic approach considering the reliability of sprinklers was a substantial improvement in life safety. Comments on the low temperature performance of elements is provided under item 25. The detailed analysis also addresses cavity fires in detail.	The analysis indicates that there will be a very large improvement in life safety using “current standard fire sprinkler systems” without enhancements, due to the large range of additional measures being adopted.

Issue	Discussion	Outcome
29: Do the NCC Deemed-to-Satisfy Provisions require the fire resistance of floor/ ceiling systems to be evaluated from above? **If so, how is this addressed for mid-rise timber buildings?**	The NCC DTS Provisions require the FRL of elements of construction to be evaluated in accordance with the standard fire resistance test method AS 1530.4. In line with most international fire resistance test methods, AS 1530.4 does not require floor/ceiling systems to be evaluated when exposed to heating from above. Therefore, it can only be concluded that the NCC Deemed-to-Satisfy Provisions do not require floor/ceiling systems to achieve an FRL when exposed to fire conditions from above. There is one potential exception to this – Clause D2.11 – which under some circumstances requires the FRL of enclosing construction of fire-isolated passageways to be evaluated when exposed to fire from outside the passageway. Some regulators extend this requirement to enclosure of the top of fire-isolated stair shafts under Spec C1.1 Clause 2.7, under some circumstances. Cases of internal downward fire spread are relatively rare in buildings, which is assumed to be the basis for the NCC DTS Provisions to not require downward fire spread to be evaluated. If it were to be required to generally evaluate the fire resistance performance of floor and ceiling systems from above, the AS 1530.4 method will require modification and many existing construction systems will require modification to ensure, for example, critical fixings are not exposed and service penetrations systems also achieve the required FRL when exposed to fire from above and do not allow burning droplets to fall to the floor below. These issues are not as critical for the enclosure of fire-isolated stairs and passageways, where service penetrations are restricted and the roof of the enclosures tends to be similar to the wall construction. There are notable examples of external downward fire spread involving cladding systems with thermoplastic components that can facilitate downward fire spread by means of burning droplets/molten material. Timber does not exhibit this behaviour because it forms a char when exposed to heat. Notwithstanding the above, the mid-rise timber building Provisions introduced into the NCC address the risk of downward fire spread in a practical way through the specification of automatic fire sprinkler protection, which substantially reduces the risk of a fully developed fire and hence downward fire spread. In addition, the requirements for cavity barriers minimise the consequences should the sprinkler system fail and fire penetrate the floor/ceiling void from above (if a void is present) prior to fire brigade intervention.	The current NCC DTS Provisions do not require floor/ceiling systems to be resistant to fire spread from above (outside) except for some fire-isolated stair and passageway configurations. Notwithstanding the above, the mid-rise timber building Provisions introduced in the NCC 2016 mitigate the risk of downward fire spread through the specification of automatic fire sprinkler protection, which substantially reduces the risk of a fully developed fire and hence downward fire spread compared to other mid-rise forms of construction. Also, as a further redundancy, the requirements for cavity barriers will restrict lateral fire spread in the low probability of failure of the sprinkler system and occurrence of downward fire spread.

Appendix C - Deemed-to-Satisfy Clauses from NCC 2015 Volume One Affected by the Introduction of a Mid-Rise Timber Building Solution in the 2016 Edition

Description	NCC clause	DTS Requirement	Comments
Non-combustible materials (concession)	C1.12	The following materials, though combustible or containing combustible fibres, may be used wherever a non-combustible material is required: (a) Plasterboard. (b) Perforated gypsum lath with a normal paper finish. (c) Fibrous-plaster sheet (d) Fibre-reinforced cement sheeting (e) Pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm thickness and where the Spread-of-Flame Index of the product is not greater than 0. (f) Bonded laminated materials where - (i) each laminate is non-combustible; and (ii) each adhesive layer does not exceed 1 mm in thickness; and (iii) the total thickness of the adhesive layers does not exceed 2 mm; and (iv) the Spread-of-Flame Index and the Smoke-Developed Index of the laminated material as a whole does not exceed 0 and 3 respectively.	Additional Clause C1.13 applies to fire-protected timber.
Vertical Separation of openings	C2.6	(a) If in a building of Type A construction, any part of a window or other opening in an external wall is above another opening in the storey next below and its vertical projection falls no further than 450 mm outside the lower opening (measured horizontally), the openings must be separated by - (i) a spandrel which - (A) is not less than 900 mm in height; and (B) extends not less than 600 mm above the upper surface of the intervening floor; and (C) is of non-combustible material having an FRL of not less than 60/60/60; or (ii) part of a curtain wall or panel wall that complies with (i); or (iii) construction that complies with (i) behind a curtain wall or panel wall and has any gaps packed with a non-combustible material that will withstand thermal expansion and structural movement of the walling without the loss of seal against fire and smoke; or (iv) a slab or other horizontal construction that - (A) projects outwards from the external face of the wall not less than 1100 mm; and (B) extends along the wall not less than 450 mm beyond the openings concerned; and (C) is non-combustible and has an FRL of not less than 60/60/60. (b) The requirements of (a) do not apply to - (i) an open-deck carpark; or (ii) an open spectator stand; or (iii) a building which has a sprinkler system complying with Specification E1.5 installed throughout; or (iv) openings within the same stairway; or (v) openings in external walls where the floor separating the storeys does not require an FRL with respect to integrity and insulation. (c) For the purposes of C2.6, window or other opening means that part of the external wall of a building that does not have an FRL of 60/60/60 or greater.	An automatic fire sprinkler system complying with Specification E1.5 is to be installed throughout, so other requirements of this clause do not apply – refer clause 2.6(b). There is therefore no variation from this clause. Analysis showed the automatic fire sprinkler option reduces the risk to life.

Description	NCC clause	DTS Requirement	Comments
Separation by Fire Walls	C2.7	(a) Construction — A fire wall must be constructed in accordance with the following: (i) The fire wall has the relevant FRL prescribed by Specification C1.1 for each of the adjoining parts, and if these are different, the greater FRL, except where Tables 3.9, 4.2 and 5.2 of Specification C1.1 permit a lower FRL on the carpark side. (ii) Any openings in a fire wall must not reduce the FRL required by Specification C1.1 for the fire wall, except where permitted by the Deemed-to-Satisfy Provisions of Part C3. (iii) Building elements, other than roof battens with dimensions of 75 mm x 50 mm or less or sarking-type material, must not pass through or cross the fire wall unless the required fire resisting performance of the fire wall is maintained. (b) Separation of buildings — A part of a building separated from the remainder of the building by a fire wall may be treated as a separate building for the purposes of the Deemed-to-Satisfy Provisions of Sections C, D and E if it is constructed in accordance with (a) and the following: (i) The fire wall extends through all storeys and spaces in the nature of storeys that are common to that part and any adjoining part of the building. (ii) The fire wall is carried through to the underside of the roof covering. (iii) Where the roof of one of the adjoining parts is lower than the roof of the other part, the fire wall extends to the underside of— (A) the covering of the higher roof, or not less than 6 m above the covering of the lower roof; or (B) the lower roof if it has an FRL not less than that of the fire wall and no openings closer than 3 m to any wall above the lower roof; or (C) the lower roof if it's covering is non-combustible and the lower part has a sprinkler system complying with Specification E1.5. (c)Separation of fire compartments — A part of a building separated from the remainder of the building by a fire wall may be treated as a separate fire compartment if it is constructed in accordance with (a) and the fire wall extends to the underside of— (i) a floor having an FRL required for a fire wall; or (ii) the roof covering.	Loadbearing fire walls are currently required to be of masonry or concrete construction and therefore the proposed new clauses in Specification C1.1 Clause 3.1 and 4.1 will permit these walls to be manufactured from timber with fire-protective coverings. Fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m have been shown to substantially reduce the risk to occupants.
Separation of Classifications in the same storey	C2.8	If a building has parts of different classifications located alongside one another in the same storey— (a) each building element in that storey must have the higher FRL prescribed in Specification C1.1 for that element for the classifications concerned; or (b) the parts must be separated in that storey by a fire wall having— (i) the higher FRL prescribed in Table 3 or 4; or (ii) the FRL prescribed in Table 5, of Specification C1.1 as applicable, for that element for the Type of construction and the classifications concerned; or (c) where one part is a carpark complying with Table 3.9, 4.2 or 5.2 of Specification C1.1, the parts may be separated	Loadbearing fire walls are currently required to be of masonry or concrete construction. The proposed new clauses in Specification C1.1 Clause 3.1 and 4.1 will permit these walls to be manufactured from timber with fire-protective coverings. Fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m have been shown to substantially reduce the risk to occupants.

Description	NCC clause	DTS Requirement	Comments
Separation of lift shafts	C2.10	(a) Any lift connecting more than 2 storeys, or more than 3 storeys if the building is sprinklered, (other than lifts which are wholly within an atrium) must be separated from the remainder of the building by enclosure in a shaft in which— (i) in a building required to be of Type A construction—the walls have the relevant FRL prescribed by Specification C1.1; and (ii) in a building required to be of Type B construction — the walls— (A) if loadbearing, have the relevant FRL prescribed by Table 4 of Specification C1.1; or. (B) if non-loadbearing, be of non-combustible construction. (b) Any lift in a patient care area in a Class 9a health-care building or a resident use area in Class 9c aged care building must be separated from the remainder of the building by a shaft having an FRL of not less than— (i) in a building of Type A or B construction — 120/120/120; or (ii) in a building of Type C construction — 60/60/60. (c) An emergency lift must be contained within a fire-resisting shaft having an FRL of not less than 120/120/120. (d) Openings for lift landing doors and services must be protected in accordance with the Deemed-to-Satisfy Provisions of Part C3.	Tables 3 and 4 of Specification C1.1 require loadbearing lift shafts to have an FRL of 90/90/90 and non-loadbearing lift shafts to have an FRL of -/90/90 for Buildings of Type A and B construction, respectively. These will be satisfied. Clauses 3 and 4 of Specification C1.1 currently require – A non-loadbearing internal wall required to be fire resisting to be of non-combustible construction and a loadbearing internal wall (including those that are part of a loadbearing shaft) to be concrete or masonry. The proposed changes will allow the use of timber protected by fire-preventative coverings. Fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m have been shown to address any potential for increased risk associated with the changes.
Separation of Equipment	C2.12	(a) Equipment other than that described in (b) and (c) must be separated from the remainder of the building with construction complying with (d), if that equipment comprises— (i) lift motors and lift control panels; or (ii) emergency generators used to sustain emergency equipment operating in the emergency mode; or (iii) central smoke control plant; or (iv) boilers; or (v) a battery or batteries installed in the building that have a voltage exceeding 24 volts and a capacity exceeding 10 ampere hours. (b) Equipment need not be separated in accordance with (a) if the equipment comprises— (i) smoke control exhaust fans located in the air stream which are constructed for high temperature operation in accordance with Specification E2.2b; or (ii) stair pressurising equipment installed in compliance with the relevant provisions of AS/NZS 1668.1; or (iii) a lift installation without a machine-room; or (iv) equipment otherwise adequately separated from the remainder of the building. (c) Separation of on-site fire pumps must comply with the requirements of AS 2419.1. (d) Separating construction must have— (i) except as provided by (ii)— (A) an FRL as required by Specification C1.1, but not less than 120/120/120; and (B) any doorway protected with a self-closing fire door having an FRL of not less than –/120/30; or (ii) when separating a lift shaft and lift motor room, an FRL not less than 120/–/–.	C2.12 requires separation by construction having an FRL not less than 120/120/120. This will be satisfied. Clauses 3 and 4 of Specification C1.1 currently require – A non-loadbearing internal wall required to be fire resisting to be of non-combustible construction and a loadbearing internal wall (including those that are part of a loadbearing shaft) to be concrete or masonry. The proposed changes will allow the use of timber protected by fire-preventative coverings. The provision of fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m is expected to address any potential for increased risk associated with the changes.

Description	NCC clause	DTS Requirement	Comments
Electricity supply System	C2.13	(a) An electricity substation located within a building must— (i) be separated from any other part of the building by construction having an FRL of not less than 120/120/120; and (ii) have any doorway in that construction protected with a self-closing fire door having an FRL of not less than –/120/30. (b) A main switchboard located within the building which sustains emergency equipment operating in the emergency mode must— (i) be separated from any other part of the building by construction having an FRL of not less than 120/120/120; and (ii) have any doorway in that construction protected with a self-closing fire door having an FRL of not less than –/120/30. (c)Electrical conductors located within a building that supply— (i)a substation located within the building which supplies a main switchboard covered by (b); or (ii) a main switchboard covered by (b), must- (iii) have a classification in accordance with AS/NZS 3013 of not less than— (A) if located in a position that could be subject to damage by motor vehicles — WS53W; or (B) otherwise — WS52W; or (iv) be enclosed or otherwise protected by construction having an FRL of not less than 120/120/120. (d) Where emergency equipment is required in a building, all switchboards in the electrical installation, which sustain the electricity supply to the emergency equipment, must be constructed so that emergency equipment switchgear is separated from non-emergency equipment switchgear by metal partitions designed to minimise the spread of a fault from the non-emergency equipment switchgear. (e)For the purposes of (d), emergency equipment includes but is not limited to the following: (i) Fire hydrant booster pumps. (ii) Pumps for automatic sprinkler systems, water spray, chemical fluid suppression systems or the like. (iii) Pumps for fire hose reels where such pumps and fire hose reels form the sole means of fire protection in the building. (iv) Air handling systems designed to exhaust and control the spread of fire and smoke.	C2.13 requires separation by construction having an FRL of 120/120/120. This will be satisfied. Clauses 3 and 4 of Specification C1.1 currently require – A non-loadbearing internal wall required to be fire resisting to be of non-combustible construction and a loadbearing internal wall (including those that are part of a loadbearing shaft) to be concrete or masonry. The proposed changes will allow the use of timber protected by fire-preventative coverings. The provision of fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m is expected to address any potential for increased risk associated with the changes.
Public corridors in Class 2 and 3 buildings	C2.14	C2.14 Public corridors in Class 2 and 3 buildings In a Class 2 or 3 building, a public corridor, if more than 40 m in length, must be divided at intervals of not more than 40 m with smoke-proof walls complying with Clause 2 of Specification C2.5	This could be required for large Class 2 and 3 buildings and Clause 2 of Specification 2.5 specifies non-combustible construction. The provision of fire-protective coverings and fire sprinklers for buildings with an effective height limit of 25 m is expected to address any potential for increased risk.
Openings in floors and ceilings for services	C3.12	(a) Where a service passes through— (i) a floor that is required to have an FRL with respect to integrity and insulation; or (ii) a ceiling required to have a resistance to the incipient spread of fire, the service must be installed in accordance with (b). (b) A service must be protected— (i) in a building of Type A construction, by a shaft complying with Specification C1.1; or (ii) in a building of Type B or C construction, by a shaft that will not reduce the fire performance of the building elements it penetrates; or (iii) in accordance with C3.15. (c) Where a service passes through a floor which is required to be protected by a fire protective covering, the penetration must not reduce the fire performance of the covering.	Service penetrations protected by a shaft complying with Specification 1.1 would comply, except that timber-framed shafts lined with fire-resistant coverings would be permitted. Also, timber elements protected by coverings would be permitted within 100 mm of an uninsulated service penetration system complying with C3.15. The provision of fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m is expected to address any potential for increased risk associated with the changes.

Description	NCC clause	DTS Requirement	Comments
Openings for Service Installations	C3.15	Where an electrical, electronic, plumbing, mechanical ventilation, air-conditioning or other service penetrates a building element (other than an external wall or roof) that is required to have an FRL with respect to integrity or insulation or a resistance to the incipient spread of fire, that installation must comply with any one of the following: (a) Tested systems (i)The service, building element and any protection method at the penetration are identical with a prototype assembly of the service, building element and protection method which has been tested in accordance with AS 4072.1 and AS 1530.4 and has achieved the required FRL or resistance to the incipient spread of fire. (ii) It complies with (i) except for the insulation criteria relating to the service if— (A) the service is a pipe system comprised entirely of metal (excluding pipe seals or the like); and (B) any combustible building element is not located within 100 mm of the service for a distance of 2 m from the penetration; and (C) combustible material is not able to be located within 100 mm of the service for a distance of 2 m from the penetration; and (D) it is not located in a required exit. (b) Ventilation and air-conditioning — In the case of ventilating or air-conditioning ducts or equipment, the installation is in accordance with AS/NZS 1668.1. (c) Compliance with Specification C3.15 (i) The service is a pipe system comprised entirely of metal (excluding pipe seals or the like) and is installed in accordance with Specification C3.15 and it— (A) penetrates a wall, floor or ceiling, but not a ceiling required to have a resistance to the incipient spread of fire; and (B) connects not more than 2 fire compartments in addition to any fire-resisting service shafts; and (C) does not contain a flammable or combustible liquid or gas. (ii) The service is sanitary plumbing installed in accordance with Specification C3.15 and it— (A) is of metal or UPVC pipe; and (B) penetrates the floors of a Class 5, 6, 7, 8 or 9b building; and (C) is in a sanitary compartment separated from other parts of the building by walls with the FRL required by Specification C1.1 for a stair shaft in the building and a self-closing –/60/30 fire door. (iii) The service is a wire or cable, or a cluster of wires or cables installed in accordance with Specification C3.15 and it— (A) penetrates a wall, floor or ceiling, but not a ceiling required to have a resistance to the incipient spread of fire; and (B) connects not more than 2 fire compartments in addition to any fire-resisting service shafts. (iv) The service is an electrical switch, outlet, or the like, and it is installed in accordance with Specification C3.15.	Service penetrations to be protected in accordance with C3.15 to maintain fire separation. Reliance will be predominantly on tested/ assessed systems. There is a slight relaxation, in that timber elements would be permitted within 100 mm of uninsulated service penetrations subject to the timber being protected by fire-protective coverings. The provision of fire-protective coverings and automatic fire sprinklers for buildings with an effective height limit of 25 m is expected to address any potential for increased risk associated with the changes.
Columns protected with lightweight construction to achieve an FRL	C3.17	A column protected by lightweight construction to achieve an FRL which passes through a building element that is required to have an FRL or a resistance to the incipient spread of fire, must be installed using a method and materials identical with a prototype assembly of the construction which has achieved the required FRL or resistance to the incipient spread of fire.	Fire-tested systems will be adopted and therefore this clause will be satisfied.

Description	NCC clause	DTS Requirement	Comments
General Concessions	Spec C1.1 Cl 2.5	(a) Steel columns — A steel column, other than one in a fire wall or common wall, need not have an FRL in a building that contains— (i) only 1 storey; or (ii) 2 storeys in some of its parts and 1 storey only in its remaining parts if the sum of the floor areas of the upper storeys of its 2 storey parts does not exceed the lesser of— (A) 1/8 of the sum of the floor areas of the 1 storey parts; or (B) in the case of a building to which one of the maximum floor areas specified in Table C2.2 is applicable — 1/10 of that area; or (C) in the case of a building to which two or more of the maximum floor areas specified in Table C2.2 is applicable — 1/10 of the lesser of those areas. (b) Timber columns — A timber column may be used in a single storey building if— (i) in a fire wall or common wall the column has an FRL not less than that listed in the appropriate Table 3, 4 or 5; and (ii) in any other case where the column is required to have an FRL in accordance with Table 3, 4 or 5, it has an FRL of not less than 30/–/–. (c) Structures on roofs — A non-combustible structure situated on a roof need not comply with the other provisions of this Specification if it only contains— (i) lift motor equipment; or (ii) one or more of the following: (A) Hot water or other water tanks. (B) Ventilating ductwork, ventilating fans and their motors. (C) Air-conditioning chillers. (D) Window cleaning equipment. (E) Other service units that are non-combustible and do not contain flammable or combustible liquids or gases. (d) Curtain walls and panel walls — A requirement for an external wall to have an FRL does not apply to a curtain wall or panel wall which is of non-combustible construction and fully protected by automatic external wall-wetting sprinklers. (e) * * * * * (f) Balconies and verandas — A balcony, veranda or the like and any incorporated supporting part, which is attached to or forms part of a building, need not comply with Tables 3, 4 and 5 if— (i) it does not form part of the only path of travel to a required exit from the building; and (ii) in Type A construction— (A) it is situated not more than 2 storeys above the lowest storey providing direct egress to a road or open space; and (B) any supporting columns are of non-combustible construction.	The proposed changes allow timber to be used in the applications where non-combustible construction is specified in these concessions (clauses d and e) subject to the use of fire-protected timber, the provision of automatic fire sprinklers throughout the building and with an effective height limit of 25 m. These precautions are expected to address any potential for increased risk associated with allowing the use of timber for these applications.
Enclosure of shafts	Spec C1.1 Cl 2.7	Shafts required to have an FRL must be enclosed at the top and bottom by construction having an FRL not less than that required for the walls of a non-loadbearing shaft in the same building, except that these provisions need not apply to— (a) the top of a shaft extending beyond the roof covering, other than one enclosing a fire-isolated stairway or ramp; or (b) the bottom of a shaft if it is non-combustible and laid directly on the ground.	FRLs will be in accordance with DTS at top and bottom of shafts. Fire-protected timber will be allowed for this application. In combination with the provision of automatic fire sprinklers, it is expected the changes will reduce the risk from fires.

Description	NCC clause	DTS Requirement	Comments
Residential aged Care Building	Spec C1.1 Cl 2.9	In a Class 3 building protected with a sprinkler system complying with Specification E1.5 and used as a residential aged care building, any FRL criterion prescribed in Tables 3, 4 or 5— (a) for any floor and any loadbearing wall, may be reduced to 60, except any FRL criterion of 90 for an external wall must be maintained when tested from the outside; and (b) for any non-loadbearing internal wall, need not apply if— (i) it is lined on each side with standard grade plasterboard not less than 13 mm thick or similar non-combustible material; and (ii) it extends— (A) to the underside of the floor next above; or (B) to the underside of a ceiling lined with standard grade plasterboard not less than 13 mm thick or a material with at least an equivalent level of fire protection; or (C) to the underside of a non-combustible roof covering; and (iii) any insulation installed in the cavity of the wall is non-combustible; and (iv) any construction joint, space or the like between the top of the wall and the floor, ceiling or roof is smoke sealed with intumescent putty or other suitable material.	This Clause relaxes the requirements for FRLs in Class 3 residential aged care buildings if protected by automatic fire sprinkler systems. The definition for fire-preventative coverings requires the element to achieve an FRL of 90/90/90 or -/90/90. Therefore, if timber with protective coverings is used in lieu of concrete, masonry or materials already Deemed-to-Satisfy non-combustible significantly higher FRLs will be provided, which would be expected to reduce the risk to life. This Clause does not apply to Class 2 buildings.
Type A Fire Resisting Construction	Spec C1.1 Cl 3.1	Fire-resistance of building elements In a building required to be of Type A construction— (a) each building element listed in Table 3 and any beam or column incorporated in it, must have an FRL not less than that listed in the Table for the particular Class of building concerned; and (b) external walls, common walls and the flooring and floor framing of lift pits must be non-combustible; and (c) any internal wall required to have an FRL with respect to integrity and insulation must extend to— (i) the underside of the floor next above; or (ii) the underside of a roof complying with Table 3; or (iii) if under Clause 3.5 the roof is not required to comply with Table 3, the underside of the non-combustible roof covering and, except for roof battens with dimensions of 75 mm x 50 mm or less or sarking-type material, must not be crossed by timber or other combustible building elements; or (iv) a ceiling that is immediately below the roof and has a resistance to the incipient spread of fire to the roof space between the ceiling and the roof of not less than 60 minutes; and (d) a loadbearing internal wall and a loadbearing fire wall (including those that are part of a loadbearing shaft) must be of concrete or masonry; and (e) a non-loadbearing— (i) internal wall required to be fire-resisting; and (ii) lift, ventilating, pipe, garbage, or similar shaft that is not for the discharge of hot products of combustion, must be of non-combustible construction; and (f) the FRLs specified in Table 3 for an external column apply also to those parts of an internal column that face and are within 1.5 m of a window and are exposed through that window to a fire-source feature.	The proposed changes allow fire-protected timber to be used, subject to the provision of automatic fire sprinklers throughout the building and an effective height limit of 25 m. These precautions are expected to address any potential for increased risk associated with allowing the use of timber for these applications. Clause 3.1(d) requires loadbearing internal and fire walls to be of masonry or concrete construction. The proposed new clause 3.1 will permit these walls to be manufactured from fire-protected timber subject to automatic fire sprinklers being provided and an effective height limit of 25 m applying, which is expected to address any potential for increased risk associated with the change.
Concessions for floors	Spec C1.1 Cl 3.2	A floor need not comply with Table 3 if— (a) it is laid directly on the ground; or (b) in a Class 2, 3, 5 or 9 building, the space below is not a storey, does not accommodate motor vehicles, is not a storage or work area, and is not used for any other ancillary purpose; or (c) it is a timber stage floor in a Class 9b building laid over a floor having the required FRL and the space below the stage is not used as a dressing room, store room, or the like; or (d) it is within a sole-occupancy unit in a Class 2 or 3 building or Class 4 part of a building; or (e) it is an open-access floor (for the accommodation of electrical and electronic services and the like) above a floor with the required FRL.	Will be applied as appropriate.

Description	NCC clause	DTS Requirement	Comments
Internal columns and walls: Concession	Spec C1.1 Cl 3.7	For a building with an effective height of not more than 25 m and having a roof without an FRL in accordance with Clause 3.5, in the storey immediately below that roof, internal columns other than those referred to in Clause 3.1(f) and internal walls other than fire walls and shaft walls may have— (a) in a Class 2 or 3 building: FRL 60/60/60; or (b) in a Class 5, 6, 7, 8 or 9 building— (i) with rise in storeys exceeding 3: FRL 60/60/60 (ii) with rise in storeys not exceeding 3: no FRL	If elements are also required to be non-combustible and fire protected timber is to be used, the FRLs of those members will be higher than those specified in this concession, tending to reduce the risk from fire
Type B Fire-Resisting Construction	Spec C1.1 Cl 4.1	In a building required to be of Type B construction— (a) each building element listed in Table 4, and any beam or column incorporated in it, must have an FRL not less than that listed in the Table for the particular Class of building concerned; and (b) the external walls, common walls, and the flooring and floor framing in any lift pit, must be non-combustible; and (c) if a stair shaft supports any floor or a structural part of it— (i) the floor or part must have an FRL of 60/–/– or more; or (ii) the junction of the stair shaft must be constructed so that the floor or part will be free to sag or fall in a fire without causing structural damage to the shaft; and (d) any internal wall which is required to have an FRL with respect to integrity and insulation, except a wall that bounds a sole-occupancy unit in the topmost (or only) storey and there is only one unit in that storey, must extend to— (i) the underside of the floor next above if that floor has an FRL of at least 30/30/30; or (ii) the underside of a ceiling having a resistance to the incipient spread of fire to the space above itself of not less than 60 minutes; or (iii) the underside of the roof covering if it is non-combustible and, except for roof battens with dimensions of 75 mm x 50 mm or less or sarking-type material, must not be crossed by timber or other combustible building elements; or (iv) 450 mm above the roof covering if it is combustible; and (e) a loadbearing internal wall and a loadbearing fire wall (including those that are part of a loadbearing shaft) must be of concrete or masonry; and (f) a non- loadbearing internal wall required to be fire-resisting must be of non-combustible construction; and (g) in a Class 5, 6, 7, 8 or 9 building, in the storey immediately below the roof, internal columns and internal walls other than fire walls and shaft walls, need not comply with Table 4; and (h) lift, subject to C2.10, ventilating, pipe, garbage, and similar shafts which are not for the discharge of hot products of combustion and not loadbearing, must be of non-combustible construction in— (i) a Class 2, 3 or 9 building; and (ii) a Class 5, 6, 7 or 8 building if the shaft connects more than 2 storeys; and (i) in a Class 2 or 3 building, except where within the one sole-occupancy unit, or a Class 9a health-care building or a Class 9b building, a floor separating storeys or above a space for the accommodation of motor vehicles or used for storage or any other ancillary purpose, must— (i) be constructed so that it is at least of the standard achieved by a floor/ceiling system incorporating a ceiling which has a resistance to the incipient spread of fire to the space above itself of not less than 60 minutes; or (ii) have an FRL of at least 30/30/30; or (iii) have a fire-protective covering on the underside of the floor, including beams incorporated in it, if the floor is combustible or of metal; and (j) in a Class 9c aged care building a floor above a space for the accommodation of motor vehicles or used for storage or any other ancillary purpose, and any column supporting the floor must— (i) be constructed so that it is at least of the standard achieved by a floor/ceiling system incorporating a ceiling which has a resistance to the incipient spread of fire to the space above itself of not less than 60 minutes; or (ii) have an FRL of at least 30/30/30; or (iii) have a fire-protective covering on the underside of the floor, including beams incorporated in it, if the floor is combustible or of metal.	The proposed changes allow fire-protected timber to be used in the applications where non-combustible construction is specified, subject to the provision of automatic fire sprinklers throughout the building and an effective height limit of 25 m (less for Type B construction). These precautions are expected to address any potential for increased risk associated with allowing the use of timber for these applications. Clause 4.1(e) requires loadbearing internal and fire walls to be of masonry or concrete construction. The proposed new clause 4.1 will permit these walls to be manufactured from fire-protected timber subject to the provision of automatic fire sprinklers and an effective height limit of 25 m, which is expected to address any potential for increased risk associated with the change.

Description	NCC clause	DTS Requirement	Comments
Smoke proof walls	Spec C2.5 Cl2	Class 9a health-care buildings Smoke-proof walls required by C2.5 in Class 9a health-care buildings must comply with the following: (a) Be non-combustible and extend to the underside of- (i) the floor above; or (ii) a non-combustible roof covering; or (iii) a ceiling having a resistance to the incipient spread of fire to the space above itself of not less than 60 minutes.	This clause also applies to Class 2 and 3 buildings via clause C2.14. For Class 2 and 3, the provision of protected timber and automatic fire sprinklers with an effective height limit of 25 m is expected to address any potential for increased risk associated with the changes.
Penetrations of walls floors and ceilings by services	Spec C3.15	1. Scope This Specification prescribes materials and methods of installation for services that penetrate walls, floors and ceilings required to have an FRL. 2. Application (a) This Specification applies to installations permitted under the Deemed-to-Satisfy Provisions of the NCC as alternatives to systems that have been demonstrated by test to fulfil the requirements of C3.15 (a). (b) This Specification does not apply to installations in ceilings required to have a resistance to the incipient spread of fire nor to the installation of piping that contains or is intended to contain a flammable liquid or gas. 3. Metal pipe systems (a) A pipe system comprised entirely of metal (excluding pipe seals or the like) that is not normally filled with liquid must not be located within 100 mm, for a distance of 2 m from the penetration, of any combustible building element or a position where combustible material may be located, and must be constructed of - (i) copper alloy or stainless steel with a wall thickness of at least 1 mm; or (ii) cast iron or steel (other than stainless steel) with a wall thickness of at least 2 mm. (b) An opening for a pipe system comprised entirely of metal (excluding pipe seals or the like) must - (i) be neatly formed, cut or drilled; and (ii) be no closer than 200 mm to any other service penetration; and (iii) accommodate only one pipe. (c) A pipe system comprised entirely of metal (excluding pipe seals or the like) must be wrapped but must not be lagged or enclosed in thermal insulation over the length of its penetration of a wall, floor or ceiling unless the lagging or thermal insulation fulfils the requirements of Clause 7. (d) The gap between a metal pipe and the wall, floor or ceiling it penetrates must be fire-stopped in accordance with Clause 7. 4. Pipes penetrating sanitary compartments If a pipe of metal or UPVC penetrates the floor of a sanitary compartment in accordance with C3.15(c)(ii) - (a) the opening must be neatly formed and no larger than is necessary to accommodate the pipe or fitting; and (b) the gap between pipe and floor must be fire-stopped in accordance with Clause 7. 5. Wires and cables If a wire or cable or cluster of wires or cables penetrates a floor, wall or ceiling— (a) the opening must be neatly formed, cut or drilled and no closer than 50 mm to any other service; and (b) the opening must be no larger in cross-sectional area than— (i) 2000 mm^2 if only a single cable is accommodated and the gap between cable and wall, floor or ceiling is no wider than 15 mm; or (ii) 500 mm^2 in any other case; and (c) the gap between the service and the wall, floor or ceiling must be fire-stopped in accordance with Clause 7.	There is a slight relaxation in that fire-protected timber would be permitted within 100 mm of uninsulated pipe penetrations, subject to the provision of automatic fire sprinklers for buildings with an effective height limit of 25 m. These requirements are expected to address any potential for increased risk associated with the change. It is noted that this specification is rarely used.

Description	NCC clause	DTS Requirement	Comments
Penetrations of walls floors and ceilings by services (continued)	Spec C3.15	6. Electrical switches and outlets If an electrical switch, outlet, socket or the like is accommodated in an opening or recess in a wall, floor or ceiling— (a) the opening or recess must not— (i) be located opposite any point within 300 mm horizontally or 600 mm vertically of any opening or recess on the opposite side of the wall; or (ii) extend beyond half the thickness of the wall; and (b) the gap between the service and the wall, floor or ceiling must be fire-stopped in accordance with Clause 7. 7. Fire-stopping (a) Material: The material used for the fire-stopping of service penetrations must be concrete, high-temperature mineral fibre, high-temperature ceramic fibre or other material that does not flow at a temperature below 1120°C when tested in accordance with ISO 540, and must have— (i) demonstrated in a system tested in accordance with C3.15(a) that it does not impair the fire-resisting performance of the building element in which it is installed; or (ii) demonstrated in a test in accordance with (e) that it does not impair the fire-resisting performance of the test slab. (b) Installation: Fire-stopping material must be packed into the gap between the service and wall, floor or ceiling in a manner, and compressed to the same degree, as adopted for testing under Clause 7(a) (i) or (ii). (c) Hollow construction: If a pipe penetrates a hollow wall (such as a stud wall, a cavity wall or a wall of hollow blockwork) or a hollow floor/ceiling system, the cavity must be so framed and packed with fire-stopping material that is— (i) installed in accordance with Clause 7(b) to a thickness of 25 mm all-round the service for the full length of the penetration; and (ii) restrained, independently of the service, from moving or parting from the surfaces of the service and of the wall, floor or ceiling. (d) Recesses: If an electrical switch, socket, outlet or the like is accommodated in a recess in a hollow wall or hollow floor/ceiling system— (i) the cavity immediately behind the service must be framed and packed with fire-stopping material in accordance with Clause 7(c); or (ii) the back and sides of the service must be protected with refractory lining board identical with and to the same thickness as that in which the service is installed. (e) Test: The test to demonstrate compliance of a fire-stopping material with this Specification must be conducted as follows: (i) The test specimen must comprise a concrete slab not less than 1 m square and not more than 100 mm thick, and appropriately reinforced if necessary for structural adequacy during manufacture, transport and testing. (ii) The slab must have a hole 50 mm in diameter through the centre and the hole must be packed with the fire-stopping material. (iii) The slab must be conditioned in accordance with AS 1530.4. (iv) Two thermocouples complying with AS 1530.4 must be attached to the upper surface of the packing each about 5 mm from its centre. (v) The slab must be tested on flat generally in accordance with Section 10 of AS 1530.4 and must achieve an FRL of 60/60/60 or as otherwise required.	
External stairways or ramps in lieu of fire- isolated exits	D1.8	External stairways or ramps in lieu of fire-isolated exits (a) An external stairway or ramp may serve as a required exit in lieu of a fire-isolated exit serving a storey below an effective height of 25 m, if the stairway or ramp is- (i) non-combustible throughout; and (ii) protected in accordance with (c) if it is within 6 m of, and exposed to any part of the external wall of the building it serves.	Fire-protected timber would probably be impractical. However, if this approach was implemented, the coverings would be expected to address any increased risk associated with the use of timber.

Description	NCC clause	DTS Requirement	Comments
Fire-isolated stairways and ramps	D2.2	Fire-isolated stairways and ramps A stairway or ramp (including any landings) that is required to be within a fire-resisting shaft must be constructed- (a) of non-combustible materials; and (b) so that if there is local failure it will not cause structural damage to, or impair the fire-resistance of, the shaft.	Fire-protected timber would probably be impractical. However, if this approach was implemented, the coverings would be expected to address any increased risk associated with the use of timber
Non-fire-isolated stairways and ramps	D2.4	Separation of rising and descending stair flights If a stairway serving as an exit is required to be fire-isolated- (a) there must be no direct connection between- (i) a flight rising from a storey below the lowest level of access to a road or open space; and (ii) a flight descending from a storey above that level; and (b) any construction that separates or is common to the rising and descending flights must be- (i) non-combustible; and (ii) smoke-proof in accordance with Clause 2 of Specification C2.S.	Fire-protected timber would be expected to address any increased risk associated with the use of timber
	D2.7	Installations in exits and paths of travel (a) Access to service shafts and services other than to fire-fighting or detection equipment as permitted in the Deemed-to-Satisfy Provisions of Section E, must not be provided from a fire-isolated stairway, fire-isolated passageway or fire isolated ramp. (b) An opening to any chute or duct intended to convey hot products of combustion from a boiler, incinerator, fireplace or the like, must not be located in any part of a required exit or any corridor, hallway, lobby or the like leading to a required exit. (c) Gas or other fuel services must not be installed in a required exit. (d) Services or equipment comprising- (i) electricity meters, distribution boards or ducts; or (ii) central telecommunications distribution boards or equipment; or (iii) electrical motors or other motors serving equipment in the building, may be installed in- (iv) a required exit, except for fire-isolated exits specified in (a); or (v) in any corridor, hallway, lobby or the like leading to a required exit, if the services or equipment are enclosed by non-combustible construction or a fire protective covering with doorways or openings suitably sealed against smoke spreading from the enclosure.	Fire-protected timber would be expected to address any increased risk associated with the use of timber if the non-combustible construction option is selected.
	D2.11	Fire-isolated passageways (a) The enclosing construction of a fire-isolated passageway must have an FRL when tested for a fire outside the passageway in another part of the building of- (i) if the passageway discharges from a fire-isolated stairway or ramp – not less than that required for the stairway or ramp shaft; or (ii) in any other case - not less than 60/60/60. (b) Notwithstanding (a){ii), the top construction of a fire-isolated passageway need not have an FRL if the walls of the fire-isolated passageway extend to the underside of- (i) a non-combustible roof covering; or (ii) a ceiling having a resistance to the incipient spread of fire of not less than 60 minutes separating the roof space or ceiling space in all areas surrounding the passageway within the fire compartment.	No specific requirement for non-combustible construction.

D

Appendix D - Performance Requirement Review

D1 Directly Relevant Performance Requirements

The NCC performance requirements directly relevant to the changes to the NCC 2015 Deemed-to-Satisfy (DtS) Provisions to permit Class 2, 3 and 5 mid-rise timber buildings are listed below:

CP1

A building must have elements which will, to the degree necessary, maintain structural stability during a fire appropriate to –

(a) the function or use of the building; and
(b) the fire load; and
(c) the potential fire intensity; and
(d) the fire hazard; and
(e) the height of the building; and
(f) its proximity to other property; and
(g) any active fire safety systems installed in the building; and
(h) the size of any fire compartment; and
(i) fire brigade intervention; and
(j) other elements they support; and
(k) the evacuation time.

CP2

(a) A building must have elements which will, to the degree necessary, avoid the spread of fire –
(i) to exits; and
(ii) to sole-occupancy units and public corridors; and

Application:
CP2(a)(ii) only applies to a Class 2 or 3 building or Class 4 part of a building.
(iii) between buildings; and
(iv) in a building.

(b) Avoidance of the spread of fire referred to in (a) must be appropriate to –
(i) the function or use of the building; and
(ii) the fire load; and
(iii) the potential fire intensity; and
(iv) the fire hazard; and
(v) the number of storeys in the building; and
(vi) its proximity to other property; and
(vii) any active fire safety systems installed in the building; and
(viii) the size of any fire compartment; and
(ix) fire brigade intervention; and
(x) other elements they support; and
(xi) the evacuation time.

CP4

To maintain tenable conditions during occupant evacuation, a material and an assembly must, to the degree necessary, resist the spread of fire and limit the generation of smoke and heat, and any toxic gases likely to be produced, appropriate to –

(a) the evacuation time; and
(b) the number, mobility and other characteristics of occupants; and
(c) the function or use of the building; and
(d) any active fire safety systems installed in the building.

Application:
CP4 applies to linings, materials and assemblies in a Class 2 to 9 building.

CP6

A building must have elements, which will, to the degree necessary, avoid the spread of fire from service equipment having –

(a) a high fire hazard; or
(b) a potential for explosion resulting from a high fire hazard.

CP7

A building must have elements, which will, to the degree necessary, avoid the spread of fire so that emergency equipment provided in a building will continue to operate for a period of time necessary to ensure that the intended function of the equipment is maintained during a fire.

DP5

To protect evacuating occupants from a fire in the building exits must be fire-isolated, to the degree necessary, appropriate to –

(a) the number of storeys connected by the exits; and
(b) the fire safety system installed in the building; and
(c) the function or use of the building; and
(d) the number of storeys passed through by the exits; and
(e) fire brigade intervention.

EP1.4

An automatic fire suppression system must be installed to the degree necessary to control the development and spread of fire appropriate to –

(a) the size of the fire compartment; and
(b) the function or use of the building; and
(c) the fire hazard; and
(d) the height of the building.

EP2.2

(a) In the event of a fire in a building the conditions in any evacuation route must be maintained for the period of time occupants take to evacuate the part of the building so that –

(i) the temperature will not endanger human life; and
(ii) the level of visibility will enable the evacuation route to be determined; and
(iii) the level of toxicity will not endanger human life.

(b) The period of time occupants take to evacuate referred to in (a) must be appropriate to –

(i) the number, mobility and other characteristics of the occupants;
(ii) and the function or use of the building; and
(iii) the travel distance and other characteristics of the building; and
(iv) the fire load; and
(v) the potential fire intensity; and
(vi) the fire hazard; and
(vii) any active fire safety systems installed in the building; and
(viii) fire brigade intervention.

Limitation: EP2.2 does not apply to an open-deck car park or open spectator stand.

D2 Parameters for Consideration

The Parameters for consideration for the directly relevant performance requirements are summarized in Table D1 and a brief description of how they have been considered in the detailed analysis is provided following the table:

Table D1: Summary of parameters for consideration.

Parameters for Consideration	**Performance Requirements**							
	CP1 – A building must have elements which will, to the degree necessary, maintain structural stability during a fire.	CP2 – A building must have elements to avoid the spread of fire (i) to exits; (ii) to sole-occupancy units and public corridors; (iii) between buildings; and (iv) in a building.	CP4 – To maintain tenable conditions during occupant evacuation a material or assembly must, resist the spread of fire and limit the generation of smoke and heat, and any toxic gases likely to be produced...	CP6 – A building must have elements which will, to the degree necessary, avoid the spread of fire from service equipment...	CP7 – A building must have elements which will, to the degree necessary, avoid spread of fire so that emergency equipment will continue to operate for a period of time necessary to ensure that the intended function of the equipment is maintained during a fire.	DP5 – To protect evacuating occupants from a fire in the building, exits must be fire-isolated, to the degree necessary, appropriate to...	EP1.4 – An automatic fire suppression system to control the development and spread of fire appropriate to...	EP2.2 – ...conditions in any evacuation route must be maintained for the period of time occupants take to evacuate the part of the building...
Function or use of building	●	●	●			●	●	●
Fire load	●	●						●
Potential fire intensity	●	●						●
Fire hazard	●	●		●			●	●
Height of building/ No. of storeys	●	●				●	●	
Proximity to other property	●	●						
Active fire safety systems	●	●	●					●
Size of fire compartment	●	●					●	
Fire brigade intervention	●	●				●		●
Other elements supported	●	●						
Evacuation time/travel distance	●	●	●					●
Occupant mobility, No. and characteristics			●					●
Building fire safety system					●	●		

Function and Use of the Building: The function and use of the building were considered in defining the building layouts and key inputs including derivation of fire loads, occupant numbers and characteristics ventilation conditions, etc.

Fire Load: This was derived based on a literature review and was used as one of the inputs to determine the fire intensity and duration.

Potential fire severity: This was calculated using the methods based on distributions derived for fire load, ventilation conditions and size of compartment together with lining properties. A multi-scenario analysis was adopted to cover a representative range of inputs.

Fire hazards: A review of fire data and literature together with discussions with relevant stakeholders was undertaken to ensure relevant fire hazards/scenarios were considered.

The impact of the height of the building/no of storeys: This was considered specifically when considering the impact of fire brigade intervention and the evacuation of occupants and inherently when estimating the consequences of fires.

Proximity to other buildings: This was addressed when considering the risk of fire spread between buildings.

Active fire safety Systems: The effectiveness of the fire detection and alarm system and automatic fire sprinklers was considered in the analysis.

Size of the compartments: This was a key input to determine the fire severity and also affected occupant numbers/evacuation and fire brigade intervention estimates for search and rescue activities.

Fire Brigade Intervention: A detailed multi-scenario analysis of fire brigade intervention was undertaken to consider the expected range of fire brigade intervention response and activity times.

Other elements supported: The analysis considered the global behaviour of the structures as far as practicable for a generic building and considered design to prevent disproportionate collapse (to be addressed through a separate FWPA Guide). The impact of fire spread through cavities was specifically addressed in relation to the risk of disproportionate collapse and hence other elements supported.

Evacuation time/travel distances: These were incorporated in the estimate of occupant evacuation including consideration of the impact of occupants encountering smoke during the evacuation process.

Occupant mobility, number and characteristics: The detailed analysis incorporated a stochastic evacuation model with distributions relating to the time to commence evacuation and the number of occupants, taking into account occupant characteristics which were compared to fire incidents and drills. The model incorporated a proportion of occupants who did not respond and evacuate unless assisted by the fire brigade.

Building fire safety system: The holistic building fire safety system was considered within the multi-scenario building analysis.

D3 Other Relevant (Supplementary) Performance Requirements

CP8 Any building element provided to resist the spread of fire must be protected, to the degree necessary, so that an adequate level of performance is maintained –

(a) where openings, construction joints and the like occur; and
(b) where penetrations occur for building services.

How CP8 was addressed – Analysis assumed that all service penetrations are protected in accordance with the NCC DTS Provisions for both the control and subject building, but the impact of defects was considered by allocating distributions around the mean FRL of a separating element.

CP9 Access must be provided to and around a building, to the degree necessary, for fire brigade vehicles and personnel to facilitate fire brigade intervention appropriate to –

(a) the function or use of the building; and
(b) the fire load; and
(c) the potential fire intensity; and
(d) the fire hazard; and
(e) any active fire safety systems installed in the building; and
(f) the size of any fire compartment.

How CP9 was addressed – Analysis assumed the same levels of access for both the control and subject building.

DP4 Exits must be provided from a building to allow occupants to evacuate safely, with their number, location and dimensions being appropriate to –

(a) the travel distance; and
(b) the number, mobility and other characteristics of occupants; and
(c) the function or use of the building; and
(d) the height of the building; and
(e) whether the exit is from above or below ground level.

How DP4 was addressed – Analysis assumed the same DtS-compliant configuration for both the control and subject building.

DP6 So that occupants can safely evacuate the building, paths of travel to exits must have dimensions appropriate to –

(a) the number, mobility and other characteristics of occupants; and
(b) the function or use of the building.

Limitation: DP6 does not apply to the internal parts of a sole-occupancy unit in a Class 2 or 3 building or Class 4 part of a building.

How DP6 was addressed - Analysis assumed the same DTS compliant configuration for both the control and subject buildings.

EP1.1 A fire hose reel system must be installed to the degree necessary to allow occupants to safely undertake initial attack on a fire appropriate to –

(a) the size of the fire compartment; and
(b) the function or use of the building; and
(c) any other fire safety systems installed in the building; and
(d) the fire hazard.

How EP1.1 was addressed – Analysis assumed the same DTS-compliant provisions for both the control and subject building and impact of manual fire-fighting by occupants and proportion of potential flashover fires derived from statistics will inherently take this into account. The NCC does not require fire hose reels in Class 2 and 3 buildings.

EP1.2 Fire extinguishers must be installed to the degree necessary to allow occupants to undertake initial attack on a fire appropriate to –

(a) the function or use of the building; and
(b) any other fire safety systems installed in the building; and
(c) the fire hazard.

How EP1.2 was addressed – Analysis assumed the same DTS-compliant provisions for both the control and subject building and impact of manual fire-fighting by occupants and proportion of potential flashover fires derived from statistics will inherently take this into account.

EP1.3 A fire hydrant system must be provided to the degree necessary to facilitate the needs of the fire brigade appropriate to –

(a) fire-fighting operations; and
(b) the floor area of the building; and
(c) the fire hazard.

Application: EP1.3 only applies to a building where a fire brigade is available to attend.

How EP1.3 was addressed – Analysis assumed the same DTS-compliant provisions for both the control and subject building and fire brigade intervention modelling will take these Provisions into account.

EP1.5 Suitable means of fire-fighting must be installed to the degree necessary in a building under construction to allow initial fire attack by construction workers and for the fire brigade to undertake attack on the fire appropriate to –

(a) the fire hazard; and
(b)the height the building has reached during its construction.

How EP1.5 was addressed – No relaxation to the DTS Provisions was sought. A broader approach to fire safety during construction is required to be taken to comply with WHS legislation normally requiring the development of a Fire Safety Plan with much broader scope than the NCC. Use of the WoodSolutions Technical Design Guide #20: Fire Precautions during Construction of Large Buildings is recommended for all buildings to supplement the NCC Deemed-to-Satisfy requirements

EP1.6 Suitable facilities must be provided to the degree necessary in a building to co-ordinate fire brigade intervention during an emergency appropriate to –

(a) the function or use of the building; and
(b) the floor area of the building; and
(c) the height of the building

How EP1.6 was addressed – Analysis assumed the same DTS-compliant provisions for both the control and subject building and fire brigade intervention modelling will take these provisions into account. Since the building is less than 25 m high, it will be assumed that these facilities will be limited to a Fire Indicator Panel close to the entrance.

EP2.1 In a building providing sleeping accommodation, occupants must be provided with automatic warning on the detection of smoke so they may evacuate in the event of a fire to a safe place.

Application: EP2.1 only applies to a Class 2, 3, 9a or 9c building or Class 4 part of a building.

How EP2.1 was addressed – Analysis assumed the same DTS-compliant provisions for both the control and subject building except that an additional alarm will be raised in the subject building upon activation of an automatic fire sprinkler system.

BP1.1

(a) A building or structure, during construction and use, with appropriate degrees of reliability must –

(i) perform adequately under all reasonably expected design actions; and

(ii) withstand extreme or frequently repeated design actions; and

(iii) be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; and

(iv) avoid causing damage to other properties, by resisting the actions to which it may reasonably expect to be subjected.

(b) The actions to be considered to satisfy (a) include but are not limited to –

(i) permanent actions (dead loads); and
(ii) imposed actions (live loads arising from occupancy and use); and
(iii) wind action; and
(iv) earthquake action; and
(v) snow action; and
(vi) liquid pressure action; and
(vii) ground water action; and
(viii) rainwater action (including ponding action); and
(ix) earth pressure action; and
(x) differential movement; and
(xi) time dependent effects (including creep and shrinkage); and
(xii) thermal effects; and
(xiii) ground movement caused by –

(A) swelling, shrinkage or freezing of the subsoil; and
(B) landslip or subsidence; and
(C) site works associated with the building or structure; and

(xiv) construction activity actions; and
(xv) termite actions.

How BP1.1 was addressed – It was assumed that the structure of the control and subject buildings will be designed in accordance with these provisions and specifically resistance to disproportionate collapse will be considered when considering the impact of a fully developed fire on the structures as required to show compliance with CP1.

E

Refer to NCC Spec A2.3 for FRL

Refer to NCC Spec A2.5 for RISF

Refer to NCC Spec A2.2 for non- combustibility

Refer to NCC Spec C1.13 for cavity barriers

Appendix E: Evidence of Suitability for Fire-protected Timber & Cavity Barriers

There are three components to the performance of fire-protected timber; all of which need to be satisfied:

- Fire-protective coverings must be non-combustible.
- The protected element must achieve the required fire resistance level – FRL.
- The protected element must achieve the required Resistance to the Incipient Spread of Fire (RISF).

E1 Non-combustible Fire-protective Covering

The NCC definition of non-combustible applies which states:

Non-combustible means –

(a) applied to a material – not deemed combustible as determined by AS 1530.1 – Combustibility Tests for Materials; and

(b) applied to construction or part of a building – constructed wholly of materials that are not deemed combustible.

This means that if the fire-protective covering is a composite or multi-layer system, each layer must be non-combustible. It is not acceptable to undertake a single combustibility test on the composite or just the facing materials and claim the fire-protective covering is non-combustible.

Typical examples of multi-layer systems are shown in Figure E1.

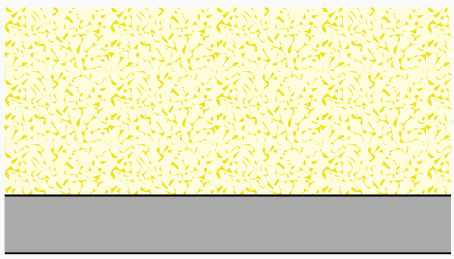
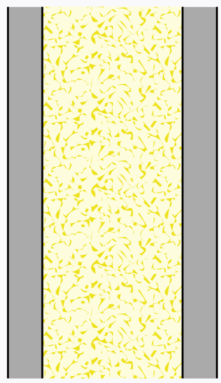
	
Multilayer –system – each layer must be non-combustible	Composite panels – each layer of the composite must be non-combustible
Commonly fire resistant board supporting non-combustible lightweight insulation used in ceilings protecting floors / beams	Commonly – non-combustible lightweight insulating core between non-combustible durable facings used for external claddings

Figure E1: Example of multi-layered fire-protective coverings (all layers).

In addition, Clause C1.12 of the NCC allows the following materials, though combustible or containing combustible fibres, to be used wherever a non-combustible material is required:

a) Plasterboard

b) Perforated gypsum lath with a normal paper finish

c) Fibrous-plaster sheet

d Fibre-reinforced cement sheeting

e) Pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm thickness and where the Spread-of-Flame Index of the product is not greater than 0

f) Bonded laminated materials where –

i. each laminate is non-combustible; and

ii. each adhesive layer does not exceed 1 mm in thickness; and

iii. the total thickness of the adhesive layers does not exceed 2 mm; and

iv. the Spread-of-Flame Index and the Smoke-Developed Index of the laminated material as a whole does not exceed 0 and 3 respectively.

All materials forming the fire-protective covering shall therefore either be permitted to be used in accordance with NCC Clause C1.12 or shall be determined to be non-combustible by testing to AS1530.1.

E2 Fire Resistance Level

A fire-protected timber element must achieve the required FRL specified in the NCC for the particular application. The fire resistance of a fire-protected timber element has to be determined in accordance with Specification A 2.3 of the NCC.

Generally, Specification A2.3 requires a prototype to be submitted to the Standard Fire Test (AS1530.4) – or an equivalent or more severe test – and the FRL achieved by the prototype, without the assistance of an active fire suppression system, is confirmed in a report from a Registered Testing Authority (RTA) that:

(i) describes the method and conditions of the test and the form of construction of the tested prototype in full; and

(ii) certifies that the application of restraint to the prototype complied with the Standard Fire Test;

or

it differs in only a minor degree from a tested prototype and the FRL attributed to the building element is confirmed in a report from an RTA that –

(i) certifies that the building element is capable of achieving the FRL despite the minor departures from the tested prototype; and

(ii) describes the materials, construction and conditions of restraint which are necessary to achieve the FRL.

The option to use AS 1720.4(1990 and 2006 edition) char-based calculation methods without additional supporting data to determine the fire resistance of fire-protected timber is not appropriate. This is due to concerns regarding the suitability of the current AS 1720.4 approach for certain types of adhesives and connections forming parts of engineered timber products, and there was insufficient data available at the time to demonstrate the suitability or otherwise of AS 1720.4.

Figure E2 through Figure E4 show a fire resistance test performed on a lightweight timber floor/ceiling system incorporating a range of lightweight engineered timber joists and trusses protected by a fire-grade plasterboard ceiling.

Figure E2: Lightweight loaded timber floor system after 90-minute fire-resistance test.

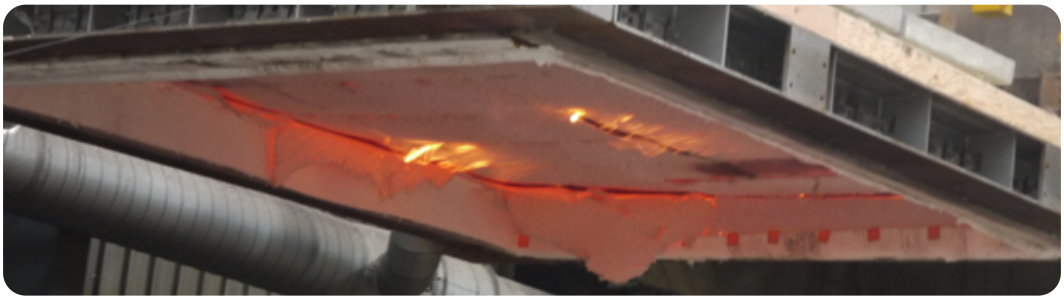


Figure E3: Fire-exposed face of lightweight loaded timber floor system after 90-minute fire resistance test.

Figure E4: Fire-exposed face of lightweight loaded timber floor system about eight minutes after 90-minute fire resistance test, after suppression with a fire hose.

E3 Resistance to the Incipient Spread of Fire

E3.1 Determine Applicable Resistance to Incipient Spread of Fire Requirements

The Resistance to the Incipient Spread of Fire (RISF) in relation to a fire-protective covering means the ability of the covering to insulate voids and the interfaces with timber elements so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The performance is expressed as the period in minutes that the covering will maintain a temperature below the specified limits when subjected to a test in accordance with AS 1530.4.

The general requirement for fire-protected timber is an RISF of 45 minutes.

The NCC permits a relaxation to the resistance to incipient spread of fire requirements for massive timber panels providing both the following additional criteria are satisfied:

- The minimum timber thickness is not less than 75 mm.
- There are no cavities between the surface of the timber and the fire-protective covering or between timber members.

The 75 mm dimensions relate to the minimum dimensions of the dressed/finished timber member. If the relaxation conditions are satisfied then the modified resistance to incipient spread of fire (MRISF) criteria are applicable. Typical examples of massive timber installations satisfying the conditions for this concession to apply are shown in Figure 4.3 in the body of this Guide.

The flow chart in Figure E5 shows the process for determining the applicable RISF requirements.

The general requirement for fire-protected timber is an RISF of 45 minutes.

The relaxed requirements for massive timber construction without voids and cavities is an MRISF that applies a higher cavity temperature limit and the time periods for which the temperature limit applies varies according to the application in accordance with Table E1.

Table E1: MRISF for massive timber construction.

Application	**Modified Resistance to Incipient Spread of Fire (MRISF)**
Inside a fire-isolated stairway or lift shaft	20 min
External walls within 1 m of an allotment boundary or 2 m of a building on the same allotment	45 min
All other applications	30 min

Note: These criteria only apply if the massive timber element has a minimum thickness of 75 mm or greater and the form of construction does not include voids and cavities

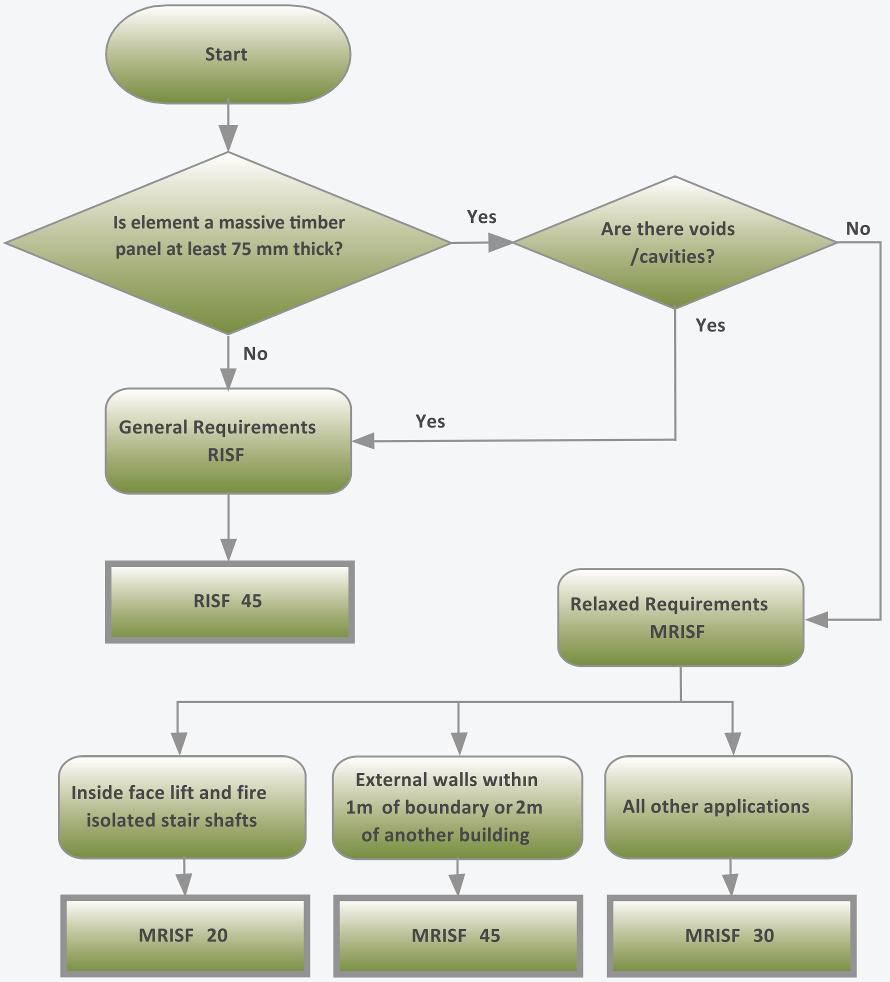


Figure E5: Determination of RISF acceptance requirements.

E3.2 Compliance Paths for Resistance to the Incipient Spread of Fire

Three paths are permitted to demonstrate compliance with the Resistance to the Incipient Spread of Fire requirements:

- simultaneous determination during a full-scale fire resistance test
- smaller scale fire resistance test (at least 1 m x 1 m specimen)
- selection of Deemed-to-Satisfy fire-resisting grade plasterboard coverings.

Simultaneous determination during a full-scale fire resistance test

When a fire resistance test is undertaken to determine the FRL of an element, additional instrumentation can be included in the test to also determine the RISF or MRISF performance – providing a cost-effective approach for new protection systems.

Smaller-scale fire resistance test

There are a large number of systems that have been tested previously to determine the FRLs, but in most cases insufficient data will have been recorded to determine the RISF or MRISF performance. Under these circumstances, the use of a smaller specimen (not less than 1 m x 1 m) is permitted to obtain supplementary data to determine the RISF or MRISF of the system in a cost-effective manner. The fire-protective covering should be fitted in the same manner as that used for the original test that determined the FRL of the system.

Deemed-to-Satisfy fire-protective-grade plasterboard coverings

Specification A1.1 deems fire-protective-grade plasterboard facings, if fixed in accordance with the requirements to achieve the required FRL of the element, to also satisfy the requirements for RISF with the performance as listed in Table E2.

Table E2: Minimum fire-protective-grade plasterboard coverings.

Requirements	Application	Performance	Minimum Deemed-to-Satisfy fire-protective-grade plasterboard
General Requirements	All applications	RISF 45min	2 layers x 13 mm thick
Relaxed requirements for timber elements not less than 75 mm x 75 mm without cavities voids or cavities voids filled with non-combustible material	Inside a fire-isolated stairway or lift shaft	MRISF 20 min	1 layer x 13 mm thick
	External walls within 1 m of an allotment boundary or 2 m of a building on the same allotment	MRISF 45 min	2 layers x 13 mm thick
	All other applications	MRIFS 30 min	1 layer x 16 mm thick

E3.3 Resistance to Incipient Spread of Fire (RISF) Test Procedures

The test procedure for determining the incipient spread of fire of horizontal elements during a full-scale fire resistance test is provided in Section 4 of AS 1530.4. Specification A1.1 of the NCC requires the relevant procedures from AS 1530.4 Section 4 to be applied to other elements.

AS 1530.4 requires walls to be full size or not less than 3 m high x 3 m wide, and floor/ceiling systems to be full size or not less than 4 m long x 3 m wide. Floor systems are exposed to furnace heating conditions (refer Figure E6) from the underside and fire-resistant walls are exposed from one side. Asymmetrical walls generally require two tests to evaluate the response to exposure to fire from either side, unless the side exposed to fire can be specified.

Smaller-scale specimens (not less than 1 m x 1 m) can be used to retrospectively determine the resistance to incipient spread of fire performance of a floor or wall system that has previously achieved the required fire resistance level in a fire resistance satisfying the minimum size requirements specified in AS 1530.4.

For universal application of results, the minimum cavity depth should be fire tested.

To determine the RISF, five thermocouple with insulating pads as prescribed in AS 1530.4. shall be fixed to the inner face of the fire-protective covering system. They shall be placed at approximately the centre, and the centre of each quarter section, as shown in Figure E7.

When testing corrugated specimens, the number of thermocouples should be increased to six to provide an equal number of thermocouples at the maximum and minimum specimen thickness.

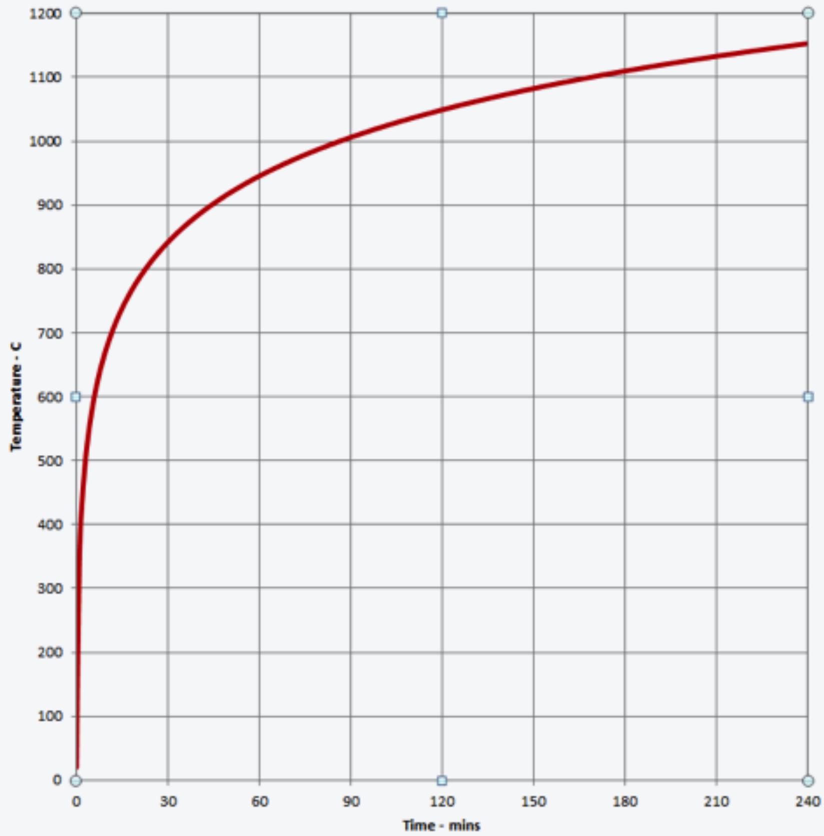


Figure E6: Standard fire resistance test heating regime.

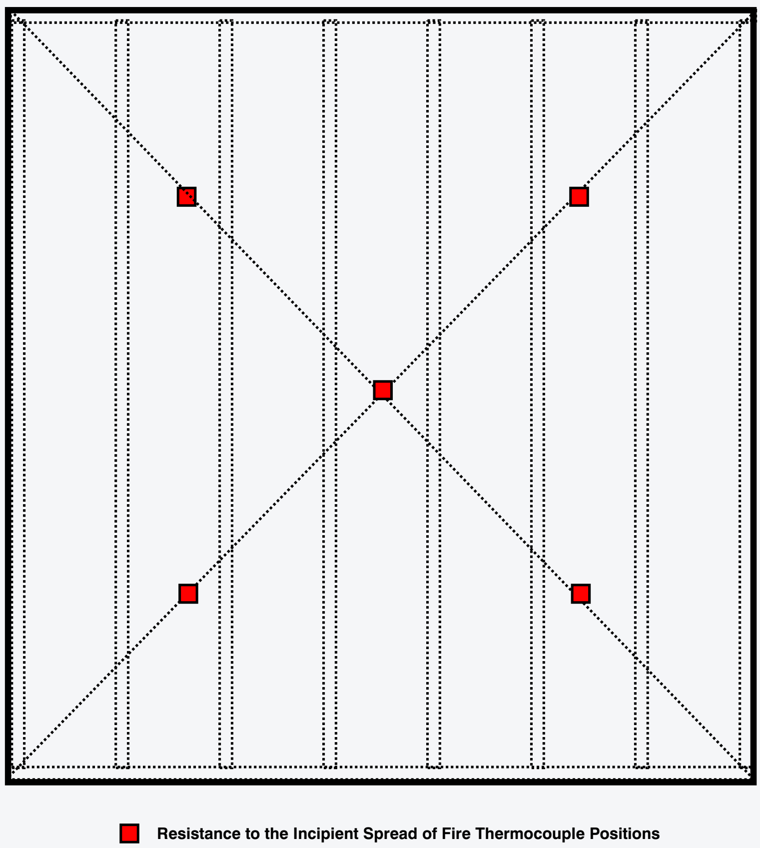


Figure E7: Elevation of a wall showing RISF thermocouple positions.

Sections through typical specimen configurations are shown in Figure E8 to illustrate the correct surfaces to apply thermocouples to determine the RISF. For fire-protected timber, the temperature has to be maintained below the prescribed temperature on the surface of the fire-protective covering facing the void and at the interface with timber elements within the wall or floor. Therefore, if a wall or floor/ceiling system is protected by a board system, for example, the temperatures are measured on the board surface within the cavity even if non-combustible insulation is applied between the timber studs or beams. However, if the non-combustible insulation forms a continuous layer between the timber elements and the board, the thermocouples should be applied to the surface of the insulation, as shown in Figure E8.

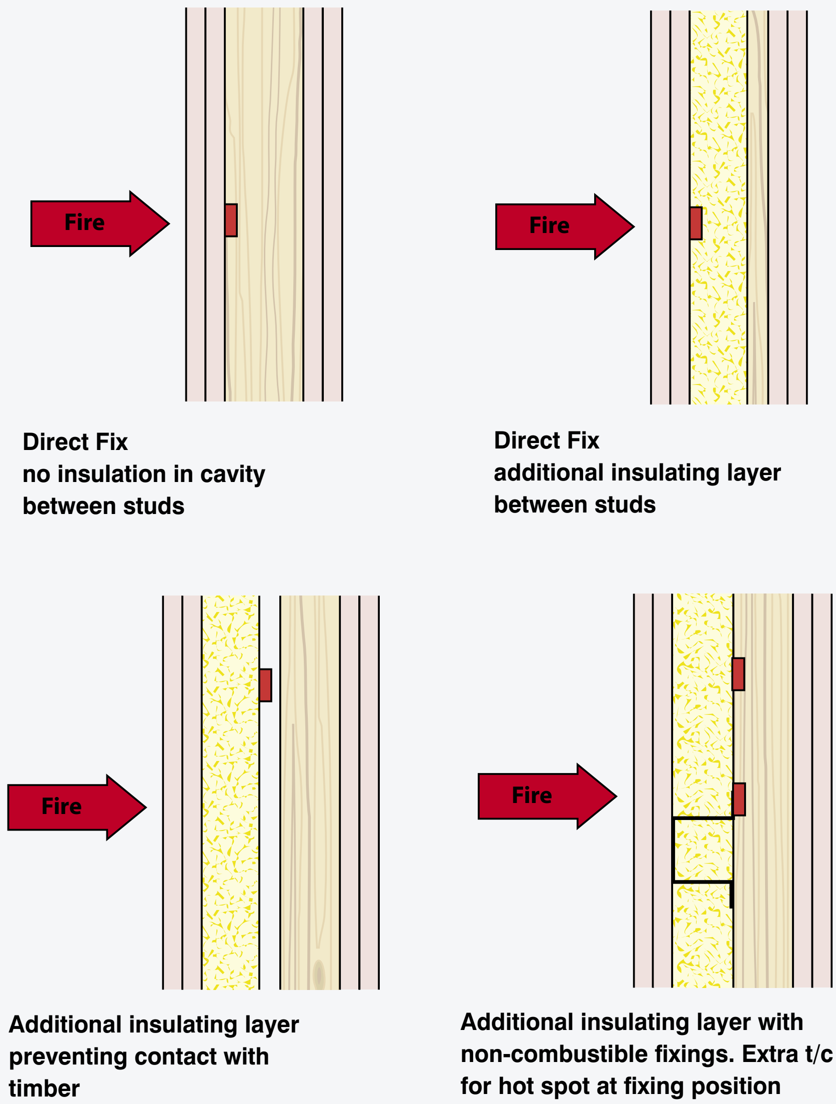


Figure E8: RISF thermocouple positions for typical timber-frame specimen configurations.

Failure in relation to incipient spread of fire is deemed to occur when the maximum temperature of the thermocouples described above exceeds 250°C.

Smaller-scale specimens (1 m x 1 m) can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations are shown in Figure E9. Additional thermocouples are shown to allow the simultaneous determination of the FRL of the service penetration system.

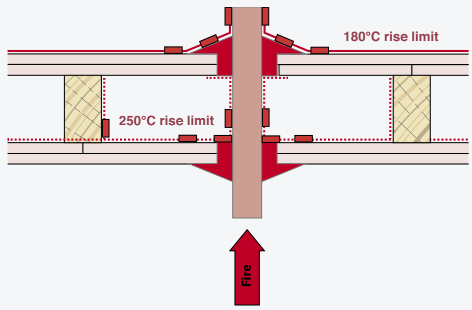
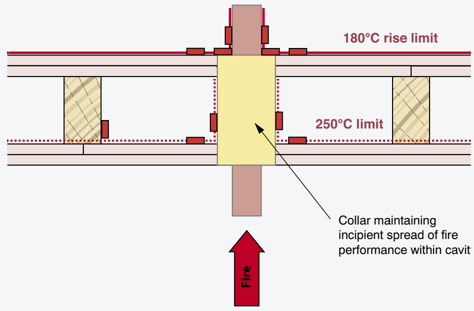
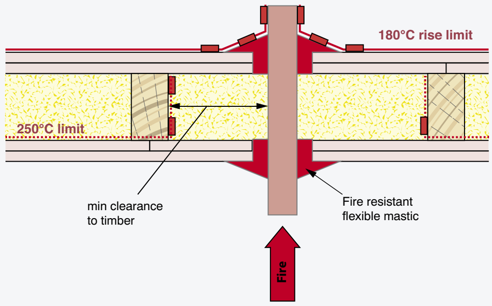
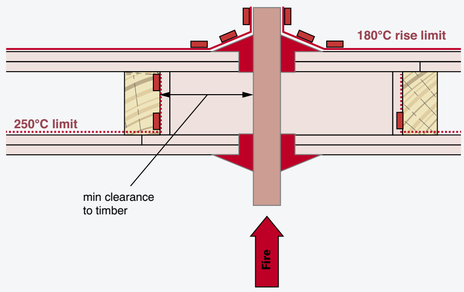
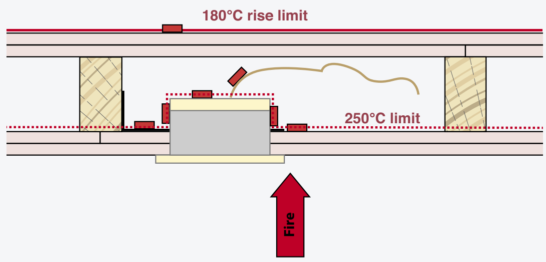
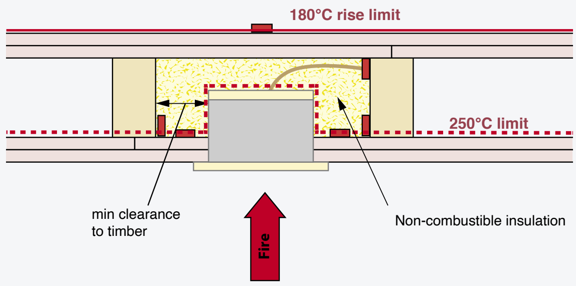
Cable / metal pipe penetration protected with fire resistant mastic.	
Plastic Pipe protected by insulating collar system.	
Cable / metal pipe penetration protected with fire resistant mastic and non-combustible cavity infill. The critical interface for RISF for the service penetration is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system.	
Cable / metal pipe penetration protected with fire resistant mastic and cavity lined with non-combustible board. The critical interface for RISF for the service penetration is the surface of the lining board where it is in contact with timber elements.	
Proprietary GPO outlet protection system. Note: Thermocouples applied to cable surface connected to the GPO, on fixing bracket and adjacent element.	
GPO outlet with non-combustible cavity infill protection. The critical interface for RISF is the surface of the insulation where it is in contact with timber elements. Note: plasterboard surface is the critical surface for determining the RISF of the wall system.	

Figure E9: Typical thermocouple positions for determining the RISF of service penetrations.

The thermocouples positions must satisfy the following requirements:

- at not less than two points located approximately 25 mm from the edge of the hole made for the passage of the service
- attached to adjacent structural members and those elements that support the penetrating service
- at points on the surface of the penetrating service or its fire stopping encasement, as follows:
  - at least 2 thermocouples located approximately 25 mm from the plane of the general surface of the covering and non-combustible insulation
  - where the seal or protection around the service is tapered or stepped, two additional thermocouples beyond the step or the end of any taper if it is expected that the temperatures will be higher at these points.
- where practicable, at two points on the seal or protection around the service
- one in the centre of the surface of the penetration nominally parallel to the plane of the fire-protective covering if it terminates within the cavity. (e.g. GPO outlets or downlights).

Failure in relation to incipient spread of fire is deemed to occur for the service penetration when the maximum temperature of the thermocouples described above exceeds 250°C.

E3.4 Modified Resistance to Incipient Spread of fire (MRISF) Test Procedures

The MRISF is applicable if all timber elements have a cross-section greater than 75 mm x 75 mm and there are no voids/cavities through which fire and smoke can spread. The MRISF, among other things, relaxes the failure temperature from 250˚C to 300˚C to reflect the reduced risk of fire spread through cavities and higher inherent fire resistance of timber with larger cross-sections. The test procedures are described in Section 3 of Specification A1.1 of the NCC and are summarised below.

Tests must be carried out in accordance with AS 1530.4 or an equivalent or more severe test on the timber element with the proposed non-combustible fire-protective coverings fixed in a representative manner.

Smaller-scale specimens (not less than 1 m x 1 m) can be used to retrospectively determine the MRISF performance of a system that has previously achieved the required fire resistance level in a fire resistance satisfying the minimum size requirements specified in AS 1530.4. If a fire protection system incorporates joints, the test specimens must incorporate representative joints.

To determine the MRISF, interface temperatures must be measured over the following features by a minimum of two thermocouples complying with Appendix C1 and Section 2 of AS 1530.4 as appropriate:

- at joint positions in the protection systems
- at least 200 mm from any joint
- at any other locations where, in the opinion of the Registered Testing Authority, the interface temperature may be higher than the above positions.

Where the fire protective covering is not in contact with the timber, the surface of the fire protective covering is deemed to be the interface.

Figure 75: Typical thermocouple positions for determining the RISF of service penetrations

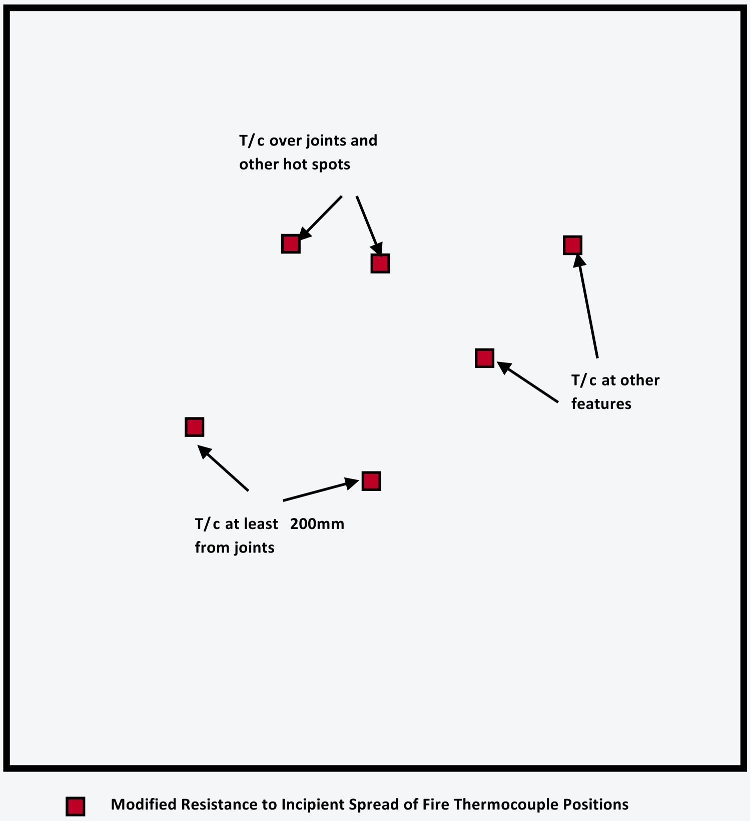


Figure E10: Elevation of a wall showing MRISF thermocouple positions.

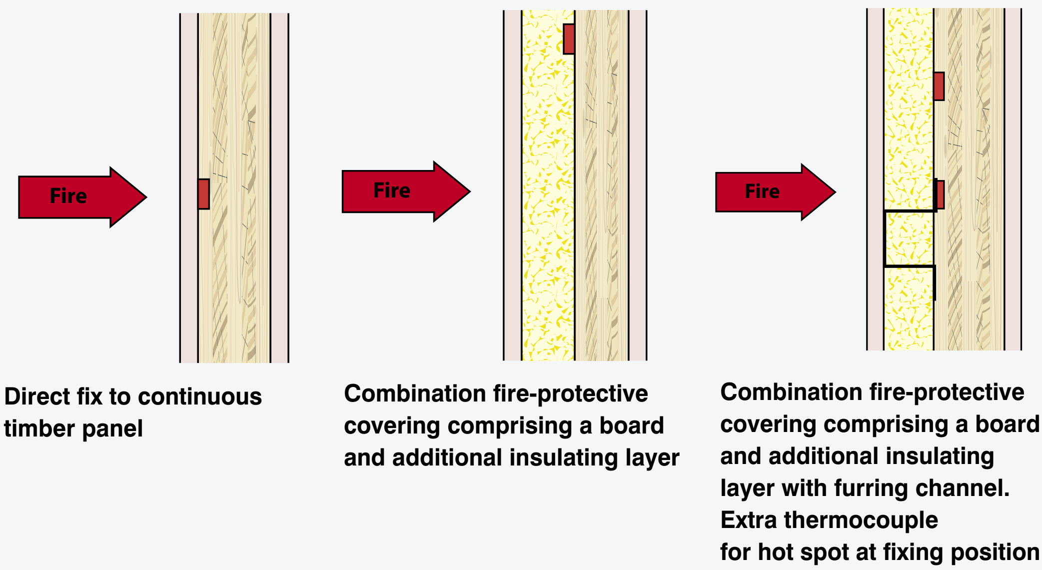


Figure E11: MRISF thermocouple positions for typical panel specimen configurations.

Failure in relation to MRISF is deemed to occur when the maximum temperature of the thermocouples described above exceeds 300°C.

Smaller-scale specimens (1 m x 1 m) can be used to determine the performance of services penetrations in fire-protected timber. Typical examples of thermocouple configurations for various types of service penetrations to determine both the MRISF and FRLs are shown in Figure E12.

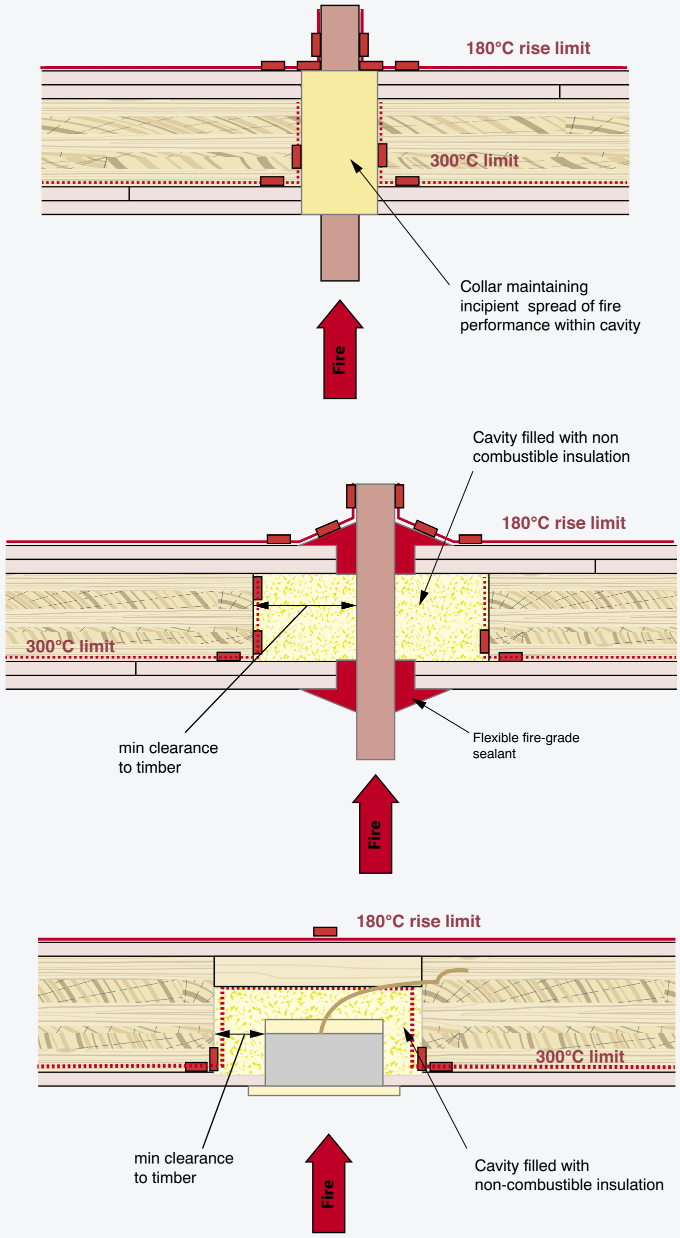


Figure E12: Typical thermocouple positions for determining the MRISF of service penetrations.

Specification C1.13 of the NCC sets out the requirements for cavity barriers in fire-protected timber construction.

Sub-clauses 2(a) to 2(d) set out the required positions of the cavity barriers that are discussed in the body of this Guide. The required performance of cavity barriers is specified in sub-clauses 2(e) to 2(h).

The following compliance options are provided for cavity barriers.

The cavity barrier system must achieve the FRLs specified in Table E3 when mounted in timber elements having the same or a lower density than the timber members in the proposed application or:

- comprise timber of minimum thickness as specified in Table E3; or
- comprise polythene-sleeved mineral wool or non-sleeved mineral wool slabs or strips placed under compression and of minimum thickness as specified in Table E3.

Another option is that, for cavity barriers around doors and windows, steel frames are also Deemed-to-Satisfy the requirements for cavity barriers, provided that wherever possible the steel frames should be tightly fitted to rigid construction and mechanically fixed. It should, however, be noted that if the windows or doors are of fire-resistant construction, the windows or door system needs to be capable of achieving the required fire resistance when mounted in the wall system, notwithstanding the requirements for cavity barriers.

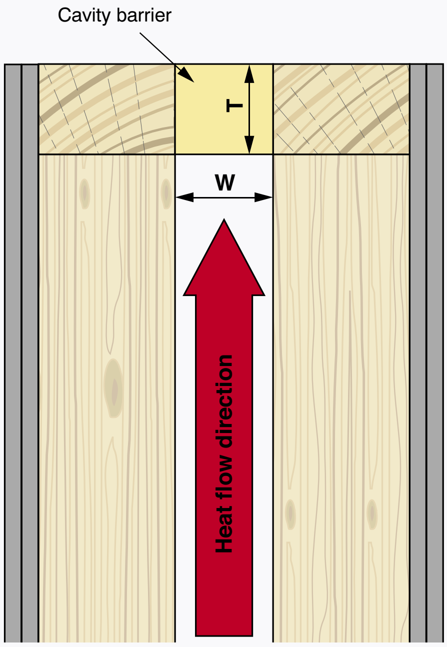
Table E3: Cavity barrier requirements for fire-protected timber.

Cavity Barrier Compliance Options	**Maximum FRL required for element cavity barrier is fitted to – min**		
	-/60/60	**-/90/90**	**-/120/120**
Cavity Barrier Required FRL – min	-45/45	-/45/45	-/60/60
Timber required minimum thickness	45mm	45mm	55mm
Mineral wool required minimum thickness	45mm	45mm	60mm

The minimum thicknesses of protection are required to be measured in the direction of heat flow. The role of a cavity barrier is normally to prevent a fire spreading from the cavity on one side of the cavity barrier to the other. The head of a double stud partition (Detail A of Figure E13) is a typical example of this, where the direction of heat flow for the cavity barrier would be from the underside to the upper face of the barrier and the thickness dimension is identified as “T” and the width of the seal would be ”W” in the Figure.

The other role for cavity barriers is to reduce the risk of fire spread to cavities occurring around openings for doors and windows within a fire-resistant wall. This configuration is shown as Detail B in Figure E13. For this scenario the heat flow direction is from the occupied area of the building through the framing to the cavity. The thickness dimension is identified as ‘T’ in Figure E13.

Detail A
Cavity Barrier at the Head of a double stud partition



Detail B
Cavity Barrier around door opening

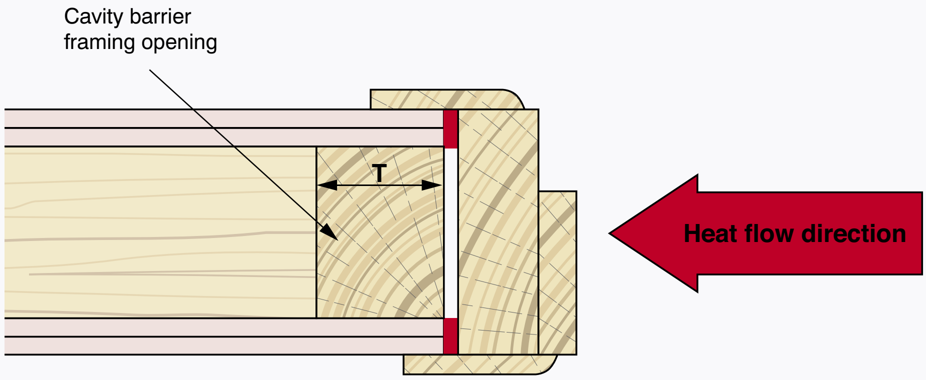


Figure E13: Heat flow direction for cavity barriers.

It is expected that proprietary cavity barrier systems may provide more practical options than the Deemed-to-Satisfy solutions for some applications. To encourage the development and use of these systems, a compliance path has been provided through the specification of FRLs. For smaller-sized cavity barriers, the performance should be determined by testing the cavity barrier as a control joint system in accordance with Section 10 of AS 1530.4:2014 using timber members as the separating element. Specification C1.13 permits the results from such a test to be used for applications where the fire-protected timber is constructed from timber having a nominal density at least equal to the tested timber.

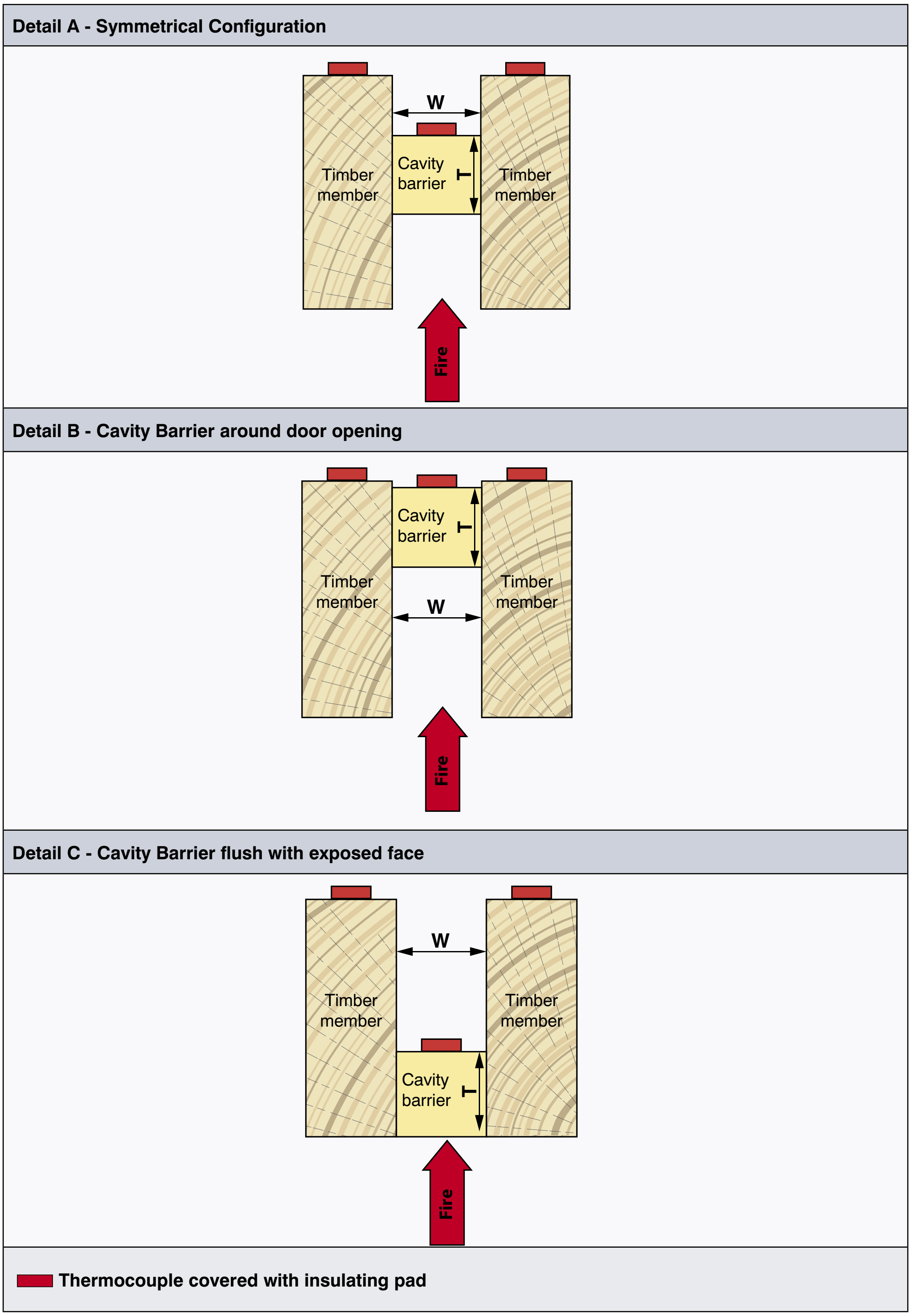


Figure E14: Typical cavity barrier test configurations.

Typical test configurations are shown in Figure E14. The selection of the test configuration(s) depends on how the cavity barrier will be mounted. If the cavity barrier system is symmetrical (e.g. the cavity barrier is to be fitted at the mid-depth of a timber member) then Detail A is appropriate. If the cavity barrier system is not symmetrical both details, B and C should be tested unless the most onerous configuration can be determined by the test laboratory or the cavity barrier use is restricted to one configuration. A report from a registered test laboratory should state the field of application for the cavity barrier based on the test results.

Cavity barriers can be of combustible construction and therefore a timber-framed partition with exposed timber members could be used, subject to the wall achieving the required FRL.

In some instances, it may be more practicable to continue the fire-resistant walls up to roof level in lieu of providing a fire-protected timber roof system with cavity barriers.

F

Appendix F: Analysis of Fire Data

F1 Fire Loss Estimates

Currently, only limited national fire statistics are published in Australia, and therefore reliance has had to be placed on older data for comparisons. Dowling and Ramsay[45] analysed Australian fire statistics for the period 1989 to 1993. The same data set was also analysed by Thomas and Verghese,[46] who calculated that there were 6.8 fatalities per 1,000 apartment fires.

Since 1993, NSW fire services have published detailed annual statistics until 2006/07. Table F1 and Table F2 have been derived from this data for the period from 2003/4 to 2006/7. [47]

Table F1: Comparison of NSW house and apartment fire fatalities and injuries.

Year	1 and 2 Family Houses					Apartments				
	Fires	Fatalities	Injuries	Fatalities / 1000 fires	Injuries / 1000 fires	Fires	Fatalities	Injuries	Fatalities / 1000 fires	Injuries / 1000 fires
2003/4	2,977	15	430	5.0	144.4	1,285	3	160	2.3	124.5
2004/5	2,879	35	431	12.2	149.7	1,185	11	142	9.3	119.8
2005/6	3,071	13	392	4.2	127.6	1,262	9	181	7.1	143.4
2006/7	2,914	10	448	3.4	153.7	1,242	6	137	4.8	110.3
Total	**11,841**	**73**	**1,701**	**6.2**	**143.7**	**4,974**	**29**	**620**	**5.8**	**124.6**

The results from Table F1 have been consolidated in Table F2 and the average loss per fire added, including an adjustment to 2014 present values.

Table F2: NSW house and apartment fire losses.

Year	1 and 2 Family Houses and Apartments					Av. loss / fire A$	Ave loss / fire A$ at 2014 value
	Fires	Fatalities	Injuries	Fatalities / 1000 fires	Injuries / 1000 fires		
2003/4	4,262	18	590	4.2	138.4	20,859	27,407
2004/5	4,064	46	573	11.3	141.0	28,017	35,920
2005/6	4,333	22	573	5.1	132.2	28,228	34,800
2006/7	4,156	16	585	3.8	140.8	26,784	32,342
Total	**16,815**	**102**	**2321**	**6.1**	**138.0**		**32,617**

The fatalities from house and apartment fires are similar (about 6/1,000 fires) and are comparable to the 6.8 fatalities/1,000 apartment fires estimated by Thomas and Vergese in their analysis of Australian Statistics for the period between 1988 and 1992.

It has not been possible to isolate sprinkler-protected apartment fires in the Australian statistics presented above, but the proportion of sprinkler-protected houses and apartments is currently very low, and therefore the above statistics are considered representative of buildings that are not protected by automatic fire sprinklers.

The average fatality rates between 6 and 6.8 fatalities/1,000 fires from the Australian data are comparable to the 7.3 fatalities /1,000 fires calculated by Hall based on the American data. It is therefore reasonable to expect a similar reduction in fatalities to that calculated from the US data, if an automatic fire sprinkler system is included in an apartment (i.e. a reduction of about 83%).

The average residential fire loss due to fires in NSW was estimated to be A$32,617 per fire at 2014 values, applying adjustments for changes to CPI. These losses can be crudely compared to the US losses by applying adjustments to CPI to estimate 2008 values (the mid-point of the data used in the Hall analysis) yielding a loss per fire of A$28,213 at 2008 values. The exchange rate at mid-2008 was approximately US$0.8 to A$1, which would value the NSW losses at US$22,570 per fire, which is comparable to the US losses of approximately US$20,000/fire. It is therefore reasonable to expect a reduction in direct property damage/home fire of the order of 69%.

F2 General Area of Fire Origin

A breakdown of the areas of fire origin for structural incidents in multi-level apartment buildings attended by the MFB between 1996–2007 was provided in a Post Incident Analysis Report after an apartment fire.[48] Figure F1 is based on this data and includes extracted areas of fire origin that were clearly within individual apartments. Some of those indicated as falling under storage areas, service facilities, means of egress and other areas may have occurred within an apartment. The data also shows a significant occurrence (7%) of fires occurring in the means of egress.

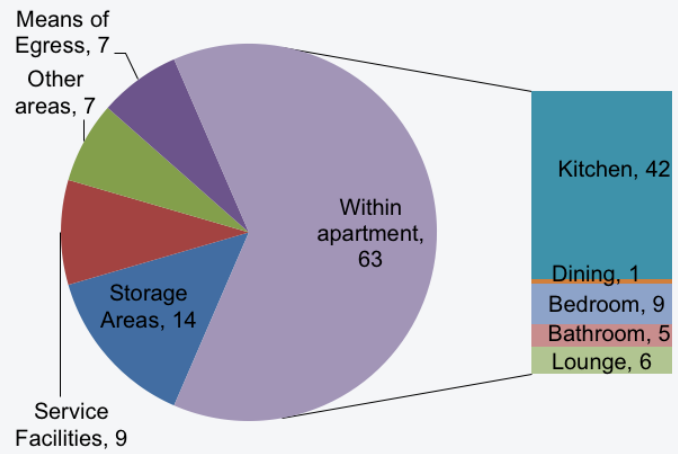


Figure F1: Area of fire origin for multi-level apartment fires attended by the MFB from 1997 to 2007.

A similar analysis has been undertaken for NSW based on published annual statistics for the period from 2003/4 to 2006/7. [47] The results are shown in Figure F2.

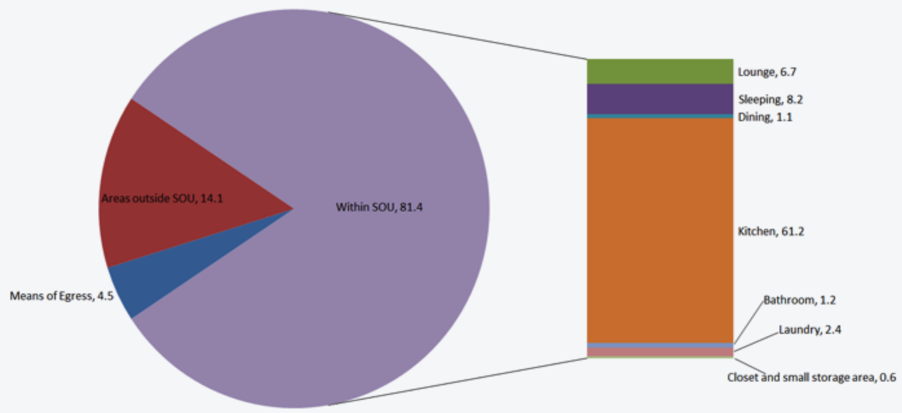


Figure F2: Area of fire origin for multi-level apartment fires attended by NSW Fire Services from 2003 to 2007.

The fires occuring in the means of egress were broken down further based on the frequency of fire starts from the NSW fire statistics and are summarised in Table F3.

Table F3: Fire starts in escape paths and shafts – NSW 2002-3 to 2006-7.

Location	Fire Starts – %
Lobby, entrance way	1.2
Hallway, corridor, mall	1.9
Exterior stairway	0.3
Interior stairway	0.8
Fire-isolated escape route	0.1
Lift, dumbwaiter	0.22
Utility shaft	0.16
Chute	0.02

F3 Fire Starts within Structural Areas

A number of surveys were undertaken, based on council records of the form of construction used for new single dwellings in the Melbourne area for the period 1979–2003. They indicated that, of the form of construction known, more than 90% of single dwellings were of timber-framed construction. It is estimated that a similarly high percentage of timber-framed construction has been adopted for single dwellings in NSW.

Fire statistics including the area of fire origin from single dwellings in NSW and Victoria can therefore provide a reasonable indication of the potential for fire starts to occur within areas of timber-framed construction; although it should be noted that there are no controls applied to internal linings and very few controls applied to external linings of single dwellings and cavity insulation, so the rates of fire starts in these areas are likely to be higher than is expected with the proposed use of fire-protected timber.

Figure F3 provides a breakdown of the area of fire origin in 1 and 2 Family Dwellings in NSW based on published annual statistics for the period from 2003/4 to 2006/7[47].

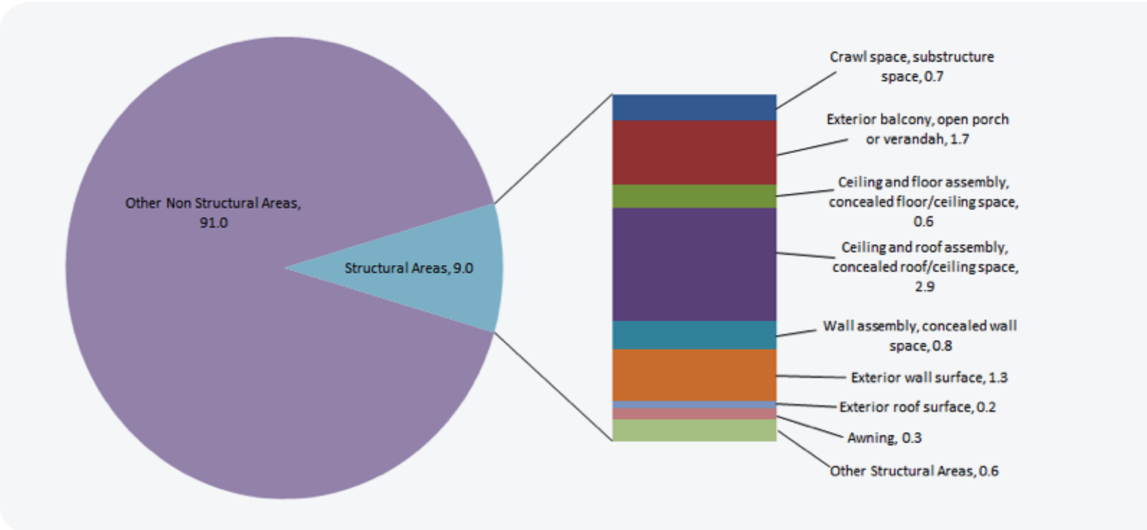


Figure F3: Detailed breakdown of structural areas of fire origin based on NSW fire statistics from 2002/3 to 2006/7.

A large number of the structural areas listed in Figure F3 relate to external areas and surfaces that are not applicable to the fire-protected timber. Since the primary focus of this analysis is concrete/masonry and non-combustible walls, the most relevant statistic is that about 0.8% of fire starts occur within a wall assembly.

Ignitions in timber-framed floors and roof constructions will also be considered in the analysis. The statistics indicate approximately 2.9% of fires occur in the concealed space between a roof and ceiling and 0.6% of fires occur in the concealed space between a floor and ceiling. The large difference in fire starts between floors and roof construction may in part be due to a fewer number of two-storey (or more) single dwellings.

G

Appendix G: Multi-scenario Quantitative Risk Assessment Supplementary Data

G1 Contribution of Timber Elements to Fire Load

G1.1 Overview

The NCC 2016 requirements for mid-rise timber buildings include additional supplementary controls to reduce the probability of a contribution from the timber. They apply the incipient spread of fire criteria from AS 1530.4 of 250°C on the inner surface of protective coverings for a period of 45 minutes for timber-framed construction, and an interface temperature of 300°C for massive timber panels with no cavities for 30 minutes. These criteria are in addition to the FRL levels of 90/90/90 for loadbearing elements and -/60/60 for non-loadbearing elements, which are also required to be met.

The potential for protected timber construction to contribute to the effective fire load and hence increase the severity was raised when an extension of the Class 2 Concession to include a Class 3 Concession for low-rise buildings was sought and addressed by means of a full-scale fire experiment, which demonstrated that there was no increase in the fire severity of an enclosure under typical natural fire conditions.[5(pp18-35)]

The Monte Carlo analysis for mid-rise buildings includes consideration of the potential consequences of incorrect installations with gross defects and other extreme circumstances, and therefore the probability and consequences of a significant increase in the fire severity from protected timber-frame members under these adverse conditions was considered.

Currently, the NCC does not directly control the fire load within individual apartments of a Class 2 building, and non-combustible elements of construction can be clad with decorative combustible materials (including timber) of any thickness. However, a conservative approach to address issues raised by stakeholders was adopted and a more detailed analysis is provided below of the potential contribution to the fire load.

With modern furnishings containing larger proportions of plastics and the increased amount of lightweight furniture, typical residential fires tend to be relatively fast growing and produce large volumes of volatiles post-flashover, leading to conditions that are heavily ventilation-controlled with long flame extensions from windows as the unburnt volatiles mix with air outside the building. During this stage of the fire, as the tests performed for the low-rise concession demonstrated[5(pp18-35)], there will be no contribution from fire-protected timber. Also, while the fire remains under ventilation-controlled conditions, any additional volatiles would not be consumed within the enclosure of fire origin and enclosure temperatures may tend to be lower, due to oxygen constraints.

Therefore, if volatiles are released, any contribution to the fire load would tend to extend the duration of a fire after it has progressed towards a fuel-controlled burning regime. During the low-rise concession tests, the fire progressed to a fuel-controlled regime and there was no evidence of an increase in severity of the fire from the protected timber frame, despite some minor charring and temperatures of the timber peaking above 350°C.

While a review of literature (e.g. Babrauskas)[49] shows that timber starts to degrade below 250°C, the rate of degradation is relatively slow and hence production of volatiles will be low. Similarly, timber can ignite at temperatures lower than 300°C, but the probability of ignition is strongly time dependent and is also dependent upon oxygen content, moisture content, the size of the specimen and other factors, and ignition temperatures can be above 400°C.

Recently published results based on cone calorimeter tests on timber samples protected by fire-protective coverings subjecting specimens to radiant heat fluxes of 50kW/m^2 and 75kW/m^2 (yielding similar timber heating rates to fully developed fires and the standard fire resistance test heating regime) have been published (Su and Lougheed[39]). This research found that the plywood substrates ignited at an average interface temperature between 320 and 350°C, indicating that both the adopted limits for general timber construction (250°C) and massive timber construction (300°C) in the NCC 2016 and used in the supporting analysis were conservative.

G1.2 Timber-framed Construction

Figure G1 shows the fire-exposed face of a test specimen about three minutes after the test in free air, prior to application of water. The specimen had been subjected to a 90-minute fire resistance test. The figure shows the openings left after a 100 mm unprotected PVC pipe penetration in the lower part of the specimen and an unprotected electrical light switch had been consumed during the test, allowing the fire to penetrate the cavity, and representing an element with severe installation faults. At this stage, the specimen was in an open laboratory area with no ventilation restrictions and – despite the fire burning within the cavity – the plasterboard facings remained intact, with flames only projecting from small openings and from the interface of the partition edge and furnace seal.

Figure G1: Exposed face of timber-framed plasterboard specimen about three mins after completion of a 90-minute fire resistance test in a free air environment.

On the basis of the above discussion and with the controls proposed above, it is likely that there will be no appreciable contribution from protected timber-frame members prior to substantial fall off of the facings – substantial fall off is likely to occur at the equivalent fire-resistant period of 12 minutes before the nominal fire resistance is achieved by the protected timber member, if there is no intervention based on the inherent fire resistance of lightweight timber-framed construction. The number of scenarios where this occurs will be very low.

A conservative approach has been adopted by assuming a significant contribution to the fire severity will occur after exposure to the equivalent of 45 minutes of the standard fire test based on the definition of fire-protected timber.

An initial Monte Carlo run was undertaken to determine the proportion of scenarios where the fire is suppressed or burns out prior to failure of a timber-framed element.

Typical results for a mean fire load of 500MJ/m2 on the 7th (top) floor are shown in Figure G2. In this example, burnout would occur before a duration equivalent to 45 minutes fire resistance in 45% of scenarios. The fire would be likely to be suppressed before achieving the equivalent of 45 minutes fire resistance in 54% of scenarios, leaving about 1% where the timber substrate is deemed to ignite.

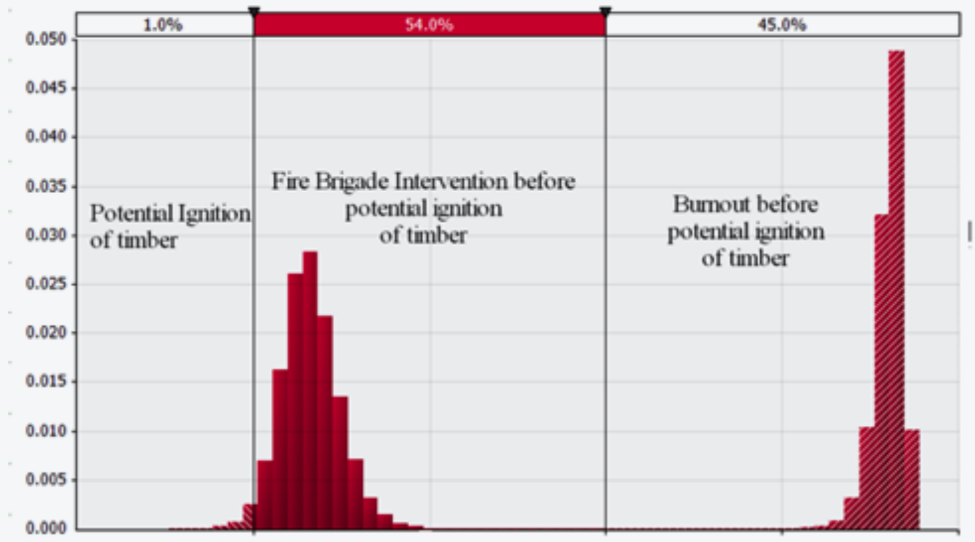


Figure G2: Typical performance of fire-protected timber coverings for timber-frame construction – mean fire load of 500 MJ/m² on the 7th floor.

The fire load was therefore increased for 1% of scenarios.

To establish an upper bound for the contribution of the potential energy that could be released from typical timber-framed elements, it was assumed all timber undergoes efficient combustion within the enclosure.

A typical wall was assumed to be 6 m x 2.4 m high and comprise 90 x 45 mm studs at 450 mm centres, with top and bottom plates and central noggings of the same dimension and of party wall design (two frames).

No of studs	(6/0.45) + 1	say 16
Length of timber	((16 x 2.4) + (3 x 6)) x 2	113 m
Mass of timber =	113 x 0.09 x 0.045 x 450	206 kg
Increased fire load / m² based on 6x4m room	206 x 18/24	155MJ/m²

A typical floor was assumed to comprise 300x50mm joists at 450 mm nominal centres spanning 4 m.

No of joists		say 16
Length of joists	16 x 4	64 m
Ring beam		40 m
Floorboards ignored assumed covered by insulation		
Mass of timber =	104 x 0.3 x 0.05 x 450	702 kg
Increased fire load / m² based on 6 x 4 m room	702 x 18/24	527MJ/m²

These increases will be expected to overestimate the contributions substantially, based on the discussion in the previous section. To simplify modelling, the fire load was increased by 500MJ/m² for the proportion of cases that the timber temperatures were estimated to exceed 250°C, which would be expected to yield conservative results.

G1.3 Massive Timber Construction

McGregor[37] undertook a series of fire tests to investigate the contribution of CLT panels to room fires. Two tests were performed using propane gas burners on protected and unprotected CLT but, due to variations in the test procedures between the tests and pre-heating as a result of restarting the first fire test, these are not discussed further. The remaining three tests were performed with representative fire loads for bedrooms. Temperature data was lost from one of these tests but the configuration was retested. Therefore, this analysis of the results has focused on Tests 4 and 5. These were performed in an enclosure constructed of CLT panels with internal dimensions 3.5 m x 4.5 x 2.5 m high with an opening 2 m high x 1.07 m wide. Furnishings/contents representing bedroom fire loads of 553MJ/m^2 and 529MJ/m^2 for Tests 4 and 5, respectively, were provided.

The CLT panels were exposed in Test 5 and protected with two layers of 13 mm fire-grade plasterboard in Test 4. Figure G3 shows the approximate average enclosure temperatures from Tests 4 and 5, with a parametric curve derived in accordance with the procedures described in this Appendix with an assumed load of 529 MJ/m^2 and the calculated temperatures of a target steel element to compare the severity of exposures.

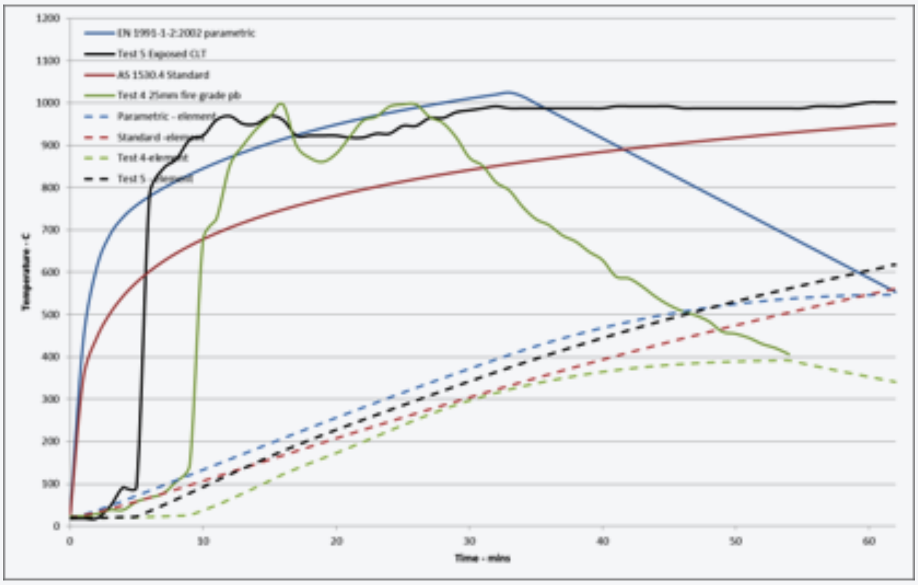


Figure G3: Average enclosure temperatures for McGregor Tests 4 and 5 compared to a parametric fire curve with a fire load of 529MJ/m^2.

In Test 4, there was no contribution from the CLT, with the plasterboard providing full protection. From examination of the average enclosure temperatures, it can be observed that until the fire load excluding the CLT had been substantially consumed the enclosure temperatures were similar, if the pre-flashover phase is excluded. For the protected enclosure, the fire burnt out and decayed; whereas, in Test 5 the CLT continued to burn, extending the duration of the fully developed fire beyond 62 minutes at which stage the test was terminated. Using the target element temperatures, the equivalent fire resistance periods were estimated from the enclosure temperatures and compared to the parametric curve. The results of this comparison are shown in Table G1. The fire severity for Test 4 was estimated to be equivalent to a 40-minute fire-resistant test.

Table G1: Equivalent fire resistance exposure periods for Tests 4 and 5 compared to parametric curves.

Scenario	Equivalent fire resistance	Comments
Test 4 Protected CLT	40 min	Burnout
Test 5 Unprotected CLT	71 min	Suppressed after 62 minutes
Parametric Curve 529MJ/m²	59 min	Full fire load from Test 4
Parametric Curve 365MJ/m²	43 min	Consumed fire load (69% of actual fire load) based on oxygen consumption calorimetry
Parametric Curve 977MJ/m²	98 min	Based on McGregor-measured energy released for Test 5
Parametric Curve 1077MJ/m²	115 min	Based on estimated timber consumed in Test 5
Parametric Curve 954MJ/m²	96 min	Test 5 estimated exposure based on proposed methods
Parametric Curve 1616MJ/m²	165 min	Simulating full burnout of CLT using proposed calculation method
Test 5 Extrapolated at 1,000°C for 105 min	119 min	Simulating full burnout of CLT at constant temperature

Oxygen consumption calorimetry was undertaken during the tests, from which it was estimated that the heat released during Test 4 was equivalent to 365 MJ/m2 (0.69 of the total fire load). Figure G4 shows the revised parametric curve based on a fire load of 365 MJ/m2 compared to Test 4 and 5 average temperatures, with the time scale offset to remove the pre-flashover growth phase. For Test 4 there is a reasonable correlation with the parametric curve, providing an equivalent fire resistance exposure of 43 minutes compared to 40 minutes for Test 4. This indicates that applying the parametric curves with high fire loads will tend to over-predict the fire severity, especially if no allowance for combustion efficiency is made.

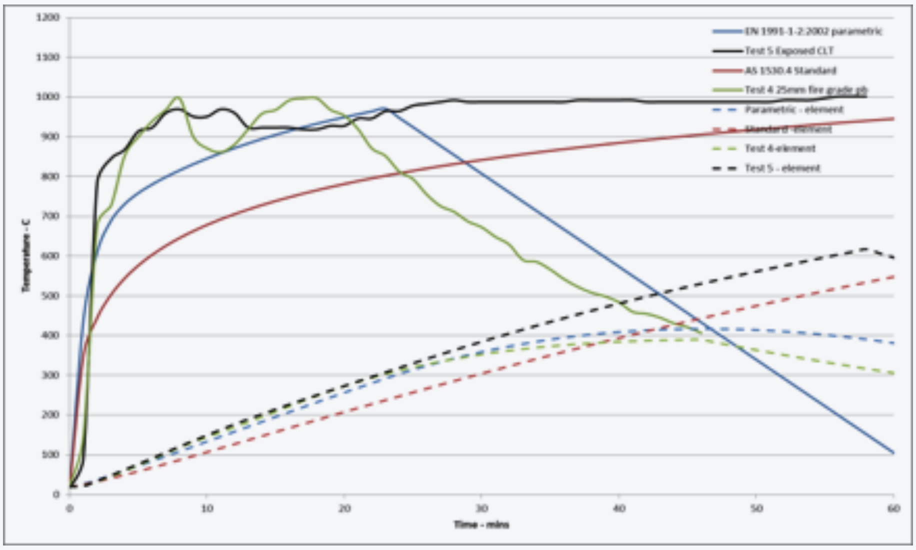


Figure G4: Average enclosure temperatures for McGregor Tests 4 and 5 compared to a parametric fire curve with fire load reduced to 365 MJ/m² and axis offset to remove pre-flashover growth.

Oxygen consumption calorimetry was also used to estimate the total heat released in Test 5, allowing the additional contribution from the CLT to be estimated. McGregor found the additional contribution to be 612 MJ/m^2 during Test 5. From Figure G4, it can be observed that the average temperatures were similar between Tests 4 and 5 for the first 20 minutes of a fully developed fire, indicating that the additional combustion must have occurred outside the enclosure. This is consistent with the fire being ventilation-controlled but, as the moveable fire load (furnishings/contents) was consumed, an increasing proportion of the volatiles produced from the CLT will burn within the enclosure. The enclosure temperature was between 900 and 1,000°C from about 5 minutes (ignoring pre-flashover stage) for a period of just less than 60 minutes, at which point Test 5 was stopped. Char depths were measured after the tests, with most of the CLT panels exhibiting char depths between 50 and 70 mm. An average char depth at the time that Test 5 was stopped of approximately 60 mm has been assumed for the following indicative calculation of the total energy available if the consumed timber undergoes complete combustion.

Internal surface of CLT = (3.5+4.5) x 2.5 x 2 + (3.5 x 4.5) – (2 x 1.07) = 53.61 m^2

Volume of CLT consumed at end of Test 5 = 53.61 x 0.06 = 3.22 m^3

Mass of timber = 3.22 x 480 = 1546 kg

Equivalent to 1546 x 18 = 27,828MJ (assuming heat of combustion 18MJ/kg)

or (27828/3.5 x 4.5) = 1766MJ/m^2

This is much higher than the additional heat release rate due to combustion of the CLT of 612MJ/m^2 calculated by McGregor. McGregor's heat release estimates were based on oxygen consumption calorimetry; collecting the gases released from the enclosure and therefore the value also includes combustion taking place outside the enclosure. Therefore, the difference in heat release must be accounted for by inefficient combustion including loss of unburnt volatiles or volatiles that have undergone partial combustion and unburnt solid residues within the enclosure, among other things. Hakkarainen[36] also investigated explanations for temperature reductions in enclosures when CLT was exposed.

The heating regimes in both tests can be idealised to steady state conditions with the enclosure at about 1,000°C followed by decay. For Test 4 (protected CLT) the steady state conditions were maintained for about 20 minutes before the fire decayed as the moveable fire load was consumed. For Test 5 (exposed CLT) the steady state conditions were maintained for 60 minutes, at which stage the test was stopped. If it is assumed that the production rate of volatiles for the CLT is constant while the enclosure is at a constant temperature, then the equivalent of 1766MJ/m^2 of fuel would be consumed over a 60-minute period at a rate of:

1766/60 = 29.43MJ/m^2/min

During the first 20 minutes of steady state burning, the moveable fire load provides sufficient energy to heat the enclosure with some volatiles burning outside the enclosure. Due to the large volumes of volatiles burning, combustion would be inefficient and the volatiles from the CLT may not undergo combustion inside or outside the enclosure but may be released as smoke (unburnt gases). This assumption is consistent with heat flux measurements taken by Hakkarainen outside test enclosures, which indicated similar peak values irrespective of whether or not the CLT was protected.

Therefore, the total heat released from the CLT assuming efficient combustion for a 40-minute period would be:

29.43 x 40 = 1,177 MJ/m^2

It is reasonable to assume a similar burning efficiency to that derived for the moveable fire load (69%) which yields an estimate of the heat released from the CLT during test 5 of:

1,177 x 0.69 = 812 MJ/m^2

This crude estimate provides an overestimate of approximately 32% compared to the measured value of 612 MJ/m^2, but has ignored the heat contribution from the fire load during the decay stage, which would further reduce this variance.

Parametric curves were generated with fire loads of 1,177 MJ/m^2 (365 +812) and 977 (365+612) and compared with the 60-minute tests. Both parametric curves overestimate the maximum enclosure temperature but the duration of the fire for the fire load of 1,177 MJ/m^2 significantly exceeds 60 minutes. From Table G1, it can be seen that the equivalent fire resistance exposure for Test 5 was 71 minutes, with the parametric curves for fire loads of 977 and 1,177 MJ/m^2 predicting equivalent fire resistance periods of 98 and 115 minutes, respectively.

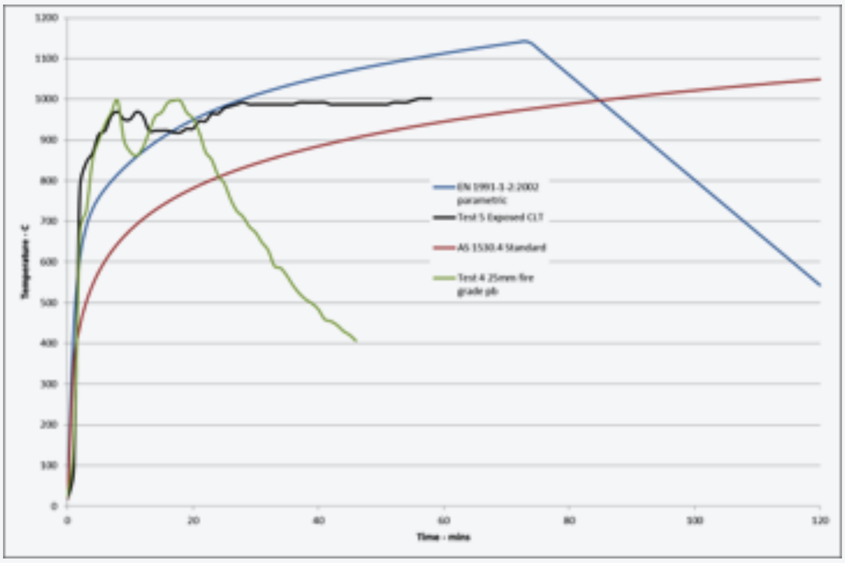


Figure G5: McGregor tests – Parametric curve with assumed fire load of 1,177 MJ/m².

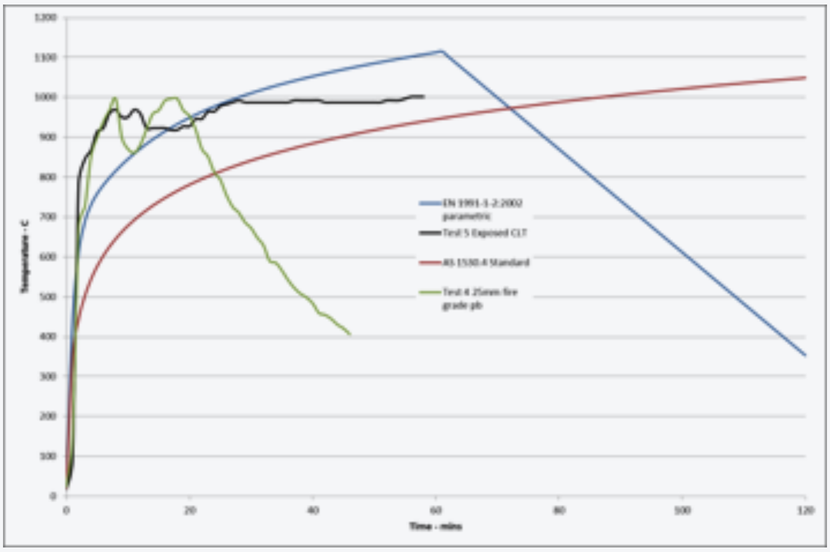


Figure G6: McGregor tests – Parametric curve with assumed fire load of 977 MJ/m².

Since the lower value still provides an overestimate of 28 minutes, it is considered appropriate to apply a burning efficiency of 50% to the CLT fire load, which would yield a contribution of 589 MJ/m² when simulating Test 5.

The fire was suppressed at the end of Test 5 before all the CLT had been consumed.

Figure G7 shows the predicted temperatures compared to actual temperature for Test 5 with a fire load of 589 MJ/m² from the CLT and 365 MJ/m² from the moveable fire load assumed (total 954 MJ/m²). The parametric curve predicts equivalent exposure for Test 5 with a fire load of 954 MJ/m² of 96 mins compared to the estimate based on the average enclosure temperature of 71 mins.

The equivalent fire resistance exposure for a parametric curve with an assumed fire load of 1,616 MJ/m² was estimated to be 165 mins. The equivalent fire resistance exposure based on Test 5 but with the enclosure temperatures extrapolated to 105 mins with assumed enclosure temperature of 100°C was estimated to be 119 mins. While this demonstrates the conservatism of the parametric curves, the use was considered reasonable having regard for the uncertainties of the inputs.

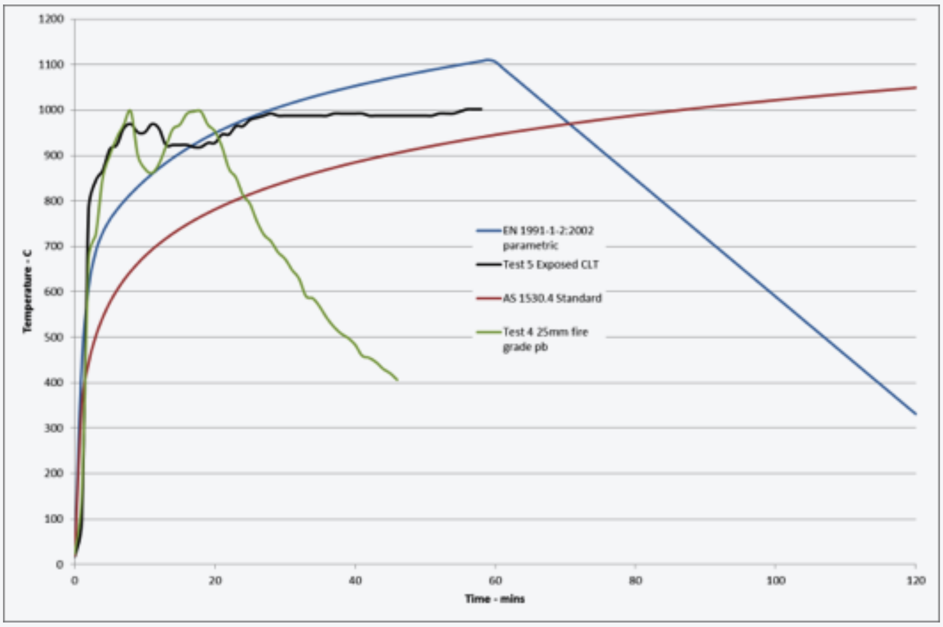


Figure G7: McGregor tests – Parametric curve with assumed fire load of 954 MJ/m².

Massive timber elements are required to be protected to reduce the probability of the element contributing to the fire severity of an enclosure fire. The minimum modified resistance to incipient spread of fire (MRISF) for most applications will be the equivalent of 30 minutes fire resistance in accordance with the NCC 2016 requirements.

The protected loadbearing elements are generally required by the NCC 2016 to achieve a fire resistance level of 90 minutes for loadbearing elements and 60 minutes for non- loadbearing elements in Class 2 band 3 buildings.

A typical CLT panel for these applications was assumed to be 150 mm thick.

Based on the above discussion, the following approach was adopted to model the performance of massive timber elements:

i. The coverings required for fire-protected timber will prevent the timber contributing to the fire severity for the equivalent fire resistance period of 30 minutes within an SOU based on the NCC 2016 requirements.

ii. Monte Carlo fire scenarios were run using parametric curves and if full burnout of the fire, automatic fire sprinkler suppression or fire brigade intervention do not occur before the elements are exposed to the equivalent fire resistance periods listed in (i) it will be assumed that the massive timber member will make a contribution to the fire load.

iii. It is assumed that no contribution is made until the interface temperature exceeds 300°C.

iv. A preliminary Monte Carlo run was undertaken to establish the proportion of fire scenarios that the massive timber may become involved in.

v. The additional fire load was based on a typical 4.5 m x 3.5 m x 2.5 m enclosure using the methods derived above as follows:

Internal surface of CLT = (3.5 + 4.5) x 2.5 x 2 + (3.5 x 4.5) – (2 x 1.07) = 53.61 m^2

Volume of CLT consumed (150 mm element) = 53.61 x 0.15 = 8.04 m^3

Mass of timber = 8.04 x 480 = 3,859 kg

Equivalent to 3,859 x 18 x 0.5 = 3,4733 MJ (assuming heat of combustion 18 MJ/kg and 50% combustion efficiency)

or 27828/3.5 x 4.5) = 2,205 MJ/m^2.

In addition, the moveable fire load will provide the fire load to overcome the coverings to the plasterboard (say 295 MJ/m²). The remainder of the moveable fire load will be assumed to be lost as excess volatiles, since the fire is ventilation-controlled.

Therefore, if the CLT panels become involved in a fire the total potential fire load was assumed to be about 2,500 MJ/m².

To demonstrate the approach, an initial Monte Carlo run was undertaken to determine the proportion of scenarios where the fire is suppressed or burns out prior to failure of a fire-protected massive timber element. The results for a mean fire load of 500MJ/m² on the 7th (top) floor are shown in Figure G8. In this example, burnout would occur before a duration equivalent to 30 minutes fire resistance in 2.6% of scenarios; the fire would be likely to be suppressed by the fire brigade before achieving the equivalent of 30 minutes fire resistance in 63.4% of scenarios; leaving about 34% where the timber substrate is deemed to ignite. The fire load will therefore be increased for this proportion of scenarios to a fixed value of 2,500 MJ/m².

Similarly, the proportion of fires where timber is deemed to ignite and contributes can be calculated for scenarios on different floors and with different fire loads.

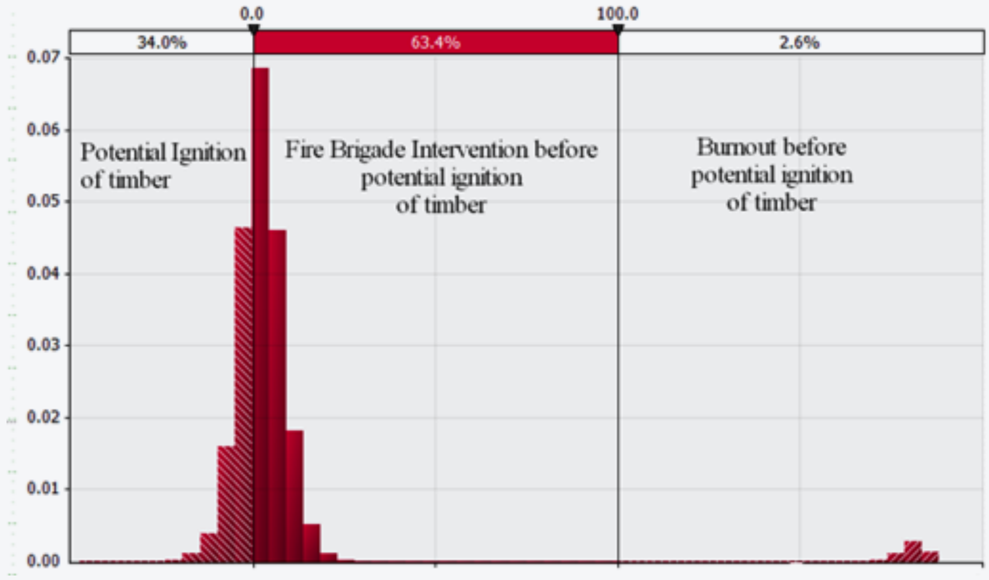


Figure G8: Typical performance of fire-protected timber coverings for massive timber construction – mean fire load of 500 MJ/m² on the 7th floor.

G2 Derivation of FRL Distributions

G2.1 Two Peak FRL Characterisation

Effectiveness can be considered to be a combination of efficacy and reliability. It is practical to express efficacy in terms of the time to failure when exposed to a standard heating regime such as AS 1530.4, ISO 834[50] or ASTM E119[51], because these methods are used for regulatory purposes and there is a very large volume of existing test data. Reliability can be considered in terms of the probability of the design performance level being achieved.

There are many factors that can affect the efficacy of passive fire protection systems. Examples are given in Table G2.

There will also be variations in the properties of materials used for structural elements and the applied loads during a fire event that can also be accounted for in the distribution.

Table G2: Factors affecting efficacy of passive systems.

Ref	Factors	Potential Impact	Est. Frequency
1	Gross defect (e.g. substitution of fire-protective coverings with standard lining materials or gross fixing errors)	Minimal protection provided by applied protection – fire resistance approximates to the inherent fire resistance of underlying structure plus a minor contribution from the lining or in concrete structures substantial spalling occurs	Relatively rare and unlikely to be systemic throughout a structure if adequate controls are in place
2	Normal variations in materials and installation practices	Typically manifests as a normal distribution of performance around the mean fire resistance	Will occur with all systems
3	Minor variations in method of fixing	Board systems tolerant of minor variations in fixing systems. Other systems such as masonry walls can be prone to premature failure due to construction errors[52]	Minor variations would occur frequently but impact on performance relatively low
4	Sensitivity to heating regimes	Fire-protective boards are normally resilient to variations in the heating rate but other systems such as glazing & intumescent coatings may be more sensitive	Low frequency of major degradation in performance would be expected
5	Aging	There is a risk of materials deteriorating with age. For board materials this impact is considered low	Low frequency
6	Unprotected large service penetrations	Could allow fire to spread through hole formed in barrier or fire spread to structural members by-passing fire-protective coverings	

A common approach is to define a normal distribution to characterise the potential variation in FRLs due to the above factors; however, some factors such as gross defects can cause very large reductions in performance. The two peak FRL characterisation proposed by England[7] was therefore adopted. Essentially, the FRL is characterised by combining two normal distributions, one with a mean value equal to the notional FRL and the other equal to a mean FRL based on the expected performance of the element with a gross defect.

This is best demonstrated by an example.

Consider an enclosure with eight loadbearing elements – six loadbearing wall elements and two loadbearing floor elements – as shown in Figure G9.

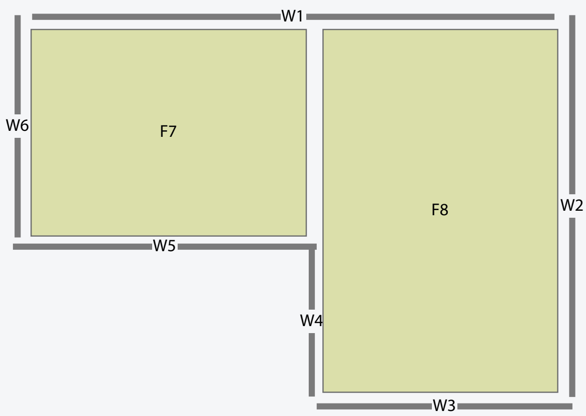
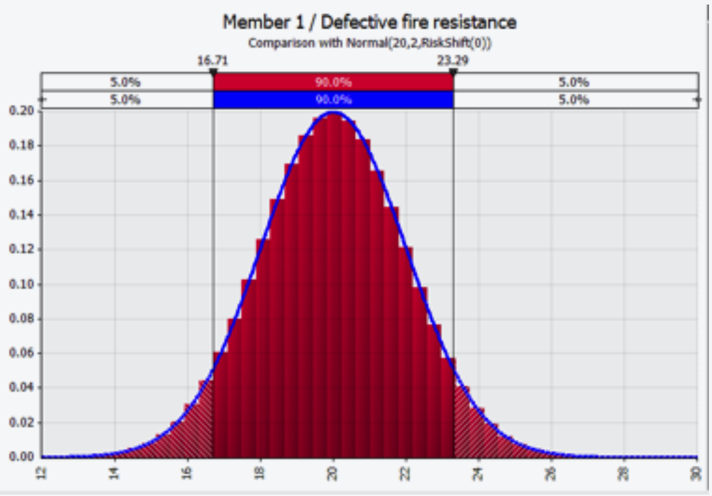
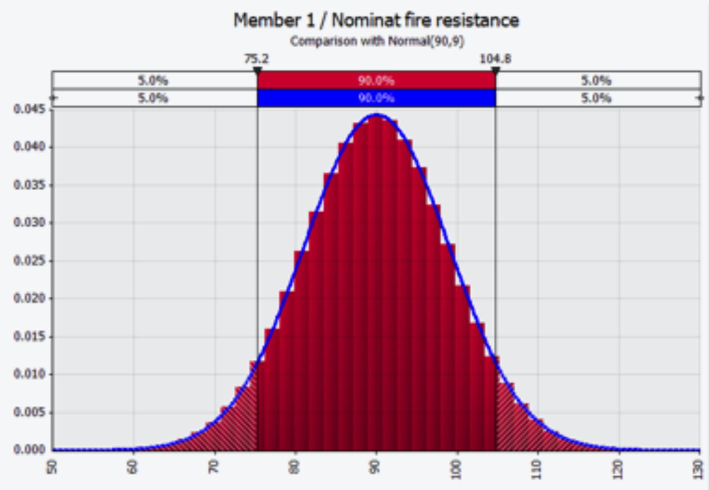


Figure G9: Example schematic layout of an enclosure.

The nominated FRL for the elements is 90/90/90 minutes and a standard deviation of 10% (9 minutes) is assumed for minor variations in performance. The probability of a gross defect is assumed to be 0.01 for each element. With a gross defect the FRL is reduced to a mean value of 20/20/20 minutes with a standard deviation of 10% (2 minutes). The assumed normal distributions are shown in Figure G10.



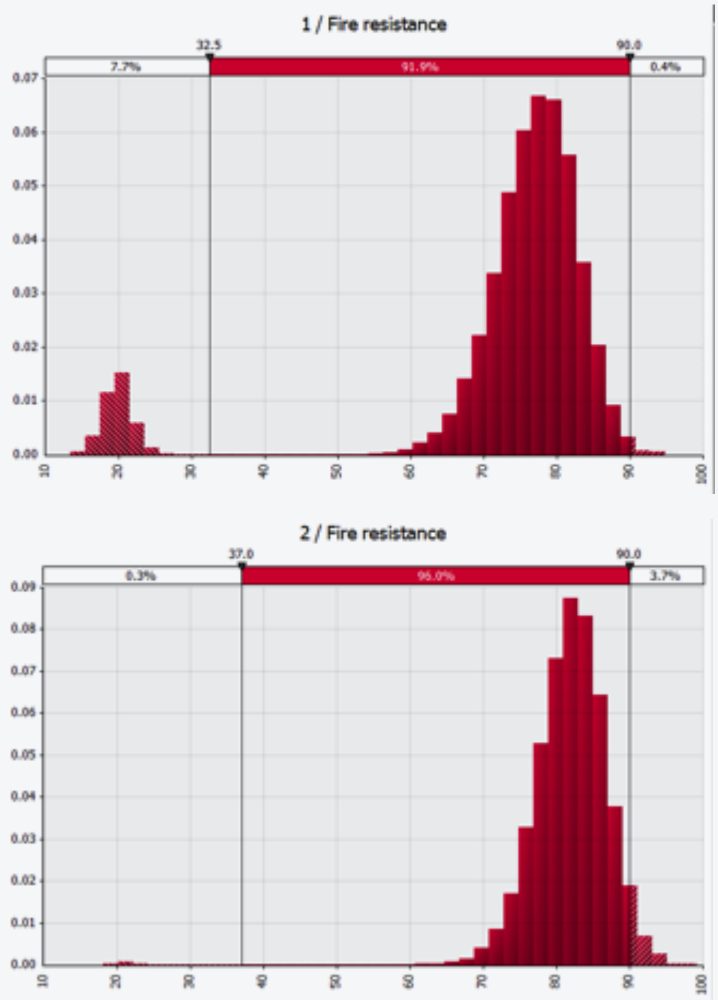
Probability v Fire Resistance Level – min with gross defect



Probability v Fire Resistance Level –min for nominated Fire Resistance Level

Figure G10: Inputs for calculation of combined distribution for simulations.

These two distributions can be combined in a Monte Carlo analysis and distributions obtained for the earliest time to failure of one element. If the time to failure of two or more elements bounding the enclosure is of interest, for example to estimate the time of a major structural failure, this can also be calculated. The results for one and two elements failing are shown in Figure G11. It should be noted that these values are for demonstration purposes and other values were used in the analysis.



Combined distribution for premature failure due to failure of 1 element only (no redundancy)

Combined distribution for premature failure due to failure of at least 2 elements (1 level of redundancy)

Figure G11: Calculated combined distributions for Monte Carlo simulations.

These results can be checked by assuming a binomial distribution using the following equation:

$Pr(X=i) = {}_{n}C_{i}p^{i}q^{n-I}$

where

$Pr(X=i)$ is the probability that there will be i successes

n is the number of independent trials

p is the probability of success for each trial, and

${}_{n}C^{i}$ is Combinatorics “Choose” function.

This relationship assumes independent trials. It could be argued that if there is one gross defect in a building it could be symptomatic of poor workmanship and supervision and, in such cases, the probability of a second fault being present may be greater. However, with good workmanship and supervision throughout the building, as required by the relevant building regulations throughout Australia, it was considered reasonable to assume independence.

Table G3 presents the probability mass function calculated in accordance with the above function for the case where there are eight primary members around a fire compartment (n=8) and the probability that the fire protection is applied correctly is 0.99 (p=0.99).

Table G3: Probability mass function for eight structural members.

Num columns ok (i)		Prob(X= i)	Prob (X≥i)
0	Pr(X=0)	1E-16	
1	Pr(X=1)	7.92E-14	
2	Pr(X=2)	2.744E-11	
3	Pr(X=3)	5.434E-09	
4	Pr(X=4)	6.724E-07	
5	Pr(X=5)	5.326E-05	0.9999993
6	Pr(X=6)	0.0026361	0.9999461
7	Pr(X=7)	0.0745652	0.9973099
8	Pr(X=8)	0.9227447	0.9227447
Checksum		1	

The probability of any one element failing prematurely can be calculated to be about 1 – 0.923 = 0.077 (i.e.7.7%), and the probability of two or more elements failing prematurely would be 1 - 0.9973 = 0.0027(0.27%), which are consistent with the secondary peaks in Figure G11.

G2.2 Estimates of impact of defects and frequency of occurrence

General variability/primary peak

Factors 2 to 5 in Table G3, together with the variabilities of the properties of the structural element and applied load, were grouped and represented as a normal distribution with a mean value equal to the nominated FRL and standard deviation of 10% of the nominated FRL.

Factor 1 could yield FRLs substantially below the mean FRL and were therefore represented by a secondary peak. The FRL of the secondary peak and probabilities were derived as detailed below. The impact of Factor 6 service penetrations can be allocated to the primary and/or secondary peaks depending upon the specific circumstances which are discussed below.

Probability of gross defects to fire protection systems protecting structural steel or timber

A typical gross defect would be a substitution of fire-protective coverings with non-fire-protective coverings encapsulating steel or timber structural elements and/or forming the boundary of a compartment. Typically, these omissions are expected to occur above false ceilings and behind false walls where they are not easily observed and would be difficult to observe by inspection.

There are very few surveys on which to base an estimate of the probability of such an occurrence. However, a survey of fire safety systems in high-rise office buildings in Melbourne was reported by Moinuddin and Thomas[14]. A sample of seventeen different buildings was considered and, since participation was on a voluntary basis, the results would be expected to be above average in most respects. Results were based predominantly on reviews of maintenance/inspection records. Of this sample, data with respect to structural steel protection was available on two buildings and the results are summarised in Table G4 extracted from the Moinuddin and Thomas paper.

Based on these two data points the number of gross defects per floor would be between 0.06 and 0.136. The report does not identify the sizes of the buildings. A typical steel-framed office approximately 63 m x 27 m could be expected to have of the order of 90 beams/floor. Using this estimate, the frequency of a gross defect would be about 0.0007 to 0.0015/element.

This range is substantially less than the assumed probability of gross defects per element of 0.005 assumed for the analysis.

Table G4: Survey results gross defects for structural steel elements.

Building Ref	No of floors	No of reports available	No of reports showing compliant	Problems reported
1	44	4	2	Report 1: At 6 locations the beams were found to be not protected Report No 2: Non-compliant (no specific details)
2	18	1	0	Fire-rated beam missing from a steel beam

Performance of timber-framed construction with gross defects

Within a residential apartment, it is unlikely that major defects such as direct exposure of unprotected structural framing would occur in occupied areas, but unprotected members may occur behind standard-grade plasterboard. This will be considered to represent a typical gross defect and a similar arrangement will be assumed for structural steel members whereby unprotected steel will be assumed to be concealed behind standard plasterboard sheets. It will be assumed that the plasterboard will provide the equivalent of an additional 10-minute FRL period in addition to the inherent resistance of the base structural member, which is consistent with the assigned contribution of 10 mm (3/8 inch) standard plasterboard[53].

It is therefore necessary to quantify the performance of unprotected timber-frame members. In 1982, a program of tests was undertaken on residential floor constructions using the ASTM E119 (similar to AS 1530.4 standard heating regime) in addition to a proposed alternative heating regime for residential fires[54], as shown in Figure G12.

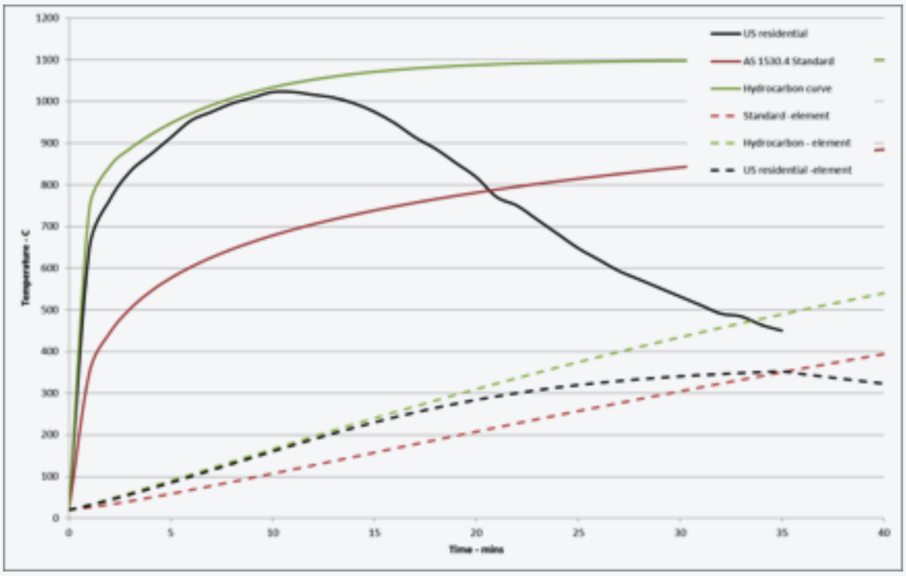


Figure G12: Graph showing NBS proposed residential heating regime with the AS 1530.4 standard and hydrocarbon heating regimes for comparison together with the calculated temperature of a target element.

In the test series, two tests were performed following the ASTM curve on an unprotected floor with 51 mm x 203 mm timber joists with structural failure estimated to occur after 14 minutes and 42 seconds in the first test and 13 minutes 10 seconds in the second test (average approx. 14 minutes). A test was performed following the proposed alternative heating regime on a similar element with failure occurring after 8min 45s. Using the conversion method described in Appendix G3: Converting Fire Resistance Time to Fire Scenario Time, a failure time of 9 minutes was predicted for the proposed alternative heating regime based on standard fire resistance failure time of 14 minutes, providing confidence in the application of the conversion method for deriving scenario times presented in Appendix G5 for low fire durations and unprotected timber members.

During the test series an unprotected lightweight steel joist was also tested with a structural failure time estimated to be 2min 48s.

The results of tests comparing the performance of engineered and more traditional solid joists exposed to the standard ASTM E119 heating regime but with failure conditions based on estimated fire fighter breach were reported by Kerber[55] and are summarised in Table G5.

Table G5: Summary of relevant results from Kerber[55].

Structural Element	Ceiling	Fire Fighter Breach – min: sec
51 x 250 solid joist	No	18:35
Timber I Joist	No	6:00
51 x 250 solid joist	12.5 mm standard plasterboard	35.30
Timber I Joist	12.5 mm standard plasterboard	26:43
Metal Gusset truss floor	12.5 mm standard plasterboard	29:00
Finger Joint truss floor	12.5 mm standard plasterboard	26:39

Su et al.[56] undertook a series of basement fire tests with representative fire loads with unprotected solid timber and engineered timber members exposed directly to the fire conditions. A series of 11 tests are reported (eight undertaken with the door open and three with the door closed, modifying the ventilation conditions and fire severity). Typical heating regimes for both these scenarios are shown in Figure G13 and Figure G14 for the door open and door closed configurations, respectively. The figures also include the AS 1530.4 Standard heating regime for comparison and calculated temperatures of a target specimen which was used to convert the structural failure times to an equivalent fire resistance time following the procedures described in G3 Converting Fire Resistance Time to Fire Scenario Time.

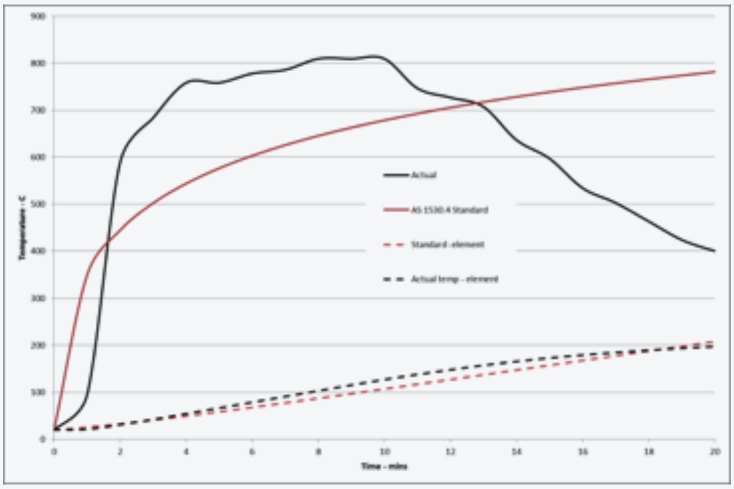


Figure G13: Typical heating regime from studies of unprotected floor assemblies – Door open (Su et al.[56])

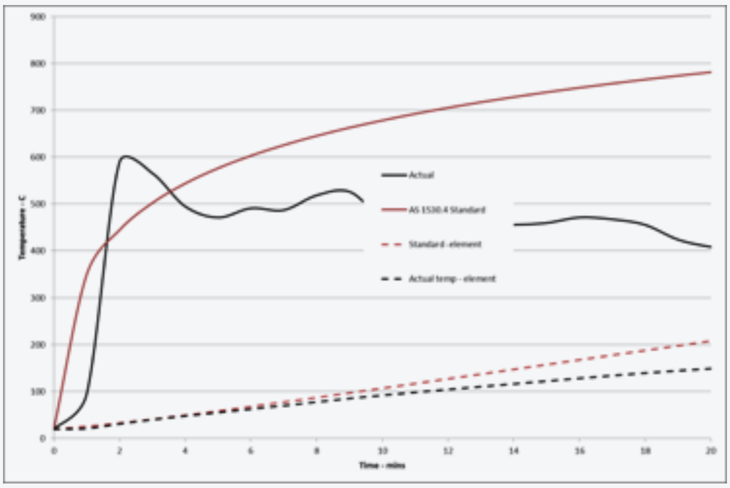


Figure G14: Typical heating regime from studies of unprotected floor assemblies – Door closed (Su et al.[56])

The results for a range of unprotected joists are summarised in Table G6. The repeatability of the results and consistency of the equivalent fire resistance period calculations are very good considering the differences in heating regimes for the door open and closed tests.

Table G6: Time to structural failure and equivalent fire resistance time for unprotected joists from Su et al.[56]

Floor Assembly Type	Test Ref	Door status	Structural failure time – min:s	Equivalent fire resistance time – min
Solid wood joist	UF-01	Open	12:20	14
	UF-02	Closed	20:00	14
Wood I-joist A	UF-03	Open	8:10	10
	UF-09	Closed	12:58	10
Steel C-joist	UF-04	Open	7:42	9
Metal–Plate wood truss	UF-05	Open	7:47	9
Wood I-joist-B	UF-06	Open	6:22	8
	UF-06R	Open	6:20	8
	UF-06RR	Open	6:54	8.5
Metal web wood truss	UF-07	Open	5:25	6.5
	UF-08	Closed	7:64	7

The above results indicate that unprotected solid timber joists will maintain structural adequacy for equivalent fire resistance periods of between 14 and 18 minutes compared to engineered lightweight timber joists, with equivalent fire resistance periods ranging from 6 mins to 10 mins. Lightweight steel joists achieved equivalent fire resistance periods for structural adequacy of 3 to 9 minutes.

The equivalent fire resistance periods for timber-framed beams (allowing 10 minutes for a covering) will be between 16 and 28 minutes, and therefore a mean value of 22 minutes will be adopted; with a standard deviation of 2.2 minutes, such that the estimated range of values will be included within three standard deviations of the mean.

For timber-framed walls, a gross defect would be a frame covered with a single layer of 10 mm or 12 mm non-fire-grade plasterboard, which would provide FRLs consistent with the above estimate of a mean value of 22 minutes with a standard deviation of 2.2 minutes.

Performance of massive timber with gross defects

The fire resistance performance of the massive timber element with a gross defect will be the sum of the inherent fire resistance of an unprotected element plus 10 minutes to allow for the partial protection of a covering. From a review of CLT test data the inherent fire resistances of the CLT tested systems can be estimated to lie in the range of 45 mins to 178 mins, excluding any contribution from a 10 mm plasterboard covering. A reasonable estimate of the performance of a CLT system with gross defects providing the element is greater than 75 mm thick, would be an equivalent fire resistance period of 60 minutes with a 10% standard deviation, assuming a normal distribution subject to adequate detailing of connections. This estimate is also consistent with the typical performance of a massive timber element with a fire-protective covering of 16 mm fire-grade plasterboard required to achieve an FRL of 90/90/90, if a 30 minutes contribution to the FRL is provided by the plasterboard and 60 minutes is provided from the inherent fire resistance of the massive timber element.

Performance of structural steel elements with gross defects

As discussed above, the impact of gross defects on the combined slab/beam assembly will be based on the performance of the steel beams. Adopting a consistent approach to that proposed for the timber-frame building, it will be assumed that fire protection has been omitted above a false ceiling or behind a false wall or column encasement. Therefore the time to failure assuming a gross defect will be based on the FRL for unprotected steel plus 10 minutes. The FRL of unprotected steel will be calculated using the correlations from AS4100 presented below and adopting a critical temperature of 550°C.

Equation 2 – three-sided exposure $t = -5.2 + 0.0221T + (0.433T/k_{sm})$
Equation 3 – four-sided exposure $t = -4.7 + 0.0263T + (0.213T/k_{sm})$

where

t = time from the start of the test, in minutes
T = steel temperatures, in °C ($500°C \leq T \leq 750°C$)
k_{sm} = exposed surface to mass ratio, in m^2/tonne ($2 \leq k_{sm} \leq 35$)

For mid-rise buildings, common sections would be expected to have surface area to mass ratios in the range of 18 to 30m^2/tonne. Table G7 shows the calculated fire resistance assuming a limiting temperature of 550°C for this range of sections.

Table G7: Calculated fire resistance for unprotected steel sections.

k_{sm} -	Fire Resistance – min	
m^2/T	three-sided	four-sided
18	20.2	16.3
19	19.5	15.9
20	18.9	15.6
21	18.3	15.3
22	17.8	15.1
23	17.3	14.9
24	16.9	14.6
25	16.5	14.5
26	16.1	14.3
27	15.8	14.1
28	15.5	13.9
29	15.2	13.8
30	14.9	13.7

For the purposes of this analysis, an equivalent fire resistance time for failure of a structural steel member with gross defects will be taken as 26 minutes (this includes a 10-minute allowance for standard plasterboard or similar coverings) with a standard deviation of 10%. With this assumed distribution, all the common values will lie within two standard deviations of the mean.

Performance of lightweight steel elements with gross defects

Test data from lightweight steel construction indicates that lightweight steel-frame construction may be more sensitive to changes in heating rates than timber-frame construction, due to the higher coefficient of thermal expansion and small cross-section increasing the risk of premature degradation of fire-protective coverings.

Since a comparative analysis is being undertaken to consider the extension of the use of timber-frame construction, it will be assumed that the method to convert fire resistance times to fire scenario times is applicable to lightweight steel-framed walls and the impact of gross defects will be similar to lightweight timber-framed construction, since such an approach will yield conservative results (ignoring potentially poorer performance of lightweight steel-frame construction compared to timber).

It will also be assumed that lightweight steel floor/ceiling systems will not be used in the control building and that structural steel will be adopted for the loadbearing frame. This approach has been adopted to provide a higher benchmark by ignoring lightweight steel floors, which tend to achieve similar or worse performance than lightweight engineered timber systems when unprotected.

Gross defects to reinforced concrete and masonry elements

Gross defects for reinforced concrete could include:

- no or minimal cover to reinforcement
- missing or insufficient reinforcement
- large openings in inaccessible places
- concrete and detailing increasing risk of excessive spalling.

Merretz[57] summarised the findings of a survey of 95 buildings in the Sydney area, which focused on durability. Of the 227 faults detected, the average cover was found to be 5.45 mm. Typically, at least 20 mm or more minimum cover is required to satisfy durability requirements. Such variations could have a very large impact on the fire resistance performance of concrete elements.

For masonry gross defects could include

- missing bricks /blocks in inaccessible places
- walls too slender / inadequate end restraint

Since the control building is predominantly of steel-frame construction a detailed analysis of the impact of gross defects relating to concrete has not been undertaken since the impact of gross defects will be based on failure of the steel beams supporting the slab due to omitted fire protection.

Service penetration defects

A report on the fire system effectiveness in major buildings in New Zealand[13] included inspection data from university, hospital, and office/retail buildings relating to more than 5,000 passive fire protection systems including service penetrations, which are summarised in Table G8.

Table G8: Summary of NZ inspections of service penetration seals.

System	Issue	% of cases in drywall systems (e.g. plasterboard)	% of cases in masonry walls	Ratio of drywall: masonry wall construction
Small penetration (e.g. single cable)	Unsealed	16.2	18.4	0.88
	Incorrect sealant	2.7	2.1	1.29
	Total	**18.9**	**20.5**	**0.92**
Large penetration (e.g. cable tray)	Unsealed	40.0	33.3	1.2
	(e.g. cable tray)	20.0	8.3	2.4
	Total	**60**	**41.6**	**1.44**
Collar system	Missing	10.8	8.3	1.30
	Incorrect installation	7.7	6.3	1.22
	Ad hoc arrangement	5.4	4.2	1.29
	Total	**23.9**	**18.8**	**1.27**

From Table G8, the frequencies of issues and types of faults with penetration seals in masonry and drywall systems are broadly similar. It cannot be ascertained if the higher frequency for plasterboard systems is a trend or just a result of the small sample size.

The ‘ad hoc arrangements’ category is assumed to represent penetrations that are sealed but there is insufficient documentation to determine the adequacy of the system. It will be assumed that the performance of these types of system will be reflected in the distributions assumed for the FRLs of the base elements of construction, and the ad hoc systems have therefore been excluded from requiring specific consideration. Table G9 shows the percentage of penetration seals with potentially significant defects from the New Zealand Study.

Table G9: Seals with potentially significant defects (derived from Table G8).

System	% of cases in drywall systems (e.g. plasterboard)	% of cases in masonry walls
Small penetration (e.g. single cable)	20	20
Large penetration (e.g. cable tray)	40	33
Collar system	19	15

These results were not obtained from apartment buildings and therefore the applicability of the data needs to be considered. In apartment buildings, the following conditions will generally apply:

- Large penetrations will generally be restricted to service risers and will therefore not be from one occupied area to another.
- Small penetrations will occur for the main power supply, telecommunications systems, sprinkler system and water supply.
- Plastic soil waste and vent pipes will be in common usage and require protection, usually employing intumescent seals in collar assemblies.
- Typically, service penetrations will occur in clusters in bathrooms/toilets and kitchen areas, and will penetrate service risers with low internal fire loads.

Defective service penetration seals can reduce the effectiveness of barrier systems. The degree to which this occurs depends on a number of factors including:

- the size of service penetration
- the type of service penetration
- the position of the service penetrations
- the separating element penetrated.

Considering the generic apartment building, the services penetrations will tend to be in three clusters around the bathrooms and kitchen areas, which are served by risers that contain only services with a limited fire load, reducing the risk of spread to other habitable spaces.

The probability and/or consequences of fire spread due to faults with small service penetrations is expected to be relatively low compared to large penetrations/plastic pipe penetrations.

Where collar systems are missing, there is a greater potential for large openings and premature fire spread to occur and/or structural adequacy to be impacted but, in many instances, the adjacent compartment may be a service shaft where the potential for fire spread would be limited by the low fire load. Since this mode of failure could impact considerably upon the size of the secondary peak in the two peak model if there is a large reduction in the FRL, an experiment was undertaken to gain an understanding of the impact of large unprotected service penetrations on timber-framed construction[58]. A nominally sized 3 m x 3 m wall system was constructed with two 90 x 45 timber stud frames faced with two layers of 13 mm thick fire-grade plasterboard on the occupancy sides of the studs. Mineral fibre insulation was fitted between studs on the non-fire side of the cavity. The party wall arrangement was selected to represent a worse case scenario because fire spread was unrestricted within the cavity, whereas the cavity was closed off with single-frame wall systems by the noggins limiting incipient fire spread. The specimen was penetrated by an unprotected 100 mm uPVC pipe, which passed through the wall to a plasterboard shaft on the non-fire side.

A load was applied to the fire-side timber frame during the test avoiding load sharing with the non-fire side frame. The specimen supported the full test load for 72 minutes, which is about two standard distributions from the mean if a 10% value for the standard distribution is assumed.

Temperatures measured on the surface of the uPVC pipe during the test close to the point where it penetrated the shaft indicated that fire spread could occur after about 5 minutes equivalent FRL period if there was a direct service penetration between occupied areas. This type of detail would, however, run through a service shaft in most installations to address noise control issues as well as fire protection.

The control building also included drywall components with steel studs compared to timber studs and, since a comparative study is being undertaken, similar performance for the control and timber-framed buildings would be expected.

Based on the above discussion, it was considered that the assumed two-peak FRL distribution will incorporate the effect of defects relating to service penetrations.

Openings in lift shafts and fire-isolated stair shafts

Moinuddin and Thomas[14] reported findings from maintenance inspections on three office buildings estimating a 16% likelihood of there being a gap/hole in each fire stair. No gaps/holes were reported in the lift shafts of the three buildings.

Considering the generic apartment building, it can be assumed that approximately three elements/floors bound the stairs or lift shaft (2 SOUs and a public area), which equates to 24 elements. Therefore, the probability of an element bounding a stair having an opening would be 0.007, which appears reasonable when compared to the estimates of the probability of openings through SOU bounding construction (0.021 to 0.03).

If unprotected openings in the shaft wall occur, the shaft will tend to fill with smoke in a similar manner – irrespective of the form of construction – if the impact of sprinklers is ignored and, under these circumstances, the consequences would be similar.

G2.3 Summary of Inputs for FRL Two Peak Distributions

The proposed inputs to the Monte Carlo analysis are summarised in Table G10.

Table G10: Fire resistant inputs for Monte Carlo simulations.

Element	Case	FRL – min	SD – %	Prob. of defect	Defect FRL – min	SD – %	Levels of redun-dancy	Notes
Apartment Fire doors and fire stair doors	All	60	10	.1 (.05)	0	0	0	Bracketed value relates to door to apartments other than fire apartment. Defects covered in 'door open' configurations
Service penetrations								Impact of unprotected service penetrations assumed to be in primary peak
Global structural collapse; loadbearing walls and floor/ceilings	Control Massive TF	90	10	.005	26 60 22	10	1	Impact of unprotected service penetrations assumed to be in primary peak
Non-loadbearing walls	Control Massive TF	60 75 75	10	.005	22 60 22	10	0	Requirements to control incipient fire spread are expected to typically provide 90 minutes FRL for TF but period has been downgraded to 75 minutes to allow for unexpected systems. Inherent FR of massive timber plus covering are expected to provide FRLS in excess of 75 minutes

Note: a single time (most critical) has been nominated for FRLs rather than separate values for structural adequacy, integrity and insulation. For example, 60 minutes has been specified for fire doors since the 30-minute insulation criteria is not considered significant for predicting fire and smoke spread.

G3 Converting Fire Resistance Time to Fire Scenario Time

In most instances, the time to failure of an element of construction ascertained in a standard fire resistance test will differ from the failure time if the element is exposed to a real or simulated fire scenario (e.g. Annex A of EN 1991-1-2:2002) because the time temperature histories will differ (see Figure G15).

If an element of construction comprises homogeneous materials with known thermal and mechanical properties at elevated temperatures (e.g. steel, concrete, timber), it is possible to determine the time to failure using simple correlations or more complex methods such as finite element analysis.

However, many fire-resistant elements or components are too complex to model reliably (such as fire doors, penetration seals, composite systems, connections, board fixings, adhesion of sprayed materials, spalling of high-strength concrete, etc) and therefore a general method for conversion of fire resistance times to scenario times was preferred. This also addressed concerns that different models are likely to have varying degrees of conservatism generating further variables in the analysis.

A detailed review of general time equivalency methods has been undertaken by Wade et al.[59] The review recommended the use of an energy-based time-equivalent approach as a general method to assess the performance of building elements exposed to compartment fires of different severities based, on Kodur's Equivalent Absorbed Energy Method[60].

Time equivalence based on the maximum temperature of protected steel was discounted in the Wade study on the basis that equivalency could only be ascertained if maximum temperatures are achieved. However, a method based on Equal Steel Temperature (EST) can be developed that does not rely on a maximum temperature being attained as detailed below:

$$\Delta T_s = \frac{k_i}{h}\left[\frac{(T_f - T_s)}{c_s\left(W/_D\right) + \frac{c_i \rho_i h}{2}}\right]\Delta t \qquad \textbf{Equation 4}$$

A 'target protected steel element' is defined with known thermal properties and the temperature at a critical point calculated when exposed to the fire scenarios and the standard heating regime. Equivalent exposure is deemed to have occurred when the critical part of the element reaches the same temperature under the different heating regimes. In this case, a lumped thermal mass approach was adopted with the mean temperature of the steel calculated using Equation 4[61] .

Where:

T_s is the steel temperature – °C

T_f is the enclosure temperature – °C

k_i is the thermal conductivity of the insulation W/m.K

c_i is the heat capacity of the insulation – K/kg.K

ρ_i is the density of the insulation –kg/m^3

c_s is the heat capacity of steel – J/kg.K

W/D is the mass per unit length divided by the heated perimeter kg/m^2

Δt is the time step – s

The process is shown graphically in Figure G15. If it is required to determine the time to failure of an element that achieved an FRL of 63/-/- when exposed to the fire scenario (parametric curve) fire, the following approach is adopted:

- the target element attains a temperature of 454°C when exposed to the standard fire resistance test for 63 minutes
- the target element would need to be exposed to the fire scenario for 45 minutes to attain the same temperature
- therefore the fire scenario failure time would be 45 minutes.

In this example, the target element would need to be exposed to the hydrocarbon heating regime for 45 minutes to attain 454°C.

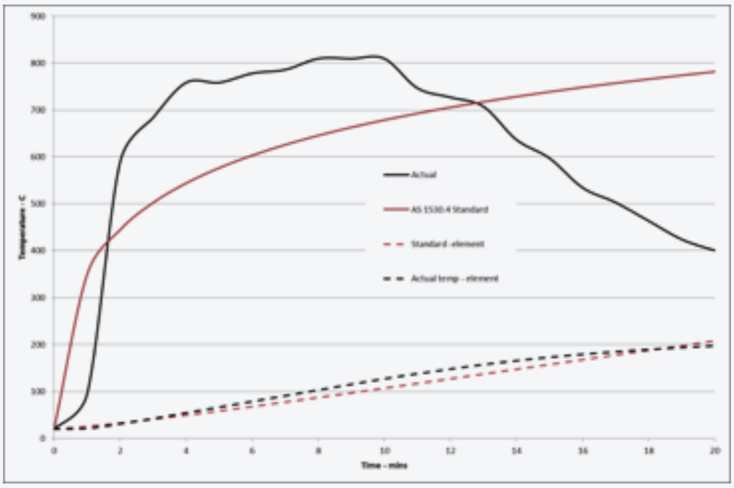


Figure G15: Conversion of fire resistance period to fire scenario time.

Three candidate methods were considered to convert fire resistance times to scenario times and vice versa based on the above discussion:

- Nyman's Method using the emissive power of Fire Gases
- Kodur's Equivalent Absorbed Energy Method
- Equal Steel Temperature (EST method) described above.

The methods were compared by using each one to convert the standard heating regime to an equivalent hydrocarbon heating regime time and then plotting results from tests on the same forms of construction that had been fire tested to both heating regimes. The data obtained for this purpose is summarised in Table G11.

The results of the comparison are summarised in Figure G16. The absorbed energy (without the modification factor) and imposed radiation methods provide comparable results (as shown in Figure G16) and tend to underestimate the hydrocarbon heating regime time compared to experimental data. Since a comparative study is being undertaken, it is preferable to estimate performance as accurately as possible. Kodur et al. calibrated their method against deflections estimated from finite element analysis and proposed a correction factor based on maximum temperature reached. For the hydrocarbon regime, a factor of 1.16 was therefore applied, improving the correlation with experimental data. Results for the Kodur method are shown with and without the correction factor derived from finite element analysis of concrete beams.

With respect to the EST method, three thicknesses of material were considered with properties approximating to ceramic fibre. The input data used is summarised in Table G12. As expected, different correlations were obtained depending upon the thickness of fire protection assumed when using the EST method. This was also reflected in the calculations based on large test programs involving loaded and unloaded steel sections protected by a sprayed vermiculite system. It is therefore important to select a material thickness representative of the fire resistance range and protection thicknesses relevant to the study. In this instance, the steel target protected by a 25 mm thickness of material with properties approximating to ceramic fibre was found to provide the most reliable conversion and was therefore adopted for the detailed analysis.

The main limitation with the above method is that it considers thermal performance only but does not directly consider the impact of factors such as thermally induced deflections and/or stresses, degradation of structural materials and materials used for protection (e.g. spalling, shrinkage, thermal shock and critical chemical reactions).

If the standard fire resistance heating regime is representative of typical fully developed fires, then the above issues may not require further consideration, but the increased use of plastics in modern furnishings and increased fuel loads among other things have tended to increase the rate of fire growth[55,62].

Table G11: Comparative data for elements exposed to standard and hydrocarbon heating regimes.

Data Sources	Type of Construction	Data used
FWPA 13 timber frame: **F91769**[63] **F91770**[64] **F91767**[65] **F91768**[66]	90 x 45 mm timber studs faced with one layer of 13 mm Australian fire-grade plasterboard non-loadbearing	Interface temperatures 300°C and stud temperatures at 7.5 mm depth 300°C
FWPA 26 steel frame: **F91780**[67] **F91782**[68] **R9112**[69] **R9113**[70]	64 mm steel stud faced with two layers of 13 mm Australian fire-grade plasterboard – non-loadbearing small scale	Upper surface of plasterboard face. Insulating pads fitted as appropriate – time to 300°C
Spray protected steel	Linear regressions of results from steel test packages undertaken on the same product to the standard and hydrocarbon heating regimes yielding correlations for fire resistance time as a function of surface area to mass ratio, and protection thickness	Comparable results generated using regression coefficients for a spray steel protection system
Solid core door: **Young and England**[18]	Comparative tests performed with corridor mounted in front of solid core timber doors fitted with smoke and intumescent seals	Time to low visibility
Concrete: **Faris et al.**[71]	Comparative tests performed on concrete slab sections under load	Reinforcement 100°C Time to 32 mm deflection
Concrete: **Cooke**[72]	Comparative tests	Deflection

Note: Results for the concrete test reported by Faris were not reported beyond 60 minutes; therefore, only limited comparative data was available.

Table G12: Input Data for EST conversion model.

Parameter	Value(s)	Units
Thickness	10,15,25	mm
Thermal conductivity (insulation)	0.2	W/mK
Heat capacity (insulation)	1000	J/kg/K
Density (insulation)	96	kg/m^3
Heat capacity (steel)	550	J/kg/K
Mass/unit length of steel section	59	kg/m
Heated perimeter	1.21	m

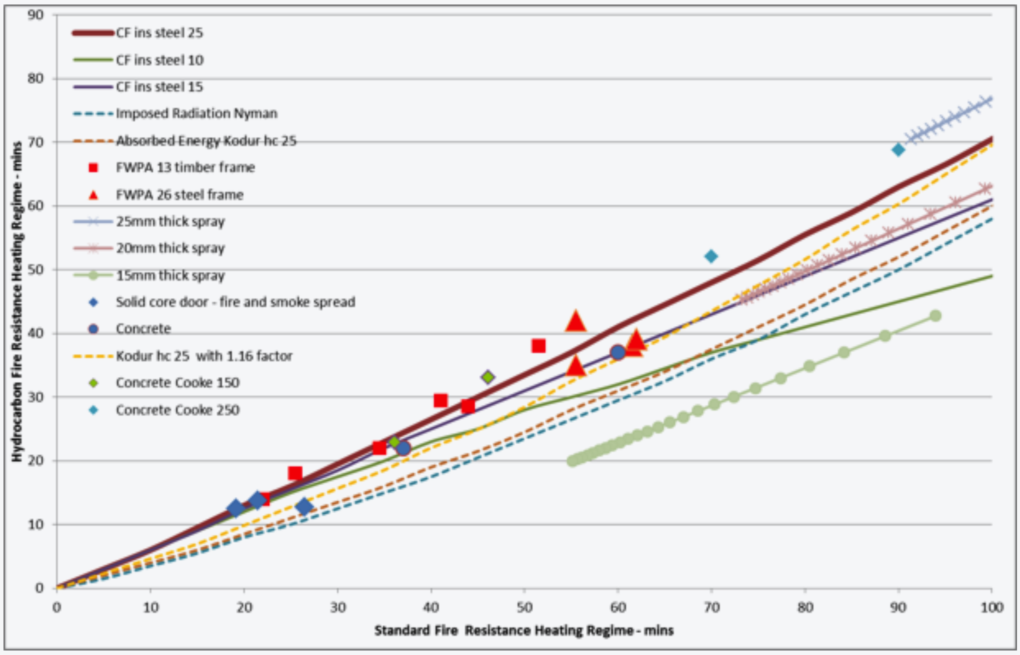


Figure G16: Comparison of conversion methods for standard and hydrocarbon fire resistance test heating regimes.

The following approach was adopted to address the above limitations:

1. The NCC 2016 requirements retain the requirement for the fire resistance of protected timber to be determined in accordance with the standard fire resistance test (AS 1530.4) and also require additional measurements for resistance to incipient spread of fire, for example. Specimens are tested at a representative size (typically 3 m minimum dimension) and, if loadbearing, under loaded conditions. The same requirements also apply to elements other than timber. These tests will demonstrate the performance of systems including thermally induced deflections and stresses, degradation of structural materials/fixings and fire protection systems.
2. To further check for sensitivities, comparisons were made against data from tests performed on similar elements of construction under different heating regimes.
3. In some instances, engineering principles can be applied to assess the impact of more severe heating rates. For example, thermally induced deflections will tend to be greater with more rapid rises in temperature. Materials with relatively high rates of thermal expansion will be prone to greater thermally induced deflections, which may cause premature structural failure or open up gaps in fire protection systems (e.g. lightweight steel systems).

Another practical limitation with the standard test method is that it does not monitor performance after the end of the fire test during the cooling phase. This will affect different forms of construction in different ways. For example, timber structural elements may continue to burn if already ignited or may self-extinguish based on the configuration and imposed heat flux; protected steel temperatures may continue to rise after the end of the test due to heat contained in the fire protection material continuing to flow towards the structural member; and concrete and masonry may degrade and sudden collapse may occur when thermally induced restraint conditions change as the element cools. While these limitations apply to most forms of construction, the mid-rise timber buildings require sprinkler protection under the Deemed-to-Satisfy Provisions in the NCC 2016, substantially reducing the frequency of exposure of the structure to fully developed fires, such that this limitation in the standard test method is less important for mid-rise timber buildings compared to non-combustible construction.

G4 Verification of Stair Climbing Component within the implementation of the Fire Brigade Intervention Model used

The stair climbing component of the model was compared against results of international studies summarised by Claridge. Figures G17 to Figure G22 show outputs from the sub-model for times to climb various levels, and Figure G23 shows these results plotted over the international study results.

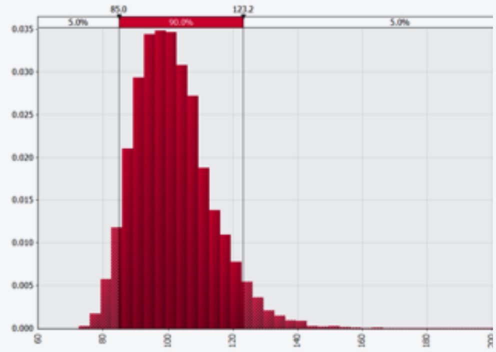


Figure G17: Time to climb 5 levels.

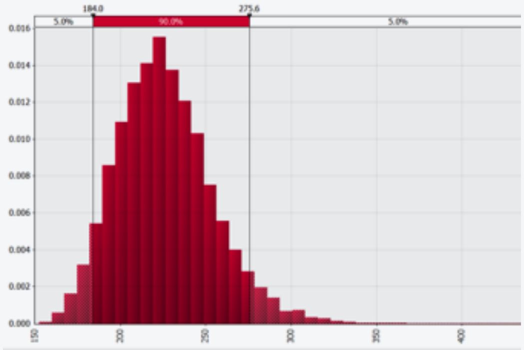


Figure G18: Time to climb 10 levels.

Figure G19: Time to climb 15 levels.

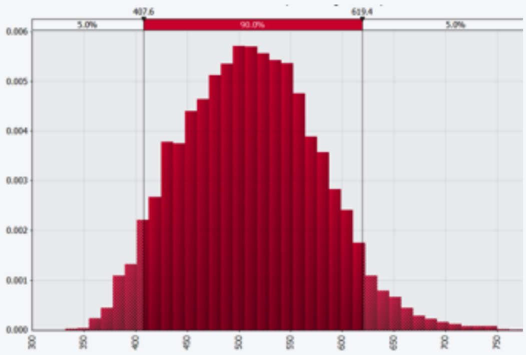


Figure G20: Time to climb 20 levels.

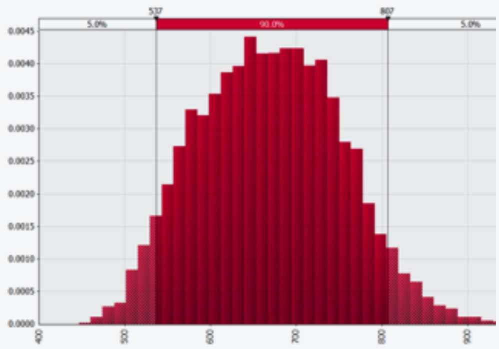


Figure G21: Time to climb 25 levels.

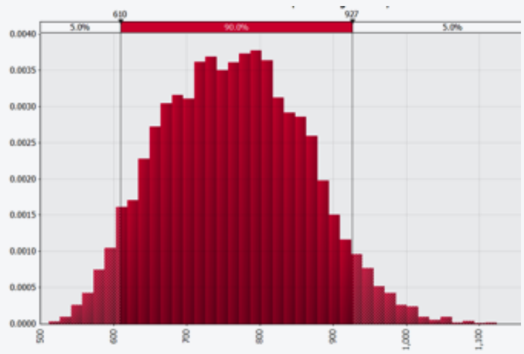


Figure G22: Time to climb 28 levels.

The results correlate well for 5 levels but tend to be conservative at higher levels, which would be expected because of the allowance of recovery periods above 6 levels. It is therefore considered reasonable to adopt the modelling approach and input data for mid-rise buildings.

However, to address the potential for hindrance to fire fighters by evacuating occupants, a 50% increase has been applied to the travel time within the stairs to the set-up position.

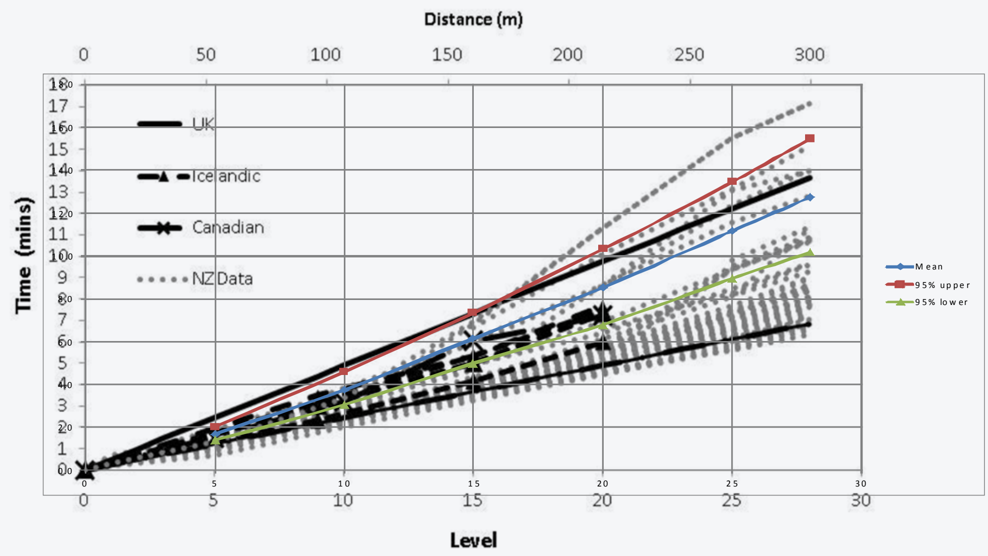


Figure G23: Stair climbing results superimposed on international study results summaries.

G5 Occupant Behaviour Review

A review of international research relating to Occupant Pre-movement Times in Fire undertaken in 2005[73] concluded that, among other things: "At best pre-movement components for specific occupancies could be estimated from statistical analysis and presented in the form of a probability distribution. At present the global database is small and the reliability of predictions based on it (are) likely to be low".

Verification Method C/VM2 prepared to support the New Zealand Building Code[24] nominates the following pre-movement times for buildings where the occupants are considered sleeping and familiar with the building (e.g. apartments):

- Enclosure of fire origin: 60 seconds.
- Remote from the enclosure of fire origin: (standard alarm signal) 0–300 seconds.

The document notes that the incipient phase of the fire growth has not been considered in the design fire, providing an implicit safety factor for the pre-travel activity time.

The above values were also proposed in a draft NCC Fire Safety Verification Method 2015 issued by the ABCB for comment in 2013[74].

Some relevant studies from fire incidents are summarised below:

Wales and Thompson[75] reported the initial stages of Kent Fire and Rescue Service's project to build a comprehensive database of the behaviours and associated motivations of those directly experiencing an accidental dwelling fire. Preliminary findings based on 140 completed surveys include:

- 70% of respondents reported entering the room of fire origin to investigate the source of cues and more than one-third attempted to fight the fire before being driven back by smoke.
- 50% of respondents waited more than one minute before calling the fire brigade (due to occupants trying to fight the fire first).
- After alerting the fire brigade, one group quickly exited alerting others on the wa,y but 40% made some attempt to fight the fire.
- 70% evacuated the building but about 40% re-entered the building for some reason.

While these results relate to single dwellings in the UK they are considered to provide a useful indicator of the likely behaviour during fires in apartment buildings in Australia, in the absence of more relevant and comprehensive data, and have been considered when determining the time the fire brigade are alerted.

Data from post-fire studies indicates that a pre-movement time of 5 minutes may not be conservative for mid-rise and high rise apartment buildings. For example, Proulx and Fahy76 reported average pre-movement (evacuation commencement) times of 2 min: 49 s (5 min: 19 s in winter) and 8 min: 35 s for residential buildings with good and poor alarms, respectively. These are significantly less than the delays in the Forest Laneway Fire (198 min) due in part to the presence of smoke in evacuation paths and a very poor detection/alarm system. A plot of the frequency against delay time to start evacuation during residential and office drills shown in Figure G24 indicates that after an initial peak there is a long tail, indicating that occupants will be evacuating the building over a lengthy period.

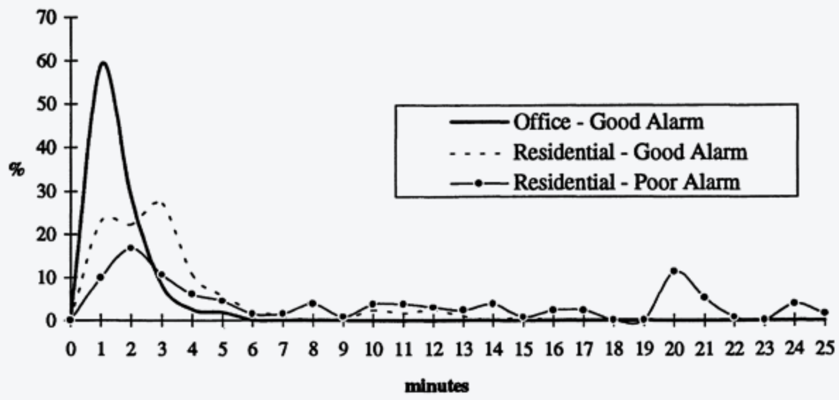


Figure G24: Delay to start evacuation during residential and office fire drills from Proulx and Fahy.

A similar but flatter distribution occurred in the Forest Laneway Fire during the first 30 minutes of the fire, as shown in Figure G25, after which evacuation could not be achieved due to fire/smoke spread, with most occupants waiting in the relative safety of their apartments. There were approximately 550 occupants at the time of the fire.

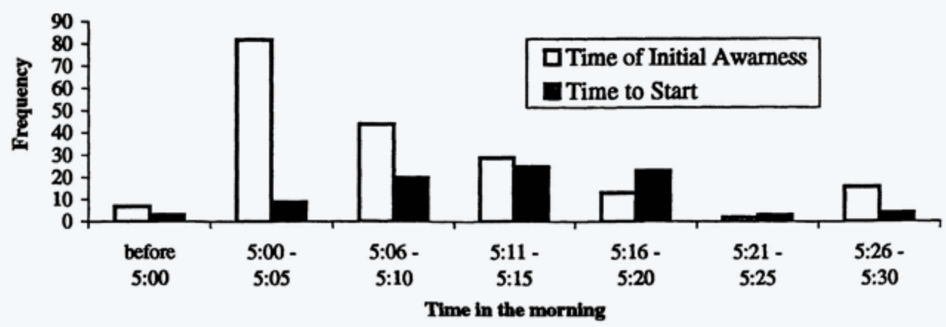


Figure G25: Time of initial awareness and time to start evacuation for the first 30 minutes of the Forest Laneway Fire from Proulx and Fahy.

An analysis was undertaken of an 8-floor apartment building fire that occurred in Rinkeby, Sweden, in 2009 with parking on the ground/basement level[77]. The building was concrete-framed with brick or lightweight concrete walls with mineral fibre insulation with a fire resistance of at least 60 minutes, and 60-minute fire doors providing direct access from the apartments to the stair. There were 12 apartments housing a total of about 33 people at the time of the fire. The fire started on the lower level of a second-level apartment and spread through an open door to the stairwell. Seven fatalities occurred in the stairwell prior to fire brigade arrival. All were trying to evacuate the 7th floor.

All occupants, with the exception of one who was asleep, initially tried to evacuate via the stairs. Some occupants returned to their apartments and awaited fire brigade intervention or climbed down the outside of the building. Others tried to evacuate through the smoke and flames in the stairwell. Of these, some were successful, but seven people perished between the 3rd and 4th levels of the stairwell on the way down from the 7th floor.

This demonstrates typical behaviour; whereby, most occupants at some stage try to evacuate a building once they realise there is a major fire. If they encounter heat and smoke, some will return to apartments and others will try to evacuate through the smoke.

H

Appendix H: Summary of UK Timber Frame 2000 Project – UK

A full-scale fire experiment was undertaken in the UK, as part of the TF2000 project on a six-storey timber-frame building constructed with the platform construction method in 1999, to demonstrate the performance of complete medium rise timber-frame buildings subject to real fires[78]. In particular, the objective of the test was to evaluate fire resistance of a medium rise six-storey timber-framed building subject to a severe natural fire exposure, to ensure that the form of construction can meet the functional requirements for such buildings of the Building Regulations for England and Wales and the Building Standards for Scotland.

The internal loadbearing walls were clad with two layers of standard (non-fire-grade) plasterboard and 9 mm OSB, Type F2 sheathing to one side, where needed for wind bracing. The internal non-loadbearing walls consisted of timber studs with one layer of plasterboard to each side. The compartment walls were a twin stud arrangement with timber studs and mineral wool insulation in between. The structural timber framing and boundary walls of the compartment were protected by plasterboard systems to provide a 60-minute fire resistance (i.e. the equivalent of an FRL of 60/60/60).

The building had four apartments on each of six storeys. The floor plan measured 24.1 m × 12.4 m. The height to the eaves of the building from the ground was approximately 14.4 m. The fire test compartment was a single flat (apartment) on the second floor (level 3) in the southwest corner of the building with a floor area of approximately 60m^2 and a fire load of 391 MJ/m^2 in the form of timber cribs[79].

The fully developed fire did not spread to involve the whole apartment (mainly due to the fire load not being distributed throughout the apartment) but was concentrated within the lounge area and adjacent kitchen. Since the fire was most severe within the living area, the following review will focus on the living area.

Key events following ignition were:

- the fire brigade broke the kitchen window after 22 minutes
- flashover after 25 minutes
- peak temperature close to ceiling in lounge area was about 1020°C after 42 minutes, based on mean of two thermocouples
- door to apartment opened for fire brigade to gain access after 63 minutes (based on observed temperature rise in lobby as door opened)
- temperature close to lounge ceiling approximately 895°C at 59 minutes.
- water applied to the lounge area after 64 minutes by fire brigade
- temperature close to lounge ceiling at 63 minutes just before application of water 730°C
- lounge temperature close to ceiling about 74°C after 68 minutes
- during or after the test, ignition of some timber framing members beneath plasterboard occurred, which was not identified or suppressed at the end of the test. The fire continued to grow within the cavity, eventually leading to the recall of the fire brigade approximately 2.5 hours later.

During the full-scale fire experiment and subsequent cavity fire, the fire did not lead to untenable conditions within adjoining apartments, although damage did occur to the ring beam and studs in the wall to the flat immediately above the fire. Reports refer to some evidence of fire spread to the flat above (presumably flaming from the window frame) but no visible damage to the wall viewed from the apartment was noted.

The following conclusions were provided in the Summary Report[81].

"The compartment fire test met the stated objectives of the programme. The following conclusions may be drawn from an analysis of the data and from observations during and after the test,

- *Derived values of time equivalence have demonstrated that the performance of the complete timber frame building subject to a real fire is at least equivalent to that obtained from standard fire tests on individual elements*

- *Results indicate that fire conditions in the living room of the flat represent an exposure approximately 10% more severe than a standard 60 minute fire resistance test.*
- *The test demonstrated that timber frame construction can meet the functional requirements of the Building regulations of England and Wales and the Building Standards of Scotland in terms of limiting internal fire spread and maintaining structural integrity.*

In meeting the requirements of the regulations and the objectives of the research programme a number of issues have arisen:

- *The standard of workmanship is of crucial importance in providing the necessary fire resistance performance especially nailing of plasterboards.*
- *Correct location of cavity barriers and fire stopping is important in maintaining the integrity of the structure.*
- *The Type of Construction is one that in the United Kingdom has a relatively low market share generally and in the medium rise terms is very recent. For this reason fire brigades are unlikely to be familiar with the type of construction details used. Clearly, education on timber frame for these bodies is necessary.*
- *The issue of vertical flame spread from floor to floor via the windows needs to be addressed."*

The relevant functional requirement is B3[80] which states:

Internal fire spread (structure)

B3.

(1) The building shall be designed and constructed so that, in the event of fire, its stability will be maintained for a reasonable period.

(2) A wall common to two or more buildings shall be designed and constructed so that it adequately resists the spread of fire between those buildings. For the purposes of this sub-paragraph a house in a terrace and a semi-detached house are each to be treated as a separate building.

(3) Where reasonably necessary to inhibit the spread of fire within the building, measures shall be taken, to an extent appropriate to the size and intended use of the building, comprising either or both of the following –

(a) sub-division of the building with fire-resisting construction;

(b) installation of suitable automatic fire suppression systems.

(4) The building shall be designed and constructed so that the unseen spread of fire and smoke within concealed spaces in its structure and fabric is inhibited.

Table H1 summarises the relevant fire safety requirements for the building on which the above conclusions were drawn, compared to the NCC 2016 Deemed-to-Satisfy Provisions.

Table H1: Comparison of fire safety requirements for the Proposal-for-Change and the TF2000 Solution.

System	NCC 2016 DTS Requirements	TF2000 UK system
Loadbearing structural elements	FRL 90/90/90 plus incipient spread of fire criteria applied for 45 minutes	FRL 60/60/60
Non-loadbearing elements	FRL (-/60/60)* plus incipient spread of fire criteria applied for 45 minutes	FRL -/60/60
Cavity barriers	FRL -/45/45	FRL -/30/15
Automatic fire detection and alarm	Required	Required
Cavity insulation required to be non-combustible	Required	Not required
Automatic fire sprinklers	Required	Not required

** Estimated impact of incipient spread of fire criteria is to increase FRL to between -/75/75 and -/90/90 depending upon form of construction*

From examination of Table H1, it can be noted that for the NCC 2016 DTS requirements:

- Protection levels to timber members have been increased, reducing the risk of fire spread to cavities.
- Incipient spread of fire criteria are also applied. Under the 2014 version of AS 1530.4, this includes enhancements for service penetration test methods in elements required to be resistant to incipient spread of fire.
- There are enhanced requirements for cavity barriers to address risk of spread should a fire initiate or spread to cavities.
- There are enhanced controls on cavity insulation materials to reduce the risk of spread via cavities.
- There is provision of automatic fire protection systems, greatly reducing the probability of flashover fires occurring and hence greatly reducing the risk of fire spread vertically between windows, in line with current NCC approaches for sprinkler-protected buildings.

Based on the above discussion, it can be observed that the NCC 2016 mid-rise timber building DTS requirements provide significantly higher levels of protection than the TF2000 building, which was considered to have demonstrated that timber-frame construction can meet the functional requirements of the Building regulations of England and Wales and the Building Standards of Scotland in terms of limiting internal fire spread and maintaining structural integrity.

The approximate temperatures close to the ceiling of the lounge area measured during the TF 2000 testing have been extracted from the project reports[78,79] and are plotted against time in Figure H1, together with the standard heating regime from AS 1530.4 and the parametric curve derived in accordance with the methods described in Section 10.6, which was then used to estimate the temperature of a standard element to compare the fire severity based on the peak element temperature. Dimensions were obtained from project reports with opening sizes scaled from drawings and photographs of the specimen.

The equivalent fire resistance exposure for the test calculated using the above approach was 58 minutes. The deterioration of parts of the structural elements was estimated to be consistent with an element exposed to the standard heating regime for approximately 66 minutes[78,79]. The parametric time temperature curve indicated an equivalent fire resistance period of 62 minutes if the enclosure had progressed to full burnout. The predicted exposures and performances using the methods of Section 10.6 were therefore within 10% of the measured/estimated performance from this experiment. The enclosure temperatures were trending downwards just prior to application of water, indicating that the fire may have been entering the decay phase, but no firm conclusions can be drawn with respect to total burnout.

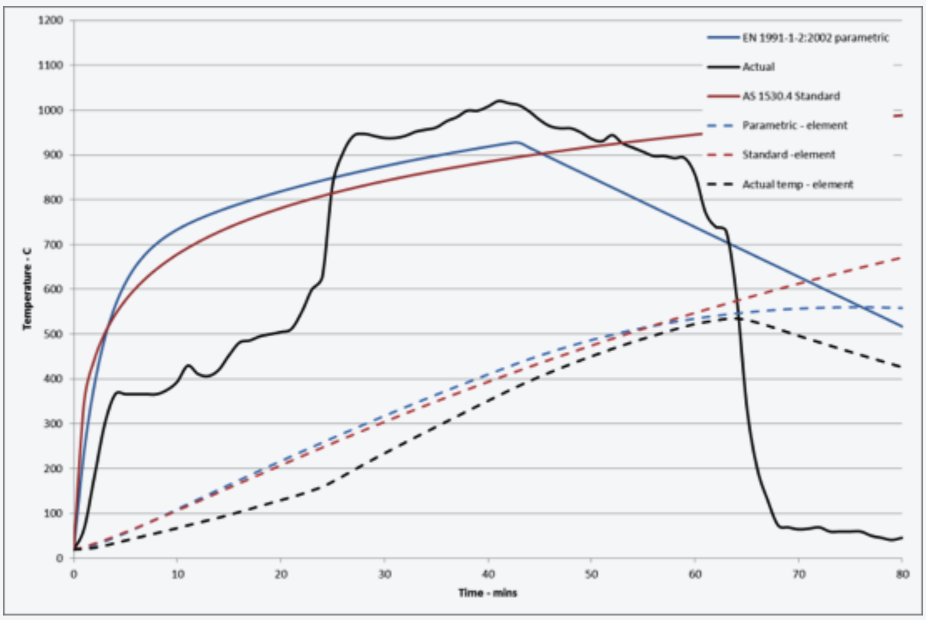


Figure H1: TF 2000 Analysis of fire severity.

Appendix I: Analysis of Fire Spread Via Concealed Spaces

I1 Frequency of Ignition in Concealed Spaces

From Section 10.2, the frequency of reported fires was estimated to be 1 x 10^{-3} fires/apartment/ year, of which 18% were estimated to be potential flashover fires.

The following fire start rates in concealed spaces were derived in Appendix F3, based on data from single dwellings for which the NCC requires fewer fire safety precautions than mid-rise buildings:

- 0.8% within a wall assembly
- 2.9% in the concealed space between a roof and ceiling
- 0.6% in the concealed space between a floor and ceiling.

The large difference in the rate of fire starts between floors and roof construction may in part be due to a fewer number of two-storey (or more) single dwellings and also the greater range of building services within roof spaces. These estimates are expected to be very conservative, since Class 1 buildings have fewer controls than medium-rise Class 2 and 3 buildings designed in accordance with the NCC 2016 fire-protected timber requirements; in particular, the proposed fire-protected timber elements require non-combustible linings to be applied to the timber, whereas combustible linings are permitted in Class 1 buildings.

It was therefore considered conservative to assume approximately 0.8% of fires initiate within cavities.

A detailed investigation into cavity fires was undertaken as part of the TF2000 project in the UK[82] that estimated that approximately 0.07% of fires are initiated in structural cavities, which is an order of magnitude smaller – implying the adopted value is very conservative.

Further discussion and background information on the findings of the TF2000 project are provided in Appendix H.

I2 Frequency of Fully Developed Fire Scenario with Spread to Cavity

This scenario was considered the most critical because of the higher frequency and the risk that a cavity fire will be coincident with a fully developed fire in a building.

A multi-tiered approach was therefore adopted to address this scenario, providing a robust fire safety strategy that is not solely reliant on any one element. The following measures prevent the fire spreading to the cavity in the first place:

- fire sprinklers installed in accordance with Specification E1.5 with an estimated reliability of 92%
- fire protection linings achieving incipient spread of fire ratings of 45 minutes
- requirements for service penetrations to meet the incipient spread of fire criteria and for cavity barriers to be fitted around windows and doors to maintain the integrity of the fire-protective linings.

Taking fire brigade intervention into account, the fire protection linings were estimated to be sufficient to prevent fire spread to the cavity in about 99% of scenarios, ignoring the impact of inadequate fire penetration seals and other defects.

If the probability of service penetration installations having major faults is taken as 0.11, the reliability of the fire protection linings in conjunction with timely fire brigade intervention with respect to preventing ignition will be taken as approximately 88%.

The probability of fire spread to the cavity from a potential flashover fire would therefore be (1-0.88) x (1-0.92) = .0096 (say 0.01). If the proportion of flashover fires is taken as 18%, then only about 0.18% of fires may spread to the cavity due to flashover fires.

The above measures substantially reduce the probability of spread to the cavity to less than the frequency of fires initiating within the cavity.

I3 Ignition Sources within Cavity

The TF 2000 analysis indicated that where the cavity construction material is the material first ignited or primarily responsible for fire growth and spread, the ignition mechanism is commonly attributable to the misuse of devices such as blow torches, paint strippers or other equipment generating similar levels of heat output or sparks. Therefore, in the majority of incidents, it is likely that the fire will be observed at or close to the time of ignition and the fire brigade alerted quickly.

Although much less likely, another ignition risk is from electrical faults and overheating. The frequency of these types of fire scenario is likely to reduce with the adoption of lower energy lighting and other services.

I4 Mitigation Measures for Fires within Cavities

The following additional mitigation measures are adopted in the NCC 2016 to mitigate the effects of scenarios where fire spreads to cavities or initiates in cavities:

- Any insulation in wall and floor/ceiling cavities must be non-combustible to ensure that if insulation is provided within the cavities it will tend to limit growth and fire spread and not introduce additional hazards.
- Cavity barriers at junctions with other fire-resistant elements of construction must be provided having FRLs of at least -/45/45 to prevent incipient fire spread to adjacent structural elements if a significant fire develops within the cavity.
- Larger floor cavities are required to have fire sprinklers fitted within the cavity in accordance with the requirements of NCC Specification E1.5 which will limit growth and fire spread within the protected areas.

I5 Performance of Cavity Barriers

Exposure during fire resistance test on partition with unprotected PVC pipe penetration

A fire resistance was undertaken on a 272-mm-thick twin-framed party wall system that included a 100 mm nominal size uPVC pipe penetration without fire protection. Horizontal and vertical cavity barriers comprising mineral fibre blanket were fitted between timber structural elements at the head and base and top of the wall system. From a review of the temperature data in the report, after about 10 minutes elevated temperatures above the ignition point of timber occurred in the cavity due to collapse/ failure of the non-fire-stopped pipe. However, the impact was very localised, so the time for the mean cavity air temperature to reach 300°C was only reduced by 10 minutes, leading to a reduction in the structural adequacy under full load conditions from the expected 90 minutes to 72 minutes, at which point the load was removed and the test continued to 90 minutes. After 90 minutes of the test, the cavity temperatures were below 600°C and the fire protection linings were still in place.

It is therefore concluded that systems capable of achieving FRLs of -45/45, or comprising mineral wool strips placed under compression when installed with a minimum depth /thickness under compression of 45 mm or 45 mm thick timber, would retard fire spread to an appropriate extent.

UK study of the fire risks in Combustible Cavities – Fire Tests

Cavity barrier provisions were considered as part of a detailed analysis undertaken for the UK Department of Trade and Industry by Lavender, Bullock and Lennon[82].

An initial test was undertaken on a standard configuration that incorporated OSB sheathing, breather membrane and a vapour barrier in addition to the timber frame, with a small ignition source comprising six 100 mm x 20 mm x 15 mm sticks and 100ml of paraffin. This type of ignition could be considered to represent a typical ignition during maintenance/construction activities or a severe scenario resulting from ignition within a concealed space due to an electrical fault. An initial peak temperature of approximately 220°C occurred shortly after ignition. This was followed by a smouldering phase until, after approximately 3.5 hours, re-ignition occurred and a peak temperature of 280°C was reached before a rapid reduction in temperature.

A test method was developed to simulate a combustible cavity construction with severe fire exposure of the cavity to evaluate different cavity barriers. The fire comprised a single 18 kg timber crib of 50 mm x 50 mm sticks capable of burning for more than 60 minutes. A small amount of paraffin was used to facilitate ignition of the crib. The average temperature within the cavity below the cavity barriers was 600°C. This scenario appears comparable to the impact of a gross defect with the fire penetrating a large opening during the early stages of a fully developed fire. Four tests were reported and the results are summarised in Table I1.

Table I1:UK cavity fire simulation tests.

Test No	Description	Result
1	PVC wrapped mineral fibre cavity barrier	Prevented fire spread >60 minutes
2	Solid timber battens	Prevented fire spread >60 minutes
3	PVC wrapped mineral fibre cavity barriers including discontinuities to simulate poor workmanship	Ignition within the cavity of the top panel occurred. Temp >250°C 2 mins (unspecified location) Approximate temperatures measured 250 mm above cavity barrier (scaled from graph) 5 min after start of growth – 250°C 15min after start of growth – 300°C 60min after start of growth – 415°C
4	Proprietary intumescent honeycomb cavity barrier	Prevented fire spread >60 minutes

Systems 1 and 2 represented current UK Deemed-to-Satisfy requirements (38 mm timber and compressed mineral wool panels). The UK-approved documents require proprietary systems to achieve the equivalent of an FRL of -/30/15. These requirements are similar to the proposed Australian requirements, except that the minimum thickness of timber is 45 mm and proprietary systems are required to achieve an FRL of -/45/45, reflecting the higher FRLs specified for loadbearing elements in Australia.

Cavity fire incident after the TF2000 fully developed apartment fire test

The potential consequences of incipient spread of fire through cavities were demonstrated during the TF 2000 test series when fire spread through cavities after the initial fire test had been extinguished[81]. Steam was released from the hot structure after suppression of the apartment fire but, after approximately 2.5 hours, hot smoke was observed being released from around the living room window area and a call was made to the fire brigade. Approximately 5.5 hours later, the fire was declared to be extinguished. The long period of time for suppression to occur can be explained by the difficulties identifying the seat of the fire and subsequently gaining access to apply water. Subsequently deficiencies were identified with the installations of cavity barriers. This event provides very useful data on which to quantify the potential consequences from fires spreading to concealed cavities with poorly installed cavity barriers.

The cavity fire occurred in an external wall, which comprised a timber frame with two layers of plasterboard lining the internal face; OSB sheathing and breather membrane was attached to the opposite face of the frame. There was a cavity separating the timber frame from the external brick veneer of the wall. This arrangement represents a severe configuration, since OSB sheathing is only required if walls require bracing. At the base of the gable wall where most of the vertical fire spread took place, the base of the cavity was open over a length of 4.8m due to a previous structural test, which may have had a significant effect on vertical flame spread due to the additional ventilation.

It is therefore considered reasonable to use data from this event to estimate the consequences of a serious event where the presence of a serious fire was overlooked for a considerable period and fire spread to and through concealed cavities. The key events on the timeline are summarised below based on a review of the reports[79,80,82,83]. These vary slightly from some reported times, due to difficulties cross-referencing different time scales.

- t=0mins: Initial fire in apartment ignited
- t=64min: Fire suppression in living area
- t=150min: Temperature rise in cavity close to living room window
- t=221min: Fire Brigade called – temperature data indicates rapid fire growth within cavity of the flat above – flaming observed from a timber window frame at approximately this stage
- t=261 min: Cavity temperature in flat above flat of fire origin peaks above 700°C – temperature within flat peaks below 45°C
- t=262min: Fire Brigade withdraw from building because cracking of brick veneer observed
- t=266min: Eaves protection removed to access cavity – suppression activity occurs
- t=275min: Additional window frames removed to provide access
- t=549min: Fire brigade confirm fire under control.

The investigations concluded that the fire had spread from the wall ceiling interface in the corner of the living area through timber studs and that the horizontal cavity barriers had not been installed effectively, allowing the fire spread to occur. The fire spread from the fire floor through the floor above, effectively removing the loadbearing capacity of the external walls at these locations.

After this severe incident with spread occurring without effective intervention for several hours, it is noteworthy that the temperature rise within the flat above was of the order of 20°C and damage was restricted to the cavity, so the impact on life safety would be expected to be minimal, provided there was no disproportionate collapse.

The NCC performance requirement BP1.1 requires a structure "to be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage" among other things. To facilitate this, a Guide has been developed and the author of the guide has provided the comment included as Appendix J on the likely structural consequences of the above event. The level of damage was considered representative of severe scenarios where: ignition occurs within a cavity; the fire grows without being constrained by lack of oxygen, non-combustible insulation or fire sprinklers within the cavity; and the cavity barriers fail, allowing spread to an adjoining element.

I6 Summary of Conclusions drawn from UK Study into Combustible Cavities

Cavity Barrier Provisions were considered as part of a detailed analysis of the fire risks in combustible cavities undertaken for the Department of Trade and Industry UK by Lavender et al.[82] The major conclusions drawn from the project were:

- Statistics indicate that, as a percentage of fires attended by the Fire Brigade in any year, cavity fire events (where the cavity construction is identified as the first material ignited or the material mostly responsible for fire development) represent a very small fraction of the total. Approximately 1 in every 1400 fires or 0.07% of these fires.
- Statistics indicate no fatalities and very few injuries resulting from cavity fires.
- A review of anecdotal reports, fire investigation records and fire statistics indicates that there is no evidence at present to suggest that a rise in the number of timber-frame residential buildings will result in an increase in the number of fire casualties. This assumes that buildings are constructed in accordance with the guidance of various published statutory instruments in support of Building Regulations.
- Fire reports, investigation records and statistics indicate where the cavity construction is the material first ignited or primarily responsible for fire growth and spread. Where this is the case the ignition mechanism is commonly attributable to the misuse of devices such as glow lamps, paint strippers or other equipment generating similar levels of heat output or sparks. This misuse of equipment needs to be addressed by relevant bodies in the provision of adequate guidance.
- When properly installed, current commonly specified cavity barrier types meet the functional requirements of Building Regulations. The workmanship involved with the installation of cavity barriers has the greatest implication on the cavity barrier meeting the functional objectives of the Building Regulations.
- Irrespective of construction type and ignition scenario, cavity fires may be difficult to locate and extinguish.
- A type of timber-frame cavity construction that utilises non-combustible materials or materials of limited combustibility helps to remove/reduce the risk of significant fire growth and spread within a concealed cavity. However, it should be noted that this form of construction has its own inherent problems. Ease of construction can be problematic, including excessive damage during construction. Exposure during inclement weather throughout erection can affect the material properties of the construction if left unprotected.
- Anecdotal reports and fire investigation records indicate that the use of combustible insulation materials in external wall cavities where both leaves are of masonry construction may give rise to a situation where fire growth and spread within the cavity is significant and where the fire service could encounter significant difficulty in dealing with the fire.
- Fire Brigades possess tools to locate the seat of a cavity fire within a short space of time after arriving at the scene. However, information/ training material on the correct method of searching a building for the fire source located within a cavity needs to be disseminated for all construction types.

- The project has highlighted that there are a number of ‘toolkit’ measures that can be employed by Design/Project Teams to ensure that the functional objectives of Building Regulation B3 are met and that the risk of fire in cavities is further reduced. These are as follows:
  - The option of designing the cavity so that it is lined with non-combustible materials or materials of limited combustibility.
  - Use of tested and approved proprietary cavity barriers fitted in accordance with manufacturers recommendations and used within the limits of the stated field of application for the product.
  - Clarification of responsibility within the construction Project Team in respect of workmanship issues relating to the installation of fire protection measures such as cavity barriers.
  - Instruction of contractors by approved bodies and appropriate supervision at key stages to ensure that cavity barriers are being installed correctly and the installation is not compromised by follow-on trades.

I7 Quantification of Risk from Cavity Fires

I7.1 Fires Spreading To Cavities of Fire-protected Timber Elements

To quantify the risk of fire spread through cavities it is necessary to establish the probability and consequences for the potential scenarios.

Fire spread to the cavity could result from:

- ignition of the timber structural elements due to heat penetration through the fire protection linings, in the event of a severe fire coinciding with slow fire brigade intervention
- inadequately fire-protected service penetrations
- gross defects in the fire protection linings leading to premature exposure of the structural frame.

In all the above scenarios, if the mitigation measures required by the NCC Deemed-to-Satisfy Provisions are effective, fire spread via the cavity will not occur and the damage will be restricted to a single element. The consequences from such events were inherently taken into account in the Monte Carlo analysis of apartment fires.

In some instances, the provision of non-combustible insulation and sprinkler coverage in larger cavities will be sufficient to prevent fire spread without reliance on cavity barriers but, for the purposes of this analysis, it will be assumed that fire spread will occur if the cavity barrier installation has serious defects. The probability of serious defects occurring in a cavity barrier will be based on the estimates for service penetrations (i.e. 0.11).

The percentage of fires spreading to cavities based on the preceding analysis is approximately 0.18% of fires and hence the percentage of fires spreading through cavities to adjacent structural elements would be 0.18 x 0.11 $\approx$ 0.02%. This equates to a frequency of 2 x10^{-7}/apartment /annum or for the subject building with 42 apartments 8.4 x 10^{-6} /annum.

The outcomes or consequences of this scenario are expected to be broadly similar to the event after the TF2000 fully developed fire test (described in Section I5) if there is no effective fire brigade intervention for several hours.

The inherent fire resistance of a loadbearing wall with gross defects has been estimated to be approximately 22 minutes. Based on the test results described above, the cavity barrier would be exposed to temperatures in the range of 400 to 600°C due to shielding of residual boards and, if there are faults with the cavity barrier at the same time, cavity temperatures are unlikely to exceed 450°C locally to the fault and would reduce considerably over the area of the partition. It is therefore considered very unlikely that the ring beam and partition in the apartment above would fail prior to fire brigade intervention. A 10% probability of significant damage to the above partition and the ring beam above will be assumed.

Under these circumstances, sufficient time would be expected to be provided for evacuation of occupants most at risk and disproportionate collapse is still unlikely to occur.

Based on this discussion, it is estimated that the frequency of fire spreading to adjacent compartments via cavities and breaking out or causing a major structural collapse is of the order of 8.4 x10^{-7} /annum (i.e. approx 1 x 10^{-6} fires per annum). If this occurs, the risk to life is expected to be low since the onset of untenable conditions and collapse would be slow; providing time for search and rescue and evacuation. If a major structural failure was to occur, the failure would be expected to be localised if the building is designed to resist disproportionate collapse and – considering the low probability of the event and number of primary fire safety systems required to fail for this outcome to eventuate – the losses were considered to be consistent with the probability of occurrence.

I7.2 Fires Initiating in Cavities of Fire-protected Timber

Based on the results from the fire tests performed in the UK with relatively small ignition sources consistent with maintenance activities, the probability of fire spread within the cavity is small and the rate of fire growth is also expected to be slower than the fire scenarios considered in I7.1. It is therefore considered likely that most fires occurring during maintenance activities would be suppressed either by occupants or the trades responsible, or fire fighters. Assuming approximately 95% of the small fires self-extinguish or are suppressed by the occupants or fire brigades before threatening structural damage, the frequency of fires growing to such an extent that significant structural damage could occur would be $1 \times 10^{-3} \times 0.8/100 \times 0.05 \times 46$ fires/annum (approximately 2×10^{-5} fires per annum).

Since these fires would be initially slow growing, the fires would be expected to be suppressed prior to causing significant damage to the apartment above, or the occupants would evacuate. Therefore, the risk to occupants would be relatively low because the fire would be contained within the cavity and spread within the cavities should be retarded by cavity barriers.

Appendix J: Comment on Structural Design Implications

J

AECOM

AECOM Australia Pty Ltd
Level 9
8 Exhibition Street
Melbourne VIC 3000
Australia
www.aecom.com

+61 3 9653 1234 tel
+61 3 9654 7117 fax
ABN 20 093 846 925

12th January 2015

Paul England

EFT Consult

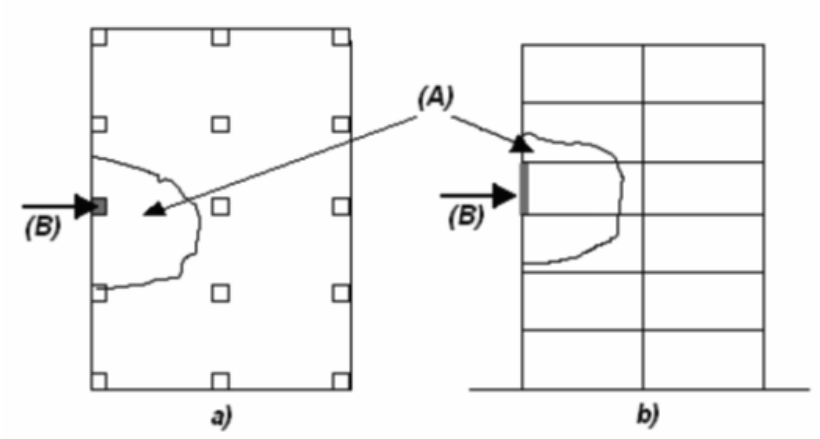
Disproportionate Collapse: Fire in Cavities

Paul,

Further to your email and our subsequent telephone conference with Boris Iskra on 19th December 2014 we provide the following letter as a summary of our approach to the disproportionate collapse of a typical stick frame building in the event of fire damage to a cavity wall system. This summary is based on the guide to structural robustness and disproportionate collapse which is currently being developed by the FWPA and AECOM.

Summary of approach to the robustness

Fundamental to the design guide is the concept of disproportionate collapse which is to say that a building should be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause. The guide is recommending to adopt the Eurocode definition of what is considered disproportionate as an area of local damage upon notional removal of a loadbearing element. The Eurocode requires that the area likely to collapse be less than $100m^2$ or 15% of the floor area (whichever is smaller), over no more than two adjacent storeys.



Key

(A) Local damage not exceeding 15 % of floor area in each of two adjacent storeys

(B) Notional column to be removed

a) Plan b) Section

Figure 1: Eurocode definition of disproportionate collapse area.

AECOM

Fire in Cavities

The following diagrams describe the cavity fire conditions assessed. Four scenarios are presented with increasing areas of fire damage and the likely structural response is described.

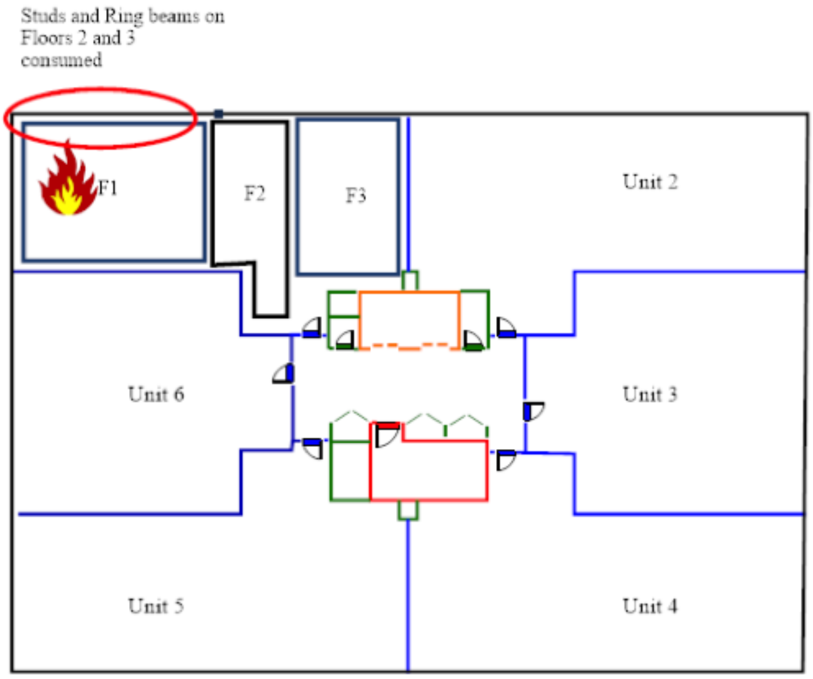


Figure 2: Plan of fire compartments

2 of 4

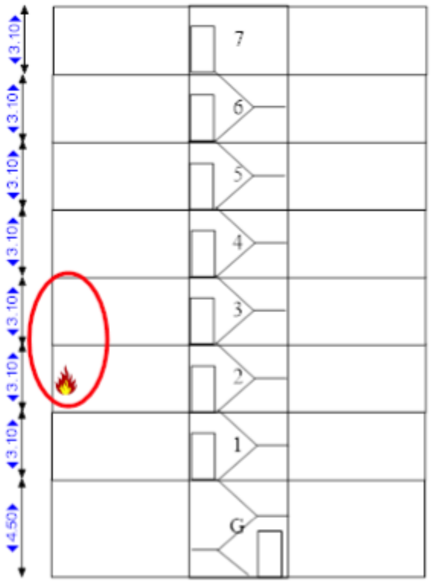


Figure 3: Sections through typical timber apartment building

The following table assumes the external wall is a loadbearing wall (fire damage to walls in the non-loadbearing direction is unlikely to cause disproportionate collapse).

Scenario	Description	Likely structural response (assuming Robust Design procedures adhered to)	Considered disproportionate?
1	Fire in lounge area of 2nd floor apartment timber stud wall frame lost but ring beam above wall frame remains substantially intact.	Loadbearing wall at level 2 ineffective but ring beam at level 3 spans over gap and distributes wall loads from above into adjacent structure. Floors remain intact	No. No floor area likely to be lost
2	Fire in lounge area of 2nd floor apartment timber stud wall frame lost and ring beam above wall frame also lost on that level.	Loadbearing walls ineffective at levels 2 and 3, ring beam at level 3 ineffective. Potential local collapse of 3rd floor. Ring beam at 4th floor spans over gap and prevents collapse of further floors above. Debris load on the 2nd floor is unlikely to exceed the design capacity of the floor and should remain intact.	No. Some area of floor lost (approx. 20m²) but not considered disproportionate to the cause
3	Fire in lounge area of 2nd floor apartment; timber stud wall frame and ring beam lost on 2nd floor and wall frame on 3rd floor lost.	As scenario 2	As scenario 2

3 of 4

AECOM

4	Fire in lounge area of 2nd floor apartment; timber stud wall frame and ring beam lost on 2nd floor and timber stud wall frame and ring beam on 3rd floor lost.	Loadbearing walls and ring beams ineffective at both levels 2 and 3. Likely local collapse at both 3rd and 4th floors. Ring beam at 5th floor spans over gap and prevents collapse of further floors above.	No. Some area of floor lost (approx. 40m²) through the loss of the loadbearing walls. Debris load on the 2nd floor of two collapsed floors to be included for in the design of the floors to prevent further collapse.

Figure 4: Table of fire in cavity scenarios (corner apartment)

Scenario 4 is the worst design case. Two floors are likely to collapse with the removal of the loadbearing walls over two levels but the floors above should be expected to remain intact. The collapse of the 3rd and 4th floors can potentially lead to a large debris load on the 2nd floor and this should be accounted for in the design and will most likely be the governing loadcase for the floor design. The guide to robustness and disproportionate collapse will recommend the inclusion of "strong floors" which will be designed to support the additional load from debris from two floors if this scenario is considered likely.

In the case of the fire in an internal wall between apartments the above table still applies. Areas of floor lost will be increased but are still likely to be less than 15% of the total floor area. With the loss of an internal loadbearing wall there is also the possibility that other alternate loadpaths are present through hanging of floors or other continuous spanning members and this will only serve to increase the robustness of the structure.

Should you wish to discuss any of these matter further then please don't hesitate to contact me.

Sincerely,

Nick Hewson
Senior Structural Engineer
Nick.hewson@aecom.com

Direct Dial: +61 3 9653 2671

cc Boris Iskra, FWPA

K

Appendix K: Class 5 Office Analysis

K1 Relevant NCC Definitions

Definition of Class 2 Buildings

Class 2: a building containing 2 or more sole-occupancy units each being a separate dwelling

Definition of Class 3 Buildings

Class 3: a residential building, other than a building of Class 1 or 2, which is a common place of long term or transient living for a number of unrelated persons, including –

(a) a boarding house, guest house, hostel, lodging house or backpackers accommodation; or

(b) a residential part of a hotel or motel; or

(c) a residential part of a school; or

(d) accommodation for the aged, children or people with disabilities; or

(e) a residential part of a health-care building which accommodates members of staff; or

(f) a residential part of a detention centre.

Definition of Class 5 Buildings

Class 5: an office building used for professional or commercial purposes excluding buildings of Class 6, 7, 8 or 9

Definition of Sole-occupancy unit (SOU)

Sole-occupancy unit means a room or other part of a building for occupation by one or joint owner, lessee, tenant, or other occupier to the exclusion of any other owner, lessee, tenant, or other occupier and includes –

(a) a dwelling; or

(b) a room or suite of rooms in a Class 3 building which includes sleeping facilities; or

(c) a room or suite of associated rooms in a Class 5, 6, 7, 8 or 9 building; or

(d) a room or suite of associated rooms in a Class 9c aged care building, which includes sleeping facilities and any area for the exclusive use of a resident.

K2 Comparison of Critical Parameters for Class 2, 3 and 5 Buildings

Review of Performance Requirements

See Appendix D for a review of relevant performance requirements.

Function and Use of the Building

The differences in the function and use of Class 2 and 3 buildings compared to Class 5 buildings have been considered in the following sections.

Fire load

Table K1 compares the design loads specified in typical codes and verification methods with the outcomes of a literature review undertaken by Ocran. It is noteworthy that the fire load for offices stated in Eurocode 1 is approximately 54% of that for dwellings, but in the NZ verification method the design value for offices is 200% of the design value for dwellings. Ocran's review of surveys of office fire loads found large variations between studies (mean values between 348-1321MJ/m^2; however, a recent survey reported in 2012 indicated a mean fire load of 557 MJ/m^2 – possibly because of the trend towards open-plan offices and less dependence on paper records and hard copy publications.

Table K1: Comparison of fire loads.

Survey/Design Code	Mean Fire Load Density or Design Fire Load Density MJ/m2		
	Dwelling (incl. Class 2)	Hotel (Class 3)	Office (Class 5)
Eurocode1 Parts 1 and 2[22]	780	310	420
NZ verification Method C/VM2[24]	400	400	800
Ocran[23] (Range from lit review and studies)	370-550		348-1321
2012 study referenced by Ocran for offices			557

The analyses for Class 2 and 3 buildings assumed a mean value for the fire load of 500MJ/m^2 with a standard deviations of 150MJ/m^2.

A sensitivity study was undertaken as part of the Class 2 and 3 analyses for a range of mean fire loads from 300 to 780 MJ/m^2 (as shown in Table K2) and the relative results were not found to be sensitive to these variations. Therefore, subject to other factors being considered, the fully developed fire modelling obtained from the Class 2 and 3 building analyses can be considered generally applicable to Class 5 buildings.

Table K2: Fire loads used in report EFT2858NCCSupplement 1-3 for sensitivity analysis.

Fire Loads	Fire Load MJ/m^2	Standard Deviation MJ/m^2	95 percentile MJ/m^2	Min MJ/m^2	Max MJ/m^2
Low sensitivity	300	90	448	100	unlimited
Design Value adopted for Class 2 and 3 buildings	500	150	747	200	unlimited
High sensitivity	780	115	970	200	unlimited

Potential fire intensity

Class 5 buildings extend the range of room geometries beyond that typical of Class 2 and 3 buildings, with potential for large open-plan offices and large length-to-width ratios for some office configurations, and there could be corresponding changes to ventilation conditions. However, significant numbers of offices will have configurations similar to those considered for Class 2 and 3 buildings. As noted above, the fire load will be similar to the range of values considered in the sensitivity analysis for Class 2 and 3 buildings. Ocran[23] identified that surveys indicated the majority of the fire load in office buildings comprised cellulosic type materials that would be expected to release volatiles at a slower rate to plastic materials, tending to extend the fire duration but reduce the peak burning intensity if a fire is not ventilation controlled.

Bennetts et al.[83] described a number of full-scale fire experiments with typical office furnishings and contents. Table K3 summarises the results from tests performed without sprinkler protection and with unprotected steel beams mounted below concrete slabs and shielded by non-fire-resistant suspended ceilings – typical of those used within office buildings. The exposure of the steel beams is expressed as an equivalent fire-resistant period, calculated in accordance with empirical correlations for unprotected steel included in Section 12 of AS 4100. An effective heat of combustion of 18 MJ/kg was used to convert the fuel load from mass to energy per unit floor area.

Table K3: Fire intensity/exposure data from office fire test experiments.

Element	Fire Load kg/m²	Fire Load MJ/m²	Max enclosure temp. °C	Ceiling tiles	Beam ESA/M m2/T	Max beam temp. °C	Calc. equiv. FRL min
Single office 1989 (4 m x 4 m)	45	810	1,100	Mineral fibre	29.3	390	10.2
140 Williams St Test 4 (12 m x 12 m area)	50	900	1,200	Plaster	26.6	530	15.1
8.35 x 3.37 enclosure	-	-	1,163	Plaster	26.6	400	11.1
Timber crib fire load (12 m x 12 m)	46	828	1,000	Plaster & mineral fibre	19.9	530	18

From Table K3 it can be observed that peak enclosure temperatures range from 1,000 to 1,200°C, which is within the range of peak enclosure temperatures previously analysed for Class 2 and 3 buildings. The exposure of the steel beams shielded by non-fire-resistant ceilings ranged from 10 to 18 minutes. For the Class 2 and 3 building analysis, non-fire-resistant ceilings were assumed to provide a contribution of approximately 10 minutes to the FRL. On this basis, the equivalent FRL exposure period for the tests summarised in Table 4.3 would have been between 20 and 28 minutes. This is at the lower end of the range of fire scenarios considered during the analysis of Class 2 and 3 buildings.

It is therefore reasonable to conclude that the potential fire intensity would not be greater than the previously analysed Class 2 and 3 buildings.

Fire hazard

The fire hazard associated with Class 5 buildings is substantially less than that associated with Class 2 and 3 buildings, with the frequency of fatalities and injuries from office fires being so low in Australia it is difficult to draw firm conclusions other than the hazard is very much lower than that associated with dwellings.

Bennetts et al.[83] considered the much larger US data base over the period 1983 to 1991 to characterise the fire hazard associated with office buildings but indicated that it showed similar trends to Australian data.

Table K4 shows data relating to the number of fires and fatalities from the Bennetts et al. report together with the calculated number of fatalities per 100,000 fires. Using probability of fire occurrence and growth (per m²/year) data derived by Fontana et al.[84] based on a survey of 40,000 fires in Switzerland, the fatality rate /m² values for residential buildings has been compared to that for offices.

Table K4: Comparison of fire hazard in residential buildings compared to office buildings.

Parameter	1 and 2 Family Dwellings	Apartments	Offices
Number of fires	1,519,848	375,551	27,679
Number of civilian fatalities	13,036	2,844	31
Civilian fatalities/1,000 fires	8.6	7.6	1.1
Probability of fire occurrence x 10-6/m²/year	33.3	33.3	10.6
Comparative risk to life /m² of floor area	24.1	21.2	1.0

Therefore, the risk to life/m² of floor area due to fire in residential buildings is expected to be of the order of 20 times higher than that of office buildings.

Recent analyses from the US undertaken by Campbell[85] for the period 2007–2011 are consistent with the above statistics indicating an average of 3,340 fires in office properties per year, 44 civilian injuries and 4 civilian fatalities.

Both Campbell and Bennetts et al. identified that fewer than one-third of fires occur outside working hours, but these fires accounted for about two-thirds of direct property damage. Bennetts et al. also identified that fires outside working hours also accounted for about two-thirds of civilian fatalities.

Other findings by Campbell were:

- 29% of fires were caused by cooking equipment but only accounted for 6% of property damage.
- 10% of fires were deliberately lit but accounted for 20% of property damage.
- Electrical distribution and lighting equipment was the second largest cause of fires (12%) and caused 15% of the property damage.
- 12% of fires in office buildings started in office areas and caused 24% of the property losses.
- 2% of office fires occurred in concealed spaces including ceiling and roof spaces but accounted for 13% of direct property damage.
- 80% of fires were confined to the room of fire origin
- Sprinklers were present in approximately 33% of fires.
- Wet pipe sprinklers operated effectively 88% of the time in fires large enough to activate the equipment.
- Deaths were 62% lower in properties with automatic wet pipe sprinkler systems (due to the small sample size this result will be sensitive to single events).
- Property losses per fire were 46% less when wet fire sprinklers were present.

Similar trends were identified by Bennetts et al. who also identified the following:

- Flame spread was limited to the area or object of fire origin 70% of the time during normal working hours compared to 48% during non-working hours.
- Early intervention of occupants was inferred by the reduced activation rates for both sprinklers and detectors during normal working hours.
- 50% of victims appeared to be intimately involved with the fire start.
- Liquid fuels were involved in 42% of fires in which fatalities occurred (mostly incendiary fires).

Based on the above discussion, it can be concluded that the overall fire hazard is substantially less in Class 5 buildings compared to Class 2 and 3 buildings, but there are differences in the nature of the fire hazard associated with office buildings that will be taken into account when applying the findings from Class 2 and 3 buildings to Class 5 buildings.

Height of the building/number of storeys

There were no changes to the height of building/number of storeys adopted for the Class 2 and 3 studies.

Proximity to other property

There were no changes to the proximity to other buildings adopted for the Class 2 and 3 studies.

Active fire safety systems

The main variations between the active fire safety systems between the Class 5 building compared to the Class 2 and 3 buildings previously analysed were:

- Internal hose reels were provided in Class 5 buildings in addition to fire extinguishers.
- Smoke hazard management provisions comprised an automatic fire detection system for the control building and an automatic sprinkler system for the subject timber buildings (without supplementary detection systems).

Size of fire compartment

Individual SOUs in Class 5 buildings cannot be considered to comprise fire compartments, as is generally the case with Class 2 and 3 buildings, and in many instances the entire floor of a building will make up a fire compartment. For the subject building with a single stair, the floor area of a typical fire compartment may therefore increase from $100m^2$ for Class 2 and 3 buildings to approximately $600m^2$. Larger compartment areas may occur in buildings with more than one fire-isolated stair but it was considered that a single stair represents a worse case with respect to life safety because of the reliance on a single evacuation path. The potential impact on fire severity was discussed above but larger compartment sizes will also impact on fire brigade intervention, occupant response and potential protection of occupants not directly involved in the fire.

Fire brigade intervention

During the periods of occupation of office (Class 5) buildings, the fire brigade are likely to be alerted substantially before flashover by the occupants, yielding a quicker response than assumed in the Class 2 analysis. Even though the control building has a fire detection system, the alarm is not monitored, and so the fire brigade response is dependent upon notification by the occupants or public. For the timber buildings, the fire brigade will receive an automatic call from a monitoring system if the fire is large enough to activate an operational sprinkler system, and the fire is likely to have been supressed or controlled by the sprinkler system before the fire brigade respond.

Outside working hours, there may be few or no occupants in a Class 5 building, and there could be a significant delay before the fire brigade are alerted for the control building without monitored alarms. This could lead to the fire brigade having to respond to a larger fire than for typical Class 2 and 3 buildings, although the numbers of occupants to evacuate will be substantially fewer. For the timber buildings, the sprinkler system will alert the fire brigade and control or suppress the fire, if it operates successfully. This provides similar conditions to those experienced with Class 2 and 3 buildings, except for sprinkler failure scenarios, where the conditions would be similar to the control building and the fire brigades may have to face a large fire.

Other elements supported

There are no changes to the structure or requirements for protection against disproportionate collapse and methods of analysis or proximity to other buildings adopted for the Class 2 and 3 studies.

Evacuation time/travel distance

The Deemed-to-Satisfy maximum distance from any point on the floor to the fire-isolated stair for the Class 5 subject building is 20 m. A different approach is adopted for Class 2 and 3 buildings by specifying the travel distance from the door of an SOU to the fire-isolated stair and a maximum distance of 6 m is specified. If a typical apartment layout is considered, the maximum travel distance from a point on the floor of an apartment to the fire-isolated stair would be comparable to the 20 m specified for a Class 5 building. However, evacuation times for occupants of a Class 5 building are likely to be less than a comparable Class 2 and 3 building, because occupants are likely to be awake and alert in Class 5 buildings and the reduced compartmentation will improve awareness of the rest of the floor in many instances.

Occupant mobility, number and characteristics

The occupant characteristics of Class 2 and 5 buildings can be considered to be broadly representative of the Australian community, with a diverse range of capabilities. Similar responses would be expected, except that occupants in Class 5 buildings would be expected to be alert, less likely to be under the influence of alcohol and drugs and more likely to assist others to evacuate and have undergone emergency evacuation training.

The analysis of Class 3 buildings considered greater proportions of occupants requiring assistance of fire brigade to evacuate.

Table D1.13 of the NCC[86] specifies the area per person based on type of use for certain occupancies. For Class 5 buildings, an occupant density of 10m^2/person is specified. The evacuation time would therefore be expected to be comparable to the values adopted for the Class 2 and 3 analyses.

Building fire safety system

The Deemed-to-Satisfy Provisions vary between Class 5 and Classes 2 and 3 to reflect the different nature of the occupancies and, in particular, higher hazard (risk to occupants in class 2 and 3 buildings).

K3 Generic Office Building Characterisation

K3.1 Building Layout and Fire Protection Details

The same general building layout as that adopted for the Class 2 building analysis was assumed. General layout details are shown in Figures K1 through to K3.

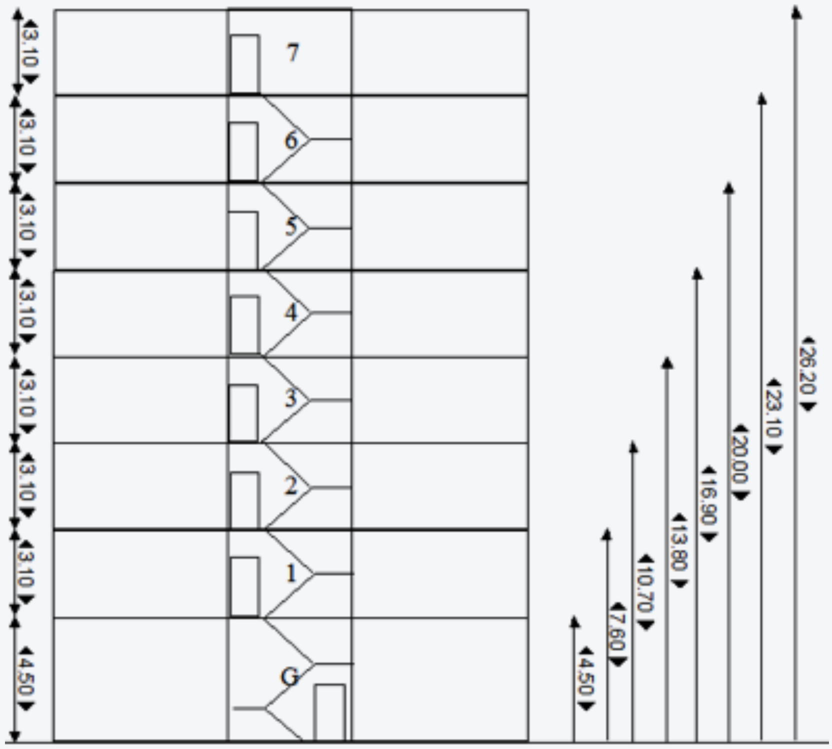


Figure K1: Vertical section through generic Class 5 building.

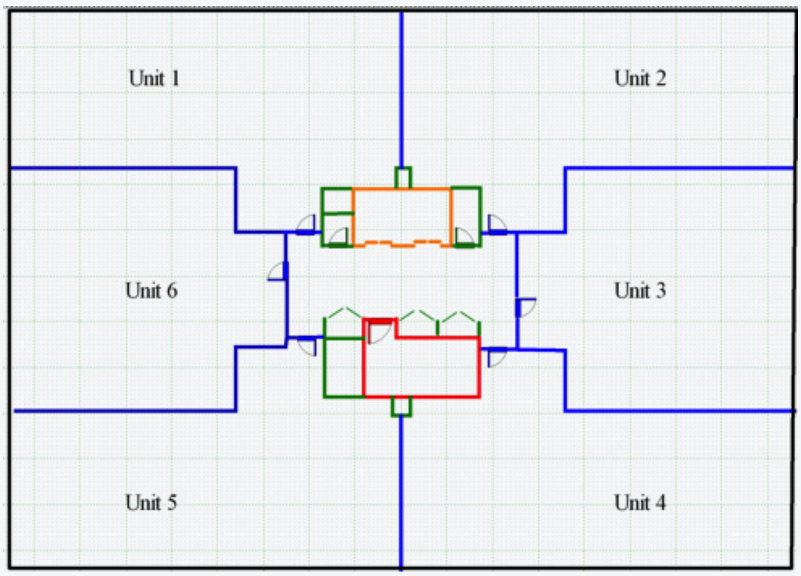


Figure K2: Horizontal section through typical upper floor of generic Class 2/3 or 5 building.

Figure K3 also shows a schematic layout of the ground floor with a typical fire indicator panel (FIP) location, fire stair access/egress, lift location and external fire brigade access.

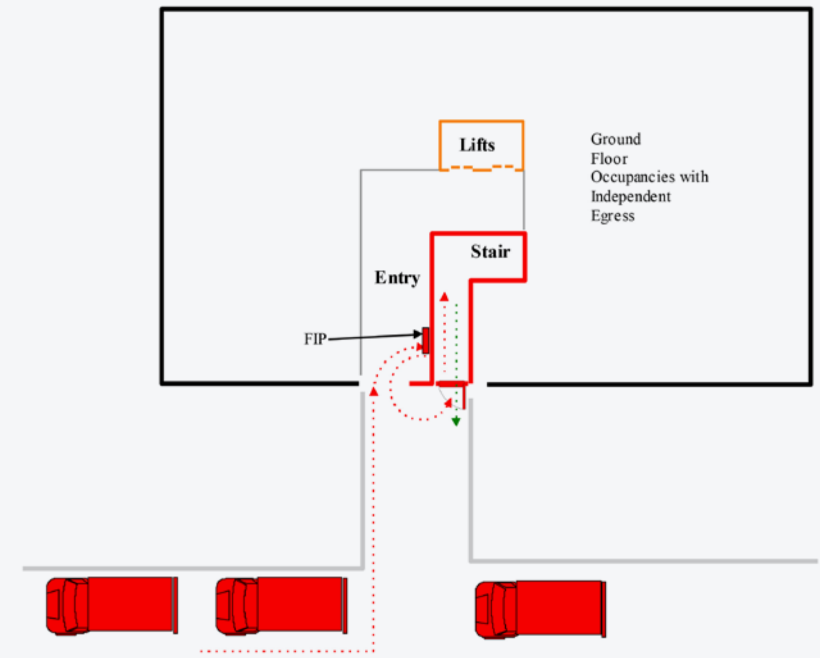


Figure K3: Ground floor plan of generic building.

Passive fire protection systems required by the NCC Deemed-to-Satisfy Provisions for the generic building are summarised in Table K5.

Table K5: Passive fire protection systems.

System	Deemed-to-Satisfy Provisions for control Class 5 building	Additional/alternative measures for timber construction
Fire resistant construction	Refer Table K6	No difference
Concrete and masonry construction	Load bearing internal walls (including shafts and fire walls)	Fire-protected timber
Non-combustible construction	External Walls Common Walls Flooring and floor framing to lift pits Non-loadbearing walls required to be fire-resisting Non-loadbearing shafts that do not discharge hot products of combustion Miscellaneous applications	Fire-protected timber
Fire hazard properties	Full compliance with Specification C1.10	No difference
Separation distances and openings in external walls	Full compliance with Deemed-to-Satisfy Provisions and non-combustible construction	Fire-protected timber in lieu of non-combustible construction

A steel-frame building has been selected for the control building since it is considered to most closely resemble the timber mid-rise buildings being considered in this report. Table K6 summarises the key elements of construction for the control and subject buildings.

Table K6: Passive systems FRLs for comparative analysis between control and timber buildings.

Element	Control building	Lightweight timber frame (Subject building 1)	Massive timber (Subject building 2)
Floor/ceiling assemblies FRL 120/120/120	Concrete slab supported on steel beams. Steel beams protected by sprayed-vermiculite to provide required FRL	Fire-protected timber floor comprising either solid joists or engineered timber beams spanning between timber-framed walls. Fire-grade plasterboard facings , 3 x 16 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber	Fire-protected cross-laminated timber horizontal panels spanning between CLT walls Fire-grade plasterboard facings, 1 x 16 mm minimum used to protect timber
False non-fire rated standard plasterboard ceiling to allow service runs above for all buildings			
Columns/ loadbearing walls 120/120/120	Steel columns protected by sprayed vermiculite and clad with non- fire-grade plasterboard	Fire-protected timber-frame loadbearing walls. Fire-grade plasterboard facings, 2 x 16 mm and timber/ mineral fibre cavity barriers (-/45/45) used to protect timber	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm
Non-loadbearing walls	Lightweight steel frame protected by 16 mm fire-grade plasterboard (-/120/120 FRL)	Lightweight timber frame protected by 2 x 16 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm minimum
Lift and stair shafts	Structural steel framework protected by vermiculite non- loadbearing plasterboard shaft wall (-/120/120)	Fire-protected timber-frame loadbearing walls. Fire-grade plasterboard facings, 2 x 16 mm and timber/ mineral fibre cavity barriers (-/45/45) used to protect timber	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm on outer face of shaft and 1 x 13 mm on inner faces, minimum
Service shafts -/90/90	Solid fire-grade plasterboard (multi-layer system)	Solid fire-grade plasterboard (multi-layer system) or fire-grade plasterboard facings, 2 x 13 mm and timber/mineral fibre cavity barriers (-/45/45) used to protect timber if integrated into apartment wall	Solid fire-grade plasterboard (multi-layer system) or cross-laminated timber protected by a minimum of 16 mm fire-grade plasterboard
External wall less than 1.5 m from fire source feature **FRLs 120/120/120 and -/120/120**	Structural steel protected by vermiculite lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard	Lightweight timber frame protected by 2 x 16 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm
External wall 1.5 m to less than 3 m from fire source feature. FRLs 120/90/90 and -/90/90	Structural steel protected by vermiculite lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard	Lightweight timber frame protected by 2 x 16 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm
External wall 3 m or more from fire source feature. FRLs 120/60/30 and -/-/-	Structural steel protected by vermiculite lightweight steel studs protected by 2 x 13 mm fire-grade plasterboard	Lightweight timber frame protected by 2 x 16 mm fire-grade plasterboard and timber/ mineral fibre cavity barriers	Fire-protected cross-laminated timber vertical panels Fire-grade plasterboard facings 1 x 16 mm
Fire doors -/60/30 modern prototypes with intumescent strips			

Table K7 summarises the active requirements for the generic Class 5 building.

Table K7: Active fire protection systems.

System	Deemed-to-Satisfy Provisions for control Class 5 building	Additional/alternative measures for timber construction
E1.3 **Fire hydrants**	Internal fire hydrants in accordance with AS 2419.1 provided for each storey	No difference
E1.4 Fire hose reels	Required for a Class 5 building (Not required in Class 2)	No difference
E1.5 Sprinklers	Not provided	System provided in accordance with Spec E1.5 (AS 2118.1)
E1.6 Portable fire extinguishers	Provided in accordance with Table E1.6 and AS 2444	No difference
E1.8 Fire control centre	Not required – building less than 25 m effective height	No difference
E2.2 Smoke hazard management **(independent exit from parts of other classes therefore no stair pressurisation required)**	Building-wide fire detection/alarm system in accordance with Spec. 2.2a.- Activation of any detector will raise alarm throughout the building	Sprinkler system provided throughout Activation of any head will raise alarm throughout the building
E2.2 System monitoring	No monitoring	Monitored with automatic notification of fire brigade

Occupant characteristics

The occupant characteristics will be identical for the timber (subject) buildings and control (Deemed-to-Satisfy Provisions).

Emergency exit provisions

Emergency exit Provisions are in accordance with the NCC Deemed-to-Satisfy Provisions and are shown in Figure K1 to Figure K3. Maximum travel distance to fire-isolated stair from any point on the floor must not be greater than 20 m.

K4 Analysis of Class 5 Buildings

K4.1 Overview

An analysis was undertaken to compare the fire performance of Class 5 mid-rise timber buildings satisfying the NCC Deemed-to-Satisfy Provisions introduced in the 2016 edition to a control building of non-combustible construction required by the Deemed-to-Satisfy Provisions.in earlier editions of the NCC.

The analysis of Class 5 buildings used the results of the analysis of Class 2 and 3 buildings where appropriate.

The fire risk in office occupancies is very small when compared to residential occupancies as demonstrated in Table K8, which has been derived from Table K4.

Table K8: Comparative risks for residential and office properties.

Parameter	1 and 2 Family Dwellings	Apartments	Offices
Comparative number of fires	55	14	1
Comparative number of civilian fatalities	421	92	1
Comparative risk to life /m² of floor area	24	21	1

It was also observed that:

- Approximately one-third of fires occur outside working hours, but these accounted for approximately two-thirds of direct property damage and civilian fatalities.
- 50% of victims appeared to be intimately involved with the fire start.
- Liquid fuels were involved in 42% of fires in which fatalities occurred (mostly incendiary fires).
- Building fire safety systems would not be expected to impact significantly with respect to injuries and fatalities where the casualties are intimately involved in the fire start.

K4.2 Impact of Fires Within the Fire Compartment of Fire Origin

Since non-loadbearing internal walls bounding corridors and SOUs are not required to be of fire-resistant construction in Class 5 buildings, the potential impact of controls specified on the combustibility or materials used to construct fire-resistant elements on the fire growth rate and fire severity of fully developed fires within the fire compartment of fire origin will be much less than in Class 2 and 3 buildings.

Both the automatic fire sprinkler system (fire-protected timber solution) and the fire detection system (control building solution) are Deemed-to-Satisfy solutions for smoke hazard management, and therefore the impact of smoke spread within the compartment of fire origin does not require further analysis. During normal working hours, occupants are more likely to identify fires quickly, irrespective of the fire detection and alarm system, and fires outside normal working hours will tend to be more critical, as indicated by fire statistics.

Outside normal working hours, if a fire is large enough and the control building fire detection operates successfully, an automatic building alarm will sound but will not automatically call the fire brigade; whereas, if the fire is large enough and the timber building's automatic fire sprinkler system operates successfully, a building alarm will sound, the fire will be controlled or suppressed and the fire brigade will be called automatically.

Therefore, in most instances, the timber building in conjunction with automatic fire sprinklers will present a lower risk than the control building with a fire detection system, since the reliability of fire sprinkler systems is similar or greater than fire detection systems. The only exception could be a fire that is large enough to activate the fire detection system but is not large enough to activate a sprinkler system. Such a fire would present a slow onset of untenable conditions and, since occupants in office accommodation can be expected to be awake and alert, they would be provided with the opportunity to evacuate and/or raise an alarm.

This conclusion was further supported by analysis of fire data indicating that fatalities were 62% lower in properties with automatic wet pipe sprinkler systems and property losses per fire were 46% less when wet fire sprinklers were present.

It was therefore concluded that the proposed timber building, in conjunction with automatic fire sprinklers, will present a lower risk to property and people than the control building within the compartment of fire origin. For both the timber and control buildings, the risk to life would be much lower than Class 2 and 3 buildings with the largest risk being to occupants in intimate contact with the fire outside normal working hours.

K4.3 Impact of Potential Fully Developed Fires Initiating in a Fire Compartment on the Remainder of the Building and Structure

The proof of concept for fire-protected timber was demonstrated in relation to Class 2 buildings with the fire-protective coverings either preventing or delaying ignition to facilitate fire brigade intervention in the low probability event of sprinkler failure.

The results showed a large improvement in life safety, which is to be expected, since a range of mitigation measures have been taken to reduce risks associated with timber structural elements and automatic fire sprinklers have been additionally provided.

It was considered reasonable to undertake a simpler supplementary analysis for office buildings, which is described below..

Risk characterisation

Review of fire data indicated that there is a difference in risk between occupied and unoccupied office buildings. In summary, more fire starts occur during normal working hours when the building is occupied, but greater losses occur outside normal working hours when the building has very few occupants.

In occupied Class 5 buildings, occupants should be alert and awake and responsive to fire cues compared to Class 2 buildings, where occupants could be asleep.

Outside normal working hours, there is greater potential for Class 5 buildings to be unoccupied, reducing the probability of early notification to the fire brigade of a fire.

Occupants are intimately involved with the fire in about 50% of cases and building fire safety systems will have minimal impact on these casualties.

A mid-rise Class 5 timber building, in accordance with the NCC 2016 DtS Provisions, will have automatic fire sprinklers but no smoke detection or smoke alarm system; whereas, for a Class 2 building, both smoke detectors or alarms and fire sprinklers are required. The smoke detection/alarm system is provided in Class 2 buildings to activate an alarm system to alert sleeping occupants. For Class 5 buildings, the Deemed-to-Satisfy Provisions infer that a fire sprinkler system adequately addresses smoke hazard management, since occupants are expected to be awake.

Normal working hours (substantially occupied office buildings)

The Class 2 and 3 building analyses previously undertaken considered a range of fire brigade call times varying from automatic notification by fire detection systems to reliance on notification by occupants or the general public after flashover has occurred. For substantially occupied office buildings, the call time to the fire brigade would be expected to lie within the range considered with a bias towards early notification.

It is therefore reasonable to apply these results for Class 5 buildings, but the improvement in life safety for timber buildings compared to the control Class 5 buildings would not be as great as that predicted for Class 2 and 3 buildings. This is due to the low base risk levels for Class 5 buildings, largely as a consequence of an alert population compared to accommodation that has a sleeping component.

The NCC Deemed-to-Satisfy FRLs for loadbearing elements are equal to or greater than those required for Class 2 and 3 buildings, typically increasing the level from 90 minutes to 120 minutes. The impact of this on timber buildings will either be to increase the protection to timber elements (further delaying or preventing ignition altogether) and/or increasing the inherent fire resistance of a massive timber element, providing a more robust structure.

Therefore, it is considered that the mid-rise timber buildings designed in accordance with the NCC 2016 DtS Provisions for mid-rise buildings would achieve a lower expected risk to life than the control building for fires occurring during normal working hours.

Outside normal working hours (unoccupied office buildings)

If the building is unoccupied, there is a significant probability that the fire brigade will not receive a prompt call in the event of a fire unless a monitored detection or sprinkler system is provided and operates correctly. The delay could be considerable, particularly if the office building is located in an area with few passers-by to observe a major fire.

If the building is unoccupied, the critical matters for consideration are facilitating fire brigade intervention and controlling property losses.

For the timber building options, the provision of a monitored automatic fire sprinkler system will control or suppress the fire and alert the fire brigade, thus facilitating fire brigade intervention and reducing losses. For the control building with no occupants or passers-by, the fire detection system will have no effect on the fire, nor will it alert people to call the fire brigade. For the control building, a greater number of fires will reach flashover and involve a whole fire compartment. Under these circumstances, the timber buildings provide substantially better performance.

A number of stakeholders indicated the importance of considering the reliability of systems and potential for fire spread and ignition of structural members. For the Class 2 and 3 building, preliminary event tree analyses were undertaken and the results subsequently confirmed through Monte Carlo analysis.

The principal differences between Class 2 and 3 and the analysis of Class 5 buildings outside normal working hours are:

- Lower occupant numbers (nil in many cases) reducing the numbers of people exposed to risk but also delaying alarm call unless there is an automatic alarm
- Increased FRLs required for some structural members requiring greater protection and hence reducing the risk of timber members igniting and improving resistance to burnout.
- Experimental data indicating that in many instances the severity of office fires may be less than that of an equivalent 30-minute standard fire resistance test.

It was therefore considered that construction of simple event trees with estimated probabilities for key events was an appropriate method to compare the Class 5 timber buildings with the control building for the 'outside normal working hours' scenario. The table of inputs and event trees and are shown in Table K9 and Figures K4 through to K6.

Table K9: Input summaries for event trees.

Input description	Control input	Lightweight timber input	Massive timber input
Sprinkler system controls/ suppresses the fire	0 probability assigned No sprinkler system provided	0.88 probability assigned from stats. Note: automatic notification of fire brigade assumed not to happen if sprinkler system fails to control the fire	0.88 probability assigned from stats. Note: automatic notification of fire brigade assumed not to happen if sprinkler system fails to control the fire
Defects reduce FRL of 1 element	0.92 probability assigned Based on Class 2 analysis and applied to all options	0.92 probability assigned Based on Class 2 analysis and applied to all options	0.92 probability assigned Based on Class 2 analysis and applied to all options
Defects reduce FRL of more than 1 element	0.999 probability assigned assigned to all options	0.999 probability assigned Assigned to all options	0.999 probability assigned Assigned to all options
Fire brigade intervention before ignition of timber element	0 non-timber option	0.98 assigned if no defects since high level of protection provided and consideration of nature of fire load 0.5 assigned with defects since greater risk of ignition but impact of nature of fire load considered	0.9 probability assigned if no defects, since less protection provided to massive timber, but nature of fire load generally (less than 30 minute equivalent FRL) reduces probability of ignition 0.5 assigned with defects, since greater risk of ignition, but impact of nature of fire load considered
Fire brigade Intervention before potential equivalent FRL exposure	0.99 assigned if no or one defect due to high levels of passive protection; reduced to 0.5 if two or more defects	0.5 assigned – generally conservative value but also takes account of proportion of fires with fire brigade intervention before ignition of timber	0.9 assigned if no defects and 0.5 assigned for other cases – generally conservative value but also takes account of proportion of fires with fire brigade intervention before ignition of timber
Compartment withstands burnout.*	0.9 no defects; 0.8 one defect; and 0.7 more than one defect values assigned, taking into account fires assumed already suppressed by fire brigade	0.9 no defects, 0.7 one defect and 0.2 more than one defect values assigned, taking into account fires assumed already suppressed by fire brigade Lower values assumed for timber to take account of potential ignition if defects are present	0.6 no defects; 0.4 one defect; and 0.2 more than one defect values assigned, taking into account fires assumed already suppressed by fire brigade Lower values assumed for massive timber to take account of potential ignition if defects are present and lower levels of protection of timber
Fire spread/ major collapse resisted	1 assigned if one or fewer members fail; 0 assigned if more than one fails	1 assigned if one or fewer members fail; 0 assigned if more than one fails	1 assigned if one or fewer members fail; 0 assigned if more than one fails

Based on full-scale fire tests summarised in Table K3, fire exposure from the simulated office fires were equivalent to exposure to the standard fire resistance test for between 20 and 28 minutes. Therefore, even with gross defects, there would be a reasonably high probability that the compartment would resist burnout.

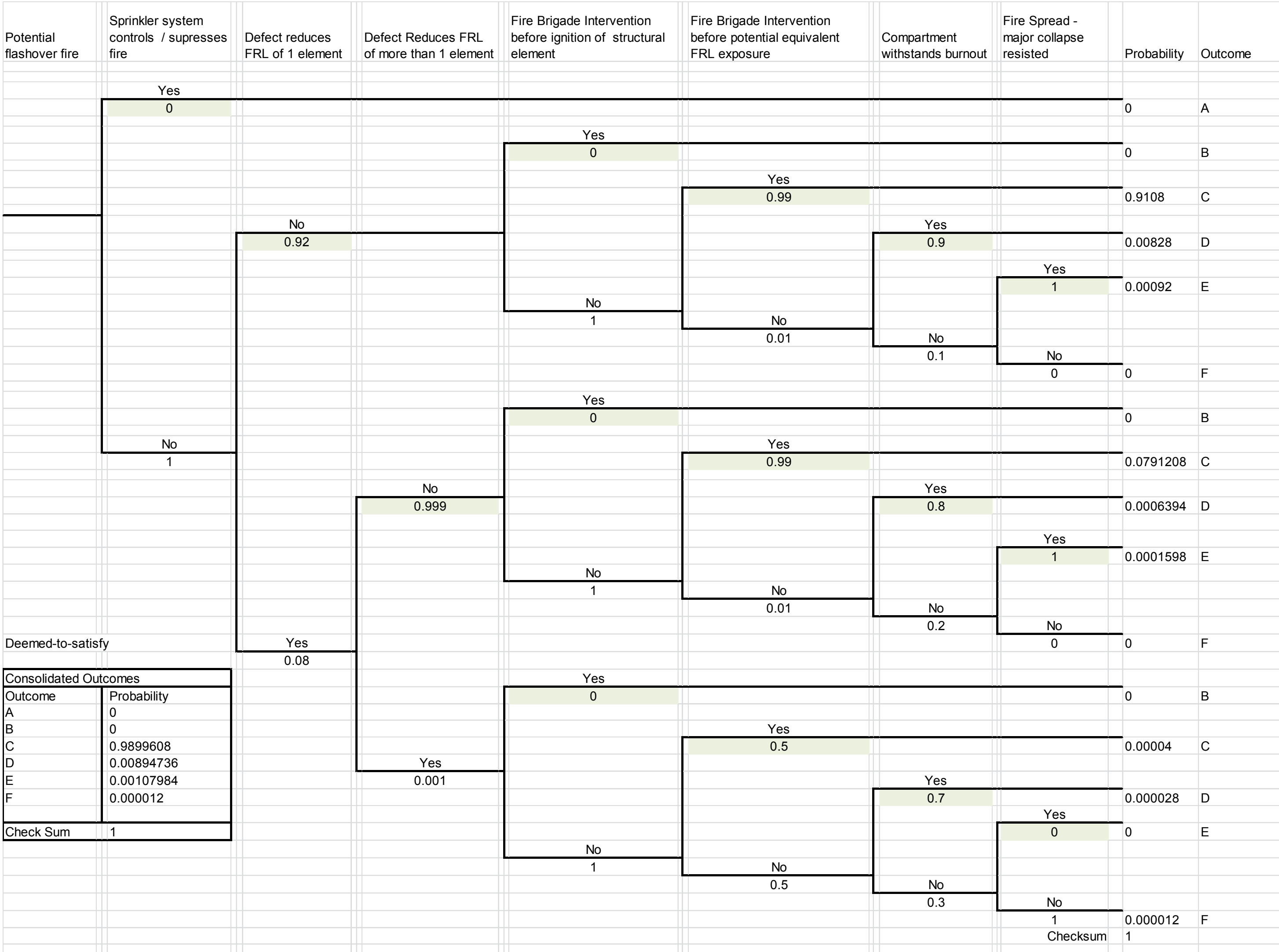


Figure K4: Event tree for control building.

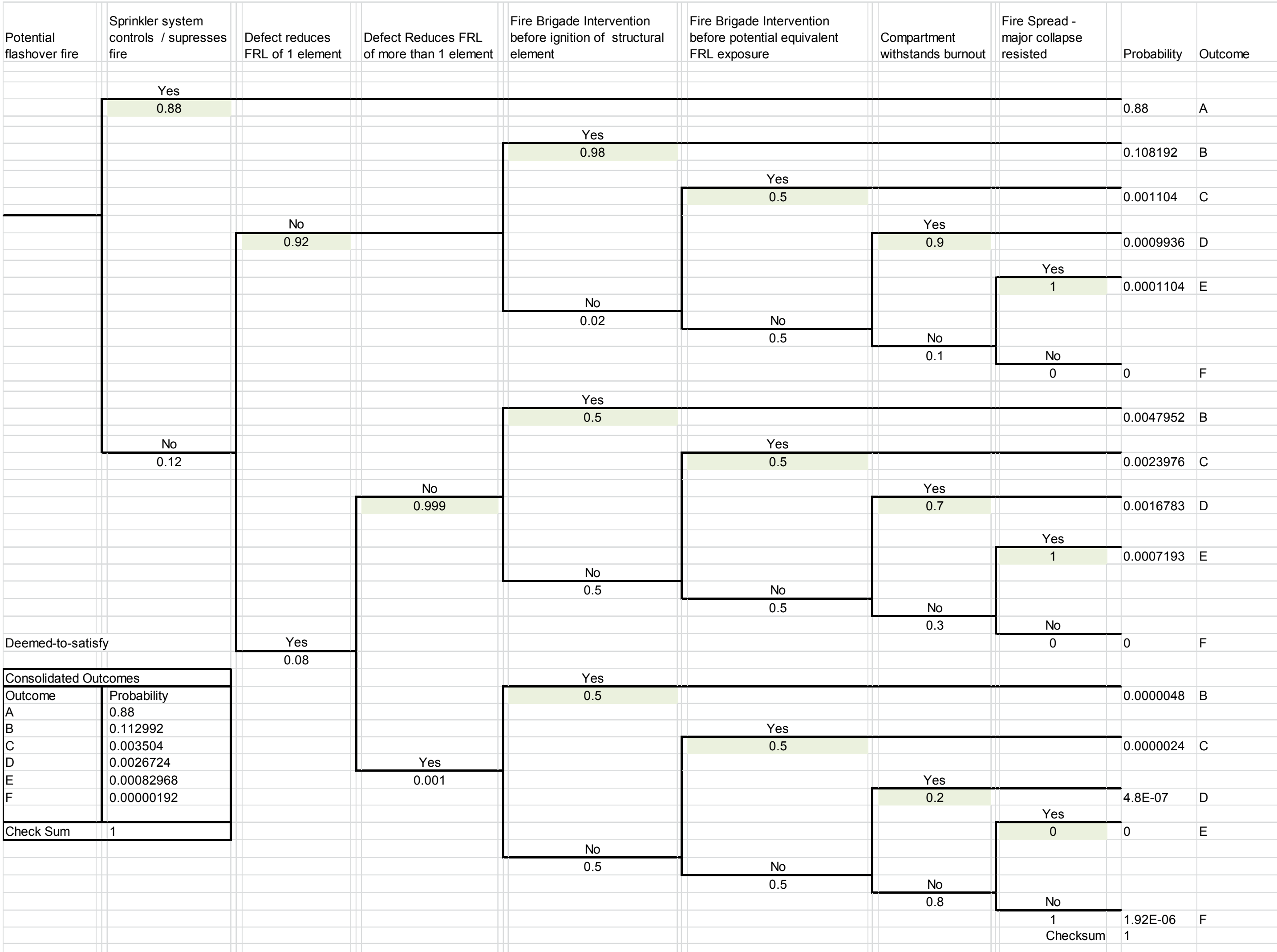


Figure K5: Event tree for lightweight timber building.

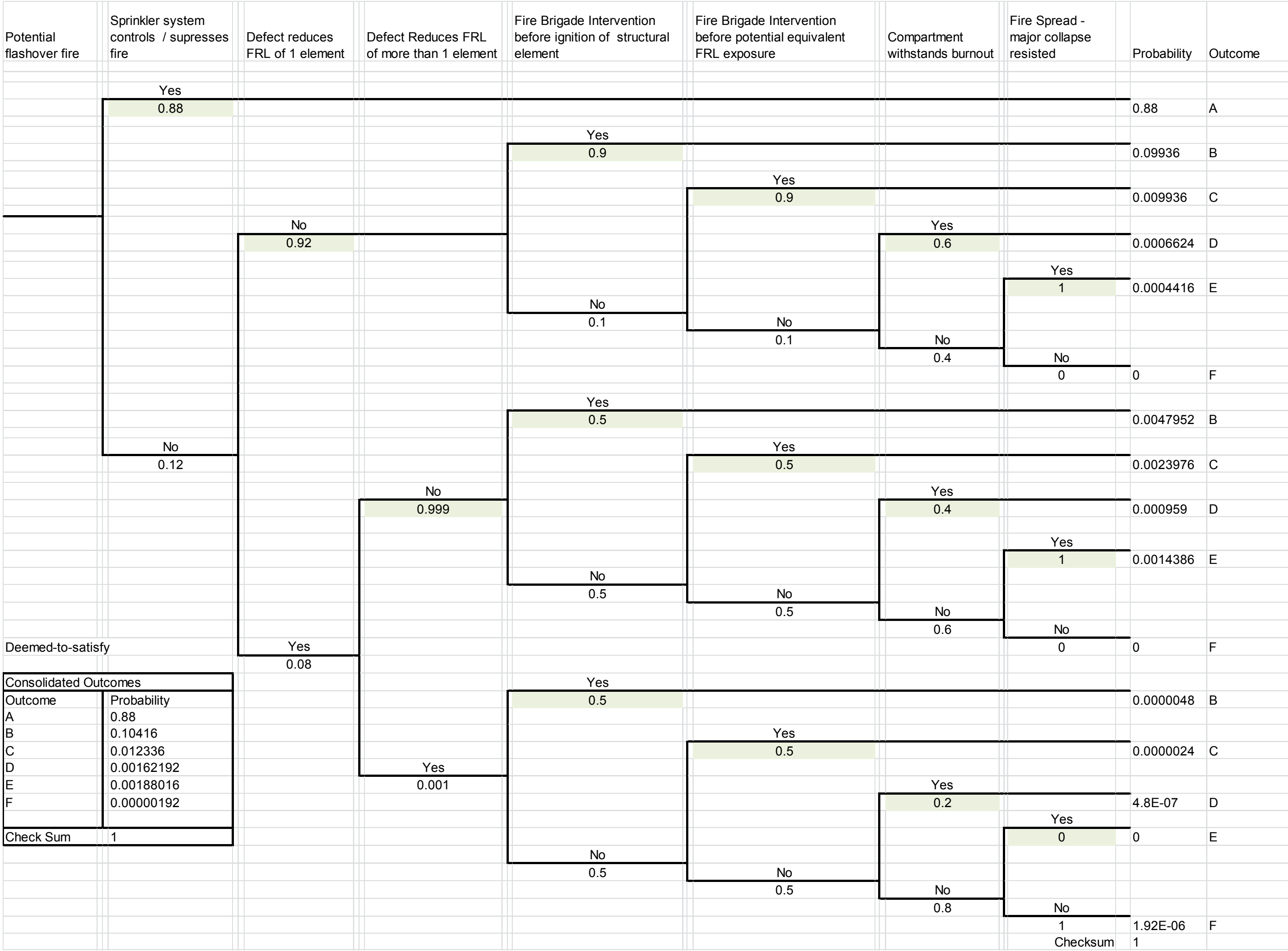


Figure K6: Event tree for massive timber building.

The results are summarised and compared in Table K10.

The sprinkler-controlled outcome represents a lower expected property loss and also minimises the risk to fire fighters, representing by far the lowest risk outcome.

Outcomes B, C and D can be viewed as being consistent with the intent of compartmentation in the NCC, i.e. containing fires within the compartment of fire origin.

Outcomes E and F occur as the result of gross defects and multiple failures of systems and, as the results confirm, are very low probability events.

Table K10: Results from comparative analysis of no-occupant scenarios.

Ref	Outcome	Probability of outcome		
		Deemed-to-Satisfy	Massive timber	Timber-framed
A	Sprinkler controlled	0	0.88	0.88
B	Fire brigade intervention before ignition of structural element	0	0.1042	0.1130
C	Fire brigade intervention before equivalent FRL period	0.989961	0.012336	0.003504
D	Compartment withstands burnout without FBI	0.00894736	0.001622	0.002672
E	Fire spread without major collapse	0.00107984	0.001880	0.000830
F	Major structural collapse	0.000012	0.000002	0.000002

Based on these results, it was concluded that the provision of automatic fire sprinkler protection in conjunction with fire-protected timber provides an acceptable level of protection.

K4.4 Impact of Fires in Fire-isolated Stairs and Passageways

The analysis undertaken for Class 2 and 3 buildings was considered valid for Class 5 Buildings.

K4.5 Fire Spread via the Façade

The analysis undertaken for Class 2 and 3 buildings was considered valid for Class 5 Buildings.

K4.6 Fire Spread between Buildings

The analysis undertaken for Class 2 and 3 buildings was considered valid for Class 5 Buildings.

K4.7 Fires in Lifts

The analysis undertaken for Class 2 and 3 buildings was considered valid for Class 5 Buildings.

K4.8 Fire Spread via Concealed Spaces

The analysis undertaken for Class 2 and 3 buildings was considered valid for Class 5 Buildings.

L

Appendix L: Peer Review Letter

SKIP

CONSULTING PTY LTD

28 April 2016

Mr Boris Iskra
National Codes and Standards Manager
Forest & Wood Products Australia
Level 4, 10-16 Queen St
Melbourne, VIC 3000

Dear Boris,

Re: **Technical Review of WoodSolutions Technical Guide #38**

Thank you for inviting us to review the document WoodSolutions Technical Guide #38 - Fire Safety Design of Mid-rise Timber Buildings which has been prepared by EFT Consulting on behalf of Forest and Wood Products Australia. We have reviewed a number of drafts of this document and provided considerable feedback to EFT Consulting with respect to technical matters and clarity of presentation. The final version of Technical Guide #38 is a significant improvement over the initial draft and has incorporated the suggested and other changes.

We understand that the purposes of Technical Guide #38 are:

(a) To clearly describe and explain the changes the National Construction Code, Volume One 2016 (NCC 2016) with respect to fire-protected timber, and
(b) To give the technical basis for these changes, and
(c) To provide guidance on the level and type of analysis required should a Performance Solution involving-fire protected timber need to be developed.

The technical basis presented in this document is the same as that described in Report No. EFT 2528 which provided the basis for the changes to NCC 2016 in relation to the use of fire-protected timber. The technical basis as presented in Wood Solutions Technical Guide #38 is both transparent, and detailed and as stated in relation to EFT 2528, is considered to provide an appropriate and adequate justification for the NCC changes.

WoodSolutions Technical Guide #38 has been structured in such a way as to assist those seeking to better understand the fire-protected timber provisions of NCC 2016 without necessarily requiring a detailed understanding of the technical basis but it also provides the detailed technical arguments, information and guidance needed by a fire-safety engineer who is considering developing a Performance Solution utilising fire-protected timber. We therefore consider that the document achieves the stated purposes.

If we can assist further, please do not hesitate to contact the undersigned.

Yours Faithfully

Dr Ian Bennetts
BE (hons), M Eng Sc, PhD
FIE (Aust) NER (civil) NER (fire safety)

FIRE SAFETY ENGINEERING AND BUILDING REGULATORY CONSULTING
P: 0438 262 400 F: (03) 5222 5672 E: consulting@skip.net.au W: www.skip.net.au
P.O. Box 397, Geelong, Vic 3220 ABN 97 123 965 079

M

Appendix M: References

1. National Construction Code. Australian Building Codes Board. Canberra, ACT, 2016.
2. National Construction Code. Australian Building Codes Board. Canberra, ACT, 2015.
3. Code of Practice; Safe Design and Structures: Safe Work Australia 2012.
4. Mass timber buildings of up to 12 storeys. *Directives and Explanatory Guide*. Government of Quebec, Canada.
5. England P, Eyre M. Extension of the concession which allows timber framed construction in Class 2 Buildings to include Class 3 Buildings. Forest and Wood Products Australia, 2011, Melbourne, Australia.
6. Hall JR. U.S. Experience with sprinklers. National Fire Protection Association, Quincy, MA U.S. 2012.
7. England JP. A framework for a risk based approach to the fire engineering design of structures. *10th International Conference on Performance Based Codes and Fire Safety Design Methods;* SFPE 2014.
8. SCRGSP (Steering Committee for the Review of Government Service Provision) 2013, Report on Government Services 2013, Productivity Commission, Canberra.
9. Apte, et al. Study Report 136 Design fires for apartment buildings – literature review; CSIRO, BRANZ, University of Canterbury, 2005.
10. Frank K, et al. A review of sprinkler system effectiveness studies, Fire Science Reviews, Vol 2; No 6; 2013.
11. Moinuddin, Thomas, Chea. Estimating the reliability of sprinkler systems in australian high-rise office buildings. IAFSS.
12. Koffel WE. Reliability of automatic sprinkler systems, Revised September 2005.
13. Marsh Limited. Fire system effectiveness in major buildings. *Research Report Number 124*, New Zealand Fire Service Commission, May 2012.
14. Moinuddin KAM, Thomas IR. A survey to estimate the reliability of fire safety components. *Engineers Australia Society of Fire Safety and SFPE Fire Safety Engineering Conference*.Charting the Course. Melbourne Australia 2009.
15. England JP, Young SA, Hui MC, Kurban N. Guide for the design of fire resistant barriers and structures. Warrington Fire Research and Building Control Commission, Victoria 2000.
16. Practice note for tenability criteria in buildings fires version 2, *Engineers Australia*, Society of Fire Safety Australia 2014.
17. Fire Brigade Intervention Model v2.2, Australasian Fire Authorities Council 2004
18. Young SA, England JP. The performance of door sets to restrict the passage of smoke when exposed to simulated fully developed fires. *Interflam*, Edinburgh 1999.
19. Supplementary Test Report. Fire Test conducted in accordance with EWFA procedure 2404400 procedure 2 Exova EWFA Report No 29685700.2 Exova Warringtonfire 2014.
20. Peacock RD, Reneke P, Forney GP. CFAST – *Consolidated model of fire growth and smoke transport (version 6) user's guide*; National Institute of Standards and Technology.
21. Hurley, Morgan. Evaluation of models of fully developed post-flashover compartment fires *Journal of Fire Protection Engineering*; Vol 15; August 2005; pp 173-197.
22. Eurocode 1: Actions on structures – Parts 1-2 General Actions – Actions on structures exposed to fire annex a parametric temperature-time curves, EN1991-1, European Committee for Standardization 2002.
23. Ocran N. Fire loads and design fires for mid-rise buildings. Thesis in partial fulfilment of requirements for a degree – Masters of Civil Engineering. Carleton University, Canada 2012.

24. Verification method: framework for fire safety design for New Zealand Building Code Clauses C1-C6 Protection from Fire; incorporating amendments 1 to 3, Ministry of Business Innovation and Employment, NZ 2013.

25. National Construction Code, Volume 1 Clause F4.2. Australian Building Codes Board, Canberra, ACT.

26. Building improvements to raise house energy ratings from 5.0 stars Constructive Concepts. Tony Isaacs Consulting, 2009.

27. Claridge, E. Assessment and validation of the Fire Brigade Intervention Model for use within New Zealand and performance-based fire engineering. Fire Engineering Report 10/4, Canterbury University, NZ 2010.

28. Report on Government Services, 2013.

29. Lennon T, Bullock M, Enjily V. DETR Framework closing report. TF2000 Stair fire test report number 200-711, Building Research Establishment, 2000, UK.

30. Bennetts ID, et al. Testing and factors relevant to the evaluation of the structural adequacy of steel members within fire-resistant elevator shafts. *Fire Safety*. 2005;40;698-727.

31. England P, Eyre M. Fire safety engineering design of combustible façades, 2011, Forest and Wood Products, Melbourne, Australia.

32. Olsson PA. Bench scale testing of light timber framed walls. *Fire Engineering Research Report 98/6*; School of Engineering, University of Canterbury, Christchurch, NZ; June 1998.

33. Tsantaridis L. Reaction to fire performance of wood and other building products – Cone Calorimeter results and analysis. Doctoral Thesis KTH – Royal Institute of Technology, Department of Civil and Architectural Engineering, Division of Building Materials, Stockholm 2003.

34. Drysdale D. An introduction to fire dynamics – Third edition. John Wiley & Sons, Ltd. UK. 2011.

35. Li X, Zang X, Hadjisophocleous G. Experimental study of combustible and non-combustible construction in a natural fire. *Fire Technology,* 51;144 -1474, 2015.

36. Hakkarainen T. Post-flashover fires in light and heavy timber construction compartments. *Journal of Fire Sciences.* Vol 20, Sage Publications, 2002.

37. McGregor CJ. Contribution of cross laminated timber panels to room fires. Thesis submitted to Carleton Faculty of graduate and post graduate affairs, Department of civil and environmental engineering, Carleton University, Ottawa, Canada 2013.

38. ISO 13785-2:2002 Reaction-to-fire tests for façades — Part 2:Large-scale test. ISO 2002.

39. Su JZ, Lougheed GD. Fire Safety Summary: Fire research conducted for the project on mid-rise wood construction. Report No A1-004377.1 National Research Council of Canada, Dec 2014.

40. Janssens, M. Fundamental thermophysical characteristics of wood and their role in enclosure fire growth (Ph.D. dissertation), Univ. Gent, Belgium (1991).

41. Janssens, M. Piloted Ignition of wood: A review. *Fire and Materials.*15;151-167 (1991).

42. Richardson J, England P. The piloted ignition of timbers exposed to radiant heat. Warrington Fire Research, Australia 2005.

43. Post Incident Analysis Report – Lacrosse Docklands, 25 November 2014. Metropolitan Fire and Emergency Services Board, Melbourne, Australia 2015.

44. White N, Delichatsios M. Fire hazards of exterior wall assemblies containing combustible components. NFPA Fire Protection Research Foundation. MA, USA, 2014.

45. Dowling VP, Ramsay CG. Building fire scenarios – some incident statistics. *Fire Safety Science Proceedings of the Fifth International Symposium* IAFFS, 1997.

46. Thomas I.R, Verghese D. Fire Risk: Summary Report, FCRC Project 4: Fire safety design solutions Part 1 core model and residential building modelling phase. *Project Report FCRC PR 01-03*. Fire Code Reform Centre 2001.

47. Annual Statistical Reports 2003/4– 2006/7. NSW Fire Brigades, Sydney, NSW.

48. Post Incident Analysis Report – Apartment/Retail Building, 26-28 Shuter Street, Moonee Ponds. Fire Date: 14 August 2007. Metropolitan Fire and Emergency Services Board 2009.

49. Babrauskas V. Ignition of wood: a review of the state of the art, pp71-88 in Interflam 2001. Interscience Communications Ltd, London, UK, 2001.

50. ISO 834-1. International Standard, Fire resistance tests – elements of building construction – Part 1. International Organisation for Standardization, 1999.

51. ASTM E119. Standard test methods for fire tests of building construction and materials. ASTM.

52. FCRC-PR 99-01. Room and furnace tests of fire rated construction. Fire Code Reform Centre, 1999.

53. White RH. Fire resistance of wood members with directly applied protection. *Fire and Materials 2009 11th International Conference and Exhibition*, Fisherman's Wharf, San Francisco, USA. Interscience Communications 2009.

54. Fang J. BNBSIR 82-2488 Fire endurance tests of selected residential floor construction. US Department of Commerce, National Bureau of Standards, Washington, US 1982.

55. Kerber S. Analysis of changing residential fire dynamics and its implications on firefighter operational timeframes. Underwriters Laboratories, US.

56. Su JZ, et al. Fire performance of houses. Phase 1. Study of Unprotected Floor Assemblies in Basement Fire Scenarios. Summary Report. NRC Canada, 2008.

57. Merretz W. Achieving concrete cover in construction. *Concrete in Australia* Vol 36 N1. Concrete Institute of Australia.

58. Exova Warrington fire test 32190200: Fire resistance loadbearing test of a nominal 272mm thick timber framed party wall system including a pipe penetration and cavity seals and tested in general accordance with AS 1530.4-2005. Exova 2014.

59. Wade CA, Gerlich JT, Abu A. The relationship between fire severity and time-equivalence. BRANZ Study Report 314. NZ 2014.

60. Kodur VK, Pakala P, Dwaikat MB. Energy based time equivalent approach for evaluating fire resistance of reinforced concrete beams. *Fire Safety Journal* Vol 45 pp211-220, 2010.

61. Milke JA. Analytical methods for determining fire resistance of steel members. Section 4, Chapter 9 *SFPE Handbook of Fire Protection Engineering,* Third Edition US 2002.

62. Dowling V. Experimental design fires for residential buildings. *10th International Conference on Performance Based Codes and Fire Safety Design Methods*; SFPE 2014.

63. Exova Report 31096300-F91769.1 Indicative fire resistance test of a section of plasterboard partition exposed to AS 1530.4 time temperature curve. Exova 2014.

64. Exova Report 31096300-F91770.1 Indicative fire resistance test of a section of plasterboard partition exposed to Hydrocarbon time-temperature curve. Exova 2014.

65. Exova Report 31096300-F91767.1 Indicative fire resistance test of a section of plasterboard partition exposed to AS 1530.4 time temperature curve. Exova 2014.

66. Exova Report 31096300-F91768.1 Indicative fire resistance test of a section of plasterboard partition exposed to Hydrocarbon time-temperature curve. Exova 2014.

67. Exova Report 31096300-F91780.1 Indicative fire resistance test of a section of plasterboard ceiling exposed to AS 1530.4 time temperature curve. Exova 2014.

68. Exova Report 31096300-F91782.1 Indicative fire resistance test of a section of plasterboard partition (ceiling) exposed to Hydrocarbon time-temperature curve. Exova 2014.

69. Exova Report 31096300-R9112.1 Indicative fire resistance test of a section of plasterboard partition exposed to AS 1530.4 Cellulosic time temperature curve" Exova 2014.

70. Exova Report 31096300-R9113.1 Indicative fire resistance test of a section of plasterboard partition exposed to AS 1530.4 Hydrocarbon time temperature curve. Exova 2014.

71. Faris A, Nadjan A, Tair A. Experimental and Numerical Study on Performance of Concrete Slabs Subjected to Severe Fire. *Fire Safety Science Proceedings of the 9th International Symposium* 2008.

72. Cooke GME. Behaviour of precast concrete floor slabs exposed to standardised fires. Fire Safety Journal Vol 36 n5 p459-475 2001.

73. Whiting P.N. A Review of international research efforts related to occupant pre-movement behaviour and response times in fire. BRANZ Study Report No 143. NZ, 2005.

74. NCC Fire Safety Verification Method 2015 Draft. Australian Building Codes Board, Canberra, Australia 2013.

75. Wales D, Thompson O. Behaviours, motivations and timescales: Towards the development of a comprehensive database of human behaviour in dwelling fires. *5th International Symposium Human Behaviour in Fire*, Cambridge, UK 2012.

76. Proulx, G. and Fahy, R.F., 1997. The Time Delay To Start Evacuation: Review Of Five Case Studies: *Fire Safety Science* 5: 783-794. doi: 10.3801/ IAFSS.FSS.5-783

77. Andree K, Bengston S.Analysis of the impact of training, communication and egress strategy in an apartment fire. *Proceedings of the 5th International Symposium on Human Behaviour in Fire*, Cambridge, UK.

78. Lennon T, Bullock MJ. The fire resistance of timber frame buildings CC1738 – Closing Report, Results and Observations. Building Research Establishment, Watford, UK, 2000.

79. Lennon T, Bullock MJ. The fire resistance of timber frame buildings CC1738 – Summary Report. Building Research Establishment, Watford, UK, 2000.

80. Building Regulations 2010 Approved Document B Volume 2 Fire safety – buildings other than dwelling houses 2006 Edition incorporating 2007 and 2010 Amendments B3 Internal fire spread (structure) HM Government 2010.

81. Lennon T, Bullock MJ. The fire resistance of timber frame buildings CC1738 – observations and implications from post-test fire spread. Building Research Establishment, Watford, UK, 2000.

82. Lavender J, Bullock M, Lennon T. Report Reference: Chilt/E01006TR Understanding fire risks in combustible cavities. Chiltern International.

83. Bennetts ID, Goh CC, Poh KW, Thomas IR. Low-rise Office Construction – A guide to fire safety. One Steel Market Mills. Sept 2006.

84. Fontana M, Favre JP, Fetz C. A survey of 40,000 building fires in Switzerland. *Fire Safety Journal* Vol32 pp137-158. Elsevier 1999.

85. Campbell R. U.S. Structure fires in office properties. National Fire Protection Association, Quincy, MA, US, 2013.

86. Table D1.13. National Construction Code Volume 1 2014. Building Code of Australia Class 2 to 9 Buildings. Australian Building Codes Board, Canberra, ACT, 2014.

WoodSolutions Technical Design Guides

WoodSolutions Technical Design Guide #1: *Timber-framed Construction for Townhouse Buildings Class 1a*. Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #2: *Timber-framed Construction for Multi-residential Buildings Class 2, 3 & 9c*. Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #3: *Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b*. Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #16: *Massive Timber Construction Systems: Cross-laminated Timber (CLT)*. Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #17: *Alternative Solution Fire Compliance, Timber –Structures.* Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #18: *Alternative Solution Fire Compliance, Façades*. Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #19: *Alternative Solution Fire Compliance, Internal Linings.* Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #20: *Fire Precautions during Construction of Large Buildings.* Forest and Wood Products Australia, Melbourne, Australia.

WoodSolutions Technical Design Guide #37: *Mid-rise Timber Buildings*. Forest and Wood Products Australia, Melbourne, Australia.

Australian Standards

AS 1720.4-2006 Timber structures – Fire resistance for structural adequacy of timber members. SAI Global 2006

AS 2118.1:1999 Automatic Fire Sprinkler Systems – General Requirements.

AS 2118.4:2012 – Automatic fire sprinkler systems – Sprinkler protection for accommodation buildings not exceeding four storeys in height.

AS 2118.6:2012 Combined sprinkler and hydrant systems in multi-storey buildings

AS 5113 Fire propagation testing and classification of external walls of buildings

AS 1530.4-2005 Methods for fire tests on building materials, components and structures – Fire-resistance test of elements of construction Standards Australia 2005

39

This guide was published in Oct 2016

Please check woodsolutions.com.au/publications to ensure you have the latest edition

Robustness in Structures

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image: Sky Believe in Better Building, Architects: Arup Associates
Photography: Simon Kennedy

WoodSolutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-37-9

Author

Nick Hewson
AECOM

First Published: October 2016

Contents

1 Introduction

As a response to structural collapses around the world, a number of international structural design codes or regulatory standards have advocated the specific consideration of robustness or avoidance of disproportionate collapse in structural design. In Australia, the National Construction Code (NCC) has always had a requirement that a structure should:

"... be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage"

Figure 1.1: NCC Clause BP1.1(a)(iii).

The interpretation of this clause has often been left up to the individual designer and rarely given much attention. Other local guidance could be found in Section 6 in the AS 1170.0 (Structural design actions, Part 0: General principles), although it deals with robustness in a relatively generic way. The supplementary commentary to AS 1170.0 gives some more information but the subject of structural robustness has remained an ambiguous and largely forgotten requirement.

The 2016 version of the NCC has given a much more explicit definition of robustness and suggest a number of ways that compliance with Clause BP1.1(a)(iii) can be demonstrated. This guide gives background on the subject of structural robustness and provides practical advice to building designers on how this new requirement can be met in a number of structural materials.

Structural robustness is about considering a variety of potential extreme events that may occur during a building's lifetime and ensuring that its design will guard against damage to the structure disproportionate to its cause. These events may be natural disasters (earthquakes, landslides, floods, etc) or man-made (vehicle impact, terrorist attack, explosions). Designing a robust structure is a key factor in designing a fire-safe structure, as fire is one of the most likely causes of accidental damage.

This guide forms part of the supporting documentation for the consideration of the fire regulations in medium-rise timber buildings. As modern engineered timber allows engineers to design ever-larger structures in wood, it is important to consider robustness in design. Most timber design codes around the world have been based on a domestic scale of construction where robustness is much less of a concern.

The guide is intended to be a best practice guide for Australian buildings and covers all structural materials, but with a particular focus on timber. It offers advice, design methodologies and details for typical building types. It is not a substitute for a detailed risk assessment to determine any likely accidental damage events that should be considered as part of the design process. It does not cover the specific requirements of large structures, such as sports stadiums, bridges, large towers and it is not intended to address buildings where there are very specific risks of certain events, e.g. terrorist attacks at government buildings or explosions at chemical plants. For these more specialist applications the appropriate expert advice should be sought.

1.1 Why Design for Robustness?

The buildings under construction today may experience changes in use, changes in environmental conditions, unforeseen events or material degradation over their lifetime, and there is a reasonable expectation that the structure should not be unduly damaged as a result. High-profile events like the World Trade Centre collapse have increased worldwide interest in the robustness of buildings and designers have a social responsibility to consider it in their work.

As buildings become larger and more pressure on designers leads to structures being designed ever more keenly there is an increasing requirement to consider robustness more explicitly. In addition, the trend towards subcontractor design for different elements of the building can mean that overall robustness of the building is lost in a fragmented design philosophy.

Figure 1.2: Ronan Point, UK – progressive collapse of one side of building due to gas explosion.

Arguably the most famous example of a collapse occurred at Ronan Point in the UK in 1968.

An explosion due to a faulty gas stove on the 18th floor of this loadbearing precast apartment building caused the flank walls in one corner to be blown out. This caused the collapse of the floors above, which subsequently overloaded the floors below and caused the progressive collapse of an entire corner of the building. Four people died and 17 people were injured. The primary fault was that the joints between the precast walls and floors were inadequate for any blast load but, more importantly, they were also not sufficiently well tied together. The entire building behaved as a stack of cards when one loadbearing element was removed.

Following this, there were a number of changes to the structural legislation in the UK. The main outcome was that buildings had to be "constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause". In addition, there were guidelines on the minimum requirements for tying building elements to one another and for various forces and pressures to be considered in a damage event.

1.2 NCC 2016

The Australian Building Codes Board (ABCB) has recognised the importance of robustness in our buildings and the 2016 edition of the NCC introduces the following clause:

BV2 Structural robustness

Compliance with BP1.1(a)(iii) is verified for structural robustness by—

(a) assessment of the structure such that upon the notional removal in isolation of—

(i) any supporting column; or

(ii) any beam supporting one or more columns; or

(iii) any segment of a load bearing wall of length equal to the height of the wall,

the building remains stable and the resulting collapse does not extend further than the immediately adjacent *storeys*; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

Figure 1.3: NCC 2016, Volume 1 – Clause BV2.

The impact of this clause is that designers need to be able to demonstrate that the structure is not overly reliant on any one particular element. The ideas of notional removal of elements and systematic risk assessments, as well as methods for demonstrating robustness, are discussed in more detail in the rest of this document. The clause shares a number of similarities with the European approach and a more detailed comparison between the two is explored later in this document. It can also give some alternative methods for demonstrating robustness.

1.3 Where AS 1170.0 Falls Short

When engineers think of robustness, they typically consider Section 6 of AS 1170.0, which deals with robustness largely by dictating a minimum lateral load to be applied to the structure (1% for buildings over 15m high or 1.5% for buildings less than this).

In the vast majority of buildings, this force will be significantly less than either the design wind loads or earthquake loads and is often neglected by the designer. The application of a minimum lateral load has as much to do with ensuring the building is able to cope with construction tolerances and column lack of verticality. Clause 6.1 uses the phrase, "Structures shall be detailed such that all parts of the structure shall be tied together both in the horizontal and the vertical planes so that the structure can withstand an event without being damaged to an extent disproportionate to that event". Neither the code nor the commentary gives much more guidance than this.

Considering robustness early in the design stages can result in a building with enhanced resistance to disproportionate collapse, without significant impact on the structure or the time required for the design.

The understanding of how to design against disproportionate collapse is well advanced in Europe, and the UK in particular, and this guide looks to their example. It takes a number of the principles of the Eurocodes and the IStructE's Practical guide to structural robustness and disproportionate collapse in buildings and discusses them in an Australian context and gives some more practical guidance on how to comply with the Section 6 of AS 1170.0 and simple ways to improve the robustness of your buildings.

2

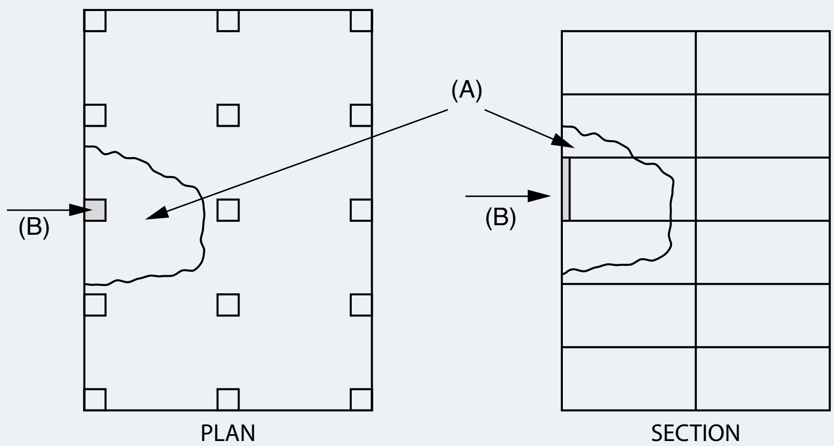
Robustness Concepts

2.1 Disproportionate Collapse

The collapse must not be excessive, considering the cause. For instance, the loss of a single wall should not cause the collapse of an entire section of the building, as in the Ronan Point example. What constitutes 'disproportionate' is a matter for consideration for the individual engineer and a judgement should take into account the probability of a collapse and the consequences of any failure. For example, a lightly trafficked industrial building should be considered quite differently to an auditorium.

The NCC requirement is that the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys. It does not provide an upper limit on the area of floor allowed to collapse, although BV2(b) places a limit of a single member carrying more than 25% of a total structure before a systematic risk assessment is required.

By comparison, the Eurocodes approach is a little more prescriptive and provides a limit of 15% of the floor or 100m^2 (whichever is the smaller) and the local collapse should be limited to not more than two adjacent storeys.



Key

(A) Local damage not exceeding 15% of floor area in each of two adjacent storeys
(B) Notional column to be removed

Figure 2.1: Eurocode definition of recommended limits for admissible damage.

In many cases, it should be possible to design and detail the building in such a way that the loss of any one supporting element does not lead to the likely collapse of any part of the structure. However, where this is impossible the designer should make their own assessment of the individual building and determine the most appropriate method to follow. While the NCC guidance could theoretically be interpreted to allow the collapse of up to 25% of a floor, this could amount to a significant loss of structure in a large building and it may be prudent to reference the European approach and limit areas of collapse to 100m^2. There are buildings where the designer may allow the entire collapse of a non-critical building (e.g. a remote, rarely used barn) or structures where no collapse is acceptable (e.g. post-disaster structures). In all cases, a conscious decision should be made and communicated to the client or certifying authorities.

Both the NCC recommendations and the Eurocodes consider the collapse of two adjacent floors to be acceptable. Although not stated in the code or associated guidance, there is the implication that the first intact floor below this level should be capable of supporting a debris load equivalent to the dead load and proportion of the design live load for these floors. This is not an assumption that should be ignored, particularly in the case of timber construction, and the concepts associated are presented in more detail in Sections 3.7 and 6.4.2. It will often prove simpler to design and detail the structure so that the collapse of two adjacent floors is prevented.

2.2 Redundancy

One of the simplest ways to improve the robustness of a structural is to design a structure that is statically indeterminate. Beams running continuously over/past supports is a simple way to ensure that the loss of one particular loadbearing element may not cause the collapse of the entire floor structure.

2.3 Insensitivity to Construction Tolerances

A large part of designing a robust structure is ensuring that it is not overly sensitive to construction tolerances, thermal movements, support settlement, etc. The minimum notional horizontal loads considered defined in AS 1170.0 will cover this in some ways but it is wise to consider the effects of tolerance in the detailing.

Consider the steel connection in Figure 2.2.

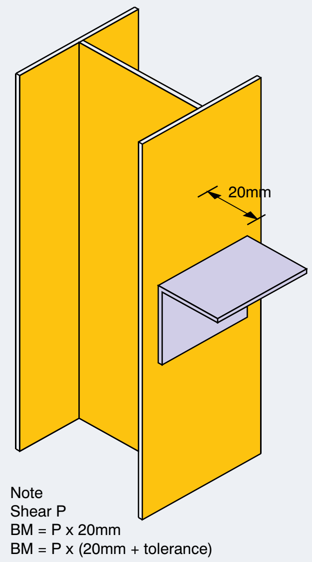


Figure 2.2: Bracket with high sensitivity to construction accuracy.

A small cantilever support bracket is designed for a column. A credible construction tolerance of 20mm will double the moment induced on the cantilever and this might be considered in the design. Clearly, if the cantilever was 1m then this tolerance is insignificant.

Consider precast beam detail in Figure 2.3.

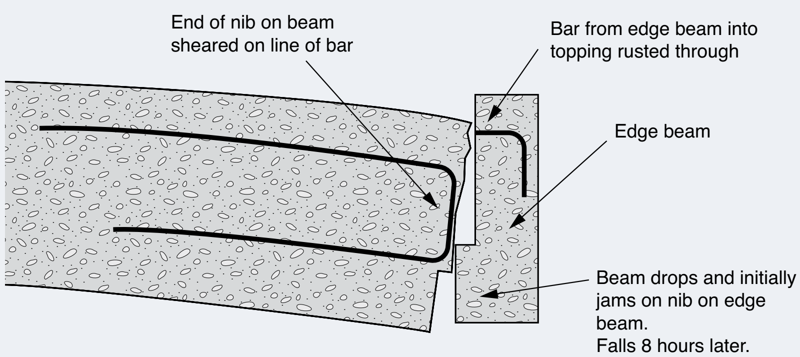


Figure 2.3: Precast connection with high sensitivity to building movement.

In this case, the precast beam is seated on a narrow corbel on the edge beam and connected with a bar at the top through an in situ joint. This detail is from a real building in the UK - the top bar became corroded through a weakness in the in situ joint and the beam became reliant on the bearing detail. The beam underwent some torsional and lateral movement and the precast slab collapsed.

Buildings should be able to accommodate these sorts of movements without disproportionate collapse.

2.4 Accidental Damage Load Case

Designing a structure for robustness involves calculating loading and member capacity for certain situations. In these accidental damage cases, it is considered acceptable to use reduced partial load factors and reduced material strength reduction factors as appropriate to each material and load as defined in the Australian Standards. In addition, the building's serviceability is no longer of concern and only the strength need be checked to ensure structure remains intact.

Given that the majority of structures are designed based on serviceability requirements, there are significant reserves of strength available in the structure before collapse is likely. The robust design of a structure shouldn't add significant – if any – costs to the structure; it is an exercise in good detailing and the engineer's clear understanding of robustness concepts.

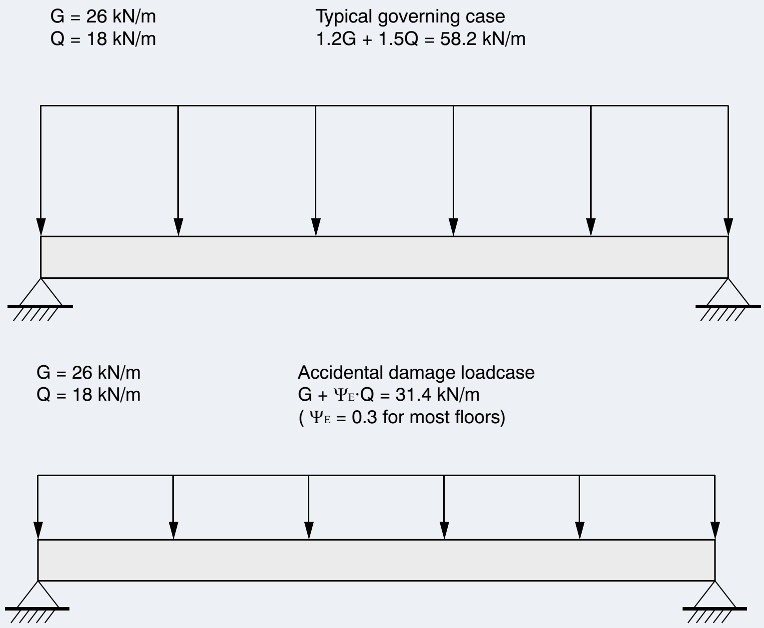


Figure 2.4: Comparison of ultimate vs accidental loads for a typical beam.

Looking at timber design as an example, in addition to the reduced loads, both AS 1720 and the Eurocodes allow you to assume a higher strength of material and a reduced load duration in the accidental damage situation. In AS 1720, the k1 factor goes from 0.8 for a medium term loading to 1 for a short-term load, effectively gaining 20% capacity. Eurocodes allow significantly more for the material strengths in an accidental design situation, with a typical nominal material safety factor increasing by up to 70%. When all these factors are considered, and with deflections no longer being a consideration, the reduction in design loads in these cases is very significant, which helps allow some very long span solutions to become viable.

3

How to Design for Robustness

3.1 Strategy

3.1.1 Building Classification

The first task is to consider the use of the building. Typically, buildings are categorised based on their size and use. Structural engineers in Australia will be most familiar with the importance levels set out in Section 3.3 of AS 1170.0. The Eurocode is based on the old UK Building Regulations designations of Classes 1, 2A, 2B and 3 in increasing consequence of failure. The two sets of classifications show a good degree of correlation so it is proposed to use the importance levels from AS 1170.0.

Consequences of failure	Description	Importance level	Comment	Eurocode Classification
Low	**Low** consequence for loss of human life, or **small or moderate** economic, social or environmental consequences	1	Minor structures (failure not likely to endanger human life)	1
Ordinary	**Medium** consequence for loss of human life, or **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels	2A
High	**High** consequence for loss of human life, or **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)	2B
		4	Post-disaster structures (post disaster functions or dangerous activities)	3
Exceptional	Circumstances where reliability must be set on a case by case basis	5	Exceptional structures	3

Table 3.1: Building classifications.

More detail on which building types fall into these categories can be found in AS 1170.0.

These classifications reflect the consequences of failure and it is fair to take a different view of the robustness requirements for an isolated industrial shed compared to that for an office building. In larger buildings, it may be sensible to consider different uses in different areas and approach robustness differently in each area (e.g. a large retail store with a restaurant may have different requirements to the attached warehouse).

Where a low classification building is immediately next to a high classification building, it may be necessary to design the low classification building to a higher standard of robustness if there is any risk of damage to the other building. This is recommended by the Eurocode where the adjacent building is within 1.5 times the height of the lower classification building.

3.1.2 Assess Extreme Event Risks

There are two basic strategies for designing a robust building; they are analogous to a deemed-to-satisfy versus a performance-based solution from the BCA. Most typical buildings without specific significant risks will be able to use the former and the building can be designed to limit the extent of localised failure. Methods for achieving this are detailed in the rest of this chapter.

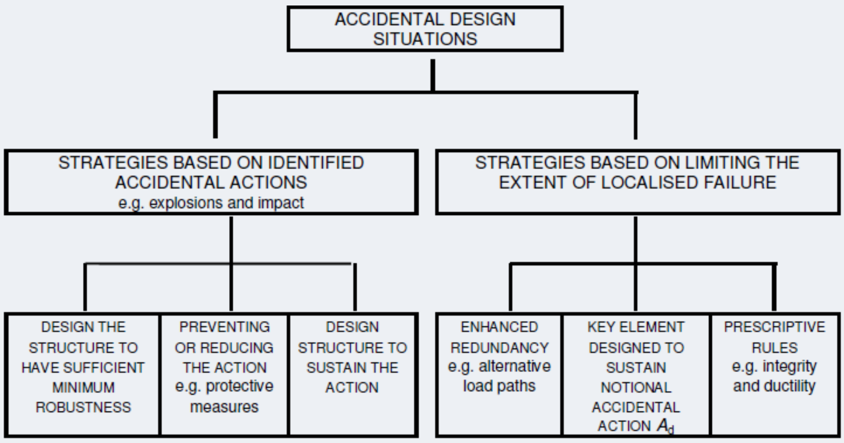


Figure 3.1: Decision flow chart for robustness design.

Buildings where there are more specific risks of certain accidental actions should be assessed in a more detailed way and look to either prevent or reduce the action. An example would be an office building built on a deck over a road. The potential risk would be a vehicle damaging a column supporting the deck putting the entire structure at risk of collapse. An appropriate response could be to move the columns away from the road edge and to place a physical barrier between the road and column to reduce the possibility of an impact. If these specific events can be reduced in likelihood then the building could be designed to comply with the relevant requirements for its class.

Particular extreme events may include (but not be limited to):

- gas/chemical explosions
- terrorist attack
- vehicular impact

Figure 3.2: Designing for disproportionate collapse is about anticipating the unexpected.

Figure 3.3: Disproportionate collapse of an apartment block in Venezuela following landslides.

- natural disaster (flood, landslide, earthquake)
- deliberate or accidental removal of a structural element
- fire.

The relationship between fire and disproportionate collapse is important; in a lot of cases it is the most likely cause of the damage of the loadbearing element. The design of a robust structure will typically also improve the performance of the building in fire.

3.1.3 Select an Appropriate Design Response

As the importance level of the building increases, the designer's response to the robustness of the structure should also increase.

Class 1

For a Class 1 building, it is possible to not specifically consider robustness, provided there is no abnormal risk of an extreme event.

Class 2

For Class 2, the building can be designed:

- with a minimum set of horizontal ties as described in section 3.4.2
- for notional element removal as described in section 3.5
- for protected elements as described in Section 3.6.

Class 3

For Class 3, the building can be designed:

- with a minimum set of horizontal ties as described in Section 3.4.2 and vertical ties as described in Section 3.4.3
- for notional element removal as described in Section 3.5
- for protected elements as described in Section 3.6.

Classes 4 and 5

Buildings in Classes 4 and 5 are outside the scope of this document and a detailed risk assessment and consideration of all potential collapse events is recommended. As a minimum they should be treated as a Class 3 building and then consider other specific risks.

For further information and guidance on the design of sensitive buildings the IStructE has produced a guide: ***http://shop.istructe.org/manual-for-the-systematic-risk-assessment-of-high-risk-structures-against-disproportionate-collapse.html***

The following flowchart details the design response to each different class.

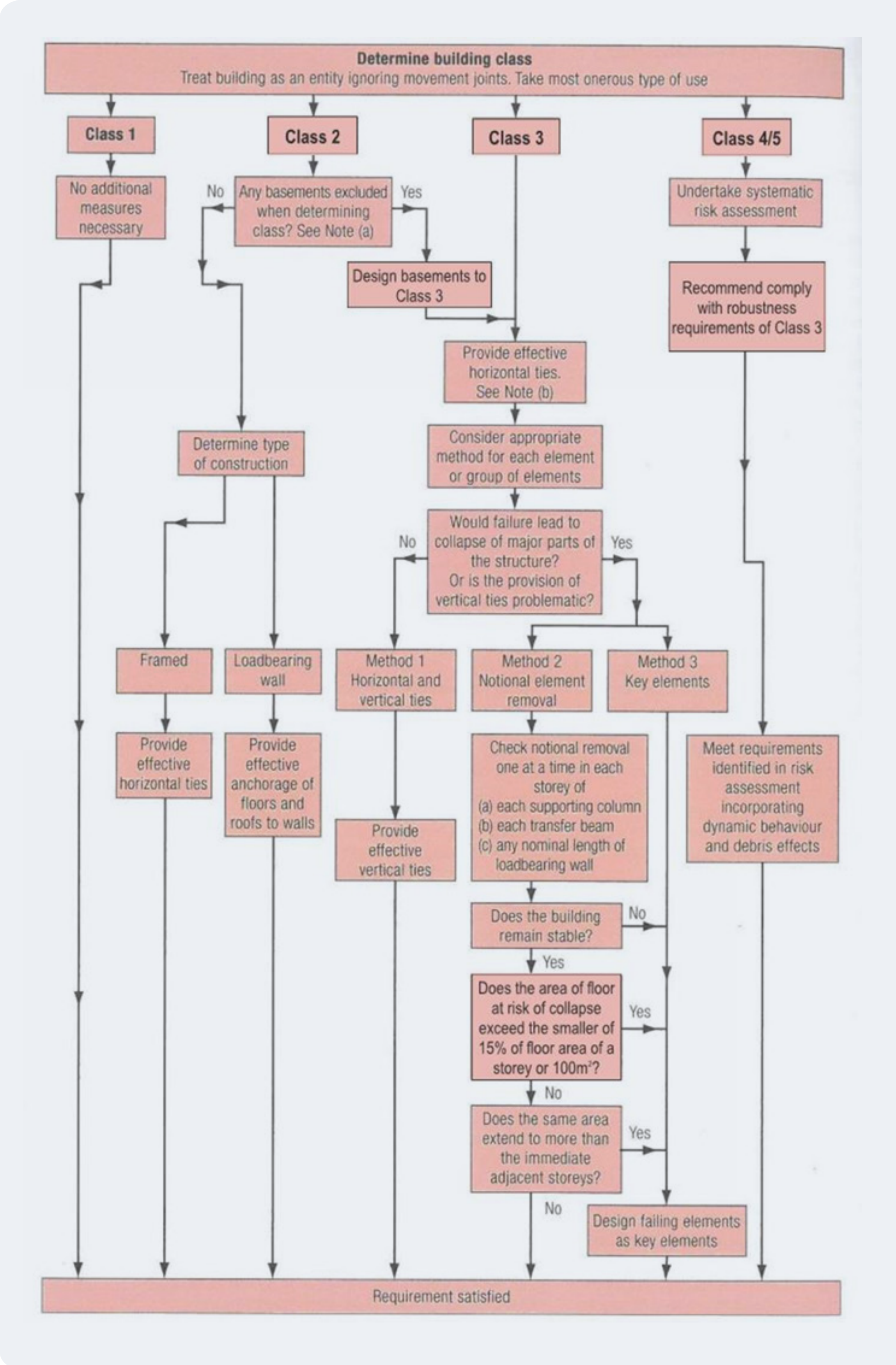


Figure 3.4: Robustness design flowchart.

3.2 Choice of Structural Form

The initial design decisions made for the structural form can have a significant effect on the robustness of a building.

3.2.1 Redundancy

Reliance on a single column to hold up a significant proportion of a building creates a potentially unnecessary dependence on a single element. Any accidental damage involving the column puts a disproportionate area of the building at risk of collapse.

Consider the following example. The central column potentially holds up the entire roof structure and heavy vehicles are constantly moving around it. Potential options to improve the robustness could be:

- place barriers around the column to reduce the chance of vehicle impact
- change the roof structure to be able to span across the building if the column is damaged
- change the roof structure from a radial structure to one spanning across the building
- design the column as a protected element to resist vehicle impact.

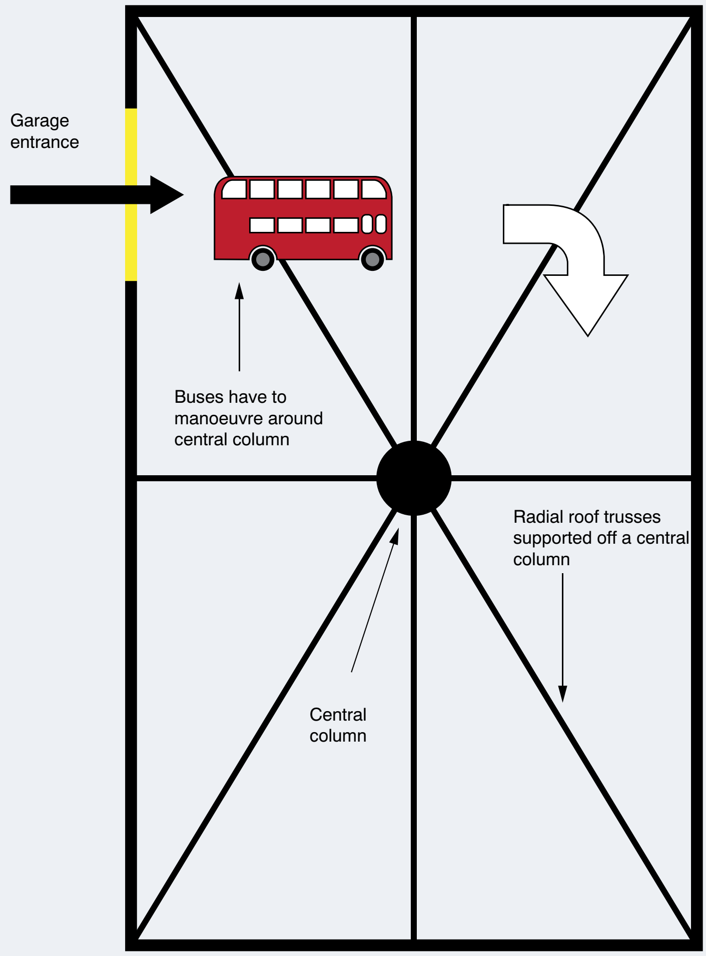


Figure 3.5: Example of a structure with a lack of redundancy.

3.2.2 Transfer Beams

Transfer beams can create a heavy reliance on a particular structural element. The Murrah Building in Oklahoma was the subject of a terrorist bombing in 1995. At the third storey there was a large transfer beam supporting the floors above where the column spacing halved. The bomb destroyed one of the lower columns and caused the transfer beam span to suddenly increase. The lack of sufficient ties caused the failure of the transfer beams and along with it all adjacent bays on the floors above.

Figure 3.6: Oklahoma City's Murrah Building after the bomb attack.

AS 1170.0 Section 6 requires that all structures are designed for minimum lateral loads equivalent to:

- 1% $(G+\psi_c Q)$for structures over 15m tall
- 1.5% $(G+\psi_c Q)$for all other structures

The origins of this load intended it to ensure that buildings that may otherwise not attract significant wind load would have some measure of resistance against unforeseen horizontal loads. In the vast majority of cases this load is likely to be significantly less than the applied wind or seismic loads and is often forgotten by most designers.

The Eurocode maintains the application of notional horizontal loads as a requirement but its rationale is based on it accounting for lack of tolerance in the building construction, particularly the out-of-plumbness of columns. It can be shown that the lateral load generated by a column with an out-of-plumb tolerance of 1/200 is equivalent to a lateral load of 0.5% of the vertical load in the column.

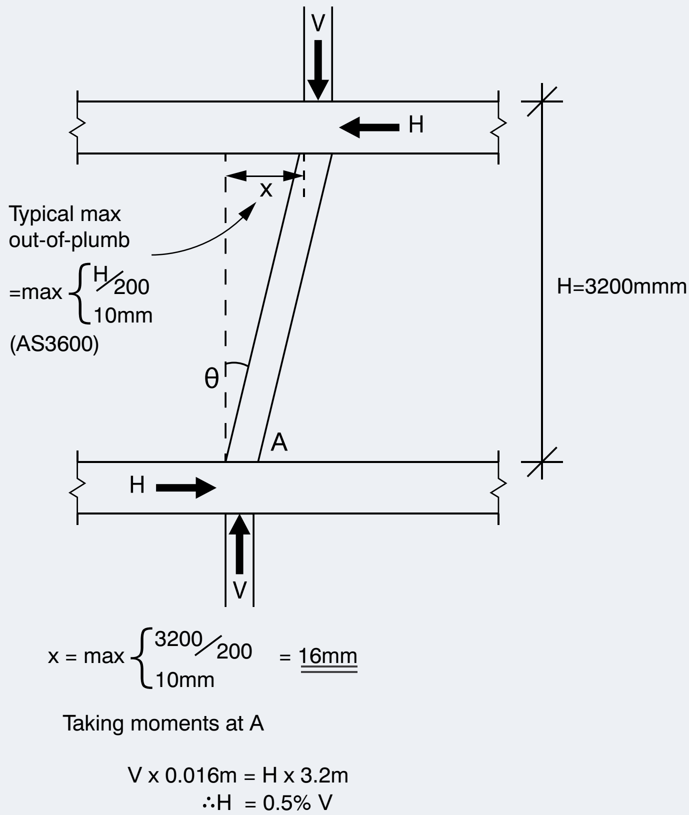


Figure 3.7: Out of plumb column leading to net lateral load.

It is for this reason that the application of notional horizontal loads is recommended for all buildings but complying with this alone does not guarantee a robust structure, more that the structure is capable of withstanding some construction tolerance.

3.4 Detailing and Tying

The concept of providing ties within a within a structure is not specifically mandated within the NCC or Australian material design codes but is used within the Eurocodes as a practical and simple way to improve building robustness. It is presented here as a potential alternative way to demonstrate compliance with the NCC and can be particularly useful in both steel and concrete buildings.

3.4.1 Why Do We Include Ties?

The principle of tying elements of a structure together is twofold: to constrain the elements during an event and to create a statically indeterminate structure that is capable of utilising alternative load paths should the need arise. The Eurocode separates them into horizontal and vertical ties, each serving a different purpose in the case of an accidental damage situation.

While the inclusion of ties within a structure is good practice and will help safeguard the structure against a majority of unknown hazards, they have potential weaknesses in irregular structures or structures with larger consequences of failure. Designers may wish to take a more thorough approach and look at the removal of notional elements as discussed in Section 3.5.

Figure 3.8 shows that in a building with ties designed into the structure, a number of different load paths can be used following the removal of a column. This distributed the loads elsewhere in the structure and avoids disproportionate collapse.

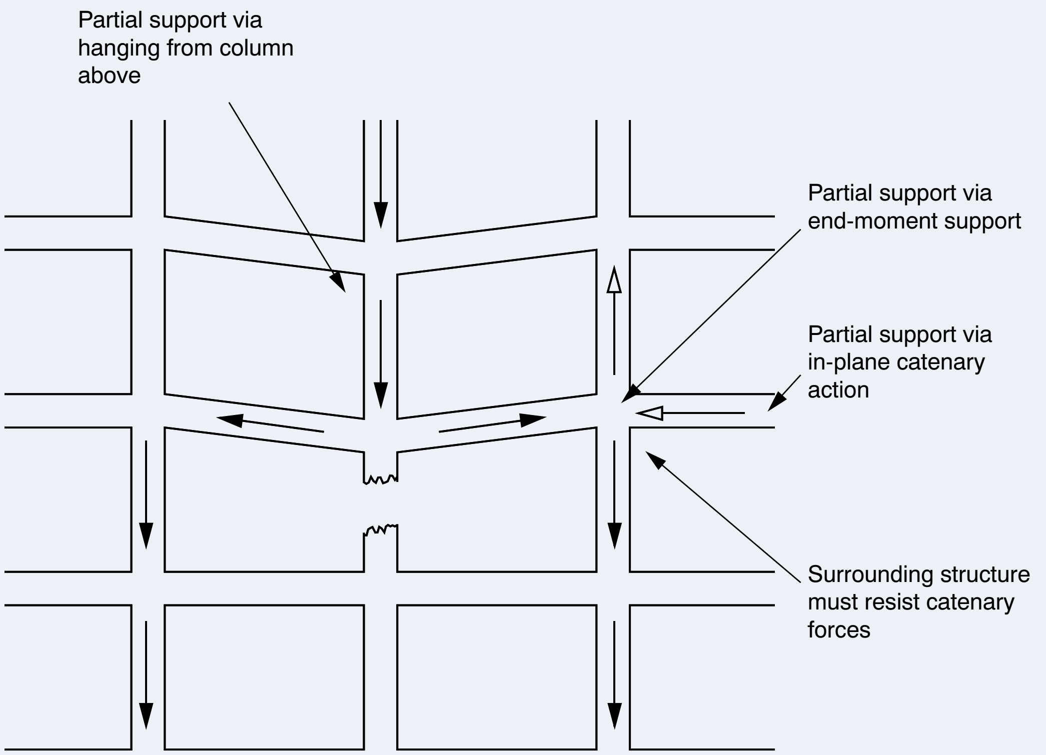


Figure 3.8: Different load paths available in a building designed with vertical and horizontal ties.

3.4.2 Horizontal Ties

The provision of ties through a structure, both around the perimeter and internally is intended to provide a measure of robustness, so that beams or slabs may be able to span across a removed support. Ties should be continuous across the building and around the perimeter and should be in a straight line where possible. Where ties are required to be cranked, their tendency to straighten should be considered and appropriate restraint provided. In a typical regular-framed structure, these ties will be largely catered for by the main floor beams and the only remaining consideration is to ensure that the connections have sufficient capacity to transfer the tie loads.

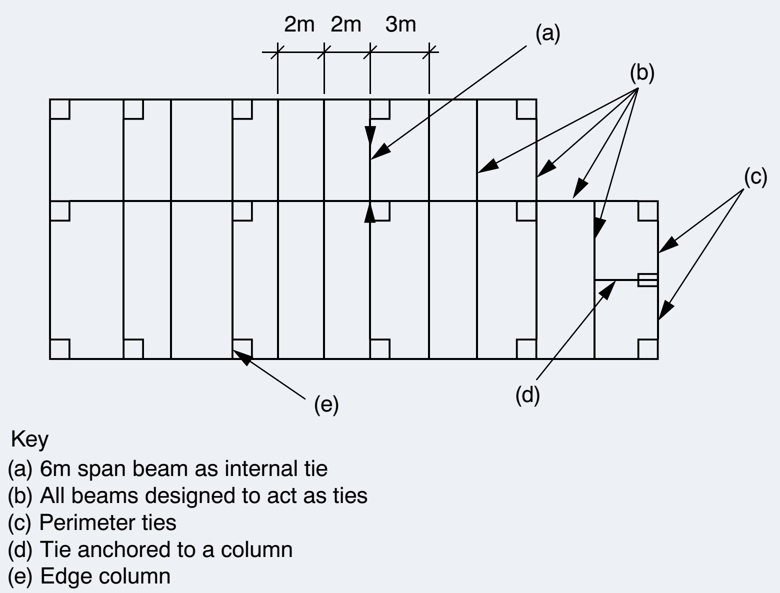


Figure 3.9: Typical tie layout for a framed building.

3.4.3 Vertical Ties

The vertical ties typical in more onerous building classifications are intended to share load over a number of levels if a loadbearing element is removed at a lower level. Ties should be continuous from the foundations to the roof level. This is usually easily achieved in framed structures, provided the tie forces are transferred through any column splices.

The additional load from any collapsed floors can be spread through other floors and reduce the likelihood of collapse of the local area. The other floors deflect significantly but the structure itself may remain intact.

3.4.4 Tie Design Forces

The tie design forces prescribed by the Eurocode are arguable and while they are based on the theories outlined above the actual forces generated are unknown. The Eurocode splits them into the following:

Horizontal ties

Tie forces recommended by the Eurocode are:

Internal Ties- $T_i = 0.8(G + \psi_E Q)sL$ *or 75kN (whichever is greater)*

Perimeter Ties- $T_p = 0.4(G + \psi_E Q)sL$ *or 75kN (whichever is greater)*

Where:

G = Characteristic dead load

Q = Characteristic live load

ψ_E = Combination factor for accidental actions

s = Tie spacing

L = Tie length

These equations relate the tie force to the area of floor supported and effectively require the tie force transferred by the connection to be a minimum of around 80% of the factored reaction at the end of the beam.

Vertical ties

Vertical ties for a framed building should be sized so that the columns and their splices are capable of resisting an accidental design tensile load equal to the largest design vertical permanent and variable load reaction applied to that column from one storey. This load need not be applied at the same time as any other permanent and variable actions.

Load-bearing wall construction

For loadbearing walls, the Eurocode recommended design tie force T is:

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \text{ Newtons or } 100 \text{ kN/m (whichever is greater)}$$

Where:

A is the cross sectional area in mm^2 of the wall measured on plan, excluding any non-loadbearing leaf

H is the wall height

T is the wall thickness

Ties should be at maximum 5 m centres along the wall and occur at no more than 2.5m from an unrestrained end of the wall.

This is fairly onerous for a lot of construction types and a great many typical loadbearing precast buildings with dowel connections would potentially not comply as it would imply that they ought to have a minimum of N20 dowels at about 1.25 m.

A fairer alternative would be to treat vertical ties in a loadbearing wall construction in the same way as that in a framed building and to design the walls and splice to resist a minimum of the design vertical permanent and variable action applied to that wall at any one level. This would allow the walls below to hang the floors in an accidental damage situation and increase the number of alternative load paths

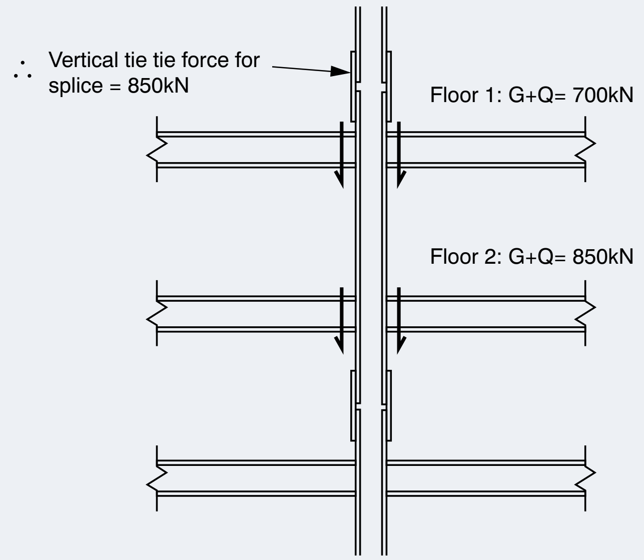


Figure 3.10: Proposed vertical tie force for a framed building.

While all these forces should be relatively achievable in the case of steel or concrete buildings, they will be much more difficult to achieve in typical masonry or timber buildings due to the connection capacity limitations inherent in these materials.

3.4.5 Tie Design Details

The tie details will differ greatly between materials. By their very nature, reinforced concrete structures are relatively robust and well tied through normal detailing practices and robustness will not be a key design consideration. By contrast, a precast concrete building connected together with weld plates or dowels is naturally significantly less robust than its in situ equivalent and the localised connections make it more difficult to demonstrate robustness. A notional element removal method may be more appropriate.

Tie details for each material is shown in more detail in the following sections of this guide.

3.5 Notional Element Removal

3.5.1 Procedure

As an alternative to providing ties a designer can look at the notional removal of each load-bearing element, one at a time. The word notional emphasizes the fact that it is an imaginary scenario.

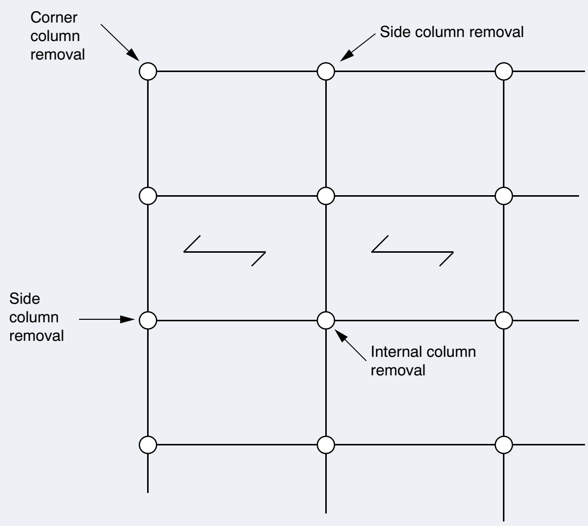


Figure 3.11: Notional loadbearing element removal – framed structures.

In the case in Figure 3.11, each individual column, length of load-bearing wall or transfer beam is imagined to be removed and the consequences examined. The building should remain stable and the connected floors should remain intact, with the exception of any localised damage permitted as discussed in Section 2.0. The lengths of loadbearing wall to be notionally removed are defined below.

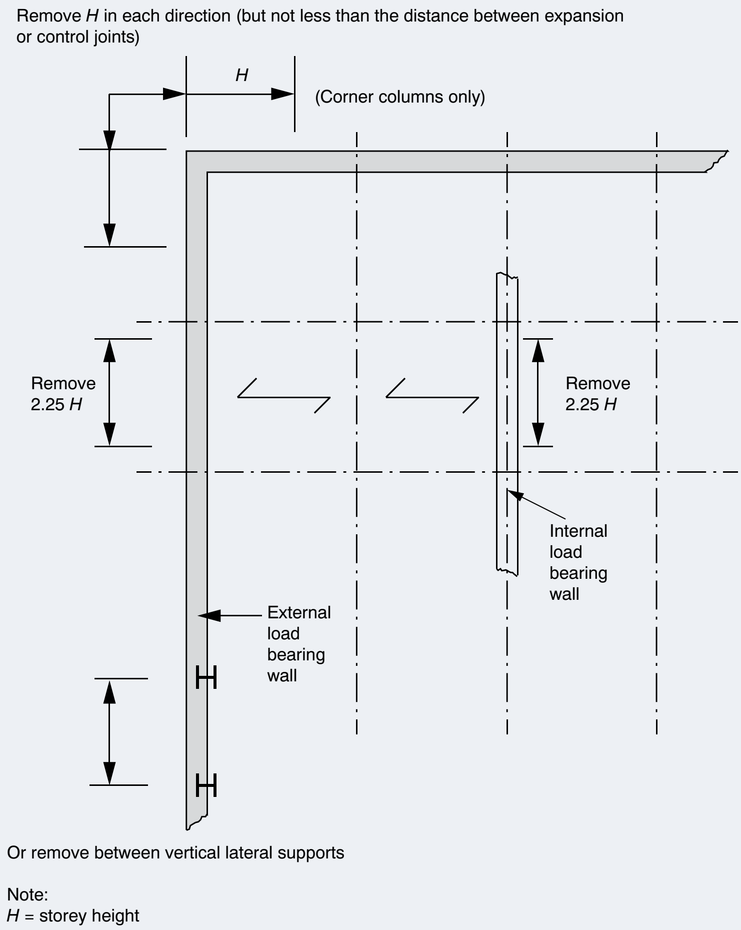


Figure 3.12: Notional element removal – loadbearing wall structures.

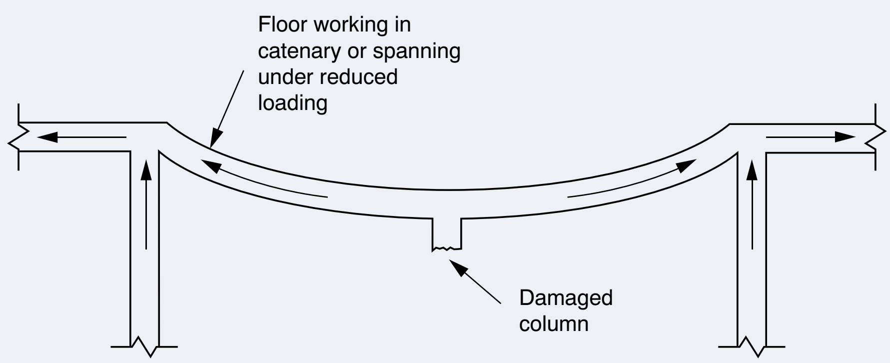


Figure 3.13: Floor working in catenary over removed column.

It is conceivable that if the beams over the columns are sufficiently well tied together, the floor may hang in catenary from the adjacent structure and remain intact, albeit with significant deformations in the floor structure. This is dependent on a number of assumptions:

- The beam-beam connection through the column remaining intact and having sufficient tie strength to support the catenary loads.
- The surrounding structure being able to support the horizontal tie forces generated.
- The connections have sufficient ductility to allow rotation at the supports to allow the beams to work in catenary action.

While the accurate calculation of the catenary action is complex in reality and dependent on a number of variables, the provision of the ties is intended to provide a feasible alternative load path. Removal of certain elements may be problematic for certain situations where there is irregular geometry, re-entrant corners, large transfer structures. In these cases, either the layout of the building may need to be changed or the element designed as a protected element (see section 3.6).

Other options for load paths are shown in Figures 3.14, 3.15 and 3.16.

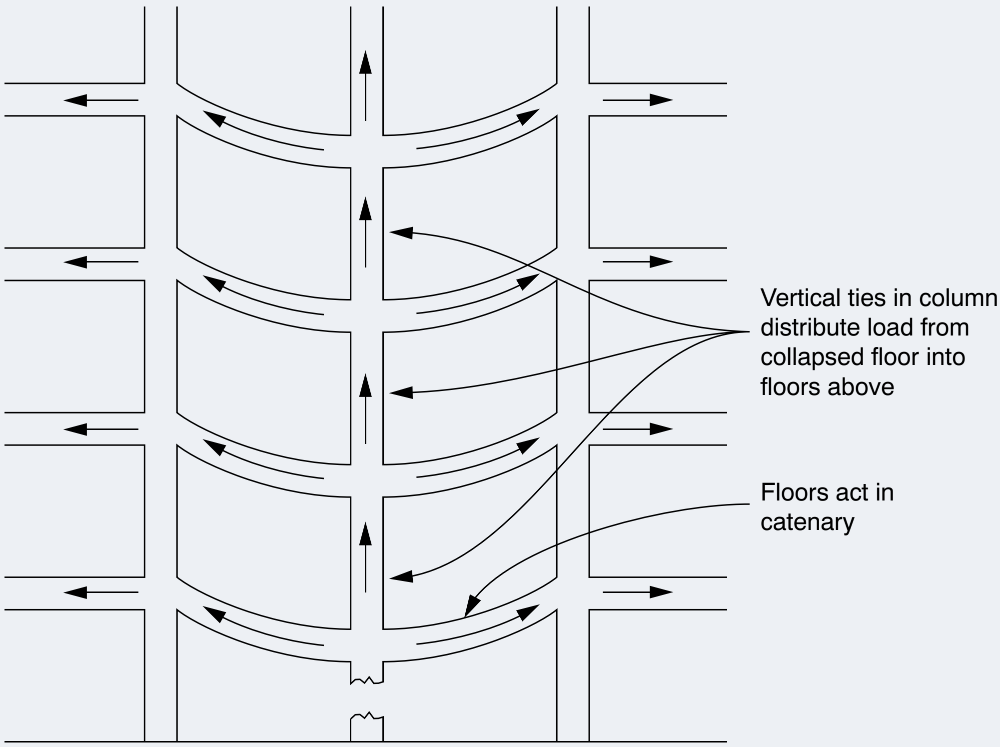


Figure 3.14: Sharing the load between floors over removed columns.

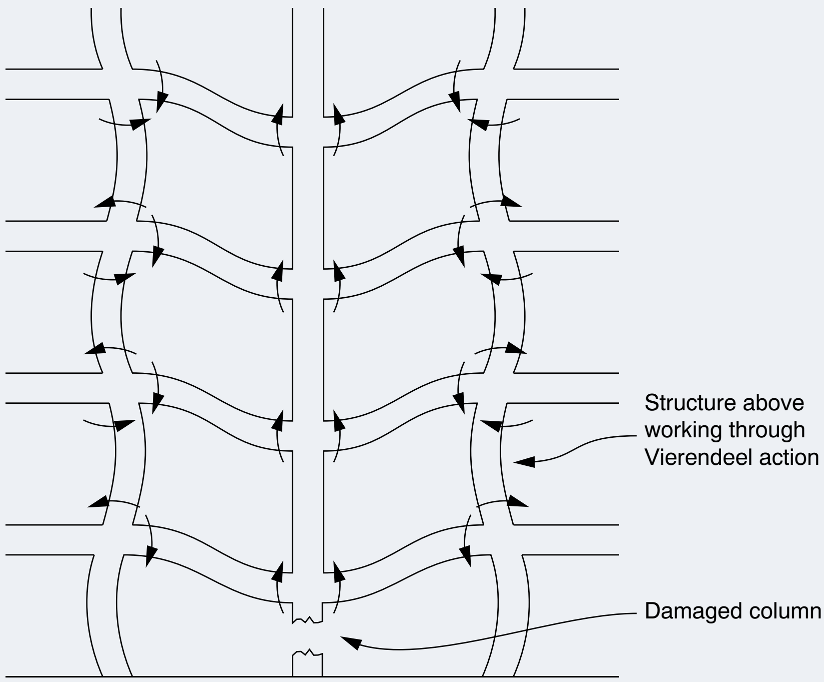


Figure 3.15: Frames acting as virendeels over removed elements.

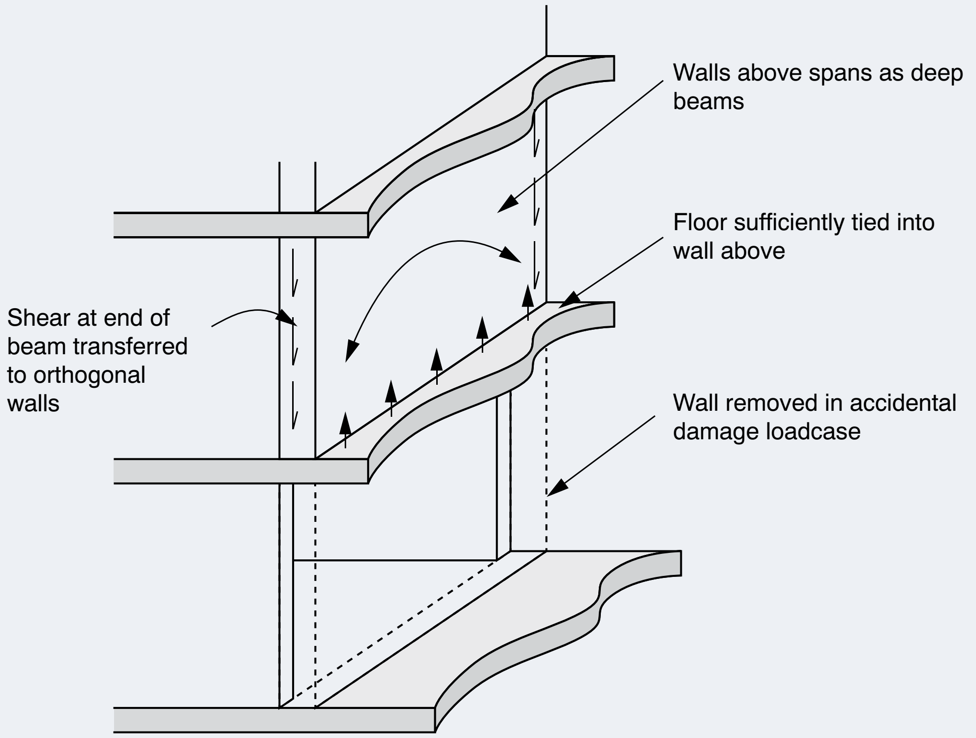


Figure 3.16: Walls acting as deep beams to span over removed elements.

3.5.2 Calculations

For most buildings in design classes 1-3, detailed calculations may not be needed for every loadbearing element removed provided sufficient simple checks are made to ensure that the structure has some feasible secondary load paths available. Both steel and concrete structures will tend to behave plastically in a failure event and redistribute load to other elements. It is a relatively complex matter to calculate things like catenary forces in floors and it is not anticipated that this will be necessary in the majority of buildings, although the sorts of structures outside the scope of this design guide may find it necessary.

Materials that behave in a much more elastic way, such as timber, will require more detailed consideration of robustness as they are much less able to redistribute forces and are significantly more reliant on discrete connections. A more detailed discussion of this is included in Section 6.

3.6 Protected Elements

In cases where certain elements are not able to be notionally removed without the risk of collapse of a disproportionate area of the building, it is necessary to design these elements as protected elements. The element should be designed to resist both their normal applied loads and the anticipated load from the accidental damage event. It is a method that a designer should use as a last resort and apply with a great deal of caution because it relies on the consideration of a lot of variables in terms of force, point of application and structural response.

The NCC requirement is for a systematic risk assessment to be undertaken and critical high-risk components identified. They then need to be designed to cope with the identified hazard or protective measures chosen to minimise risk. Accidental damage events are, by their nature, difficult to define and the forces applied in these events are hard to assess. Specialist advice may be needed. It is anticipated that designing a member as a protected element in an accidental damage load case will be the critical load case for this member, sometimes by an order of magnitude larger than the normal in service design condition.

The Eurocode gives some guidance on the forces to apply, although the actual application of these forces is poorly defined. The code calls for an accidental damage force of 34kPa to be applied in any direction to the member and any attached components. Defined following the Ronan Point disaster, 34kPa is a static force representing the force generated by a gas explosion. There is no real guidance on how best to apply this load and it is clearly a large force that will be onerous in a number of cases and un-conservative in others.

Take the example of an isolated internal column. If a 34kPa force is applied horizontally to the face of the column, it clearly has a very small net load due to the small surface area. However, if we apply the same 34kPa force vertically to the slab that transfers load to the column, this load is very large indeed and most likely would cause the slab and connections to fail long before the load reaches the column.

The IStructE's guide notes that this is a limitation and recommends taking the upper boundary of the connection capacity for the floor to the column as the largest vertical load transferred to the column. The lateral load on the column is also recommended to be a point load applied at the worst case location, 250kN at ground floor and 150kN at all other floors.

Designing protected elements requires a large amount of risk assessment and engineering judgement. Consultation with specialists in the field of blasts engineering and transport impacts may be prudent in a number of cases.

3.7 Strong Floors

As discussed in Section 2.0, the definition of admissible damage including areas on two adjacent floors implies that the floor immediately beneath the collapsed ones will need to support the weight of the debris from the floors above. It will often be simpler to design the building in such a way that only one floor is lost because in most cases the in-service ultimate loads will be larger than the accidental loads and reduced factors. However, in some cases (in particular some fire cases where cavity barriers are at risk of failure and fire can spread vertically over two floors) this may be unavoidable (See Section 6.4.1).

The actual failure mechanism of each floor is subject to a large number of variables and it is impossible to predict with any certainty.

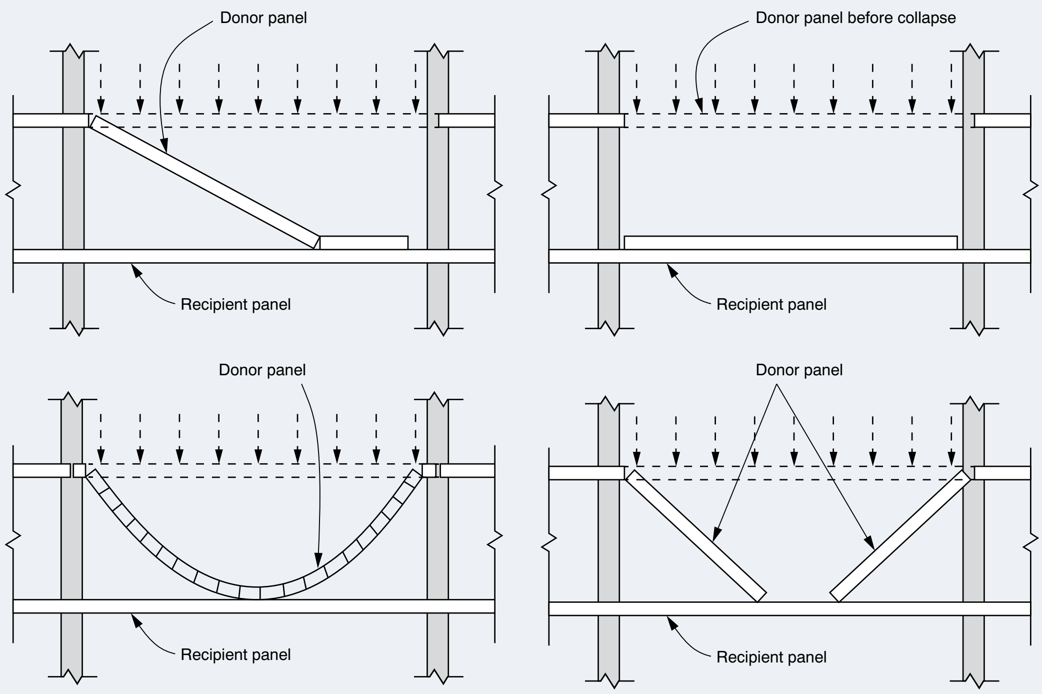


Figure 3.17: Potential failure mechanisms for floors.

A reasonable approach is to design the floor to support the dead load of two floors and a reduced live load based on accidental load factors and to consider this a short-term design load case. Rather than design every floor for this load case and significantly increase the material in each floor, an alternative may be to include a strong floor at every other floor that is capable of supporting this load. As demonstrated in Section 2.4, the accidental damage load case is often around half the typical ultimate design load case, so designing a floor to support a single collapsed floor is comparable to the normal ultimate design and designing a strong floor to support two floors would represent an increase of 50% over the normal ultimate load case check.

3.8 Hybrid Structures

Many buildings will have a combination of materials throughout the structure and will potentially involve a number of different designers. It is important that one engineer takes responsibility for the structure's robustness requirements and communicates this effectively with other designers. In most cases, the base build engineer should retain an overview of the requirements.

The following sections discuss some individual design requirements and details for the main structural materials and when dealing with hybrid structures we should ensure that the robustness design principles remain consistent and non-contradictory for the whole structure and that compatibility of each material is considered.

4

Concrete

4.1 In Situ vs PT vs Precast

In situ reinforced concrete frames designed and detailed in line with best practice guidelines are inherently very robust. The robustness strategies outlined in this guide will apply to concrete buildings although in most cases providing horizontal and/or vertical ties will be the most efficient strategy. The tie provisions will normally be met by ensuring that the structure complies with the minimum reinforcement requirements of AS 3600 and that the structure is designed as suitably ductile. The ties should be continuous and subject to good practice requirements, such as ensuring that tie bars have a full development lap to ensure full ductility is achieved. Interaction between ties is critical to be able to develop catenary action and both horizontal and vertical ties will need to be lapped, particularly through columns at supports. The bottom reinforcement is much more effective in catenary action and it is recommended that ties are designed in these layers. It is worth noting that in the case of the loss of a column and an upper slab hanging a lower one, the slab will need to be able to resist punching shear in the reverse of its normal direction, which may require a design check.

Post-tensioned structures with bonded tendons will tend to show inherent robustness. The continuous tendons will normally comfortably provide the tying force for the structure and there are significantly fewer laps. The main challenge is to ensure sufficient interaction between vertical and horizontal ties. This can normally be achieved if the ducts pass through the centre of a column or at least as close as possible either side.

Unbonded post-tensioned structures are rare in Australia outside civil applications. Unbonded tendons are not suitable for consideration as ties; their failure in one span may cause failure in adjacent spans.

Precast construction lacks the inherent continuity of ties that is present in in situ construction and a more explicit consideration of robustness is required. The vast majority of precast elements used in Australia will only make up part of an in situ structure and it is rare for a building to be primarily constructed from precast. There are plenty of opportunities to adequately tie precast elements into the rest of the structure in most cases and this guide looks to identify some of the less typical cases where precast requires more overt consideration.

The construction conditions can often be the most critical for precast and the temporary stability of the structure should be considered at all stages until the final connections are made.

4.2 Typical Methodologies

The following table is a recommendation for the typical methods for dealing with concrete structures in each of the importance level classifications. For the majority of concrete structures the simplest method would be to design for ties although the alternatives of notional element removal or protected elements are also viable.

Importance Level (IL)	Method
1	No requirements
2	Continuity of Ties in each principal direction. Where vertical continuity cannot be achieved, design element for bridging over failed zone.
3	Alternate Load Path Method or Continuity of Ties and Enhancement of Corner and first internal columns at ground floor. Compliance with moderate detailing requirements of AS3600 (Appendix C).
4	Alternate Load Path Method or Continuity of Ties and Enhancement of all supports at ground floor. Compliance with moderate detailing requirements of AS3600 (Appendix C).
5	As per IL 4 with increased Local Element shear and ductility enhancement for all ground floor and publically exposed load bearing elements.

Figure 4.1: Proposed methodology for concrete buildings designed for robustness.

4.3 Ties in Concrete Buildings

Ties should be designed as per the principles set out in Section 3.4. Figure 4.2 shows the different types of tie present in a concrete building.

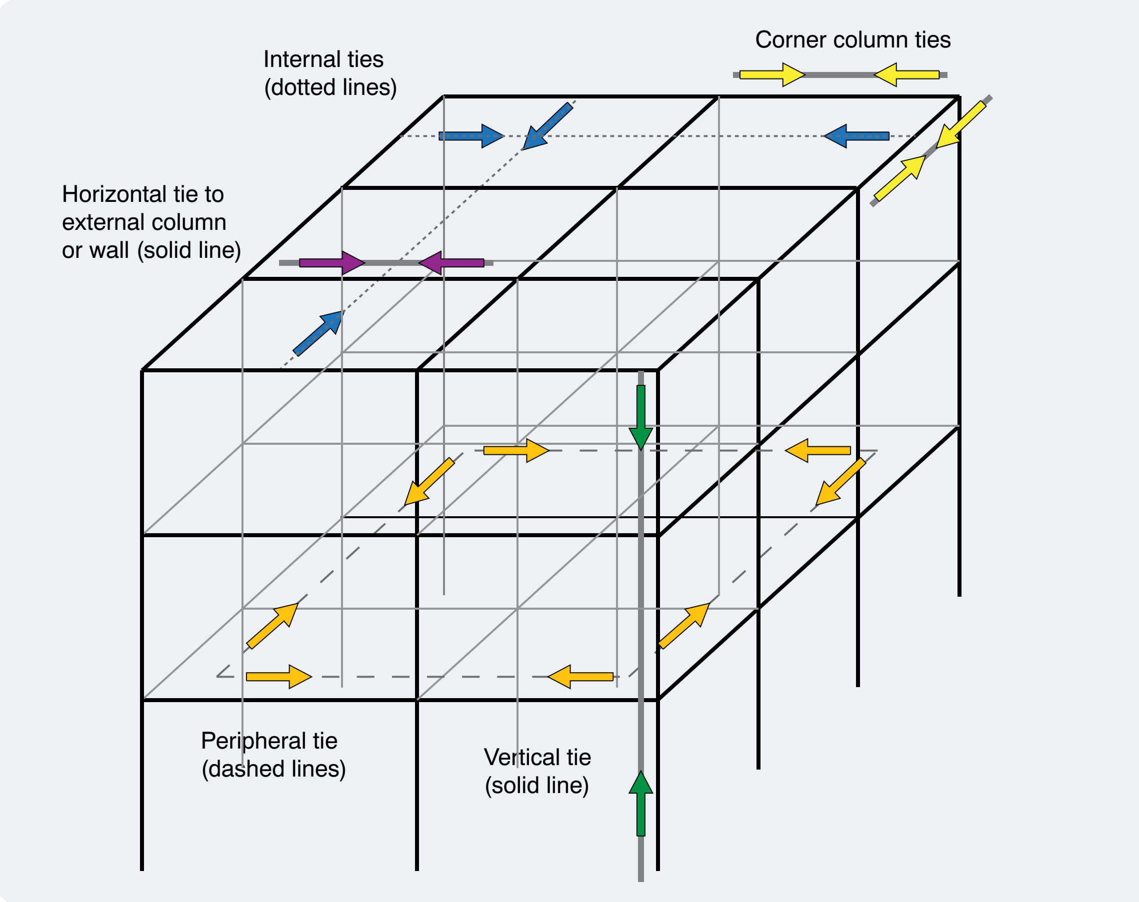


Figure 4.2: Typical ties in a concrete-framed building.

4.3.1 Peripheral Ties

Peripheral ties are treated differently in the Eurocodes as they are both the most at risk of damage and also less readily able to utilise alternate load paths. No other forces need to be considered at the same time.

$$Peripheral\ tie\ force, F_{tp}(kN) = (20 + 4n_0) \leq 60kN\ where\ n_0 is\ the\ number\ of\ storeys$$

This is effectively only a small bar around the perimeter and they should be situated within 1.2m of the slab edge or within the external walls or slab. This is a simple matter for most concrete structures with the exception of precast, and further details can be found in Section 4.4.1 for these structures.

4.3.2 Internal Ties

Internal ties should be placed in two directions and spacing between ties should not exceed the column spacing. Note: the internal tie forces are given in kN/m and may be either spread across the slab or concentrated in beam strips as best suits the detailing.

$$Internal\ tie\ force, F_{ti}(kN/m) = max \begin{cases} \left[\left(\frac{1}{7.5}\right)(G+Q)\left(\frac{s}{5}\right)\right] F_{tp} \\ 1.0\ F_{tp} \end{cases}$$

Where:

G = Characteristic dead load

Q = Characteristic live load

s = Typical tie spacing

n_0 = Number of storeys

4.3.3 Vertical Ties

Vertical ties for a framed building should be sized so that the columns and their splices are capable of resisting an accidental design tensile load equal to the largest design vertical permanent and variable load reaction applied to that column from one storey. This load need not be applied at the same time as any other permanent and variable actions.

4.4 Typical Details

As discussed earlier, the main issues with robustness in concrete buildings will occur where precast sections are used. Most commonly the precast elements in buildings are restricted to the cores/ external walls or to precast columns

4.4.1 Precast Cores/Walls

Most vertical precast connections will include grout tubes and tie bars and ensuring the bars are continuous through floors is important to develop effective ties. The tie bars should be capable of supporting the weight of the floor should the panel below be removed (see section 4.3.3).

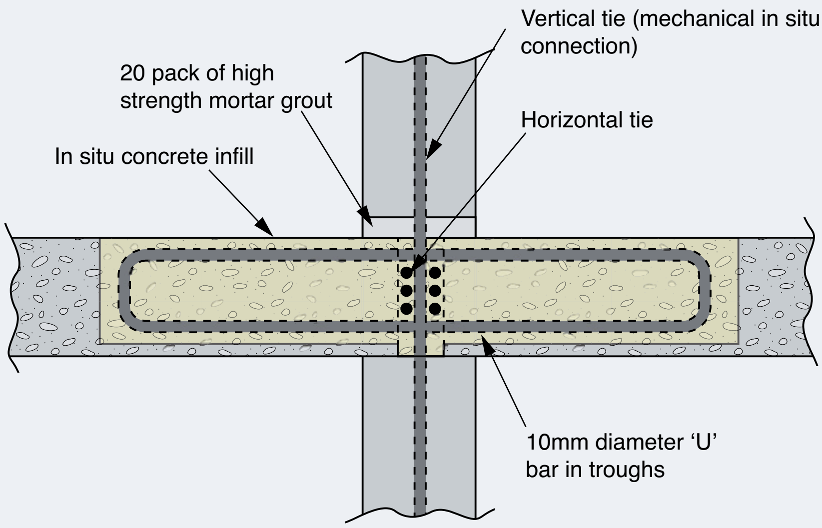


Figure 4.2: Precast wall connection with ties. Typical section through internal wall (load-bearing).

Figure 4.2 shows a continuous vertical tie through the floor and also a precast slab with an in situ joint to allow horizontal bars ties to be maintained through the floor structure.

Most often, the core walls in a concrete building will be made from precast and joined together with weld plates and dowel joints. The removal of a precast panel from a core will require the check that the stability of the building is still viable under reduced loading.

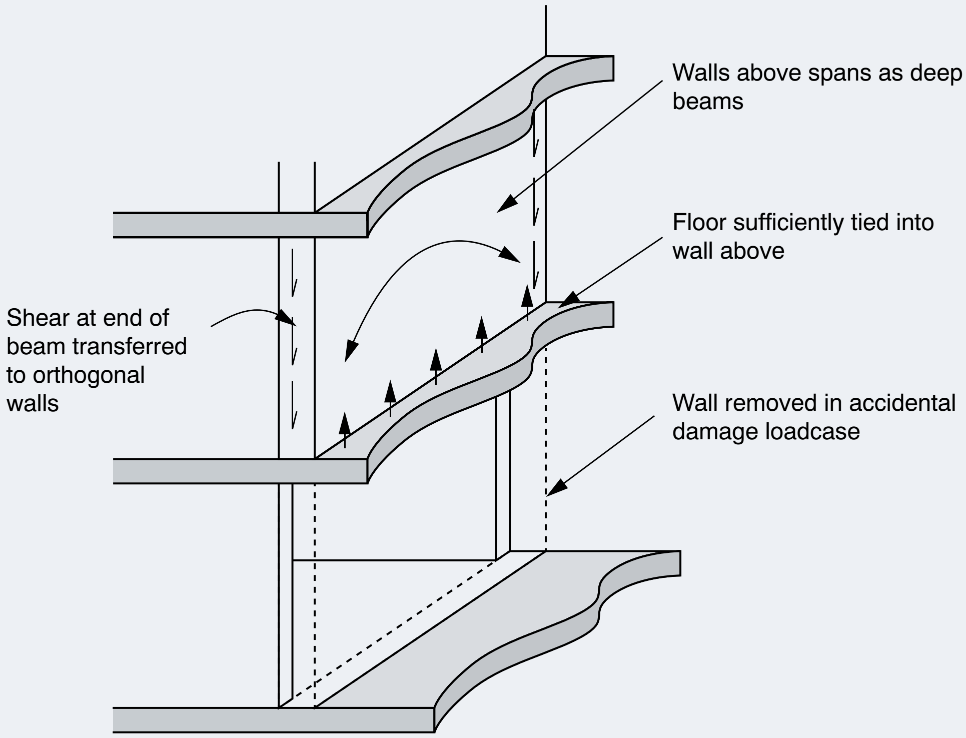


Figure 4.3: Precast core walls spanning as deep beams in accidental damage situation.

4.4.2 Other Precast Connections

In concrete structures built largely from precast elements the designer will need to ensure that ties are maintained through careful detailing. Typically, this will involve in situ joints between precast elements to allow continuous ties to be effective throughout the structure.

Figure 4.4 shows a precast beam and precast slab connection with continuous bars in a wet joint between the elements forming the horizontal ties for the structure. The U-bars ensure both the slab and beam are effectively anchored to the tie bars.

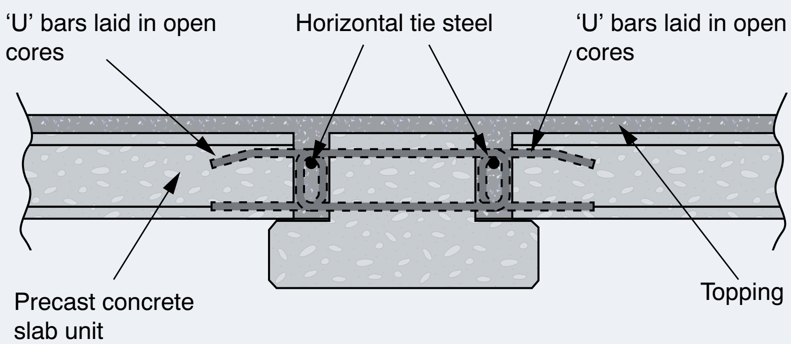


Figure 4.4: Precast concrete beam-precast slab connection detail incorporating ties. Central beam detail.

Good detailing of the precast beams to allow sufficient rotation of the floor structure and avoid a brittle failure can help ensure the beams have some capacity to work in catenary action.

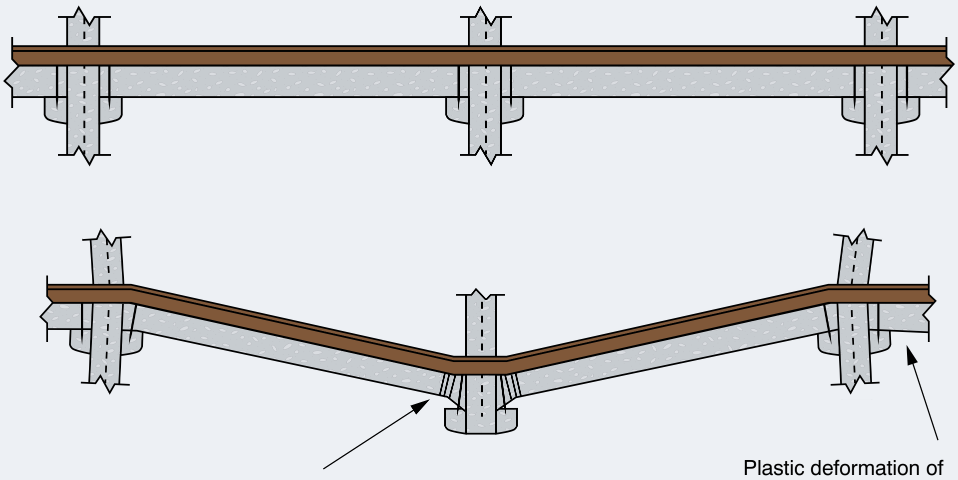


Figure 4.5: Concentration of links at supports to ensure high ductility.

5

Steel

5.1 Typical Methodologies

Much like concrete-framed buildings, most well-detailed steel buildings will possess good inherent structural robustness for the following reasons:

- steel often performs in a very ductile way in extreme events
- connection capacities are typically high and will often work in tension as well as shear
- most steel-framed buildings in Australia will have an in situ concrete slab that will aid alternative load paths and distribution of loads.

All the design procedures detailed in Section 3 are relevant for steel design but most frequently it will be simplest to design for ties. If ties cannot be provided, the option is to design for notional removal.

By far the most likely time for the disproportionate collapse of a steel structure is during construction as the final load paths may not be viable until a large amount of the adjacent structure is complete. It is the designer's responsibility to ensure there is a safe method of construction for the entire building and draw the builder's attention to any requirements for temporary works assumed within the design.

5.2 Best Practice

Some points to consider when detailing steel frames to improve robustness of a structure:

- In pin-jointed braced frames it is wise to consider some form of redundancy to the stability system.
- Detailing connections in proportion to the member sizes, regardless of applied forces, will allow reserves of capacity in accidental damage cases.
- Columns should be tied in two approximately orthogonal directions where possible.
- Avoid details that are sensitive to small changes in alignment.

5.3 Horizontal Ties

Horizontal ties are the simplest way to demonstrate robustness for a Class 2 building and are most often concentrated in the primary structural members.

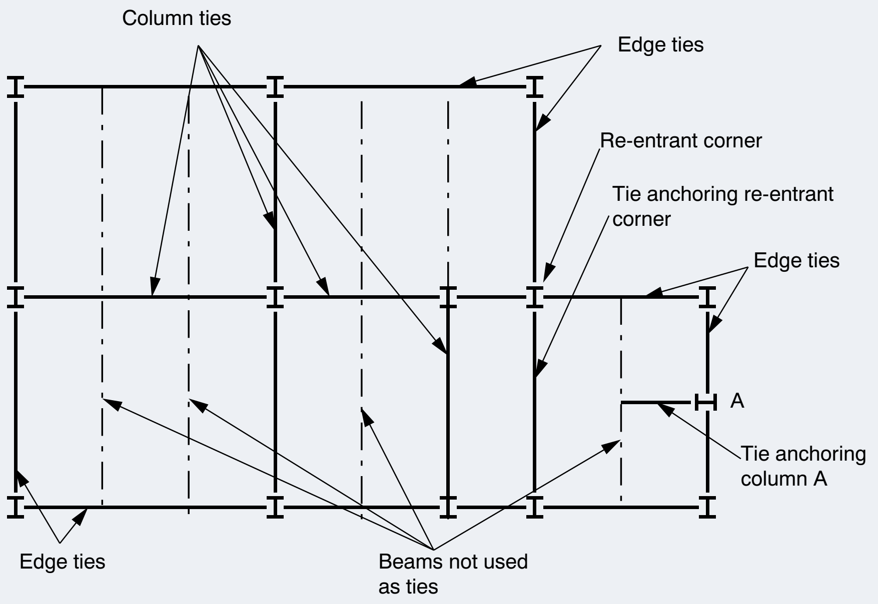


Figure 5.1: Typical horizontal tie arrangement for a steel frame.

Tie forces recommended by the Eurocode are:

$$Internal\ Ties - T_i = 0.8(G + \psi_E Q)sL\ or\ 75kN\ (whichever\ is\ greater)$$

$$Perimeter\ Ties - T_p = 0.4(G + \psi_E Q)sL\ or\ 75kN\ (whichever\ is\ greater)$$

Where:

G = Characteristic dead load

Q = Characteristic live load

ψ_E = Combination factor for accidental actions

s = Tie spacing

L = Tie length

These equations relate the tie force to the area of floor supported and effectively require the tie force transferred by the connection to be a minimum of around 80% of the factored reaction at the end of the beam. Many typical steelwork connections will be able to resist this naturally.

5.4 Vertical Ties

Vertical ties for a steel frame building should be continuous through the building from the foundations to the roof. The vertical tie loads specified in Section 3.4.4 should be easily achievable in most steel-framed structures and the column splices should be checked for the tie loads.

5.5 Bracing

Pin-jointed steel framed buildings with isolated bracing are potentially at risk of collapse if there is some damage to the bracing system. Designers should consider if there are other potential lateral load systems available if this should occur and if there is the potential to ensure a minimum degree of moment capacity in some frames in the building to allow for a secondary lateral load system.

5.6 Transfer Structures

Transfer structures are sometimes used in steel structures to reconcile structural grids. If they are damaged, they create a risk to large areas of the building. There are a few options for improving the resilience of transfer structures if they cannot be avoided:

- design the transfer structure for continuity over internal supports if possible
- allow for transfers on consecutive floors such that one can do the job for both in the accidental load case
- design the transfer structure as a protected element
- ensure the transfer structure has sufficient ties and ductility to behave as a catenary.

5.7 Floor Slab Details

Most steel-framed buildings in Australia will have a composite concrete deck slab that is, by its nature, well tied to the steel frame and provides excellent opportunity for alternate load paths and catenary action.

For other types of floor slab, such as precast planks, there is a clear necessity to ensure the floors are well tied to the steel structure. In class 2 buildings and lower-rise, less-important structures, the horizontal tie requirements could be achieved through friction between the slab and beam although this is difficult to demonstrate and a more direct connection may be better suited. For higher importance level buildings an in situ joint and physical tie between the floor and steel is the preferred option.

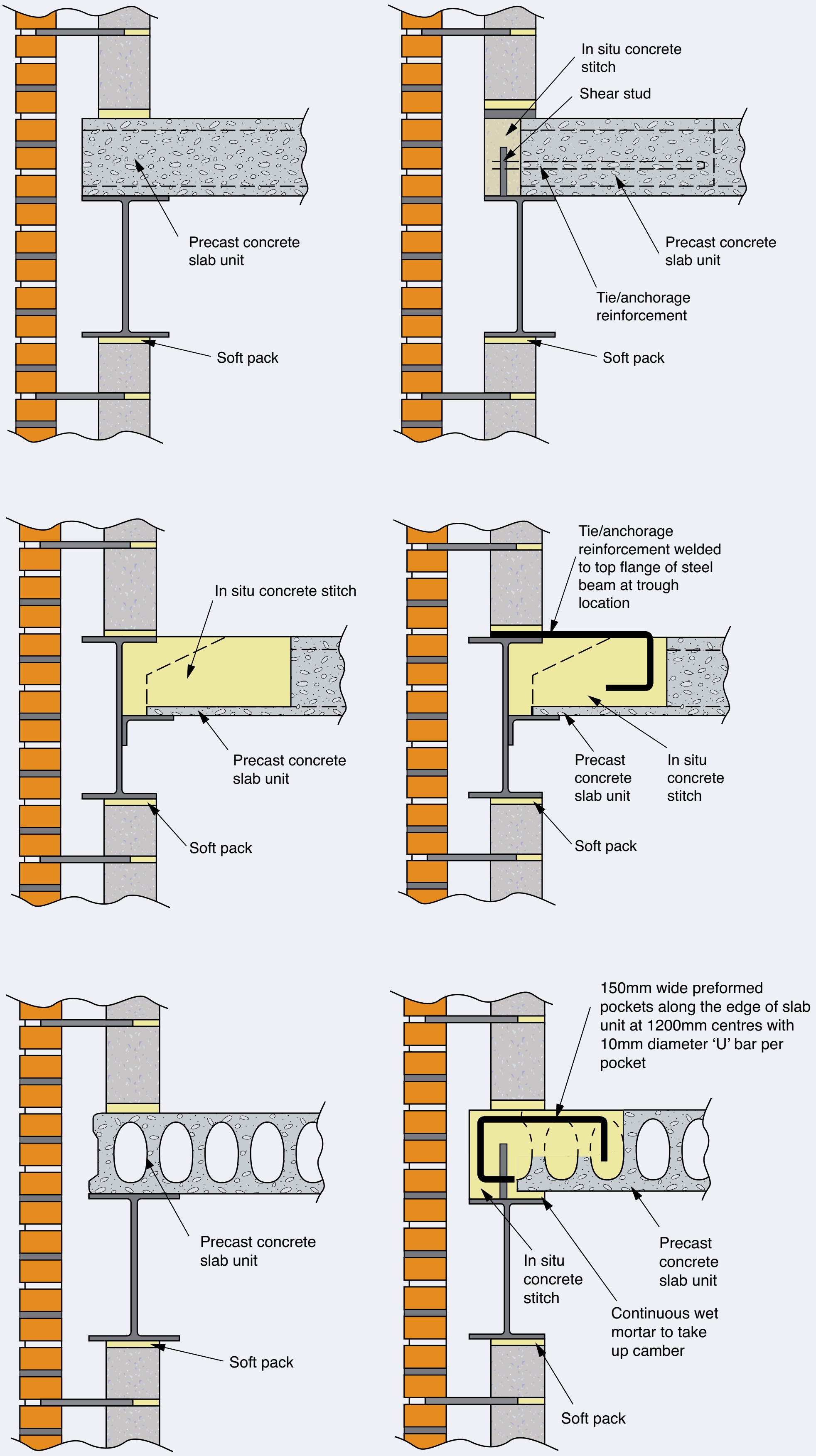


Figure 5.2: Typical tie details for steel-precast concrete structures.

6

Timber

6.1 Stick or Mass Timber Construction

Modern timber construction can mean anything from domestic residential to large-scale commercial buildings. Each material type and building use can require a different approach to robust design. In general, this guide groups them into two categories:

6.1.1 Stick Frame Construction

This covers traditional timber frames, joisted floors, stud walls, prefabricated roof trusses. This construction is most often used in domestic residential and low-rise multi-residential buildings.

Figure 6.1: Typical timber-framed domestic construction.

6.1.2 Mass Timber Construction

Mass timber refers to a more recent trend in larger-scale engineered-timber buildings. Engineered timber products like glulam, laminated veneer lumber (LVL) and cross-laminated timber (CLT) are making it possible to build much larger buildings – and ones that are much more likely to need consideration of robustness in their design. Mass timber construction can be used for a wide array of building types and sizes: medium-rise, multi-residential (like Forte, Melbourne), public buildings (like Library at the Dock, Melbourne), offices, schools, etc.

Figure 6.2: Forte, Melbourne, nine storeys of CLT on concrete podium.

The inherent robustness and recommended design procedures are significantly different for each methodology.

6.2 How is Timber Different?

The main difference between timber structures and concrete or steel structures is that timber does not behave in the same plastic manner and is therefore much less able to redistribute load or utilise catenary action. It is also made up of a number of individual components that are connected at discrete locations. These connections represent a limitation on how the structure is able to transfer load in an accidental damage situation. For these reasons, the design of robust structures in timber often requires more attention and some more explicit design checks to be made.

In timber structures' favour are that they are typically much lighter weight than the equivalent building in other materials and the forces generated during accidental damage events are much less severe and sometimes easier to design against.

6.3 Mass Timber Construction

Massive timber construction is a relatively new form of construction in Australia. Buildings like Forte and the Docklands Library in Melbourne are leading the way and more timber buildings are in construction. Timber construction has been relatively commonplace in Europe for a few decades now, although robustness concepts have only really been taken on board with the adoption of the Eurocodes.

While the different materials are often used in combination, this chapter splits its consideration of mass timber construction into multi-residential platform construction with CLT and framed construction with glulam/LVL.

6.3.1 Cross-laminated Timber

CLT is perhaps the most inherently robust of all the massive timber construction forms for a few reasons:

- the panels have the ability to work in two directions
- wall panels have the ability to span as beams
- high connection capacities can be developed along panel joints.

It does, however, require some careful design and detailing to ensure that it will perform sufficiently robustly in an accidental case and the locations of panel joints will be crucial to establishing this. CLT structures are capable of being designed either to comply with the Deemed-to-Satisfy tie loads or with the performance-based element removal methods but, in most cases, the notional element removal method will lead to the most economic answer in terms of connection design and speed of construction.

Figure 6.3 represents a typical residential CLT structure. All walls and floors are constructed from CLT and are connected with screws and brackets.

Figure 6.3: Typical CLT apartment layout.

Consider the removal of a typical wall panel. As noted in section 3.5 the minimum length of wall to be considered as removed should be the larger of 2.25*H (about 6.75m in this case) or the length of the wall between joints (in this case, the length of the CLT panel). For this check, we will base it on the length of the entire CLT wall being lost.

The floor is now spanning across double the span and has a central line load on it from the wall above. It is reasonable to assume that each of the floors above will do the same and we can design each floor to support the load from a single wall.

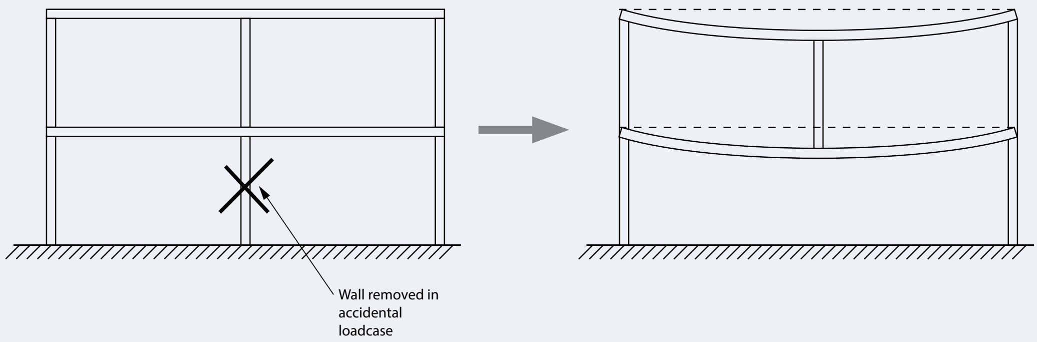


Figure 6.4: Removal of loadbearing wall in a CLT building.

For the permanent design case our floor was designed as 162 mm thick, based on deflection and dynamic requirements. In the accidental case, the floor can be designed based on a reduced factors and loads and now the deflection of the floor is unimportant and the governing design check will be the moment. Note that this check is carried out based on the Eurocode recommendations for partial load factors and material strengths; these bear a close relationship to the earthquake load case design factors in the Australian Standards.

G_k = 1.89kPa, Q_k = 1.5kPa, Span = 5.5m (two spans), Wall G_k = 2.5kN/m

$$w_{acc} = 1.05\times(G_k + 0.3\times Q_k)$$

$$w_{acc} = 2.46 kPa$$

So for a 1m wide CLT panel this gives a moment and bending stress of:

$$M_{acc,d} = 44.3\ kNm$$

$$\sigma_{acc,d} = 12.43\ MPa$$

Based on the short-term loading factor and material partial factors and a characteristic bending strength of 24MPa this gives and allowable bending stress of:

$$f_{m,y,acc} = \frac{k_{mod}\times f_{m,k}}{\gamma_m}$$

$$f_{m,y,acc} = \frac{1.1\times 24}{1.25} = 21.12\ MPa$$

The bending stresses of the material are not exceeded in the accidental case and the floor remains standing, albeit with large deflections. It is possible to show a similar magnitude of moment is developed in the floor if the wall had happened to be at a joint in the floor panel. It is good practice to stagger the joints in floor panels where possible as shown in Figure 6.5 (this has the added bonus of making it easier to demonstrate diaphragm action in the floors). A similar check can be carried out for the external walls.

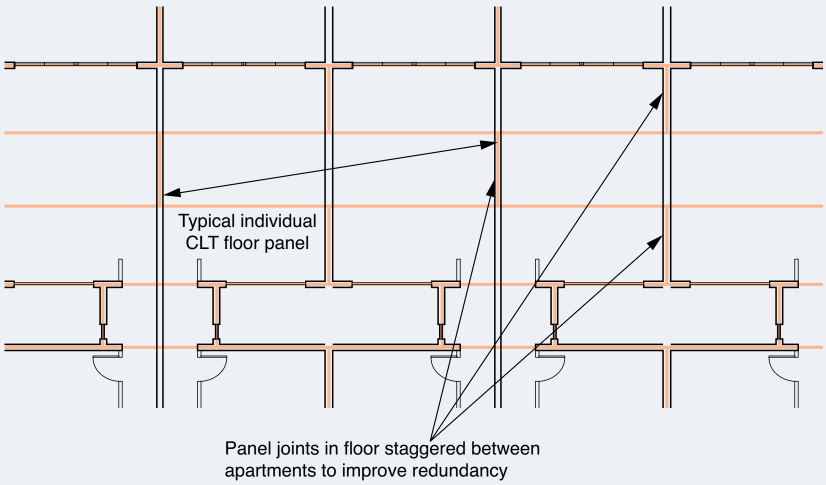


Figure 6.5: Staggering floor joints in CLT buildings is good practice for robustness.

Another useful benefit of CLT construction is being able to use walls as deep beams to span over walls removed below. The depth of the walls means they will work well as spanning elements if called for and it remains to ensure the connections are capable of resisting the loads and the walls are sufficiently well restrained.

Consider the example in Figure 6.6.

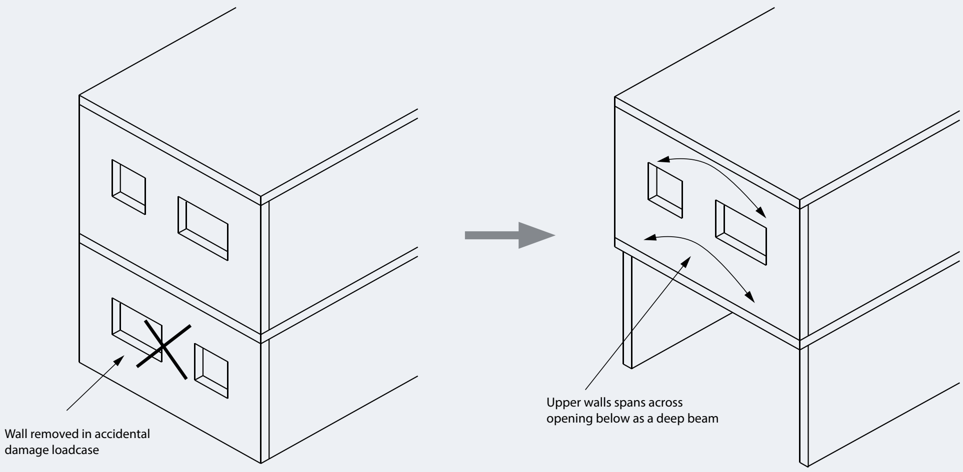


Figure 6.6: Spanning of walls in accidental damage load cases.

The loadbearing wall at the lower level has been removed. The floor is also connected into the wall above. A very basic design check on the wall shows the stresses are very low and we're unlikely to be at risk of the wall buckling as it is restrained top and bottom by the floors.

G_k = 6m x 1.89kPa + 3m x 0.75kPa, Qk = 6m x 1.5kPa, Span = 7m

$$w_{acc} = 17.1 kN/m$$

$$M_{wall,acc} = \frac{17.1 \times 7^2}{8} = 104.7\ kNm$$

$$\sigma_{wall,acc} = 0.78\ MPa$$

Next, it is important to check the floor-wall connection is capable of hanging the lower floor. In the accidental load case the weight of the floor hanging from the wall will be:

$$F = 17.1\ kN/m$$

This load is within the right magnitude for the capacity of a typical floor-wall connection detail and is also notable for being significantly lower than the vertical tie force, had that method been chosen. It is important that if you are using proprietary brackets, you ensure that the chosen bracket has sufficient tension capacity. Some are only rated for tension loads when installed on both sides of a wall. Additionally, it is wiser to install screws rather than nails in these brackets as nails have nominally little tension capacity.

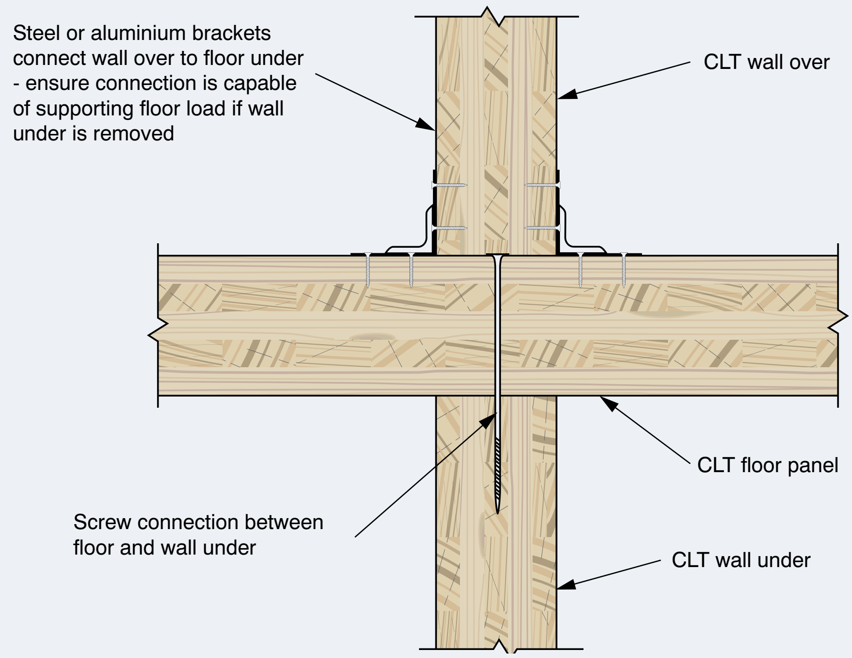


Figure 6.7: Typical wall-floor connection in a CLT building.

These are two of the primary methods for establishing alternative load paths in CLT platform type construction. They are equally as applicable for CLT elements in other timber buildings.

6.3.2 Post and Beam Construction

At a basic level, the construction of a timber post and beam building is similar to a steel-framed one. The building is built a similar way – with columns, primary beams and a strongly single-direction spanning floor system.

Figure 6.8: Timber post and beam construction – College of Creative Arts, Wellington, NZ.

The main challenge in attempting to design a robust timber post and beam building is that it will perform fundamentally differently to a steel building. It will be more elastic and there is much less capacity for rotation at joints, redistribution, catenary action, etc. This means we need to make some more explicit design checks.

The following example is based on a similar structure to the Library at the Dock in Melbourne, although some changes have been made to demonstrate robustness concepts. The beams are detailed as double sections running either side of the column, which is notched to partially seat the beams on. Using a double beam arrangement either side of the column is an efficient way to detail the floor structure. The continuity helps reduce the beam deflections and also makes the structure more robust because it provides some potential secondary load paths.

The robustness principles set out in Section 3 still apply to these structures but, for the tie method to be effective there needs to be some consideration given to the detailing of connections to allow sufficient rotation at these joints to let the structure hang and work in catenary. In practice, this is relatively difficult, although the post-tensioned frames favoured in New Zealand would likely achieve it and the PT cables would also provide excellent ties. The robustness of post-tensioned timber frames is outside the scope of this guide but further information on this type of construction is available through the Expan website.

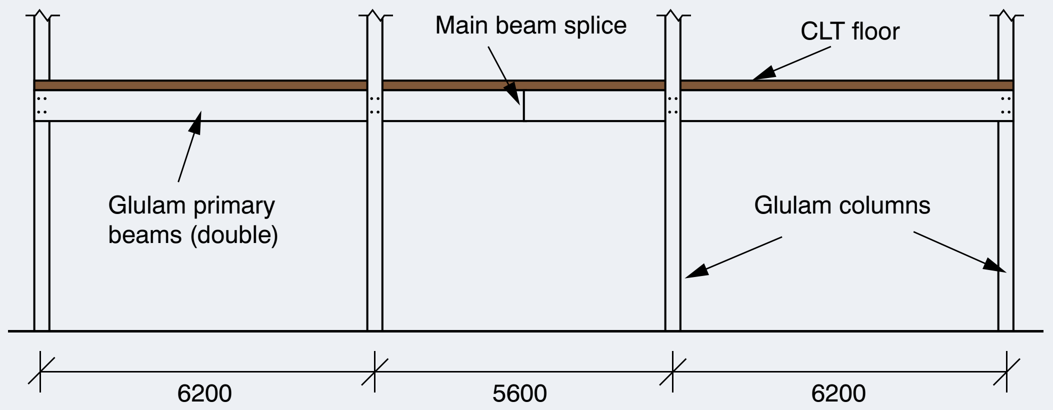


Figure 6.9: Cross section of post and beam timber frame.

Consider the removal of a central column in Figure 6.9. In a simply supported structural system, the loss of a column would potentially lead to the loss of both connecting primary beams and likely two entire floor bays. This would likely extend through the building and would clearly constitute a disproportionate collapse. By making the structure continuous past the columns we have introduced a potential alternative load path.

In a collapse load case, the beams now span to the central splice location while the other beam supports a significant additional load on the end of the cantilever. Figure 6.10 shows the moments in the main beams under both permanent and the accidental cases.

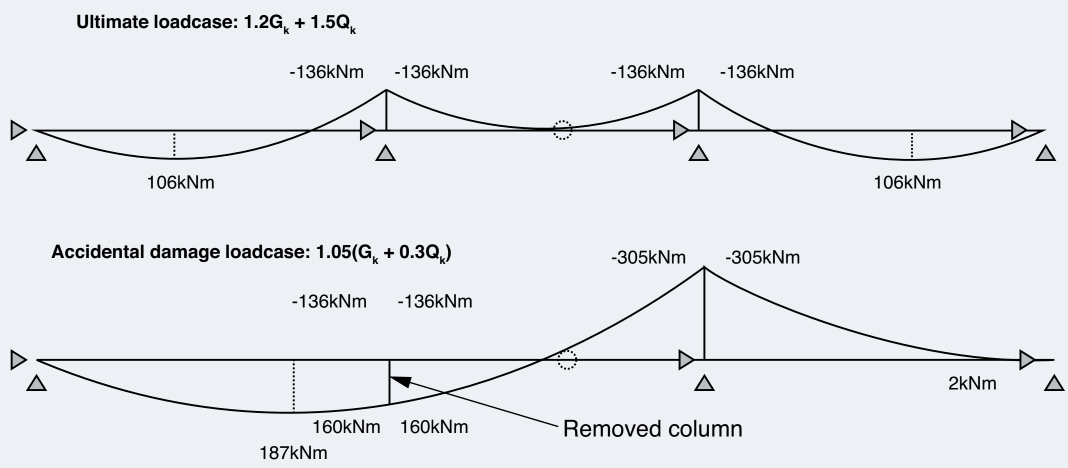


Figure 6.10: Comparison of bending moments for in service ultimate case and accidental damage case.

While the left hand primary beam shows a small increase in design moment, the right hand beam has a large increase in moment over the internal support. Although typically the floor design will be governed by serviceability requirements under normal conditions and beams will tend to have reserves of moment capacity, this is too onerous to base a member design on. Additional options to reduce this moment would be look at detailing floors to maintain some continuity to reduce the loads or to detail some composite capacity in the floors to increase the effective section depth over the column.

6.4 Stick Frame

Stick frame refers to the frame construction familiar to most engineers and builders. Built from stud walls, joisted floors and roof trusses, it is most often seen on domestic and multi-residential buildings, sometimes making up the top few storeys on a concrete-framed apartment building. Currently the tallest multi-res project in Australia constructed in this manner is Fraser Property's 'The Green' in Parkville, Melbourne. Its five storeys were built from largely prefabricated components and shows this form of construction is viable for larger buildings.

Robustness design principles established in this document will apply to this type of construction but it is anticipated that the most effective method would be to provide horizontal and vertical ties in accordance with Section 3.4 and to utilise a system of rim beams around the top of wall panels to support the floors in the event of the loss of the wall below. The tie forces established are lower than the corresponding values for steel and concrete and this is a reflection of both the lightweight nature of these buildings and the fact that they are unlikely to be economic above 5-6 storeys and therefore the consequences of failure are less. The provision of ties method and the tie forces would need to be revisited if the floor becomes more heavyweight or the consequences of failure increase (either the building use becomes more sensitive of the height increases).

In the UK in 1999, the BRE (Building Research Establishment) carried out a series of tests on a full-scale, 6-storey, timber-framed, brick-clad building. This largely focused on fire resistance testing and disproportionate collapse testing. A number of fire scenarios as the complete removal of certain walls were tested to test see how the structure performed. The results led to a number of typical details being developed to constitute best practice for multi-storey timber frame. These are included in Section 6.4.2.

6.4.1 Stick Frame Robustness Concepts

All the typical robustness concepts detailed in Section 3 are relevant for stick-framed buildings although some can be difficult to apply.

Tie forces

Tie forces considered in Section 3.4.4 are unlikely to be viable in timber frames buildings due to the magnitude of forces and the difficulty in achieving high capacity connections with small timber sections. The tie forces have been generated through consideration of heavyweight concrete and steel construction and the forces likely to be experienced by a stick frame structure are significantly less. The UK Annex to the Eurocode recognises this and reduces the maximum tie forces to be considered in timber framed buildings as below:

Class 2 Buildings

$$Internal\ Ties - T_i = 0.8(G + \psi_E Q)sL\ or\ 7.5kN\ (whichever\ is\ greater)$$

$$Perimeter\ Ties - T_p = 0.4(G + \psi_E Q)sL\ or\ 7.5kN\ (whichever\ is\ greater)$$

Class 3 Buildings

$$Internal\ Ties - T_i = 0.8(G + \psi_E Q)sL\ or\ 15kN\ (whichever\ is\ greater)$$

$$Perimeter\ Ties - T_p = 0.4(G + \psi_E Q)sL\ or\ 15kN\ (whichever\ is\ greater)$$

Where:

G = Characteristic dead load

Q = Characteristic live load

ψ_E = Combination factor for accidental actions

s = Tie spacing

L = Tie length

Ties in timber buildings

The UK Timber Frame Association (UKTFA) have developed a set of typical details designed to ensure that the buildings maintain a minimum degree of robustness. These include minimum requirements for fixing density, member sizes, etc, and were detailed as a Deemed-to-Satisfy response to robustness. Two of the primary details included in Figure 6.11 show the sorts of sizes required but Australian designers should seek to satisfy themselves that these details are appropriate for their own situation.

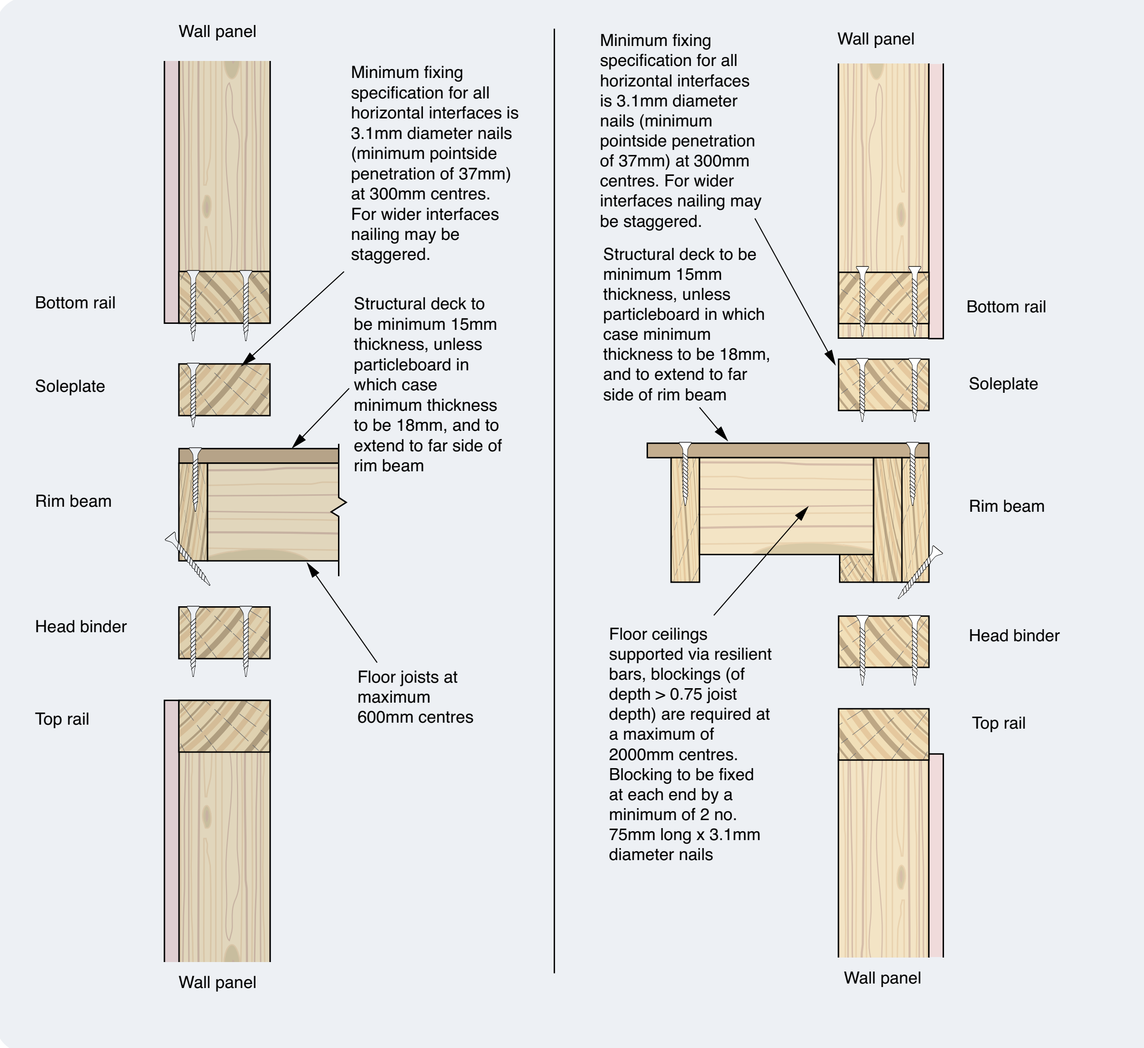


Figure 6.11: Extract from British Standards showing minimum sizes for wall-floor connections.

Rim beams

The most commonly used robustness measure in timber stick-framed buildings is to detail continuous rim beams around the top of the walls. These allow for the support of a floor if the loadbearing wall is damaged by spanning over any gaps and redistributing loads into the adjacent walls.

Figure 6.12 shows typical requirements for rim beams in a timber-framed building.

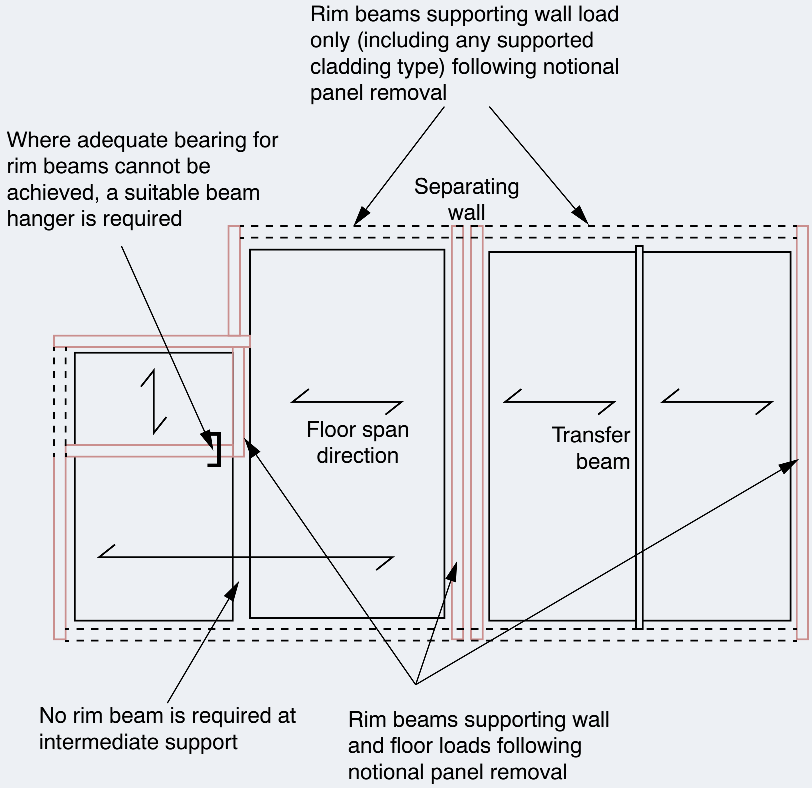


Figure 6.12: Plan of a typical timber-framed apartment building.

Section 3.5 defines the removal of a notional length of loadbearing wall as 2.25 x H so for a typical apartment building with a floor-floor height of 3.2m this translates to a maximum length of 7.2m.

G_k = 1.5kPa, Q_k = 1.5kPa, Loaded width = 4.5m (external wall), rim beam 90x300 LVL

$$w_{acc} = 9.21kN/m$$

$$M_{acc} = \frac{18.43\times7.2^2}{8} = 59.7kNm$$

$$\sigma_{acc} = \frac{59.7\times10^6}{1.35\times10^6} = 44.2MPa$$

This applied stress is high but achievable in a high grade engineered timber. It should be noted that this calculation is for the maximum length of wall removed and in most wall designs the length of wall between returns is likely to be less than 7.2m. There is also the potential to mobilise the walls above as a deep box beam with the rim beam acting as the bottom flange. Although this is difficult to demonstrate, it was an observed phenomenon in testing of the TF2000 framed building.

When detailing the rim beams, ensure that the beams would be supported if the loadbearing panel was removed (Figure 6.13). If this is difficult to achieve with a corner stud group detail, it may be necessary to include a joist hanger or another mechanical fixing to ensure the support to the rim beam is maintained.

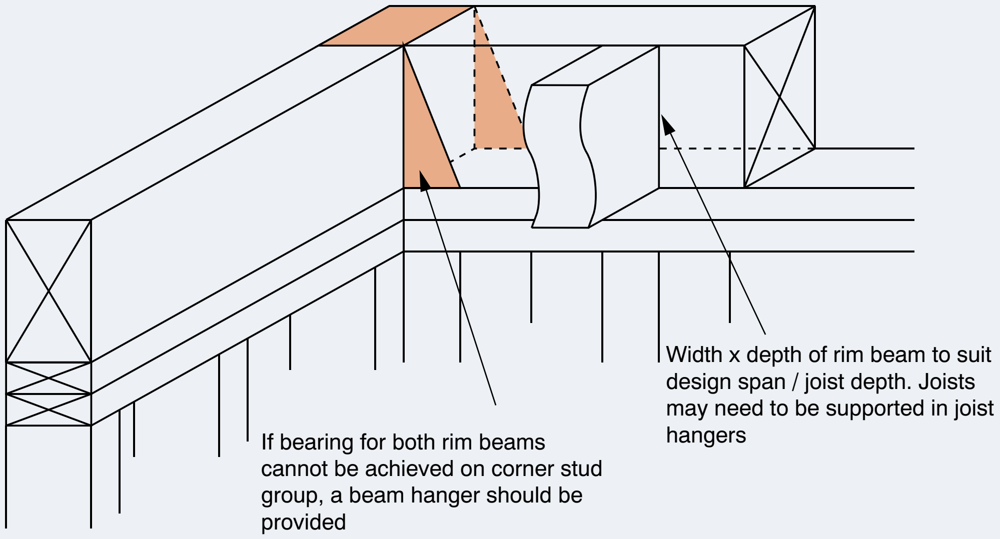


Figure 6.13: Rim beam connection with joist hanger at corner.

Notional removal of loadbearing walls

One fire test on the TF2000 project in the UK looked at the fire resistance of a wall. Following the test, the fire brigade put the fire out. Several hours later, the fire re-appeared on the floor above having been smouldering within the wall cavity. It is thought that this was due to poor installation of the cavity fire barriers.

In multi-storey timber frame construction, there is a possibility that the loss of the loadbearing stud wall could extend over more than one floor if a fire does penetrate into the cavity. It is recommended that Class 3 and above buildings should consider the removal of loadbearing walls and rim beams over two floors. The normal concepts of rim beams and alternate load paths/ties still apply for the structure but it is likely that the loss of two floors will need to be accounted for within the design. The building should still comply with the limits set out in Section 2.1.

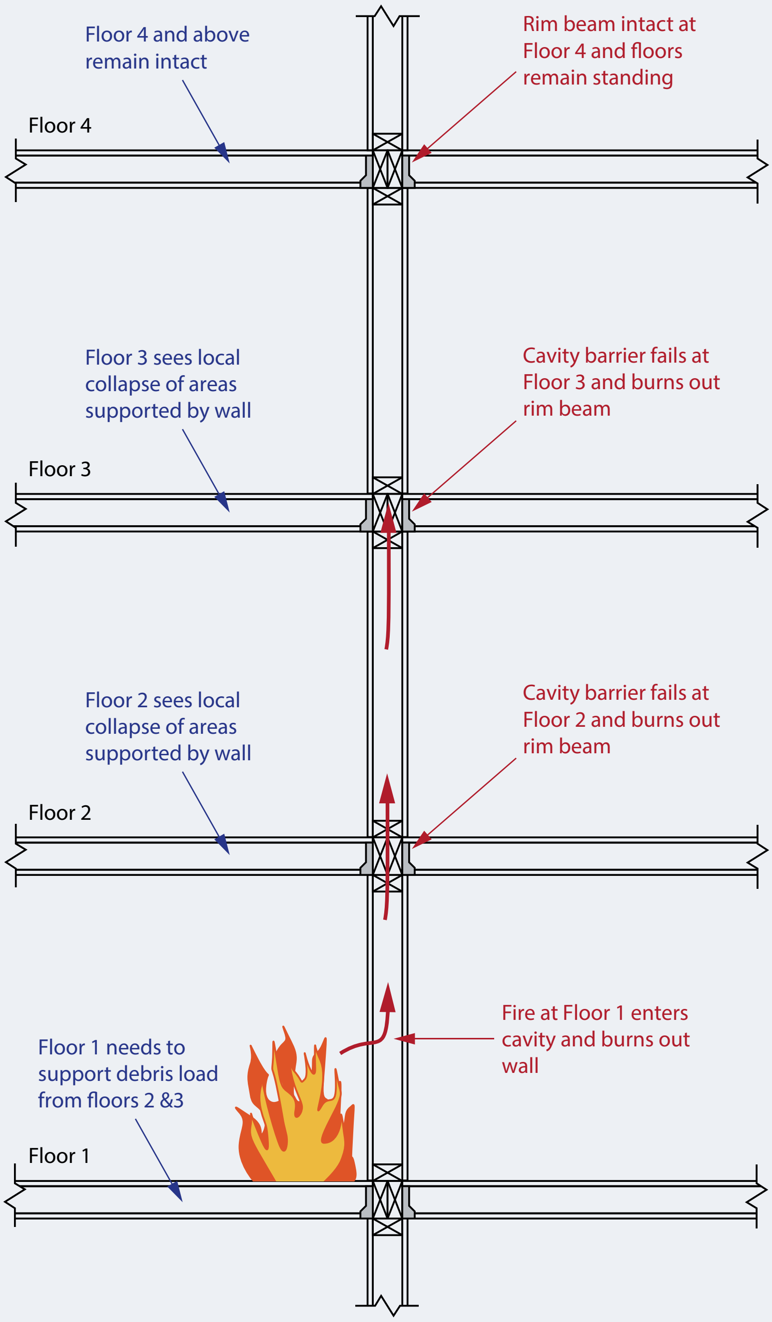


Figure 6.14: Loss of two storeys of loadbearing walls in a loadbearing timber stick-framed building.

As discussed in Section 3.7, the concept of strong floors requires consideration in this particular accidental damage load case. Allowing the collapse of two floors implies that the lower floor should be capable of supporting the debris load from the floors above, which is likely to govern the floor design. Rather than increase the design load on every floor, it is possible to design a strong floor every other floor to allow for the debris loading and reduce the impact of any potential increase in materials.

Diaphragm action

Suitable diaphragm action should be provided in all timber buildings but as timber stick-framed buildings start to increase in size, the diaphragm action should be checked more explicitly. It is important to ensure that all the loads can be transferred out of the floors and into the walls. The design of diaphragms in these buildings is well covered in the EWPAA's *Structural Plywood and LVL Design Manual*, which sets out the member and connection checks required.

Other methods of designing against disproportionate collapse

Other methods that can be potentially used to design buildings against disproportionate collapse include:

- wall panels designed to work as deep beams (difficult to do once openings are introduced but useful for party walls)
- floor cassettes designed to double span in accidental damage load cases.

6.4.2 Typical Stick Frame Details

The following details were developed from the TF2000 project in the UK and represent typical robust wall/floor connections for different classes of timber stick-framed building.

Figure 6.15 shows details for Class 2 buildings.

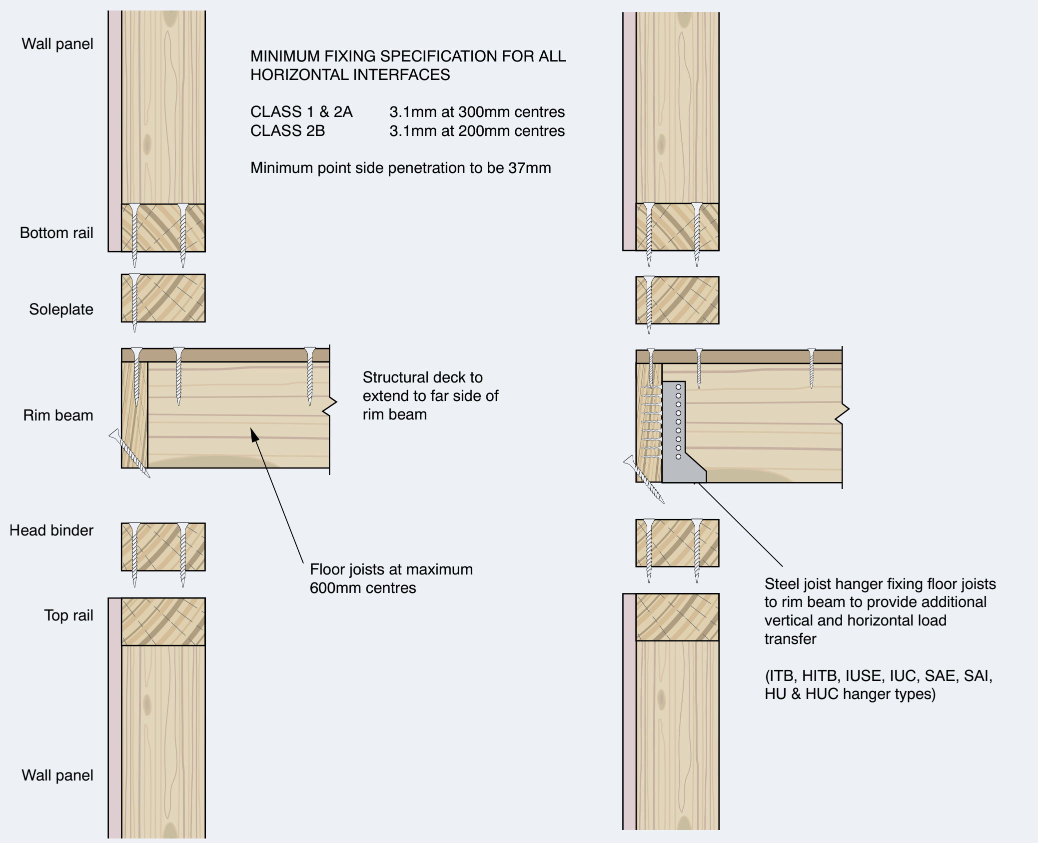


Figure 6.15: Class 2 building, typical floor-wall connection details.

Figure 6.16 shows the same detail for Class 3 buildings and it introduces an additional rim beam member as part of the floor system. This provides some additional redundancy and strength to the system as well as making it well suited to prefabricated floor units as it provides a robust edge member to the unit.

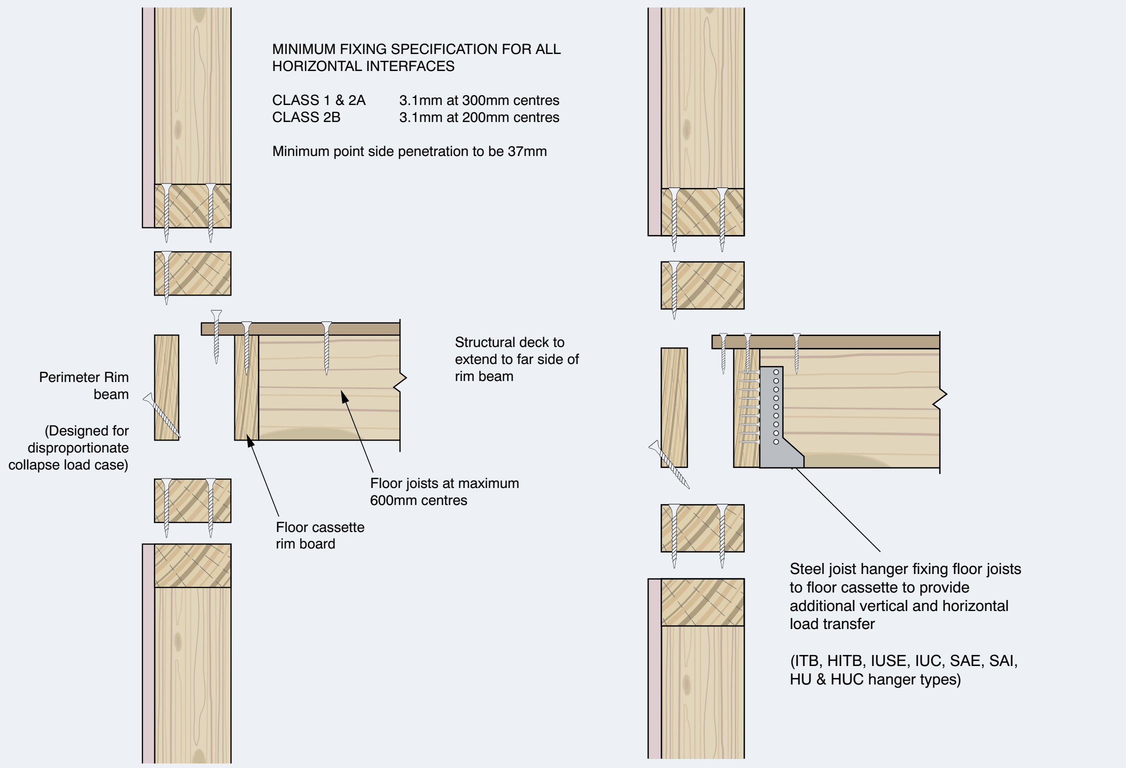


Figure 6.16: Class 3 building, typical floor-wall connection details.

Detailing of rim beams is critical to the robustness of stick-framed buildings in accidental damage load cases. Ensuring that they remain supported when the wall panels are notionally removed is vital. The details in Figure 6.17 show typical details of rim beam connections with joist hangers at corners to provide support in the event of removal of the supporting walls below.

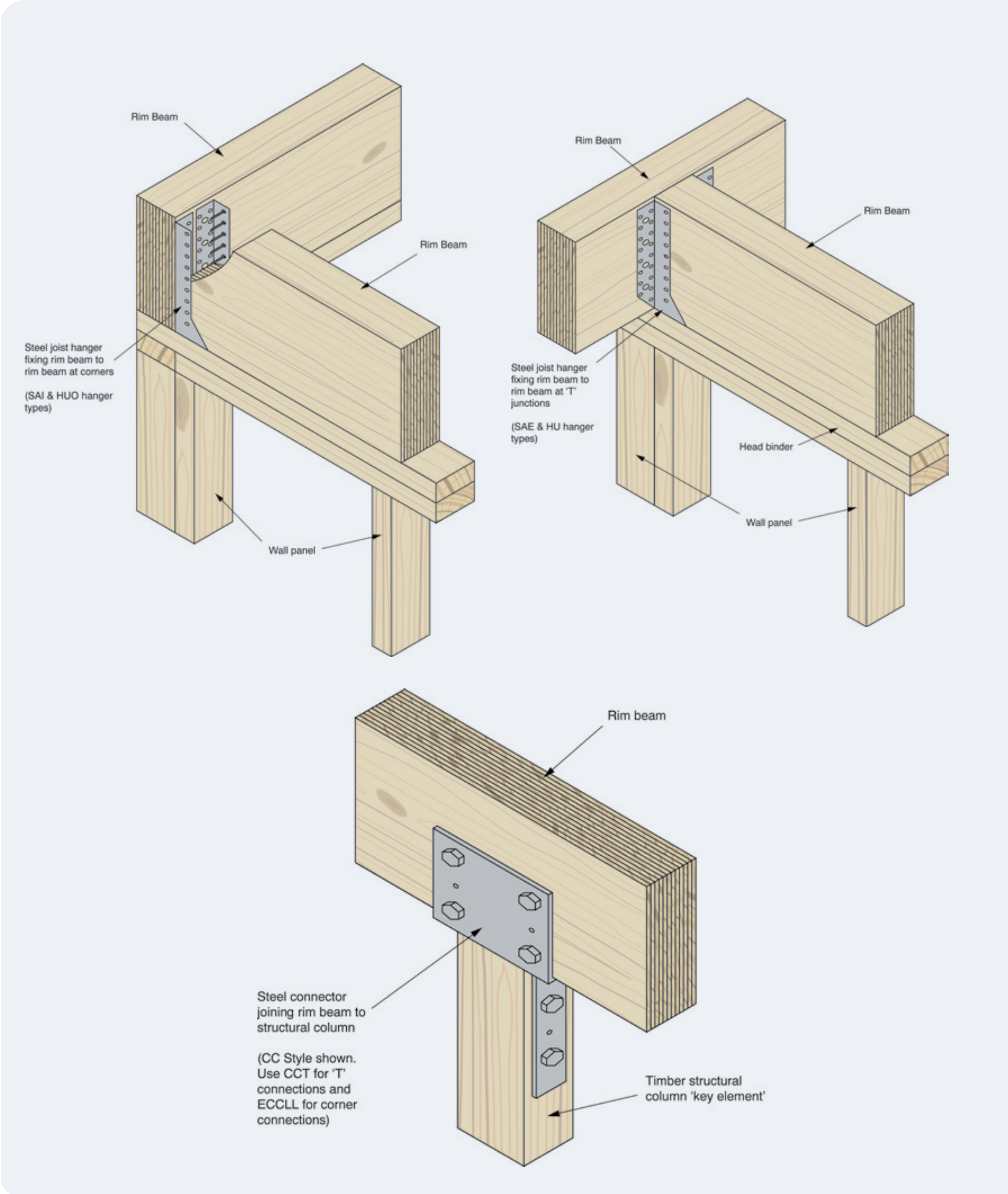


Figure 6.17: Rim beam connection details.

Transfer beams will require the same sorts of precautions as other structures and, where an accidental damage load case causes the potential loss of a transfer beam supporting more than two storeys, this may require designing as a key element. The details in Figure 6.18 show such a case.

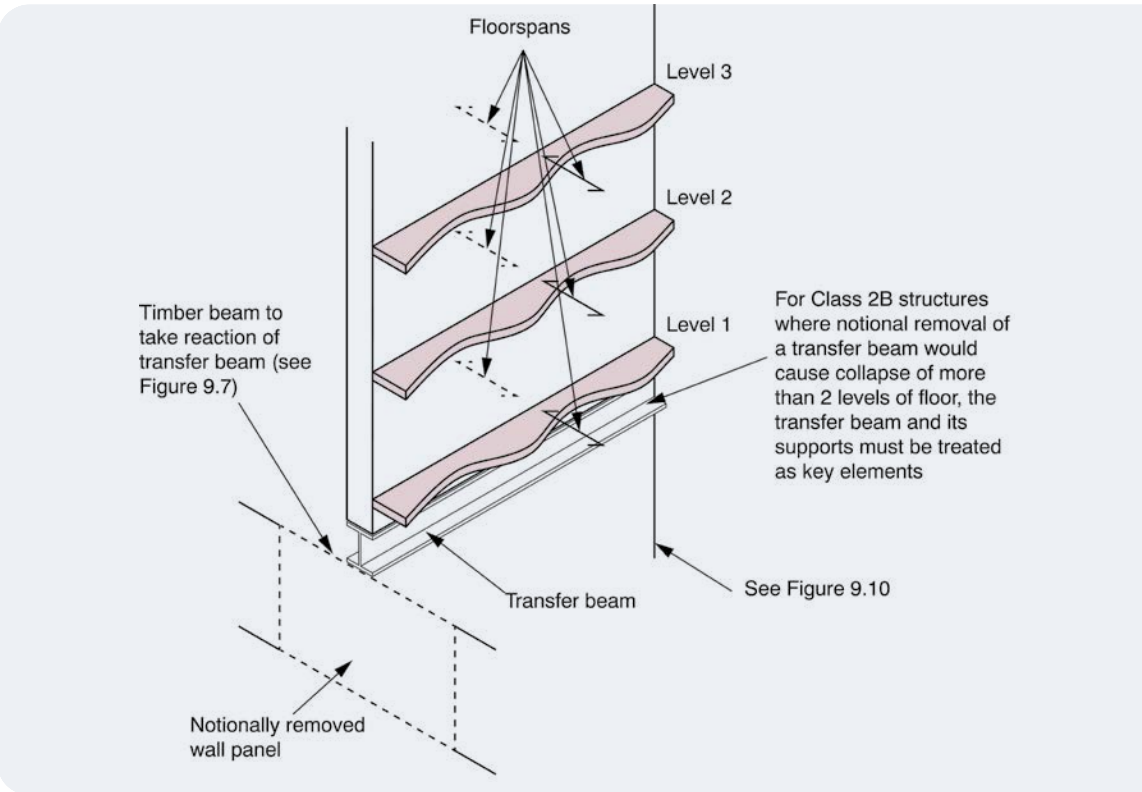


Figure 6.18: Transfer beam in timber frame multi-storey building.

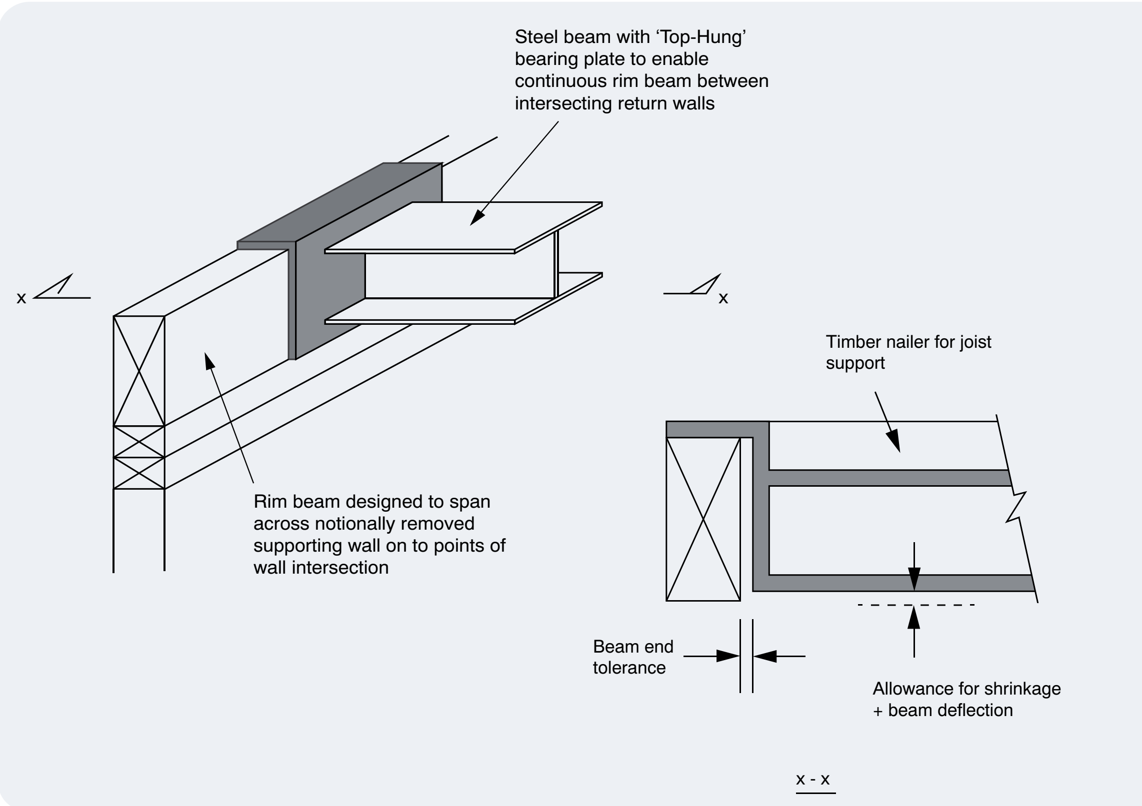


Figure 6.19: Transfer beam connection to allow rim beam to span continuously to supports.

40

WoodSolutions™
design and build
woodsolutions.com.au

This guide was published in Dec 2016
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Building Timber-framed Houses to Resist Wind

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

A growing suite of information, technical and training resources, the Design Guides have been created to support the use of wood in the design and construction of the built environment.

Each title has been written by experts in the field and is the accumulated result of years of experience in working with wood and wood products.

Some of the popular topics covered by the Technical Design Guides include:

- Timber-framed construction
- Building with timber in bushfire-prone areas
- Designing for durability
- Timber finishes
- Stairs, balustrades and handrails
- Timber flooring and decking
- Timber windows and doors
- Fire compliance
- Acoustics
- Thermal performance

More WoodSolutions Resources

The WoodSolutions website provides a comprehensive range of resources for architects, building designers, engineers and other design and construction professionals.

To discover more, please visit
www.woodsolutions.com.au
The website for wood.

Cover image: Beach House - Mornington, Clare Cousins Architects
Photography: Shannon McGrath

Wood Solutions™
design and build
woodsolutions.com.au

WoodSolutions is an industry initiative designed to provide independent, non-proprietary information about timber and wood products to professionals and companies involved in building design and construction.

WoodSolutions is resourced by Forest and Wood Products Australia (FWPA – www.fwpa.com.au). It is a collaborative effort between FWPA members and levy payers, supported by industry bodies and technical associations.

This work is supported by funding provided to FWPA by the Commonwealth Government.

ISBN 978-1-925213-42-3

Prepared by:
TimberED Services Pty Ltd
Geoff Boughton
Debbie Falck

Acknowledgements
Author: Dr Geoff Boughton

Reviewed by:
Colin MacKenzie (Technical consultant)

First Published: December 2016

Contents

Contents

1 Introduction

Wind causes lateral and uplift forces on houses that must be resisted by the appropriately designed and installed tie-downs and bracing detailed in AS 1684.

Under normal circumstances, the house structure carries the gravity loads caused by the weight of the materials in the house itself and the loads applied by furniture and occupants. However, wind loads apply forces in different directions (lateral, over-turning and uplift) that can load a house in a completely different way. Unless elements and connections in the building have been designed and constructed with sufficient capacity to resist wind loads, significant damage may result, as illustrated in Figure 1.1.

Figure 1.1: Wind damage to houses.

This Technical Design Guide provides assistance in the design and construction of houses to provide adequate performance during wind events close to the design wind speed.

1.1 Scope

This Guide outlines key design and construction considerations for wind-resistance of houses with structural timber roofs in all areas of Australia.

This Guide covers design and construction of tie-down and bracing systems for houses that use timber trusses or timber-framed roofs. It can be used for timber-framed walls or masonry walls and is suitable for cyclonic and non-cyclonic areas.

It includes:

- houses with stick-built timber-framed roofs, or timber roof trusses
- houses with timber-framed walls, or masonry walls
- houses in non-cyclonic areas (Wind Regions A and B) or cyclonic areas (Wind Regions C and D).

This Guide only applies to Class 1 and Class 10a buildings within the size limits in AS 1684 and AS 4055 (see Section 2.3.1). The Guide does not include Class 2, 3 or 4 dwellings, which can be designed using the Standards indicated in Section 3.5.1.

It provides guidance on compliance with AS 4055, AS 1684.2 or AS 1684.3, which are all primary referenced documents in the National Construction Code. AS 1684.4 is not addressed in this Guide.

Some states may have additional regulatory requirements for timber-framed construction, which are published as state or territory variations in the National Construction Code (NCC) or via other state or territory Acts and Codes.

1.2 How to Use this Guide

1.2.1 Outline of the Guide

Section 2 contains background on the Codes and Standards used in designing houses and Section 3 contains background on wind loads on houses. These sections provide the context for the other sections in this Guide.

Section 4 provides guidance for determining the wind classification for a house. Even if someone else has responsibility for the evaluation of the wind classification, make sure you understand how it is selected, and develop a 'feel' for what is a sensible classification for each site. Determining the correct site wind classification is essential to ensure every house has the appropriate details to resist the wind loads.

Section 5 gives guidance on selecting the correct tie-down connections for the house. The tie-downs form a chain of strength from the roof cladding to the ground.

Section 6 gives guidance on evaluating bracing loads and capacity in the two principle directions – parallel and perpendicular to the main ridgeline.

Section 7 contains advice for selecting elements to resist wind in new houses. It covers structural framing elements such as rafters or trusses and connections, and provides some guidance on envelope elements such as windows and garage doors. It cross-references Sections 4, 5 and 6.

Section 8 contains useful information for planning repairs or renovations so that the resilience of the house to wind actions is improved. It also cross-references Sections 4, 5 and 6.

1.2.2 Other Documents

It is not possible to design for wind resistance using this Guide alone, but it explains the steps required to design and build houses that comply with relevant Codes and Standards.

This Guide is used in conjunction with AS 1684.2, AS 1684.3 and AS 4055.

To understand this Design Guide, it is essential to have access to copies of AS 1684 (Parts 2 or 3) and AS 4055.

This Guide is part of a suite of WoodSolutions resources, including Technical Guides and AS 1684 User Guides. Other appropriate resources to assist with the use of AS 1684 are listed in the References section.

2 Building Codes and Standards

2.1 National Construction Code and Building Code of Australia

All buildings must comply with the National Construction Code (NCC); Volume 1 for most buildings, and Volume 2 for houses only (Class 1 and class 10a). Volumes 1 and 2 of the NCC are the Building Code of Australia (BCA). Volume 3 of the NCC concerns plumbing and is not referred to in this Guide. Any reference to the BCA in this Guide includes the NCC.

AS 4055 and AS 1684.2 or AS 1684.3 are acceptable construction manuals and are used to design most houses in Australia.

The BCA provides Deemed-to-Satisfy provisions that reference Australian Standards and other documents. It represents a minimum standard for construction. In some cases, owners may want higher levels of performance than the minimum set out in the BCA.

The BCA nominates acceptable construction manuals. For housing, wind loads are evaluated using AS 4055 or AS/NZS 1170.2, and the timber framing must comply with AS 1684 parts 2, 3 or 4, as illustrated in Figure 2.1.

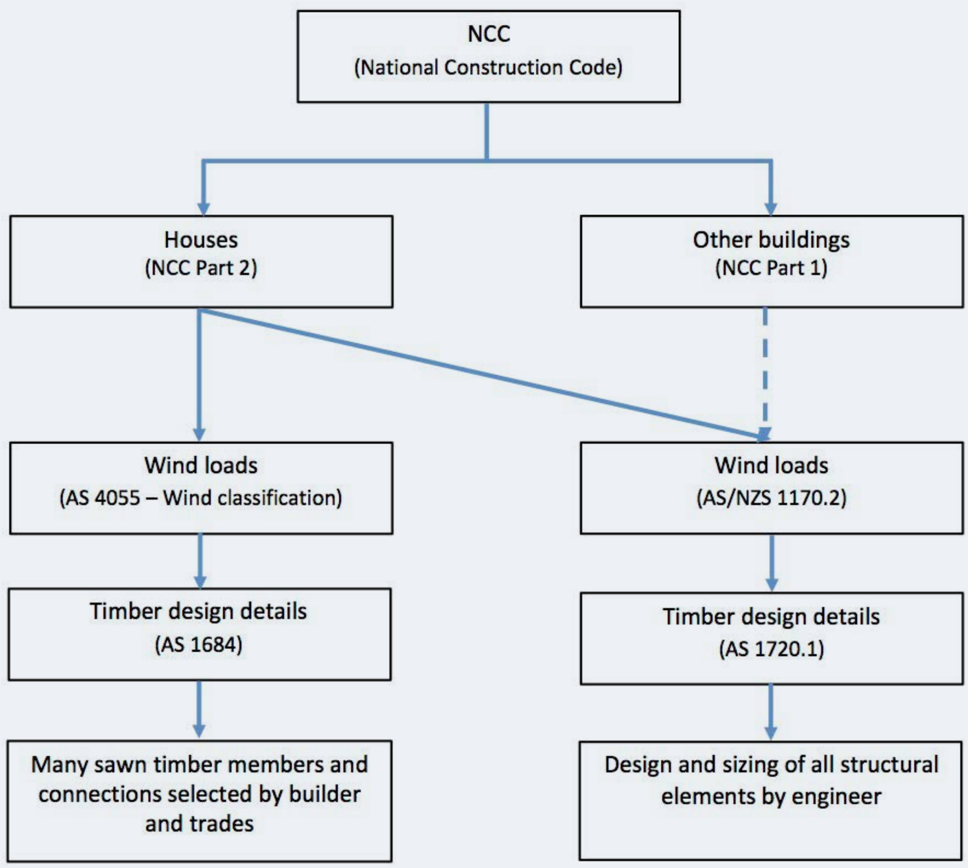


Figure 2.1: Deemed-to-Satisfy acceptable construction manuals for wind loads on houses.

The BCA's objectives relevant to wind loads are to:

- safeguard people from injury caused by structural failure
- safeguard people from loss of amenity caused by structural failure
- protect other property from physical damage caused by structural failure
- safeguard people from injury that may be caused by failure of, or debris impact with, glazing.

To meet these objectives, the building must:

- remain stable and not collapse
- prevent failure from progressing following minor damage
- minimise loss of amenity
- avoid causing damage to other properties.

The BCA requires all buildings to protect occupants by remaining stable, preventing failures, and avoiding loss of amenity and damage to other buildings.

The BCA indicates the need for tie-downs and bracing for sites in high wind areas (N3 and above, and all C classifications), as shown in Figure 2.2. However, even in N1 and N2 wind classifications, specific tie-down and bracing systems are required.

Construction in high-wind areas

The intent of building construction in high-wind areas is to ensure the structure has sufficient strength to transfer wind forces to the ground with an adequate safety margin to prevent the collapse of the building and the building being lifted, or slid off its foundations.

To resist these forces it is necessary to have:

- *an anchorage system, where the roof is connected by the walls to the footings by a chain of connections*
- *a bracing system to prevent horizontal collapse due to wind forces*
- *continuity of the system where each structural element is interlocked to its adjoining structural element throughout the building.*

Anchorage of the system is achieved by using a variety of connectors. Each connector must be capable of carrying the uplift force, because the ability of the building to resist the wind forces is directly related to its weakest link.

Figure 2.2: Explanatory information on construction in high wind areas. Source: BCA Volume 2 Clause 3.10.1.0

The BCA sets the design level for housing (Importance Level 2) as a minimum annual probability of exceedance of 1:500 for wind. (The 1:500 probability of exceedance is equivalent to a 10% chance in 50 years that the wind speed will exceed the design level.) This should be the minimum V_R (Design wind speed in metres/second) selected for houses if AS/NZS 1170.2 is used to calculate wind loads. It is the design level used in AS 4055.

2.2 Standards Used to Determine Wind Loads

The Australian and New Zealand Standard for structural design wind actions, AS/NZS 1170.2, as shown in Figure 2.3, and the Australian Standard for wind loads on houses, AS 4055, divides Australia into Wind Regions. A similar map is published in the BCA. AS/NZS 1170.2 and the BCA maps sub-divide Wind Region A in Australia into A1 to A5 depending on wind direction. The regional design wind speed is the same for A1 to A5. AS 4055 does not differentiate between the sub-regions.

Wind Regions are defined in maps in the BCA, AS/NZS 1170.2, and AS 4055.

Wind Regions A and B are classified as 'non-cyclonic areas' and Wind Regions C and D as 'cyclonic areas'.

Strong winds can be generated by severe thunderstorms in any part of Australia, and frontal systems south of the tropics. Tropical cyclones only occur in northern coastal areas of Australia, but can travel further south into non-cyclonic areas as they weaken.

- Wind Region A design winds are associated with severe thunderstorms, large frontal systems or significantly weakened tropical cyclones.
- Wind Region B design winds are generally associated with severe thunderstorms, or tropical cyclones that have weakened a little.
- Wind Region C design winds are associated with tropical cyclones as they cross the coast.
- Wind Region D design winds are associated with severe tropical cyclones.

The scopes of AS/NZS 1170.2 and AS 4055 do not include tornados, due to difficulties in evaluating probabilities of their occurrence at any site (the US wind standard also excludes tornados for the same reason). However, tornados have occurred in all Wind Regions in Australia and estimated wind speeds generated by the tornados were close to the design wind speed in many cases. High-intensity tornados have the potential to damage houses that comply with AS 1684.2 or AS 1684.3.

There are different design wind speeds for each Wind Region. Damage investigations following severe wind events in all parts of Australia have shown that recently constructed houses can be structurally damaged by winds less than the design wind speed if design and construction do not meet the requirements of the relevant Standards.

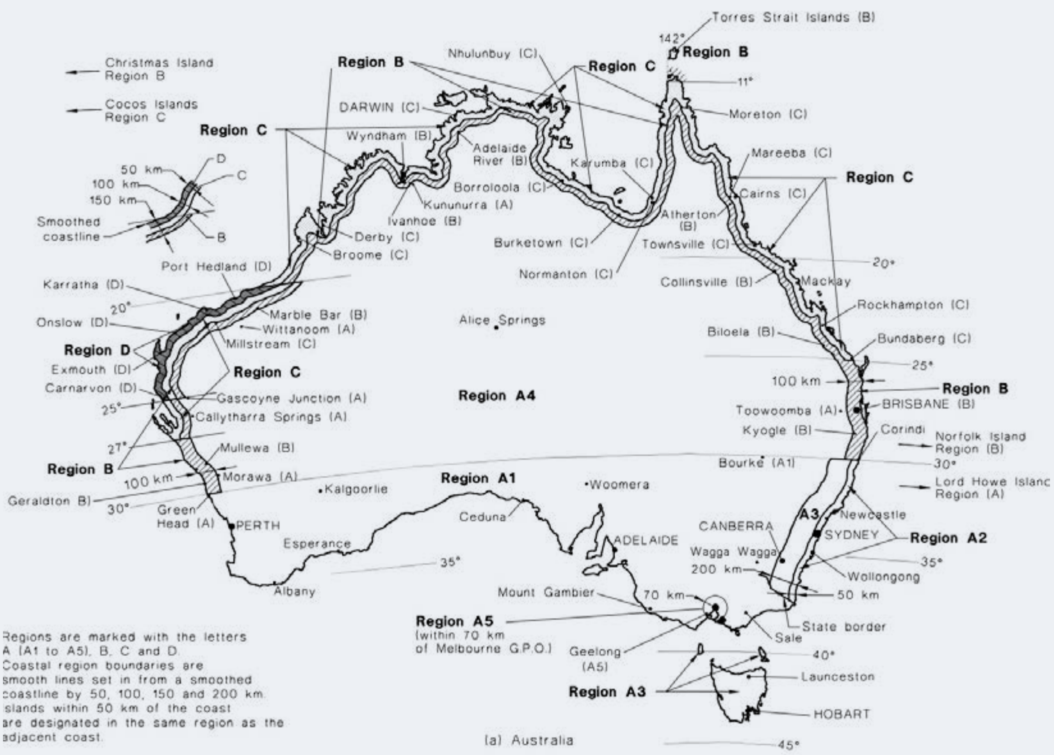


FIGURE 3.1 (in part) WIND REGIONS

Figure 2.3: Wind Regions of Australia. Source: AS/NZS 1170.2 Figure 3.1 (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

2.2.1 Gust Wind Speeds

Design wind speeds are a notional peak gust speed that the building must be designed to resist. It is estimated that any house has 1-in-10 chance of experiencing the design wind speed during its 50-year life. Gusts may only last less than a second or two, and larger gusts can completely envelope a house. All gusts can apply significant loads to the structure.

2.2.2 Cyclonic Areas

Tropical cyclones develop over the warm oceans to Australia's north during the hotter months from November to April, and can bring destructive winds, heavy rain and flooding to many coastal areas in northern Western Australia, the Northern Territory and northern Queensland. The winds in tropical cyclones rotate clockwise around a central low pressure area ('eye') that generally has a diameter of about 15–50 km. Cyclones can cross the coast and track overland at varying speeds as they decay into low-pressure systems. As a cyclone passes over a given location, it generates increasing then decreasing winds from different directions, and may subject communities to severe weather for several hours. The longer a community is exposed to these high winds, the more severe the damage is likely to be due to increased levels of wind-borne debris and sustained cyclic loading on cladding materials.

Cyclonic areas are a strip at least 50 km wide around the northern Australian coastline.

Tropical cyclones are usually hundreds of kilometres wide, can travel hundreds of kilometres and can affect an area of hundreds of square kilometres. The highest winds are experienced just outside the eye, and the peak gust speed decreases with distance from the edge of the eye. The Bureau of Meteorology categorises cyclones as shown in Table 2.1 with increasing severity from Category 1 to 5 according to the maximum expected gust wind speed near the centre of the cyclone.

Table 2.1: Bureau of Meteorology Cyclone Categories.

Cyclone Category	**Gust Wind Speed near the centre of the cyclone at 10 m height in flat open terrain**			**Bottom of wind speed range is within the design wind speed**
	km/h	**Knots**	**m/s**	**Wind Regions**
1	**90–125**	49–68	25–35	A, B, C, D
2	**125–164**	68–89	35–46	A, B, C, D
3	**165–224**	89–121	46–62	B, C, D
4	**225–279**	121–151	62–78	C, D
5	**>280**	>151	>78	D

For example, a Category 4 tropical cyclone crossing the coast may cause wind speeds near the centre that are close to the design wind speed for Wind Region C housing. At that time, the winds will be significantly lower for houses that are around 50 km from the centre, which may experience winds equivalent to a Category 3 event. The winds even further from the centre will be the equivalent of a Category 2 event or, hundreds of kilometres away, a Category 1 event.

As the eye of the tropical cyclone progresses over land, the central wind speeds reduce, so that after travelling across the land for 40 to 50 km, the wind speeds near the centre will be equivalent to a Category 3 event. The wind speeds further from the centre will be even less. This is illustrated in Figure 2.4

Weakened tropical cyclones can pass into non-cyclonic areas and still not exceed the design wind speeds for those Wind Regions.

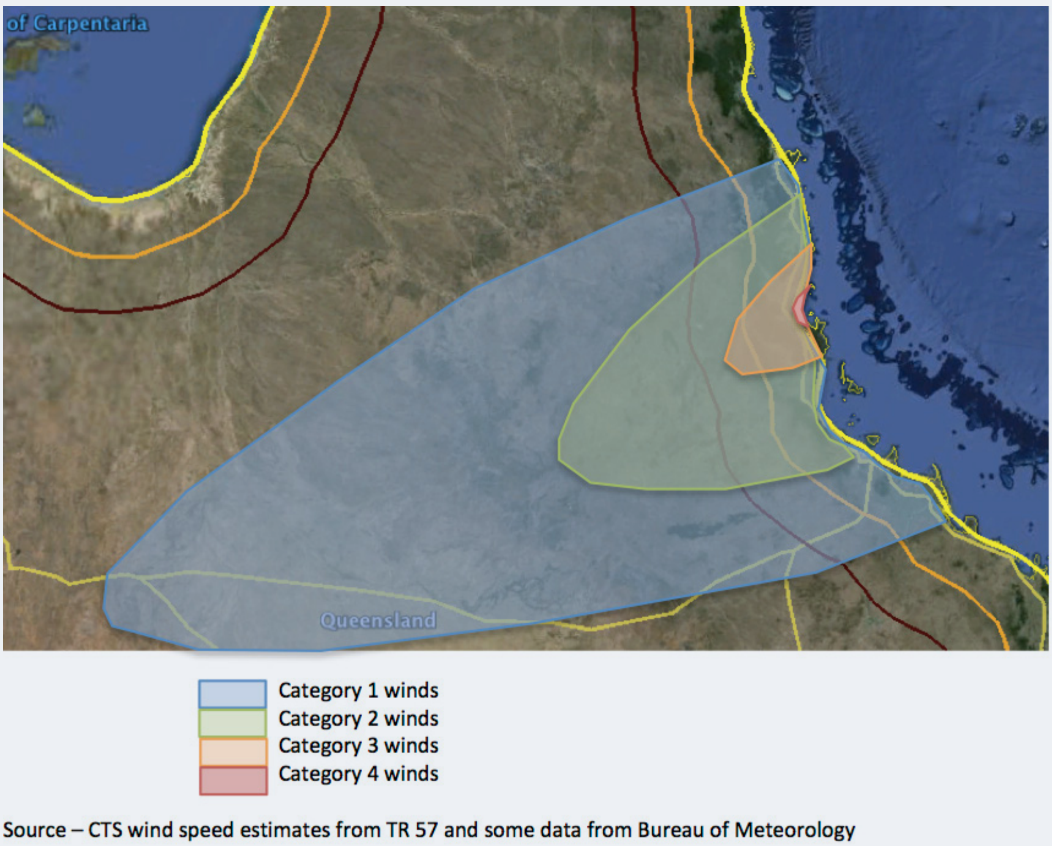


Figure 2.4: Peak gust wind categories at different locations during Tropical Cyclone Yasi in February 2011.

2.2.3 Non-cyclonic Areas

The design wind event in Wind Regions A and B is usually associated with a localised frontal weather system or severe thunderstorm, both of which typically last for less than an hour. Designers and builders should not underestimate the damage that even 120 km/h winds can inflict on houses with inadequate roof connections in these Wind Regions.

Weakened tropical cyclones often pass through Wind Regions B and A. Table 2.1 shows that houses in Wind Regions A and B are designed to resist wind speeds greater than those in Category 1 and most Category 2 tropical cyclones.

2.2.4 AS/NZS 1170.2 Structural Design Actions – Part 2: Wind Actions

This Standard is the primary wind loading Standard for Australia and New Zealand. It is a referenced document in the BCA and can be used by engineers to select wind loads for any structural members on buildings (see Section 3.5.1). If this Standard is used to evaluate wind loads on buildings, a wind classification (e.g. N1 or C2) is not given.

2.2.5 AS 4055 Wind Loads for Housing

This Standard is a simplified wind loading Standard restricted for use with houses. It is compatible with AS/NZS 1170.2 and gives similar wind load effects to AS/NZS 1170.2. Both AS 4055 and AS/NZS 1170.2 use the same Wind Regions shown in Figure 2.3. Either Standard can be used to select wind loads on houses. AS 1684 uses wind classifications consistent with AS 4055.

AS 4055 is compatible with AS/NZS 1170.2, but uses simplified calculations and is only suitable for design of houses.

AS 4055 presents methods for:

- evaluating the wind classification to be used in the design and construction of a house
- calculating wind pressures on elements such as roofing, windows and wall cladding
- determining the tie-down forces at the top of walls
- evaluating racking loads to be resisted by bracing elements in the house.

The first two of these steps are detailed in Section 4, and the other two are presented in Section 5 and Section 6, respectively.

2.3 Residential Timber-framed Construction Standards

Australia has a suite of Standards that support the design and construction of timber-framed houses. Each contains:

- building practice notes on arrangements of members, connection methods and geometric limits
- information on determining member sizes using span tables, the building plans, timber grade and wind classification
- methods for calculating bracing loads on the house and establishing the bracing resistance of the walls in the house
- tie-down requirements for all structural elements on the load path between the roof cladding and the ground.

AS 1684 is a suite of Standards that provides comprehensive specifications for the design of timber-framed housing.

2.3.1 Geometric Limitations

AS 1684 and AS 4055 have geometric limitations on houses that can be designed using these documents. These limitations ensure that the simplifying assumptions that enable the use of the deemed-to-satisfy construction manuals on the left hand path in Figure 2.1 are valid. If any house does not fall within these limitations, it must be designed using AS/NZS 1170.2 and AS 1720.1, as shown in the right hand path in Figure 2.1.

AS 1684 and AS 4055 can only be used to design houses within size and height limitations.

Limitations on the use of AS 4055 and AS 1684 include:

- maximum number of storeys anywhere in the building = two
- maximum wall height under floors or under the lowest point of the ceiling = 3 m
- maximum distance between external walls across a ridge line or under a skillion roof = 16 m
- maximum distance between any two bracing walls in the same direction = 9 m
- maximum roof slope = 35°.

2.3.2 AS 1684.2 Non-cyclonic Areas

This volume contains the information needed for design and construction of houses in Wind Regions A and B. The format of AS 1684.2 and its span table supplements is similar to that of AS 1684.3.

2.3.3 AS 1684.3 Cyclonic Areas

This volume contains the information needed for design and construction of houses in Wind Regions C and D. The higher wind speeds in these regions mean that the tie-downs and bracing must have higher capacity than those in Wind Regions A and B. The format of AS 1684.3 and its span table supplements is similar to that of AS 1684.2. AS 1684.3 only provides design solutions for wind classifications up to C3. Higher wind classifications are possible and will need to be engineer-designed and certified using AS/NZS 1170.2 and AS 1720.1, as shown in the right-hand compliance path in Figure 2.1.

2.3.4 AS 1684.4 Simplified – Non-cyclonic Areas

This document is applicable to houses with N1 or N2 classifications only. It is set out differently to the other parts of AS 1684 and has span tables in a different format. Because this part is restricted to very low-wind classifications, it has simplified tie-downs and bracing details. AS 1684.4 is not addressed in this Design Guide.

This Design Guide does not include the use of AS 1684.4.

2.4 Residential Timber-framed Construction

2.4.1 Different Construction Methods

Houses can be designed and constructed using a variety of different combinations of cladding and structural elements. Different Standards apply to different construction methods and materials:

- Timber floor frame, timber wall frames and timber-framed roof – AS 1684 applies.
- Timber floor cassette system, timber wall frames and timber-framed roof – AS 1684 applies to the wall and roof frames. Cassette floor manufacturers provide details on the floor system.
- Concrete slab, timber wall frames and timber-framed roof – AS 1684 applies to the wall frames and roof frames. Forces in tie-downs to the slab can be calculated using AS 1684.
- Concrete slab, solid masonry walls and timber-framed roof – AS 1684 applies to the roof frames only. Forces in tie-downs from the roof to the walls can be calculated using AS 1684.
- Timber floor frame, timber wall frames and timber-trussed roof – AS 1684 applies to all but the trusses (use AS 1720.5 and AS 4440 for the trusses).
- Concrete slab, timber wall frames and timber-trussed roof – AS 1684 applies to wall frames. Forces in tie-downs to the slab can be calculated using AS 1684. Use AS 1720.5 and AS 4440 for the trusses.
- Concrete slab, masonry walls and timber-trussed roof – AS 1684 does not apply. Use AS 1720.5 and AS 4440 for the trusses.

Lightweight timber-framed construction systems can be prefabricated off-site into wall frames, floor and roof trusses, or cassette floor modules, and then erected on-site. AS 1684 applies to timber wall frames and tie-down and bracing requirements for the whole house. AS 1720.5 and AS 4440 provide requirements for trusses.

2.4.2 Timber Materials

A number of different sawn and manufactured timber products are used in timber-framed house construction.

Sawn timber products

Sawn timber products include seasoned structural softwood (MGP10 and MGP12 with limited amounts of MGP15 available) and seasoned or unseasoned structural hardwood (typically F8 to F27 or A17). Commonly used thicknesses are 35 mm and 45mm for seasoned timber, and 38 mm, 50 mm and 75 mm for unseasoned timber.

AS 1684 includes span tables and construction practices that are based on commonly available stress-graded sawn timber.

Other timber materials used in domestic construction are shown in Figure 2.5.

AS 1684 span tables only include sawn timber products. Manufacturers of engineered wood products provide design support for their products.

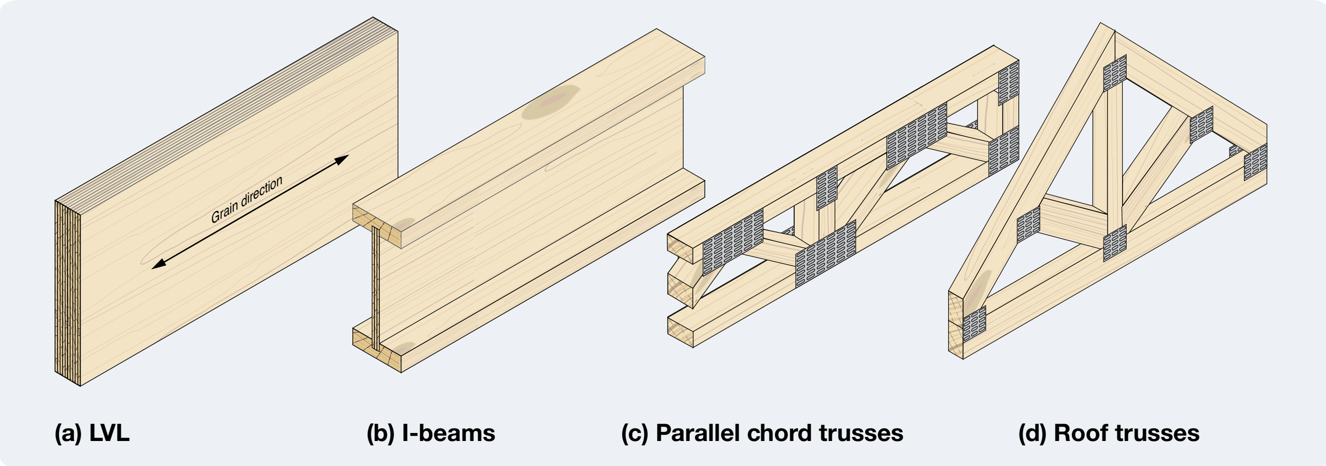


Figure 2.5: Other timber products used in house construction in Australia.

Laminated Veneer Lumber (LVL)

Some lightweight framing elements are also available in Laminated Veneer Lumber (LVL), an engineered wood product available in standard framing sizes. LVL is manufactured by bonding together rotary peeled or thinly sliced wood veneers under heat and pressure. The grain in the veneers is all oriented in the same direction and LVL elements are suitable for beam or stud applications.

LVL should be marked as complying with AS/NZS 4357. Individual LVL manufacturers provide design information that may include span tables consistent with AS 1684, and additional construction practice information that is specific to their LVL products. Extra requirements for nail lamination of LVL and connection of rim-boards to resist bracing loads are provided in Appendix J of AS 1684.2 and AS 1684.3.

I-beams

I-beams are lightweight, high-strength, long-span structural timber beams. They typically comprise top and bottom flanges of LVL or solid timber that make the distinct 'I' shape. The flanges are separated by a vertical web, usually manufactured from structural plywood, oriented strand board (OSB) or light gauge steel.

They can be used in floor systems as joists and/or bearers, or in roofs as rafters. Manufacturers provide span tables and information on the variations to construction practice that may be required in order to use their products. Appendix J of AS 1684.2 and AS 1684.3 give information on modifications to construction practices for I-shaped sections.

Parallel chord trusses

Parallel chord trusses are similar to I-beams in that they have top and bottom chords made from LVL or solid timber. However, instead of solid webs, web struts made from either timber or light gauge steel are secured to the chords with toothed plates. The struts may be diagonal (more common for steel struts) or a mix of vertical and diagonal (more common for timber struts).

Although parallel chord trusses already have openings in the web, many of the limitations for I-beams on splays and birdmouths in Appendix J of AS 1684.2 and AS 1684.3 also apply to parallel chord trusses.

Figure 2.6: Hybrid floor system with parallel chord trusses supported from a steel I-beam.

Roof trusses

Timber roof trusses provide an engineered roof frame system designed to carry the roof or roof and ceiling loads, usually without the support of internal walls. Light truss roofs typically span 2–16 m, are manufactured from sawn or LVL timber elements connected with nailplates or other mechanical fixings, and are designed and supplied by frame and truss manufacturers.

Roof trusses are designed to AS 1720.5 and installed to AS 4440. The truss manufacturer specifies connections between the trusses and the remaining structure. In the absence of other information, Section 9 of AS 1684.2 and AS 1684.3 provide some detail on connections between the trusses and the wall structure.

3 Wind Effects on Buildings

Wind exerts positive pressures on the windward wall, and negative pressures (suction) on the roof, leeward and side walls of buildings, as illustrated in Figure 3.1.

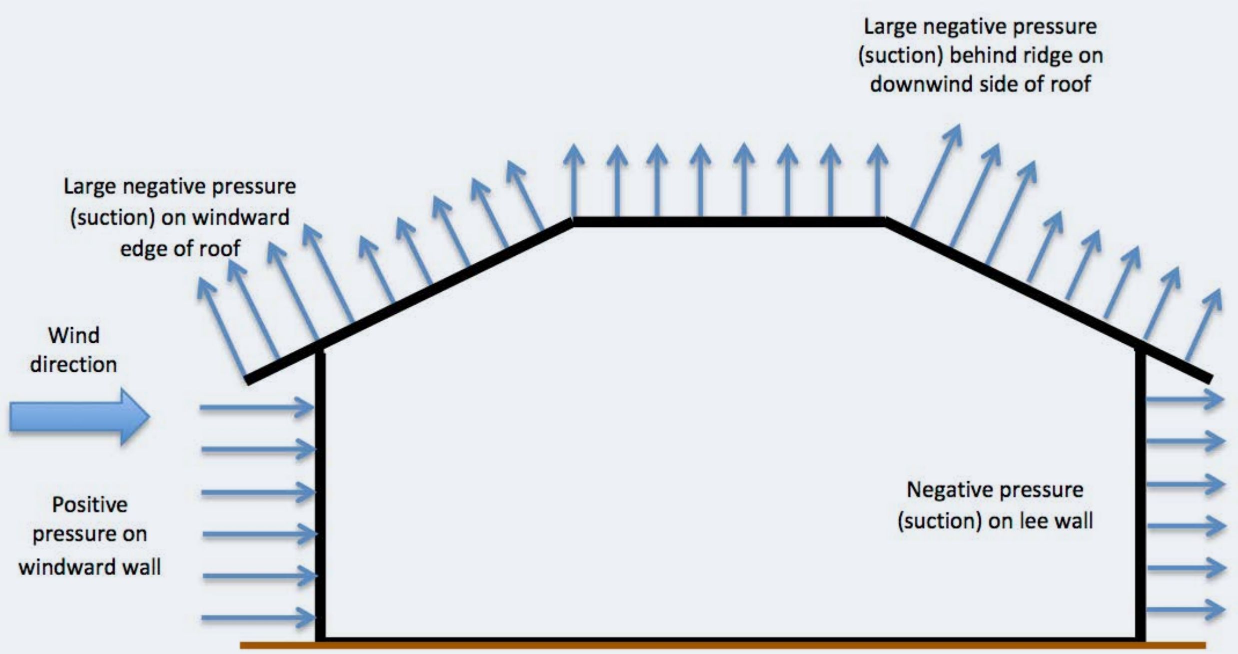


Figure 3.1: Representation of wind pressures on external surfaces of a building.

Wind pressure on a surface causes a force that is equal to the pressure times the area over which it acts. Wind loads or forces on individual elements can be calculated by multiplying the wind pressure by the contributing area of each structural element. Buildings must be designed to resist the forces that these pressures cause.

Wind causes positive pressure on the wall facing the wind and negative pressure (suction) on most other surfaces.

3.1 Pressure on Windward Walls

The only external surface of the house that experiences positive pressure from the wind is the wall facing the wind – the windward wall. Pressure on the windward wall can cause a number of problems:

- The pressure of the wind can damage some cladding elements and push them into the house. This type of failure has been observed in windows and doors where elements such as connections of the door or window frame to the wall, glazing or latches and hinges are not strong enough to carry the loads.
- Debris such as branches, outdoor furniture, boats, wheelie bins or sections of damaged buildings can be picked up by the wind and strike the windward wall of the house. This can contribute to the damage. (While wind-borne debris is frequently associated with tropical cyclones, it is also often seen in high winds in non-cyclonic areas.)
- Wind is often accompanied by rain, and wind-driven rain is blown horizontally onto the windward walls. If water is driven through the building envelope it causes water damage to building linings, contents and furnishings.

Any damage to the windward wall has the potential to increase the pressure inside the house – the internal pressure.

3.2 Internal Pressures

Figure 3.1 shows the pressures and suctions generated by the wind on the outside of the building. However, at the same time, there will also be positive or negative pressures on the inside of the house. These pressures are negative or positive depending on whether there are openings in the building envelope. Openings may be caused by:

- small gaps around doors and windows throughout the house
- occupants leaving doors or windows open
- wind pressure forcing a door, window or garage door open
- wind-borne debris breaking cladding or glazing elements on the windward wall.

Where the openings are roughly the same on all walls (e.g. small gaps around all doors and windows, or if all of the windows are left open by the occupant), the internal pressures tend to be negative. This is because the majority of the building surfaces have negative external pressure (suction) on them, as shown in Figure 3.1.

Any opening in the building envelope will cause internal pressures that contribute to the loads on the structure.

Wind-borne debris only damages the building envelope on the windward side, and doors, windows and garage doors can blow inward on the windward side. Figure 3.2 shows internal pressures on a house where a large opening has developed in the windward wall. This opening means that the high positive pressure on the windward wall is applied to all internal surfaces (red arrows). The internal pressures cause forces that act in the same direction as the external suction forces on the roof and the leeward wall. This increases the net forces on roof and wall elements (see Section 3.3).

The principles for designing to resist internal pressures are as follows:

- When designing to AS/NZS 1170.2, designers must anticipate the worst likely opening configuration for each structural element. For example, if designing roof members, the worst opening scenario is an opening in the windward wall as illustrated in Figure 3.2.
- When designing houses in cyclonic areas (Wind Regions C and D), AS 4055 assumes high internal pressures caused by large openings in the critical walls. For example, in sizing roof tie-downs, an opening in a windward wall as shown in Figure 3.2 is assumed.
- When designing houses in non-cyclonic areas (Wind Regions A and B), AS 4055 assumes low internal pressures (no doors and windows open at all) and does not model the effect of openings caused by damage to the windward wall. Selecting wind-rated components for all elements in the external envelope of a house will contribute to the resilience of houses in Wind Regions A and B. This supports the assumptions in AS 4055 of low internal pressure.

AS 4055 assumes high internal pressures for houses in cyclonic areas and low internal pressures for houses in non-cyclonic areas.

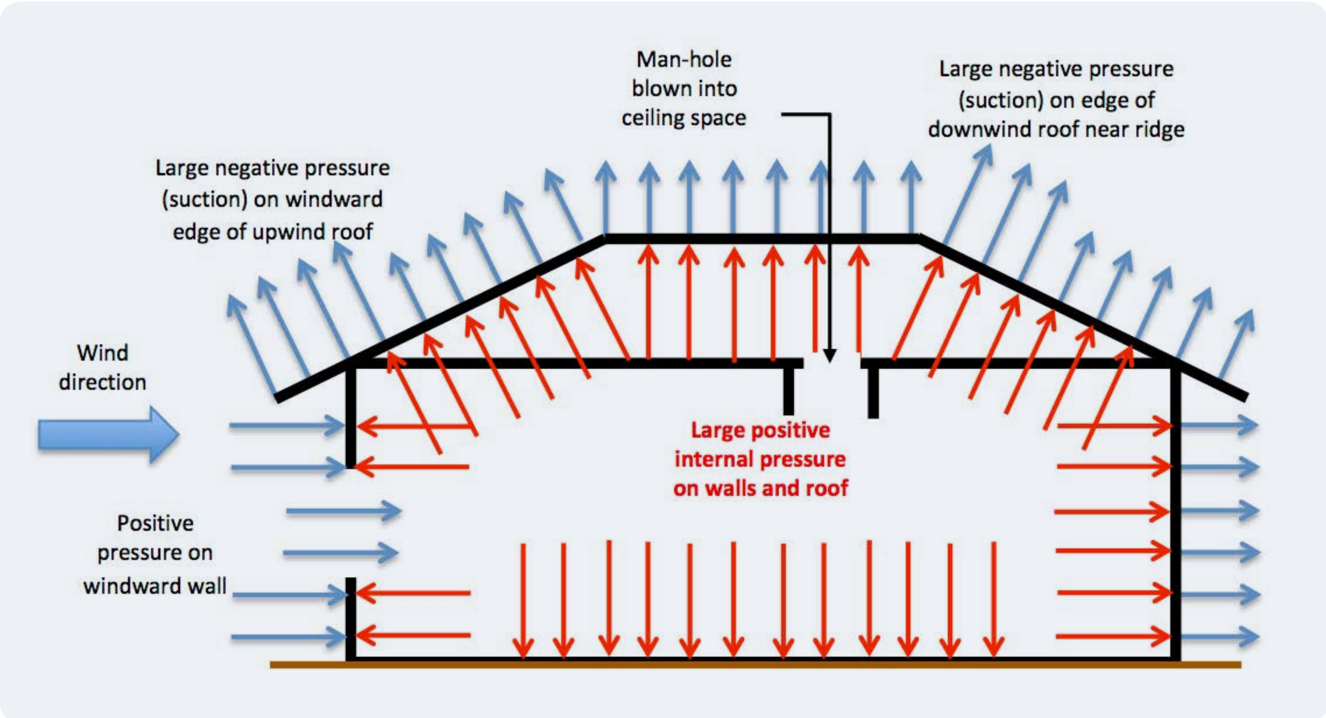


Figure 3.2: Wind forces with a dominant opening in windward wall.

3.3 Uplift on External Surfaces

All roofs (flat, pitched, hips, gables, simple or complex) experience wind uplift due to suctions created by the wind as it passes over the top of the roof surface. Figure 3.1 shows that the suctions on the roof create forces in an upward direction that tend to lift the roof off the walls. Figure 3.2 shows that an opening on the windward wall creates internal pressures acting in the same direction. The combination of positive internal pressure and external suction on roof surfaces can almost double the uplift loads on the roof. Many roof failures have been triggered by failure of elements on the windward wall of houses.

These upward forces can be large. For example, the lowest design wind speed for houses in AS 4055 produces a net upward force on a suburban house of 0.7 tonnes per 10 m^2 of roof area. On other houses, the loads can be significantly more (e.g. up to 6.4 tonnes per 10 m^2 of roof area for houses on hill tops in cyclone areas).

The upward forces from the wind exceed the weight of the materials in lightweight roofs, so tie-down systems are necessary to keep the roof on during high winds. The same is true for tiled roofs in many locations around Australia. The main cause of roof loss in strong wind events is inadequate connections between roof elements.

3.4 Bracing Loads

The combination of positive external pressures on the windward wall and negative pressures (suctions) on the opposite wall cause a net lateral force on the whole house. The blue arrows on the windward wall in Figure 3.1 and Figure 3.2 point to the right, and so do the blue arrows on the leeward or right hand wall. There is a net effect tending to push the whole house to the right. (Internal pressures act equally on windward and leeward walls and cancel each other out, i.e. internal pressures do not contribute to bracing loads.)

If the house does not have sufficient strength and rigidity to resist those forces, it can cause the house to fold up (racking) as shown in Figure 3.3(a), slide sideways as shown in Figure 3.3(b) or overturn as shown in Figure 3.3(c).

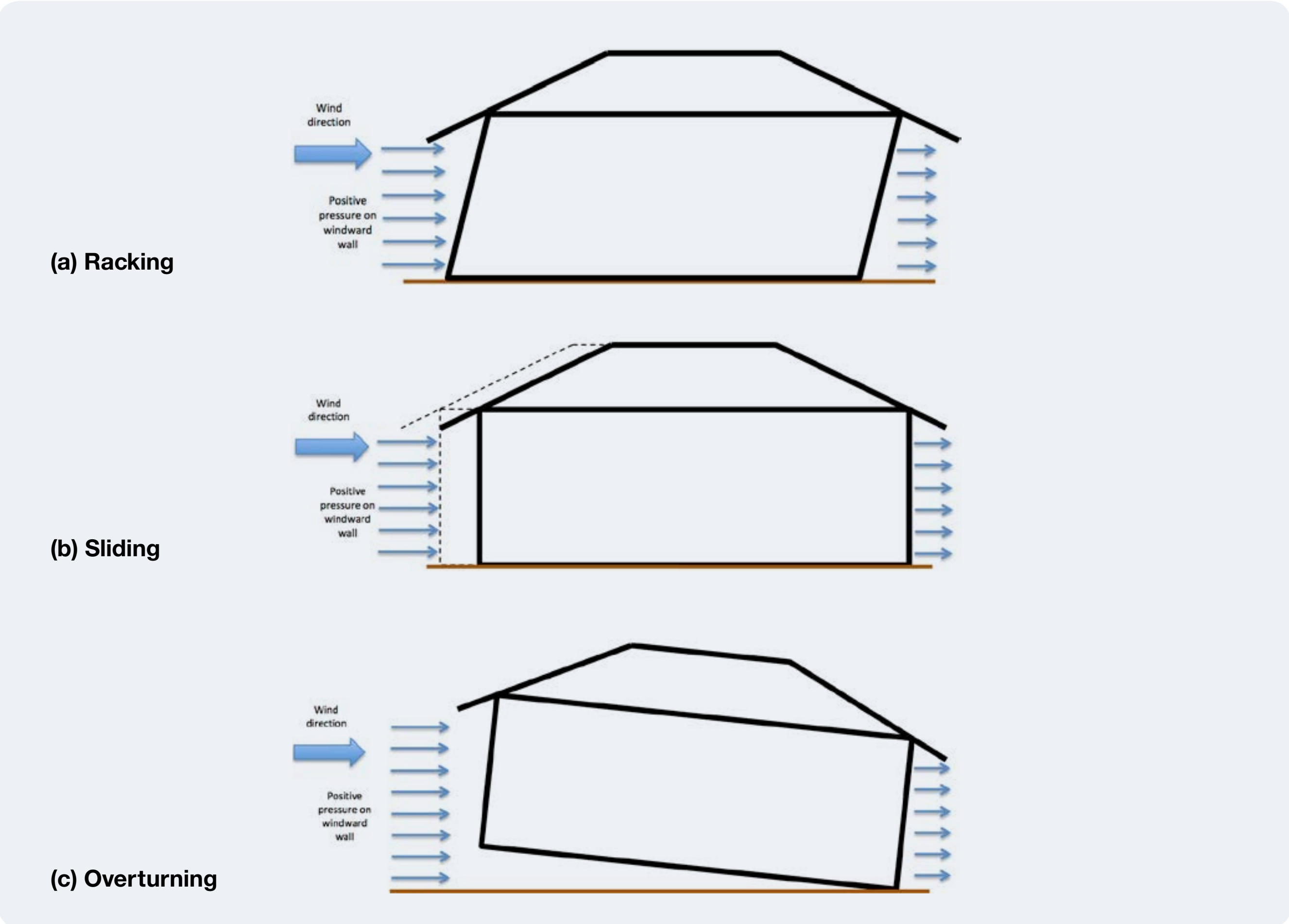


Figure 3.3: Effects of lateral forces on houses.

Providing adequate bracing walls will prevent racking failures, and ensuring appropriate connections at all levels of the building from roof to the ground will prevent sliding and overturning failures (see Section 6).

3.5 Design to Resist Wind Effects

The process of designing to resist wind effects involves:

a) determining the design wind speed on the house
b) evaluating wind pressures on external and internal surfaces
c) calculating the tributary wind area for each structural element
d) calculating the wind force on each structural element
e) combining wind actions with other actions (self-weight, occupancy loads) on the element
f) selecting appropriate structural elements with sufficient capacity to resist the loads.

This is accomplished differently for the two design paths shown in Figure 2.1.

Designing to resist wind involves evaluating wind effects and loads, then detailing the structure to resist them.

3.5.1 AS/NZS 1170.2 and AS 1720.1

Structural engineers often use AS/NZS 1170.2 and AS 1720.1 to select and specify elements that have sufficient strength and stiffness to carry the required wind loads:

- AS/NZS 1170.2 is used to calculate the wind forces on each structural element used in Steps a) to d) in Section 3.5.
- AS/NZS 1170.0 is used to combine wind loads with other actions for Step e).
- AS 1720.1 gives member and connection capacities to match the combined loads for Step f).

Some elements such as windows and garage doors can be specified using the calculated net wind pressure from Step b).

Any building can be designed to resist wind actions using these two Standards. However, houses can also be designed using AS 4055 and AS 1684 (the left-hand pathway in Figure 2.1).

3.5.2 AS 4055 and AS 1684

AS 4055 and AS 1684 can be used together to select elements and members without the need for engineering calculations:

- Use the characteristics of the site to select a wind classification in AS 4055 – Step a).
- Use span tables in AS 1684 together with spacing and spans of elements to select members – Step f); and
- Use load tables in AS 1684 to establish tie-down or bracing loads – Step e) and combine with connection or bracing wall capacities also given in AS 1684 to select the elements required – Step f).

Using this approach, Steps b) to e) are incorporated into the tables in AS 1684.

For example, to design structural elements of houses:

- Use AS 4055 to select the wind classification (see Section 4).
- For windows, order by size, wind classification and consider whether the window is at an external corner.
- For structural timber members, look up appropriate span tables (wind classification and timber stress-grade) to select member sizes.
- For tie-down connections, look up net uplift forces from tables in AS 1684 and match with connection capacities also given in AS 1684 (see Section 5).
- For bracing walls, for the two different directions, look up racking pressures in tables in AS 1684 using the house configuration and wind classification, then convert this to a force by multiplying the pressure by the face area. Ensure that there is sufficient bracing wall capacity parallel to each direction to match the force calculated (see Section 6).

AS 4055 provides different wind speeds and pressures for:

- the ultimate limit state, which relates to the strength of structural elements
- the serviceability limit state, which relates to deflection of structural elements.

Prevention of failure in uplift or racking relates to the ultimate limit state, and the tables in Clauses 8 and 9 in AS 1684.2 or AS 1684.3 are based on ultimate limit states pressures from AS 4055.

4 Site Wind Classification

For houses that satisfy the geometric limitations in AS 1684 and AS 4055 (see Section 2.3.1), it is possible to determine the wind classification using AS 4055. Both AS 4055 and AS/NZS 1170.2 make it clear that wind loads can be determined using either Standard, but not using a mixture of both.

If the house does not meet those geometric limitations or if the designer has chosen to follow the right hand Deemed-to-Satisfy path shown in Figure 2.1, engineers can use AS/NZS 1170.2 to determine the appropriate wind pressures to be resisted by structural elements.

4.1 AS 4055 – Wind Loads for Housing

The Wind Classification for the house is determined by following steps in AS 4055:

- select the Wind Region for the house
- evaluate the Terrain Category
- select the Topographic Class
- select the Shielding Class
- look up the Wind Classification.

4.1.1 Stage of Development

For home renovations, extensions and in-fill development, the characteristics of the surrounding properties will not change significantly in the future. The shielding (see Section 4.1.5) offered by neighbouring houses and the terrain category (see Section 4.1.3) of the surrounding suburb will remain much the same for decades.

Assess the terrain category and shielding based on expected development within 5 years.

However, houses built on a new subdivision may have no shielding at all when built, but will be surrounded by other houses within a few years. In order to make a reasonable assessment on the wind environment on a house over most of its life, assess the wind classification based on how the suburb is likely to be developed in five years' time.

4.1.2 Wind Regions

Figure 2.3 shows the Wind Regions defined by both AS/NZS 1170.2 and AS 4055. (AS/NZS 1170.2 divides Wind Region A into sub-regions based on wind direction, but these are not used in AS 4055.)

Identify the Wind Region of the town or district in which the house will be built. In some cases, a town may be close to a border between two Wind Regions. In these cases, it is appropriate to choose the higher of the two Wind Regions.

Decide on Wind Region based on the location of the town or district that the house is to be built in.

The primary Wind Regions are A, B, C or D:

- Wind Region D – within 50 km of a smoothed coast line in WA between latitudes 20°S and 25°S (e.g. Onslow and Port Hedland, WA)
- Wind Region C – within 50 km of a smoothed coastline between 20°S in WA and 25°S in Queensland (e.g. Darwin, NT, Townsville, Qld, and Broome, WA), within 50 km of a smoothed coastline between 25°S in WA and 27°S (e.g. Denham, WA), and between 50 km and 100 km of a smoothed coast line in WA between latitudes 20°S and 25°S (e.g. Pannawonica, WA)
- Wind Region B – between 50 km and 100 km of a smoothed coast line between latitudes 20°S in WA and 25°S in Queensland (e.g. Collinsville, Qld), within 100 km of a smoothed coast line between 25°S and 30°S in Queensland (e.g. Brisbane, Qld), within 100 km of a smoothed coast line between 27°S and 30°S in WA (e.g. Geraldton, WA), and between 100 km and 150 km of a smoothed coast line in WA between latitudes 20°S and 25°S (e.g. Nannutarra, WA)
- Wind Region A – everywhere else. All of Australia south of 30°S (e.g. Melbourne, Vic; Sydney, NSW; Launceston, Tas) and inland regions north of 30°S (e.g. Coober Pedy, SA; Katherine, NT).

4.1.3 Terrain Categories

The terrain category (TC) for a site is used to indicate the 'aerodynamic roughness' of the land within a radius of 500 m around the site. Aerodynamic roughness refers to objects between 2 m and 20 m high on the ground breaking up the wind flow and reducing the speed of the air moving at house height.

Select terrain category as the most open within a 500 m radius circle centred on the house site.

To assess the terrain category for a site, look at a satellite view of the suburb and overlay a 500 m radius circle centred on the house. This circle will have a diameter of 1 km on the ground (check with the scale on the satellite image). Examine the character of the most open section of land (at least 200 m wide) within the circle. Part of its width can extend outside of the circle, as shown in Figure 4.1(a), Figure 4.1(b) and Figure 4.1(c). This will indicate the TC for the site.

If the circle contains:

- any open water in lakes, canals, rivers, inlets, or small bays, then the terrain category is TC1. For this category, the water should extend for more than 200 m and less than 10 km, see Figure 4.1(a).
- any ocean, large harbour or a large bay, then the terrain category is TC1.5. For this category, the water should extend away from the house site by more than 10 km, see Figure 4.1(b).
- any open country typical of paddocks, ovals, playing fields, golf courses, cleared sub-divisions, then the terrain category is TC2. For this category, the open area should be wider than 200 m and have an area of roughly a quarter the size of the circle, see Figure 4.1(c).
- any large lot developments with fewer than 10 houses per hectare, then the terrain category is TC 2.5, see Figure 4.1(d).
- only suburban development with allotment sizes less than 1000 m^2 that may also contain small parks, lakes, canals or road reserves (each less than 200 m wide), then the terrain category is TC3, see Figure 4.1(e). It is possible to include heavily wooded areas as TC3 in Wind Regions A and B only. (In Wind Regions C and D, heavily wooded areas should be considered as TC2).

The TC is based on the likely terrain in 5 years' time, which allows for reasonable urban development early in the life of the house.

(a) TC1 – lake

(b) TC 1.5 – ocean

(c) TC2 – golf course

(d) TC 2.5 – large lot development

(e) TC3 – closely spaced lots in suburban development

Figure 4.1: Examples of terrain categories from satellite photos.

4.1.4 Topographic Classes

Houses located on elevated sites such as hills, ridges or escarpments experience higher wind speeds than those built on low, flat terrain. The topographic class (T0 to T5) is determined by the height and steepness of the hill and how far up the hill the house is situated.

Use the characteristics of the hill or ridge the house is to be constructed on to determine the topographic classification.

In order to select the topographic classification using Table 2.3 in AS 4055 (excerpt shown in Figure 4.2[c]), the following information is required:

- height of the hill (H)
- maximum slope of the hill (Φ_a)
- location of the house on the hill (L – lower third, M – mid third, T – top third).

Steps to select topographic class

Use a contour map of the area surrounding the house:

1. Locate the house site on the contour map.
2. If there is no hill within 500 m of the site, then the topographic class is T0.
3. Otherwise, move upwards across the contours at right angles from the house site until the top of the topographic feature is found. Then from the top of the feature select the lowest point on the contour map by moving downwards away from the top in any direction. The lowest point defines the bottom of the hill, see Figure 4.2(a).
4. Find the height of the lower third of the hill and the upper third of the hill.

$$H_{1/_3} = \frac{1}{3} H_{top} + \frac{2}{3} H_{bottom}$$

$$H_{2/_3} = \frac{2}{3} H_{top} + \frac{1}{3} H_{bottom}$$

If the height of the house is lower than $H_{1/_3}$ then the topographic class is T0, otherwise continue.

5. Find the contour at half hill height.

$$H_{2/_3} = \frac{2}{3} H_{top} + \frac{1}{3} H_{bottom}$$

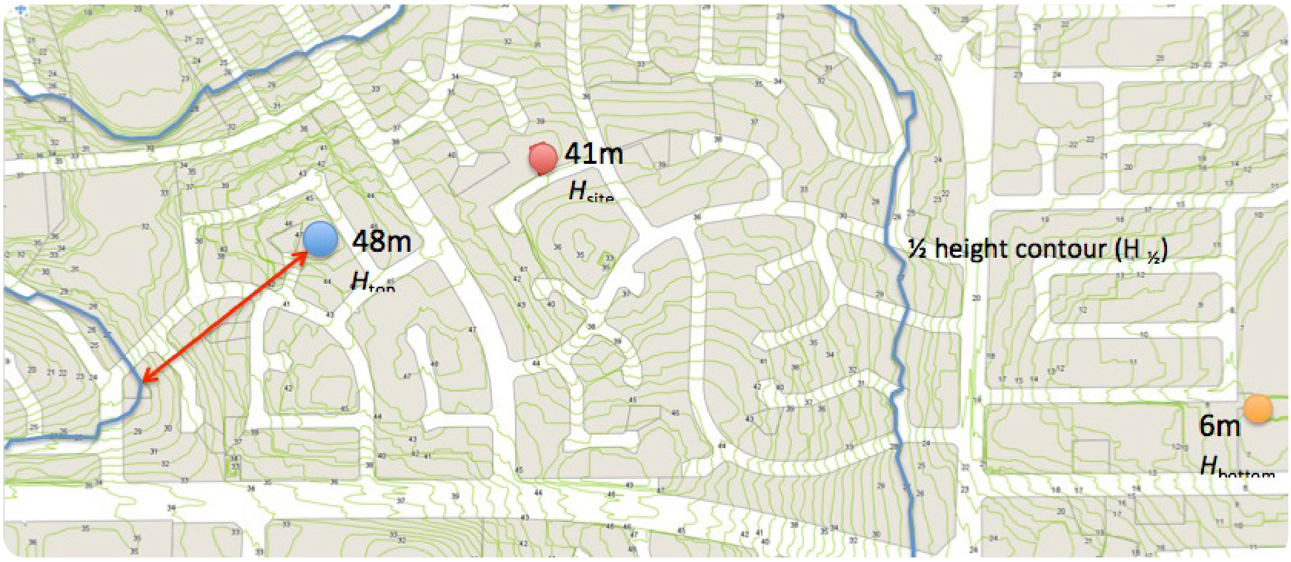
Find the closest point on this contour to the top of the hill and scale off the distance between the contour and the top of the hill ($D_{1/_2}$). (See Figure 4.2[b])

The hill slope (Φ_a) is 1: $\dfrac{2\times\left(D_{1/_2}\right)}{\left(H_{top} - H_{bottom}\right)}$

6. Look up Table 2.3 in AS 4055 to find the topographic class. (See Figure 4.2[c])

AS 4055 provides some guidance on the calculation of the slope of hills and other factors to determine the appropriate topographic classification in an Appendix.

(a) Top and bottom of topographic feature



(b) Hill slope

TOPOGRAPHIC CLASSIFICATION FOR HILLS, RIDGES OR ESCARPMENTS

Maximum slope (ϕ_a)		Site location (see Figure 2.2) Lower-third zone (L)	Mid-third zone (M)	Top-third zone (T)			Over-top zone (O) (for 4H past crest of escarpments only)
				$H \leq 10$ m	10 m < $H \leq 30$ m	$H > 30$ m	
<1:20 (<2.9°)		T0	T0	T0	T0	T0	T0
≥1:20 (≥2.9°)	<1:10 (<5.7°)	T0	T0	T1	T1	T1	T0
≥1:10 (≥5.7)	<1:7.5 (<7.6°)	T0	T1	T1	T2	T2	T0
≥1:7.5 (≥7.6°)	<1:5 (<11.3°)	T0	T1	T2	T2	T3	T1

(c) Excerpt from Table 2.3 in AS 4055. (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

Figure 4.2: Example of selection of topographic class.

Figure 4.2 illustrates an example using the steps above:

Steps 1 to 3:

The house is on a hill and the contour map indicates that

H_{top} = 48 m, H_{bottom} = 6 m, and the house site is at 41 m. (Figure 4.2[a])

The hill height (H) = 42 m.

Step 4:

$H_{2/3}$ = 34 m, $H_{1/3}$ = 20 m, so the house is in the top third of the hill.

Step 5:

Half height contour is 27 m and is marked in Figure 4.2(b).

The shortest distance between the $H_{1/2}$ contour and the top of the hill is shown by the red arrow and scales to 280 m.

The slope of the hill = 21:280 = 1:13.

Step 6:

Table 2.3 in AS 4055 gives terrain classification T1, as shown in Figure 4.2(c).

Although the steps indicated above can be undertaken as a desk-top evaluation, the site should be visited to confirm the decisions made and conduct a rough check on whether the topographic classification for houses in suburban areas is appropriate.

It is recommended that an engineer is involved in all aspects of the structural design of houses on T5 sites.

4.1.5 Shielding

The wind speed on a house is influenced by the size and number of structures that are close to it, either across the road or on adjoining allotments. Houses can be shielded by buildings of a similar size, or in Wind Regions A and B only, by heavily wooded areas (vegetation with height equal to or greater than the house and dense enough to prevent the site from being seen through it). Shielding is evaluated for all directions, and the worst case applies. AS 4055 categorises the amount of shielding at a site as follows:

- Full shielding (FS) – if there are at least two rows of houses at greater than or equal to 10 houses per hectare between the site and any open terrain such as parks or water in all directions, see Figure 4.3(a)
- Partial shielding (PS) – if there is at least one direction with only one row of houses between the site and open land such as a park. This class also applies to houses in large lot size developments with between 2.5 and 10 houses per hectare, see Figure 4.3(b)
- No shielding (NS) – if there are no houses within 100 metres of the house site in at least one direction, see Figure 4.3(c). Houses adjacent to open areas such as parkland or open water with the smallest dimension larger than 100 m are considered to have No shielding.

At least two rows of houses must surround the site on all sides to achieve Full shielding.

(a) Full shielding

(b) Partial shielding

(c) No shielding

Figure 4.3: Examples of selection of shielding class.

If, in any direction, there are no houses next to the site, then the site has No shielding.

4.1.6 Wind Classification

Once the Wind Region, terrain category, topographic class and shielding class have been selected, it is possible to determine the wind classification for the site using Table 2.2 in AS 4055.

On steeply sloping land, it is not possible to consider Full or Partial shielding, so for T4 and T5, the only shielding option is NS. Likewise, Full shielding is not possible for the intermediate topographic class of T3.

An example for Wind Region A, with terrain category 3, topographic class T2 and No shielding (NS) is given in Figure 4.4 as N2.

WIND CLASSIFICATION FROM WIND REGION AND SITE CONDITIONS

Wind region	TC	Topographic class												
		T0			T1			T2			T3		T4	T5
		FS	PS	NS	FS	PS	NS	FS	PS	NS	PS	NS	NS	NS
A	3	N1	N1	N1	N1	N2	N2	N2	N2	N2	N3	N3	N3	N4
	2.5	N1	N1	N2	N1	N2	N2	N2	N3	N3	N3	N3	N4	N4
	2	N1	N2	N2	N2	N2	N3	N2	N3	N3	N3	N3	N4	N4
	1.5	N2	N2	N2	N2	N3	N3	N3	N3	N3	N3	N4	N4	N5
	1	N2	N3	N3	N2	N3	N3	N3	N3	N4	N4	N4	N4	N5
B	3	N2	N2	N3	N2	N3	N3	N3	N3	N4	N4	N4	N4	N5
	2.5	N2	N3	N3	N3	N3	N3	N3	N4	N4	N4	N4	N5	N5
	2	N2	N3	N3	N3	N3	N4	N3	N4	N4	N4	N5	N5	N6
	1.5	N3	N3	N4	N3	N4	N4	N4	N4	N4	N5	N5	N5	N6
	1	N3	N4	N4	N4	N4	N4	N4	N5	N5	N5	N5	N6	N6
C	3	C1	C1	C2	C1	C2	C2	C2	C2	C3	C3	C3	C3	C4
	2.5	C1	C2	C2	C2	C2	C2	C2	C3	C3	C3	C3	C4	NA
	2	C1	C2	C2	C2	C2	C3	C2	C3	C3	C3	C4	C4	NA
	1.5	C2	C2	C3	C2	C3	C3	C3	C3	C4	C4	C4	NA	NA
	1	C2	C3	C3	C3	C3	C3	C3	C4	C4	C4	NA	NA	NA

FS = Full shielding
PS = Partial shielding
NS = No shielding
N = Non-cyclonic
C = Cyclonic
N/A = Not applicable, that is, beyond the scope of this Standard (use AS/NZS 1170.2)
TC = Terrain category

Figure 4.4: Example of selection of wind classification – Excerpt from Table 2.2 in AS 4055.
Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.

4.1.7 View Check of Wind Classification

More exposed sites have higher wind classifications. This includes homes close to the ocean or other open areas, high on hills or facing open areas such as parks. These houses usually enjoy good views. The view from the site can be used as a useful rough check on whether the wind classification specified on the plans for the house is likely to be correct:

- House has no view – generally N1 or C1
- House has a view past two rows of houses – generally N2 or C2
- House has a view over two suburban street blocks – usually N3 or C3
- House has a view beyond the suburb or out to sea – often N4 or higher, or C4 – an engineer should design all structural elements.

Houses with a great view generally have a high wind classification.

4.2 AS/NZS 1170.2

Where wind actions on a house are calculated using AS/NZS1170.2, engineers have to undertake a more detailed study to determine wind speeds and the pressures on building elements:

- The Wind Region for the site and a V_R, regional design wind velocity is selected.
- Terrain category, topography and shielding are evaluated as velocity multipliers independently for eight cardinal wind directions centred on the site.
- These multipliers are used to determine eight directional design wind velocities.
- Appropriate design velocities are calculated for the face orientations of the house.
- The roof shape and wall configurations are used to evaluate pressure coefficients.
- Pressures on each external surface are calculated using the design wind velocity for the orientation and the appropriate pressure coefficient for the surface.

5 Tie-downs

As discussed in Section 3.3, wind exerts significant uplift forces on a roof. Working to resist the wind uplift is the weight of the roof and the strength of all of the connections in the roof. For tiled roofs, the weight of the relatively heavy tiles helps to counteract the uplift forces. However, for lightweight roofs, the weight of the thin sheeting on its own is only just enough to resist the uplift created by a breeze, and high winds must be resisted by stronger connections within the roof system.

A continuous chain of tie-down details between the roofing and the ground is required for almost all wind classifications

Uplift forces act on the roof cladding, and can only be resisted by transferring the uplift forces through the complete structure to the ground. A secure chain of structural elements and connections is required to transmit the forces from the upper surface of the roof to the ground. This is illustrated in Figure 5.1.

The elements that may form part of this chain include:

- roof sheeting – spanning between battens
- sheeting fasteners – carrying loads from the sheeting to the battens
- battens – spanning between rafters or roof trusses
- batten fasteners – carrying loads from battens to the rafters
- roof structure or roof trusses – carrying loads from the batten fasteners to the tops of the walls. Trusses are planar structural elements that transmit these loads. A combination of rafters, roof beams, underpurlins, struts, collar ties and ceiling joists may be involved in the load transfer in framed roofs
- roof tie-downs to the top of walls
- uplift load transfer within the wall from the top plate to the base of the wall
- uplift load transfer from the bottom of the wall to the floor system
- uplift load transfer through the floor and sub-floor systems to the ground.

As the loads pass down through the structure, the weight of all elements above is engaged, which reduces the net uplift that has to be carried by elements lower in the chain.

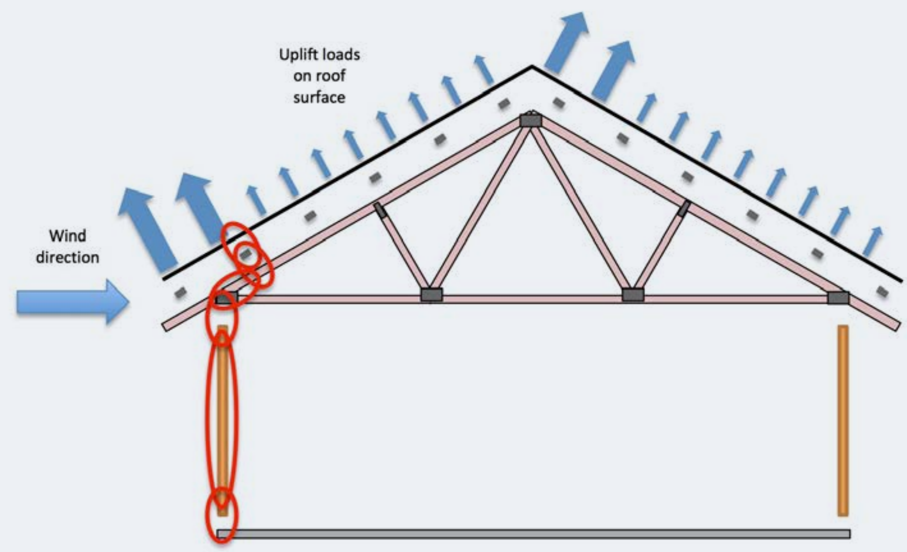


Figure 5.1: Secure tie-down chain of structural elements.

Damage investigations following severe wind events have indicated that if any of the elements in the tie-down chain are not capable of resisting the uplift loads, then failures result, with the loss of all elements above the element that failed. Figure 5.2 illustrates the consequence of failures of tie-down connections.

(a) Batten-to-rafter or truss connection

(b) Roof structure to wall connection

Figure 5.2: Failure of structural elements in the tie-down chain.

In Figure 5.2(a), the batten-to-rafter connection failed and the roofing and battens lifted off the house. These are all of the elements higher than the defective element in the tie-down chain. In Figure 5.2(b), the roof-to-wall connection failed and the roofing, battens and roof structure (all higher than the weak link in the tie-down chain) lifted off the walls.

Winds less than the design wind speed can damage houses with inadequate tie-down details.

5.1 Corrosion Protection of Tie-down Connections

Metal connectors including straps, framing anchors, ties, joist hangers, nailplates, nails and screws are extensively used in the tie-down chain. It is important to ensure that the metal connectors have durability appropriate for the environment in which they are used. They must remain serviceable for at least 50 years to provide resistance against the design winds for the life of the house.

Metal connectors with appropriate protection against corrosion are required in coastal or industrial areas.

Manufacturers of metal connectors provide guidance for their use in different environments. Timber Queensland's Technical Data Sheet 35 describes the following steps to selecting appropriate materials for metal connectors:

- Select the Corrosion Zone:
  - Industrial zones – close proximity to industrial complexes where corrosive gases may be emitted, e.g. smelters, galvanising plants, oil refineries, swimming pools
  - Sea spray zone – within 1 km of a surf coast or 100 m from bayside areas
  - Coastal zones – 1 km to 10 km from a surf coast or 100 m to 1 km from bayside areas.
  - Low hazard zones – locations not included above.
- Determine the Exposure Conditions:
  - Exposed – outside the building envelope, in the open
  - Sheltered – outside the building envelope but under cover
  - Enclosed – within the building envelope.
- Look up the appropriate corrosion protection from the table provided in the Technical Data sheet.

For connections that require nails or screws to be in contact with metal plate elements, e.g. nails into triple grips, the corrosion-protective coating on the nails or screws should be compatible with the material used for the metal plate element.

Metal connectors should also be compatible with the treatment (e.g. CCA – Copper Chrome Arsenic, ACQ – Alkaline Copper Quaternary, or Copper Azole) used on the timber elements. Synthetic pyrethroids used in blue H2 treatments for softwood framing do not affect the corrosion resistance of metal fasteners.

WoodSolutions Technical Design Guide #5 – *Timber Service Life Design* provides some guidance on the selection of corrosion resistant fasteners.

5.2 General, Edge and Corner Areas of Roofs

Figure 3.1 shows the very high local uplift pressures that develop near the eaves and just behind the ridge. They act over relatively small areas and are modelled using compatible, but slightly different, ways in AS 4055 and AS/NZS 1170.2.

- AS 4055 defines different areas of the roof. AS 1684 uses the edge and general regions defined in AS 4055 and illustrated in Figure 5.3. Different tie-down requirements are specified for the different roof areas:
  - Edge areas are within 1.2 m of an external roof edge (eaves or gable) and within 1.2 m of a hip or ridge
  - General areas are all areas more than 1.2 m from a roof edge, hip or ridge.
- AS/NZS 1170.2 defines local pressure factors and the roof areas on which they act. The roof local pressure areas are only slightly different to those illustrated in Figure 5.3.

Wind forces are highest within 1200 mm of the edges of roofs. Higher capacity tie-down connections are required in edge areas.

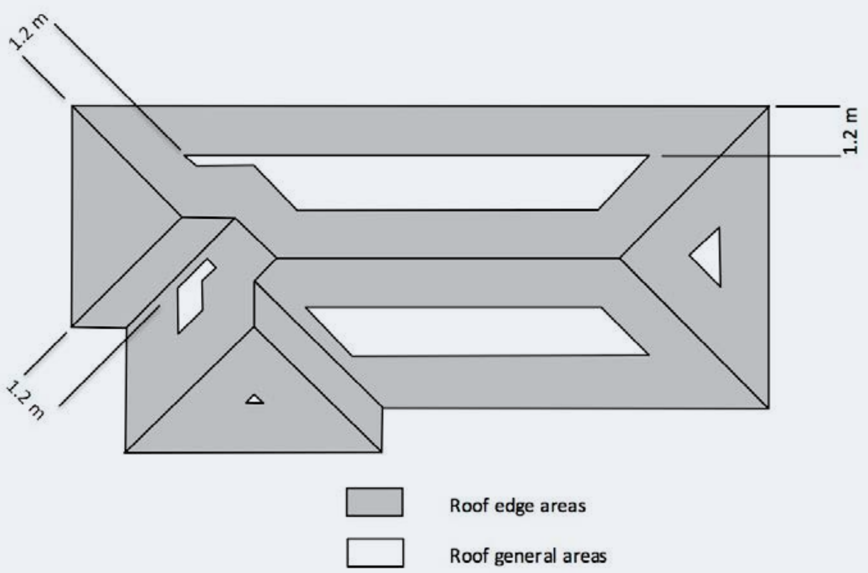


Figure 5.3: Edge and general roof areas.

Because the high local uplift pressures act over relatively small areas, they only affect elements that are loaded by relatively small areas of roofing. They only apply to:

- roof cladding – sheeting or tiles
- roof cladding to batten fasteners
- roof battens
- batten-to-rafter/truss connections.

5.3 Roof Cladding to Battens

Manufacturers usually provide recommendations on fixing their roofing products to battens. Their details may vary with wind classification and can include specification of:

- fastener type and strength
- frequency of connection or spacing of fasteners along a batten
- maximum batten spacing
- special requirements for fixings in the edge or corner areas
- special requirements for fixings in the cyclonic areas (Wind Regions C and D)
- special requirements for corrosion resistance for sites near salt water.

Roof cladding manufacturers provide recommendations for connections of cladding to battens. There will be different requirements for cyclonic and non-cyclonic areas.

5.3.1 Tiled Roofs

In all wind classifications, the uplift forces at the design wind speed on a single tile can exceed the weight of a tile. Therefore, some type of tile anchorage is required for all roofs. The anchorage forces are higher in edge areas – near eaves, gables, hips or ridges.

Where these anchorages have been omitted, removed or are inadequate, loss of tiles at winds near the design wind speed has been observed, as shown in Figure 5.4, where both events were a little lower than the design wind speed.

(a) Loss of tiles in Wind Region A (non-cyclonic)

(b) Loss of tiles in Wind Region C (cyclonic)

Figure 5.4: Consequences of not anchoring tiles.

5.3.2 Metal Sheet Roofs

Metal roof sheeting has a much lower mass per area than tiles, hence the weight forces that oppose the wind uplift forces are less, and tie-down of the sheeting and roof structure is even more important.

Metal roof cladding is lighter than tiles and requires higher capacity tie-downs to resist the wind uplift forces.

The higher wind uplift forces in cyclonic areas require fasteners at closer centres along the battens than those in non-cyclonic areas, as shown in Figure 5.5. Also, in severe tropical cyclones, repeated gusting over many hours can cause fatigue in the metal roofing or in the connection. The BCA requires sheet roofing for cyclonic areas to demonstrate resistance to this fatigue by using a 'Low-high-low cyclic load test'. In cyclonic areas, there may need to be some additional features of the connection to alleviate cracking under repeated loading, e.g. the cyclone washers shown in Figure 5.5(b).

(a) Non-cyclonic areas

(b) Cyclonic areas

Figure 5.5: Requirements for connection of metal sheeting.

In all Wind Regions, it is important to provide cladding connections that can satisfactorily resist the uplift forces accompanying high winds. Warranties for cladding materials may be voided if inadequate or incorrect connection has been used.

5.4 Roof Members

Timber roof members are sized using span tables for the house wind classification (N or C classification) and measures of the tributary uplift area on the member.

The span tables are presented in Supplements to AS 1684.2 and AS 1684.3. Each supplement has tables for a single timber grade and a single wind classification. For example, AS 1684.2 N3 Supplement 4 gives all of the span tables (53 tables covering all timber elements in a frame – battens to bearers) for N3 wind classification for houses using MGP10 timber.

Span tables for the appropriate site wind classification are used to select roof members.

Battens are sized using the batten span tables. These are presented in Table 32 in the appropriate Supplements to AS 1684.2 and AS 1684.3 and use the following inputs:

- wind classification
- batten spacing
- rafter or truss spacing (the span of the batten).

(An example is shown in Figure 5.6.)

AS 1684.2 N3 SUPP. 4 - 2010 | Wind classification N3 – Seasoned softwood – Stress grade MGP 10

TABLE 32 **ROOF BATTENS – Supporting roofing only**

Size DxB (mm)	Batten Spacing (mm)									
	330		450		600		900		1200	
	Maximum Batten Span and Overhang (mm)									
	Span	Overhang	Span	Overhang	Span	Overhang	Span	Overhang	Span	Overhang
Sheet Roof										
35x70	1200	350	1100	300	800	275	650	250	650	225
35x90	1200	500	1200	375	1050	325	800	300	800	275
45x70	1200	600	1200	450	1200	400	1050	350	1050	325
45x90	1200	600	1200	600	1200	500	1200	425	1200	400
Tile Roof										
35x70	1200	325	1000	300	750	250	600	225	600	225
35x90	1200	450	1200	350	950	300	750	275	750	250

Figure 5.6: Excerpt from Table 32 in AS 1684.2 N3 Supp 4. (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

5.5 Connections between Structural Roof Members

Selection of appropriate tie-down connections to resist uplift forces using Section 9 of either AS 1684.2 or AS 1684.3 is a simple two-step process:

1. Look up the net uplift load on the specific roof structure connection.
2. Select a connection that provides enough capacity to meet or exceed the net uplift load using tables in AS 1684 or guidance provided by connection systems manufacturers. AS 1684 is not specific about the Joint Group of MGP10 timber. AS 1720.1 classifies MGP10 timber as Joint Group JD5. (The grade can include pieces that contain heart-in material.)

Use Joint Group JD5 for MGP10 when selecting tie-down connection capacities.

5.5.1 Connections between Battens and Rafters or Trusses

Damage investigations following high wind events in all regions of Australia have often shown that poor connections between batten and trusses or rafters can lead to loss of significant areas of roofing. However, selecting the correct connection is as simple as referring to the manufacturer's information or looking up two tables in AS 1684.2 or AS 1684.3, as shown in Sections 5.5.2 and 5.5.3.

For connections between metal battens and timber rafters or trusses, some manufacturers specify two screws. An experimental study (Boughton et al, 2015) indicated that gun-driven nails do not have sufficient capacity for edge areas in any wind classification.

5.5.2 Batten-to-Rafter Connection Load

The batten-to-rafter or truss connection loads are found in Table 9.14 in AS 1684.2 for non-cyclonic areas (N wind classifications), shown in Figure 5.7 or AS 1684.3 for cyclonic areas (C wind classifications).

The wind classification indicates the pair of columns to be considered, and the combination of rafter (or truss) spacing and maximum batten spacing provide the row to use. The table presents the uplift force on a single batten-to-rafter or truss connection in general and edge areas for tiled or sheet roofs.

The process is illustrated in the following example (Figure 5.7) of the connection between battens and rafters or trusses. The roof is a sheet roof on a house with an N2 classification. The roof has trusses (MGP10) at 600 mm centres, and 35 mm thick battens with maximum batten spacing of 1200 mm. The connection force in general areas of the roof is 0.71 kN and in edge areas 1.3 kN.

NET UPLIFT FORCE ON ROOF BATTENS

Rafter or truss spacing	Batten spacing	Uplift force, kN							
		Wind classification							
		N1		N2		N3		N4	
mm	mm	General area	Edges	General area	Edges	General area	Edges	General area	Edges
Tile roof									
450	330	0.04	0.14	0.09	0.21	0.17	0.37	0.29	0.59
600	330	0.06	0.18	0.11	0.29	0.23	0.50	0.39	0.79
900	330	0.08	0.27	0.17	0.43	0.35	0.75	0.59	1.1
1200	330	0.12	0.36	0.22	0.58	0.46	1.0	0.78	1.6
Sheet roof									
600	370	0.15	0.29	0.22	0.41	0.35	0.65	0.53	0.97
	450	0.18	0.35	0.26	0.50	0.42	0.79	0.64	1.1
	600	0.24	0.47	0.35	0.66	0.57	1.0	0.86	1.5
	750	0.31	0.59	0.44	0.83	0.71	1.3	1.0	1.9
	900	0.37	0.71	0.53	1.0	0.85	1.5	1.2	2.3
	1200	0.49	0.94	0.71	1.3	1.1	2.1	1.7	3.1
900	370	0.23	0.44	0.33	0.61	0.52	0.97	0.79	1.4
	450	0.28	0.53	0.40	0.75	0.64	1.1	0.96	1.7
	600	0.37	0.71	0.53	1.0	0.85	1.5	1.2	2.3
	750	0.46	0.88	0.66	1.2	1.0	1.9	1.6	2.9
	900	0.55	1.0	0.79	1.5	1.2	2.3	1.9	3.5
	1200	0.73	1.4	1.0	2.0	1.7	3.1	2.5	4.7

Figure 5.7: Example of batten-to-rafter connection load – Table 9.14 in AS 1684.2. (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

5.5.3 Batten-to-Rafter Connection Capacity

The connection selected must have enough capacity to meet or exceed the net uplift loads on that connection. For the example in Figure 5.7, the batten-to-rafter connections used in the general areas of the roof must exceed 0.71 kN, and those used in the edge areas must exceed 1.3 kN. AS 1720.1 classifies MGP10 timber as Joint Group JD5. (The grade can include pieces that contain heart-in material.)

Plain shank nails do not have sufficient capacity to resist the wind uplift forces for sheet roofs on MGP10 trusses or rafters in all wind classifications.

Table 9.25 in AS 1684.2 and AS 1684.3 (Figure 5.8) indicates that:

- diagrams (b) and (d) apply to 35 mm battens. (It is acceptable to use connection capacities for battens in the table that are thicker than the battens to be used on the house, but it is not OK to use the capacity if the batten thickness in the table is thinner than the battens that will be installed. That is, capacities listed in Table 9.25 for 38 mm battens can be used if 35 mm battens are to be installed, but not if 45 mm thick battens are to be used.)

- two 75 mm deformed shank nails with a diameter of 3.05 mm or more can be used in the general areas of the roof only (the connection capacity = 0.86 kN, which is greater than the load in general roof area = 0.71 kN)
- two 75 mm deformed shank nails with a diameter of 3.75 mm or more can be used in the general areas of the roof only (the connection capacity = 1.0 kN, which is greater than the load in general roof area = 0.71 kN)
- one 75 mm long No 14 Type 17 screw is the minimum fastener that can be used in both the edge and general areas of the roof (the connection capacity = 3.6 kN, which is greater than the load in edge roof area = 1.3 kN and load in general roof area = 0.71 kN).

The batten span tables include 45 mm thick battens, but Table 9.25 in AS 1684.2 and AS 1684.3 has strength data for connections into 38 mm battens only. An experimental study (Boughton et al, 2015) has shown that the connection capacities for 45 mm thick battens should be evaluated using Equation 1.

AS 1684 does not include connections through 45 mm battens. Use Equation 1 to calculate the capacity.

Equation 1: Capacity in 45 mm batten = Capacity in 38 mm batten × $\frac{length\ of\ fastener - 45}{length\ of\ fastener - 38}$

TABLE 9.25

UPLIFT CAPACITY OF ROOF BATTEN TIE-DOWN CONNECTIONS

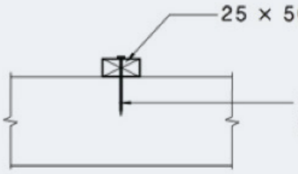
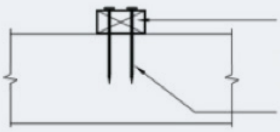
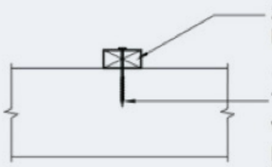
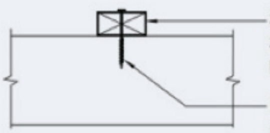
Position of tie-down connection		Uplift capacity, kN					
		Unseasoned timber			Seasoned timber		
Roof battens to rafters/trusses		J2	J3	J4	JD4	JD5	JD6
(a)  25 × 50 mm batten; Nails as per table	Plain shank						
	1/50 × 2.8Ø	0.36	0.30	0.28	0.20	0.13	0.09
	1/65 × 2.8Ø	0.58	0.48	0.44	0.32	0.20	0.14
	1/65 × 3.05Ø	0.65	0.54	0.48	0.34	0.22	0.16
	1/75 × 3.05Ø	0.81	0.68	0.60	0.43	0.28	0.20
	Deformed shank						
	1/65 ×3.05	1.3	1.1	0.95	0.68	0.45	0.32
	1/75 ×3.05	1.6	1.4	1.2	0.85	0.56	0.40
(b)  38 × 75 or 38 × 50 mm batten; Nailed as per table; Two screws shall be used only with 75 mm wide batten	Plain shank						
	1/75 × 3.05	0.61	0.52	0.45	0.32	0.21	0.15
	2/75 × 3.05	1.2	1.0	0.90	0.64	0.42	0.30
	Deformed shank						
	1/75 × 3.05	1.2	1.0	0.90	0.65	0.43	0.30
	2/75 × 3.05	2.5	2.1	1.8	1.3	0.86	0.60
	2/75 × 3.75	2.8	2.5	2.2	1.7	1.0	0.72
(c)  25 × 50 mm batten; 1/75 No. 14 Type 17 screw, with 50 mm penetration into receiving member		7.4	5.5	3.2	6.0	4.7	3.6
(d)  38 × 75 or 38 × 50 mm batten; No. 14 Type 17 screws as per table	Screws (length)						
	1/75 mm long	5.7	4.2	2.4	4.5	3.6	2.7
	1/90 mm long	7.4	5.5	3.2	6.0	4.7	3.6
	2/75 mm long	11	8.4	4.8	9.0	7.2	5.4

Figure 5.8: Example of batten-to-rafter connection capacity – Table 9.25 in AS 1684.2 or AS 1684.3. (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

5.6 Tie-down through Roof Structure to Walls

Different tie-downs from the roof to the walls may be required, depending on the wall structure and material. In general, similar tie-downs can be used for houses with timber wall frames and cladding, and those with timber wall frames and brick veneer. AS 1684.2 and AS 1684.3 can be used for selecting tie-downs for both wall systems. However, houses with double brick walls or reinforced masonry require a connection from the timber roof structure to a masonry wall system which is not covered in AS 1684.2 and AS 1684.3.

5.6.1 Roof Trusses

Timber roof trusses are designed and manufactured (prefabricated off-site) to comply with AS 1720.5 and installed to comply with AS 4440. Trussed roof systems use the principle of triangulation to provide a rigid structural system that spans between the support points (usually the external walls). The trusses are loaded in uplift by the battens and transmit those loads to tie-downs at the support point. Trusses are designed using the wind classification for the house and design documentation provided by the manufacturer should outline the tie-down requirements at the support points.

Roof truss manufacturers provide information on tie-downs for trusses. Girder trusses require tie-downs with higher capacity.

Other tie-downs in the walls below the truss tie-downs must have the capacity to carry the uplift loads from the trusses.

If the tie-downs are not specified, then the uplift force in kN can be evaluated using Table 9.13 in AS 1684.2 or AS 1684.3. The Uplift Load Width (ULW) for a single tie-down point is half of the distance in plan between the tie-down points plus the width of any nearby overhang. A tie-down detail with the appropriate capacity can be selected using Table 9.21 in AS 1684.2 or AS 1684.3, and using the techniques outlined in Section 5.4.

Girder trusses

The wind uplift forces on girder trusses are significantly higher than those on standard trusses, due to the larger roof area supported by girder trusses. All girder trusses must have tie-downs with higher capacities than tie-downs for common trusses. All tie-downs through wall and floor systems below girder trusses must be sized for the higher uplift forces from the girder trusses.

Internal wall tie-down

In some situations, particularly in high wind locations, trusses may be designed to have tie- downs installed at intermediate support locations (i.e. not just at external walls, but at some internal walls as well). The capacity of tie-down required at these locations may be significantly higher than at the external wall tie-down points. The tie-down chain should transmit those forces all the way down to the foundations through the internal walls.

Internal support points will be marked on the truss and it is crucial to ensure that the internal walls line up with those points. If support for the truss is provided away from the marked location, the bottom chord of the truss may be damaged.

5.6.2 Stick-built Coupled Roofs

Elements for stick-built roofs, which include rafters, underpurlins, collar ties, struts and strutting beams, are assembled on-site. All members for stick-built roofs are sized using the appropriate span tables for the wind classification. There are some requirements that are different for coupled or non-coupled roofs. AS 1684 has definitions for these terms, but many houses seem to fall between the two categories. The explanatory material below helps identify the principle behind the classification.

Coupled roofs are defined in Clause 7.1.2.2 of AS 1684.2 and AS 1684.3.

"The roof pitch in a coupled roof construction shall be not less than 10° and ceiling joists and collar ties shall be fixed to opposing pairs of rafters, in accordance with Section 9.

Rafters shall be continuous in length from ridge to wall plate, or shall be lapped or spliced at their support points (see Clause 7.1.2.1). Rafters may be supported on underpurlins."

The key point in the definition is that coupled roofs use triangulation to behave as pseudo trusses. The triangulation in the roof structure means that part of the roof uplift load can be carried using tension and compression in the roof members to the top of the external walls and tie-down connections can be similar to those for trussed roofs. Tie-downs for stick-built coupled roofs in wind classifications N1 and N2 can be evaluated using the AS 1684.2 tables for trusses (see Clause 9.6.3 in AS 1684.2).

Coupled roofs use collar ties and ceiling joists to achieve triangulation in the roof structure. Some special tie-down options can be used.

Some roofs may behave like coupled roofs although they do not meet the definition in Clause 7.1.2.2 of AS 1684.2 and AS 1684.3. For example, rafters at 900 mm spacings can only be tied to every second ceiling joist (at 600 mm spacings). While the definition uses the word "shall" for direct connection between rafters and ceiling joists, other references in the Standard indicate that it is possible to have some rafters that are not connected to ceiling joists within coupled roofs. The basic test of a coupled roof is that collar ties provide triangulation and there are good connections between rafters and ceiling joists where possible. So, if collar ties are installed on every pair of opposing rafters, the roof framing can still form a rigid structural system and may be considered to be a coupled roof. The tie-downs for coupled roofs can be used.

5.6.3 Stick-built Non-coupled Roofs

AS 1684 defines non-coupled roofs as:

"A non-coupled roof (including cathedral and skillion) shall have rafters (raking beams) supported off walls, ridge beams and/ or intermediate beams. It may have ceilings in the same plane as the roof. Rafters, ridge and intermediate beams may be exposed internally."

The ceiling configuration in some houses prevents the use of collar ties and ceiling joists in a separate plane to the rafters, making triangulation within the roof structure impossible. Although the roof system of coupled roofs can transfer the uplift forces through triangulation to the external walls, all individual elements in non-coupled roofs must have their own tie-down path to resist the uplift forces. For example, in roofs with cathedral ceilings, each connection between the rafters and the roof or intermediate beams must be selected using the appropriate tables in AS 1684 to resist the uplift forces. Each end of the roof and intermediate beams must also have appropriate tie-downs to transfer those forces through the walls to the ground. The process of selecting these connections is outlined in Section 5.5 and illustrated for batten connections.

Non-coupled roofs are typically in houses with cathedral ceilings, which prevent the use of triangulation in the roof structure. Each connection must be a tie-down connection.

5.6.4 Ribbon Plates

Ribbon plates can be nailed to the top of wall frame top plates. Tie-downs such as straps, triple grips or framing anchors should connect the truss or rafter to the top plate, not the ribbon plate. Connecting only to the ribbon plate will not provide load transfer to the wall.

5.7 Tie-down of Walls

For houses with tiled roofs and wind classifications of N3 and above, or sheet roofs and wind classifications of N2 and above, there is net wind uplift at the underside of the wall that requires continuation of the tie-down chain to the ground. The principles outlined in Section 5.5 can be extended to include the wall and floor systems.

Table 9.19 in AS 1684.2 and AS 1684.3 shows that tie-down forces can be carried through a timber wall frame using either connections between the studs and top and bottom plates or steel rods or plywood to carry uplift forces directly from the top plate to the bottom plate.

Plywood adjacent to openings cannot be used for tie-down at these locations, and other tie-down methods (see Table 9.20 in AS 1684.2 or AS 1684.3) are required. Plywood in structural bracing walls should not be used for tie-down.

5.7.1 Tie-down of Walls to Concrete Slab

Timber wall frames are tied-down to concrete slabs using either:

- hardened nails
- cast-in bolts; or
- chemical or other proprietary fasteners.

Tables 9.9 or 9.11 in AS 1684.2 and AS 1684.3 enable calculation of the tie-down forces required for connections between the walls and the slab and Table 9.18 in AS 1684.2 and AS 1684.3 provides capacities of connections.

5.7.2 Tie-down of Walls to Suspended Timber Floor Systems

Tie-down of walls to suspended timber floor systems requires the evaluation of forces and selection of connections for three positions in the floor system:

- walls to floor joists
- floor joists to bearers
- bearers to columns, posts or piers.

Appropriate connection details for all of these connections are presented in Clause 9 in AS 1684.2 and AS 1684.3. The connections of columns to their footings also have to transmit the net uplift force at the bottom of the floor system.

6 Bracing Houses to Resist Lateral Forces

Bracing is required to enable the roof, wall and floor systems to resist net horizontal forces on the whole building from wind effects (See Section 3.4.) Appropriate materials and connections are required to transfer these forces through the timber frames to the foundations of the building.

Bracing is required to resist the net horizontal forces caused by wind on windward and leeward walls.

The magnitude of the total lateral wind force on the surface of a building depends on the wind classification, width of the building, the pitch of the roof and whether the house is single or two storeys. It will be different for winds parallel and perpendicular to the main ridge line.

6.1 Principles of Bracing in Houses

6.1.1 Bracing Load Paths

For design of bracing systems, two principle directions are considered:

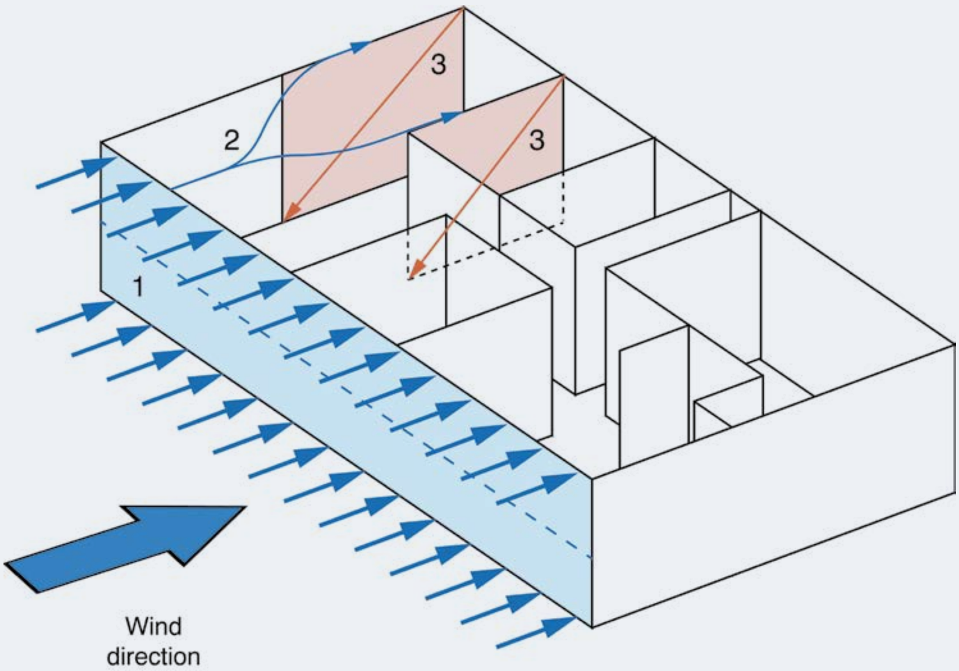
- wind direction parallel to the larger dimension of the floor plan (generally parallel to the main ridge line in hip and gable roofs)
- wind direction perpendicular to the larger dimension of the floor plan.

Evaluate bracing and load capacity for wind in two directions.

For each of these wind directions, the net horizontal loads on the house are transferred to the ground as follows:

- Lateral loads caused by windward wall pressures put studs in the wall frames into bending. For upper storeys or single-storey houses, this transfers half of the load downward to the floor structure and half of it upward to the roof structure.
- The lateral loads in the roof structure are applied over the full length of the windward and leeward walls. The ceiling diaphragm (which structurally behaves as a plate or deep beam) transfers those loads to the top of walls that are parallel to the direction of the wind.
- The in-plane rigidity of the walls that are parallel to the wind transfers the horizontal forces in the ceiling diaphragm through the walls to the floor structure. Walls perpendicular to the wind direction are not counted as contributing to bracing in the house for that wind direction.
- On two-storey houses, the loads must then be added to the loads from the lower storey and transferred using bracing walls on the lower storey to the floor.
- Where the ground floor is a suspended floor, then the loads must be carried through the subfloor structure to the ground.

Figure 6.1 illustrates the load paths for one wind direction through two bracing walls in a single-storey house. It shows the critical role of the walls (both internal and external) in bracing the building against the horizontal forces generated by the wind.



1. Horizontal forces on the windward wall are carried by studs to the top and bottom of the wall.
2. Horizontal forces are carried by the ceiling diaphragm to the top of the bracing walls. (The ceiling diaphragm is a horizontal or near-horizontal plate that remains relatively ridged in-plane and can transmit lateral loads through it.)
3. Bracing walls parallel to the wind direction carry horizontal forces from the ceiling to the floor.

Figure 6.1: Horizontal load paths through the building.

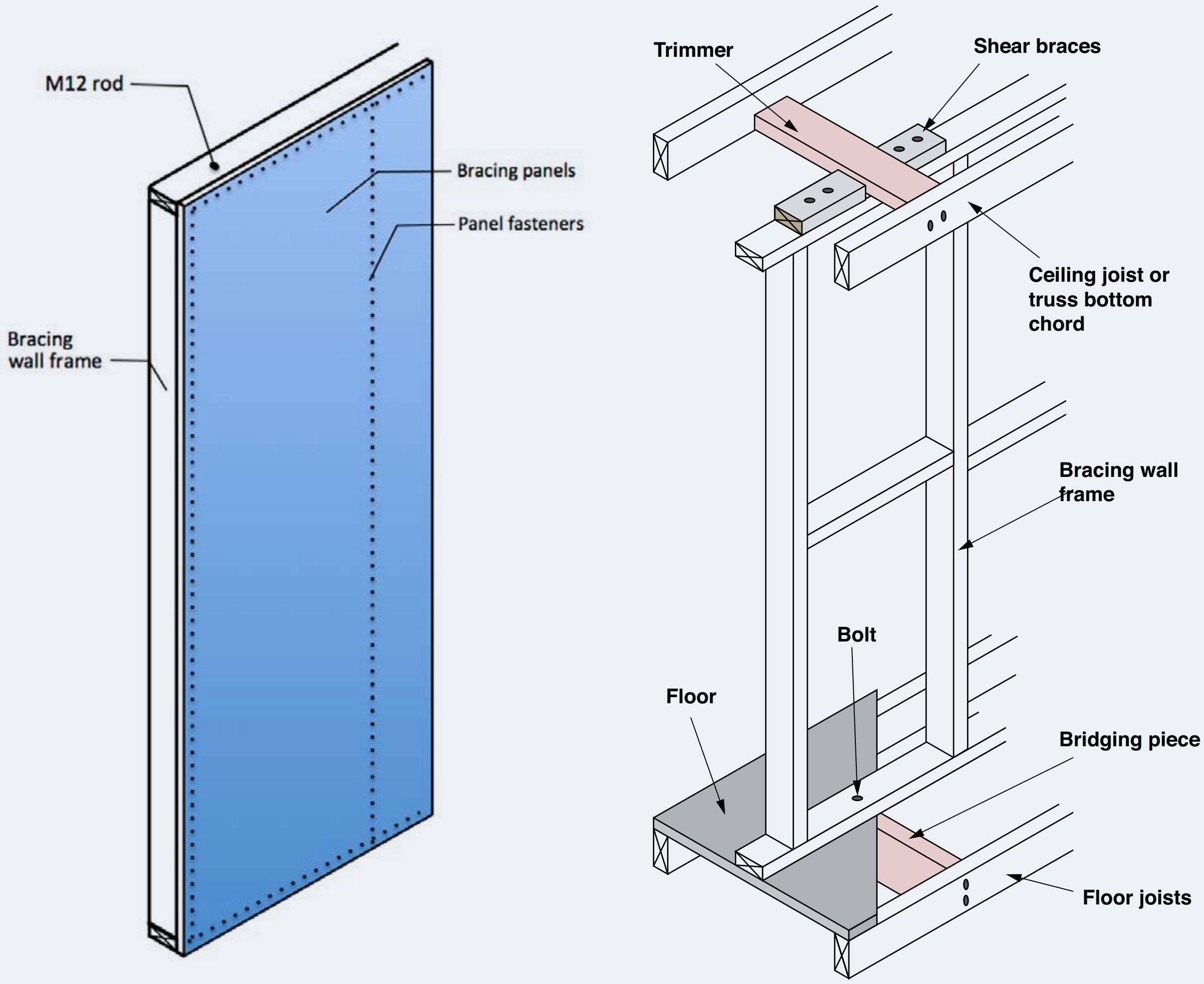
6.1.2 Bracing Walls

All walls that are parallel to the wind direction have the potential to resist the bracing forces on a house. However, there are two classifications of walls that relate to their capacity to carry bracing loads.

- Nominal wall bracing – are walls in which the cladding is fixed using the nominal fasteners. They have some bracing capacity, but it is incidental.
- Structural wall bracing – are walls in which the cladding is fixed using reduced spacings, and special structural details are incorporated into the top and bottom of the wall to transfer bracing loads into and out of the wall, respectively. These walls have a significantly higher bracing capacity, even though the material can be similar to that of the nominal wall bracing. Both AS 1684.2 and AS 1684.3 require that structural wall bracing provide at least 50% of the required bracing capacity.

Structural wall bracing is specifically designed to provide stability for the house. It must provide at least 50% of the required wall bracing.

Figure 6.2 illustrates the details that must be incorporated into the construction of structural bracing walls to resist the racking and overturning failure modes.



(a) Typical connection details between sheeting and frame in structural wall bracing

(b) Connections between ceiling and floor diaphragms and the bracing wall

Figure 6.2: Features of structural wall bracing.

Figure 6.2(a) shows that the rigidity of the frame that prevents racking comes from the use of rigid cladding and effective closely spaced connections. Figure 6.2(b) shows the details that transfer shear from the ceiling plane into the wall and effectively tie the wall frame to the floor system. Unless the wall is effectively tied to the floor system, the rigid bracing wall will overturn under horizontal loads applied at the top plate, as illustrated in Figure 3.3(c).

6.2 Calculation of Bracing Wall Requirements

A method to calculate bracing forces is given in Section 8 of AS 1684 and is outlined in this section. (A simplified method for calculating bracing forces on hip or gable roofs for N1 to N4 is also presented in Appendix F. The forces calculated from the tables in Appendix F are usually higher than those calculated using the method in Section 8 in AS 1684.)

Bracing forces must be evaluated for two wind directions: wind perpendicular to the ridge (giving bracing requirements for walls perpendicular to the ridge) and wind parallel to the ridge (giving bracing requirements for walls parallel to the ridge).

6.2.1 Area of Elevation

The area of elevation is illustrated in the diagrams at the top of Tables 8.2 to 8.5 in AS 1684.2 or AS 1684.3 (see Figure 6.3 with an example in Figure 6.4). The area of the elevation starts at mid-height of the bracing level under consideration and goes to the top of the roof.

6.2.2 Total Racking Force on a House for a Given Wind Direction

The total racking force at each level in the house can be calculated using Equation 2. This uses the area of elevation evaluated in Section 6.2.1, and the pressures taken from the relevant table (Tables 8.2 to 8.5 in AS 1684.2 or AS 1684.3).

Equation 2: Total racking force (kN) = Area of elevation (m^2) x Lateral wind pressure (kPa)

Tables 8.1 to 8.5 in AS 1684.2 or AS 1684.3 can be used to determine the pressures on the house for the two required wind directions using the following steps:

1. Select the table with the appropriate wind direction: Tables 8.2 and 8.3 for wind perpendicular to the ridge and Tables 8.4 and 8.5 for wind parallel to the ridge.
2. Select the table appropriate for the level of the building. (Both the lower and upper storeys must be evaluated for two-storey buildings). Tables 8.2 and 8.4 apply to single or upper storeys and Tables 8.3 and 8.5 apply to lower storeys.
3. Select the section of the table corresponding to the wind classification for the house.
4. Select the row corresponding to the width of the house, measured parallel to the wind direction from external wall to external wall.
5. Select the column corresponding to the roof slope (pitch). If the roof has a number of different pitches, the pitch of the roof panel facing the wind is used.

An example of determining bracing loads

The bracing requirements for a N2 single-storey house with a 25° pitch hip-roof, 600 mm eaves, wall height of 2.7 m and external dimensions 16 m x 12 m can be found for wind perpendicular to the ridge as illustrated in Figure 6.3. (A similar calculation is required for wind parallel to the ridge.)

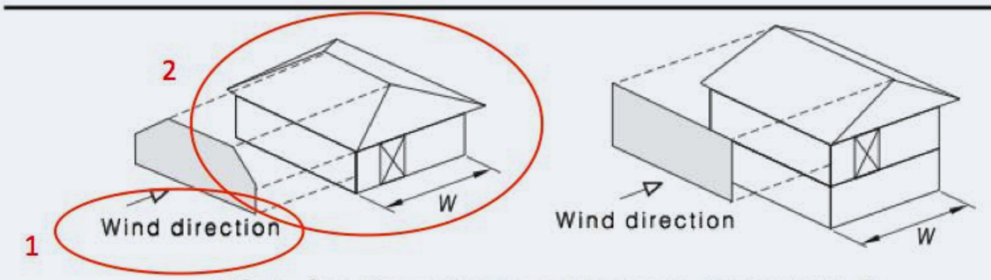
The lateral wind pressure on the elevation is determined using the steps given above. In this case:

1. Tables 8.2 or 8.3 are appropriate tables for wind perpendicular to the main ridge.
2. Table 8.2 is appropriate for single-storey houses, so should be used in this case.
3. The second section of the table is for N2 wind classification.
4. The width of the house is 12 m (the dimension perpendicular to the main ridge).
5. The column for 25° roof pitch is selected.

The lateral wind pressure is given at the intersection of the appropriate rows and columns in the table; 0.72 kPa.

TABLE 8.2

PRESSURE (kPa) ON AREA OF ELEVATION (m^2)—SINGLE STOREY OR UPPER STOREY OF TWO STOREYS—LONG LENGTH OF BUILDING—HIP OR GABLE ENDS



NOTE: See Figure 1.1 for guidance on determining *W*.

W	Roof pitch, degrees							
m	0	5	10	15	20	25	30	35
N1								
4.0	0.61	0.53	0.48	0.44	0.44	0.52	0.56	0.55
5.0	0.61	0.52	0.46	0.41	0.42	0.50	0.54	0.53
6.0	0.61	0.50	0.44	0.39	0.42	0.50	0.53	0.54
7.0	0.61	0.49	0.42	0.38	0.43	0.51	0.53	0.54
8.0	0.61	0.47	0.40	0.37	0.43	0.51	0.52	0.54
9.0	0.61	0.46	0.39	0.36	0.44	0.52	0.51	0.54
10.0	0.61	0.45	0.38	0.35	0.44	0.52	0.51	0.54
11.0	0.61	0.44	0.36	0.35	0.45	0.52	0.51	0.55
12.0	0.61	0.42	0.34	0.35	0.45	0.52	0.51	0.55
13.0	0.61	0.41	0.33	0.36	0.46	0.52	0.52	0.55
14.0	0.61	0.40	0.31	0.36	0.46	0.53	0.52	0.56
15.0	0.61	0.39	0.30	0.36	0.47	0.53	0.52	0.56
16.0	0.61	0.39	0.29	0.37	0.47	0.53	0.52	0.56
N2								
4.0	0.84	0.74	0.67	0.61	0.61	0.72	0.77	0.76
5.0	0.84	0.71	0.64	0.57	0.58	0.69	0.75	0.74
6.0	0.84	0.69	0.61	0.55	0.59	0.70	0.74	0.74
7.0	0.84	0.67	0.58	0.53	0.59	0.70	0.73	0.74
8.0	0.84	0.65	0.56	0.51	0.60	0.71	0.72	0.75
9.0	0.84	0.64	0.54	0.49	0.61	0.71	0.71	0.75
10.0	0.84	0.62	0.52	0.48	0.61	0.72	0.70	0.75
11.0	0.84	0.60	0.50	0.48	0.62	0.72	0.71	0.75
12.0	0.84	0.59	0.47	0.49	0.63	0.72	0.71	0.76
13.0	0.84	0.57	0.45	0.49	0.63	0.73	0.71	0.77
14.0	0.84	0.56	0.43	0.50	0.64	0.73	0.72	0.77
15.0	0.84	0.55	0.42	0.50	0.65	0.73	0.72	0.77
16.0	0.84	0.53	0.40	0.51	0.65	0.73	0.72	0.78

NOTE: 0° pitch is provided for interpolation purposes only.

(continued)

Figure 6.3: Example of calculation of pressures for bracing wall calculations – Excerpt from Table 8.2 in AS 1684.2. (Reproduced by WoodSolutions with the permission of Standards Australia under Licence 1606-c031.)

The area of elevation is calculated corresponding to the illustration selected at the top of the table. In this case, it is the left-hand diagram in Figure 6.3 for a single-storey hip-roof house.

The width of the elevation is the length of the building (16 m) and the roof pitch gives a roof height of 2.79 m. The wall height of 2.7 m means that there is 1.35 m of wall above the mid-height of the bracing walls as shown in Figure 6.4.

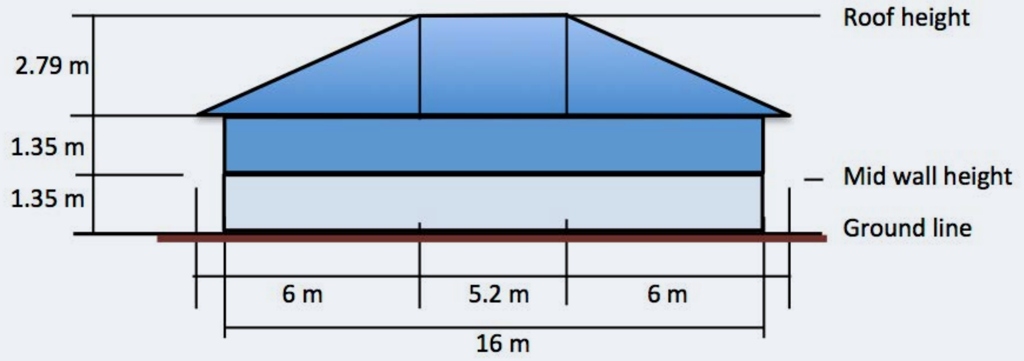


Figure 6.4: Example of calculation of area of elevation.

The area of elevation is the total area of the dark shading comprising the roof and the upper half of the wall.

Area of elevation of upper half of wall = 16 x 1.35 = 21.6 m^2

Area of elevation of roof = 6 x 2.79/2 + 5.2 x 2.79 + 6 x 2.79/2 = 31.2 m^2

(Area of triangle on left + area of rectangle in centre + area of triangle on right)

i.e The total area of elevation for the bracing wall calculations is 21.6 + 31.2 = 52.8 m^2

Hence the total bracing force from Equation 2 is:

Area of elevation x lateral wind pressure = 52.8 x 0.72 = 38.0 kN.

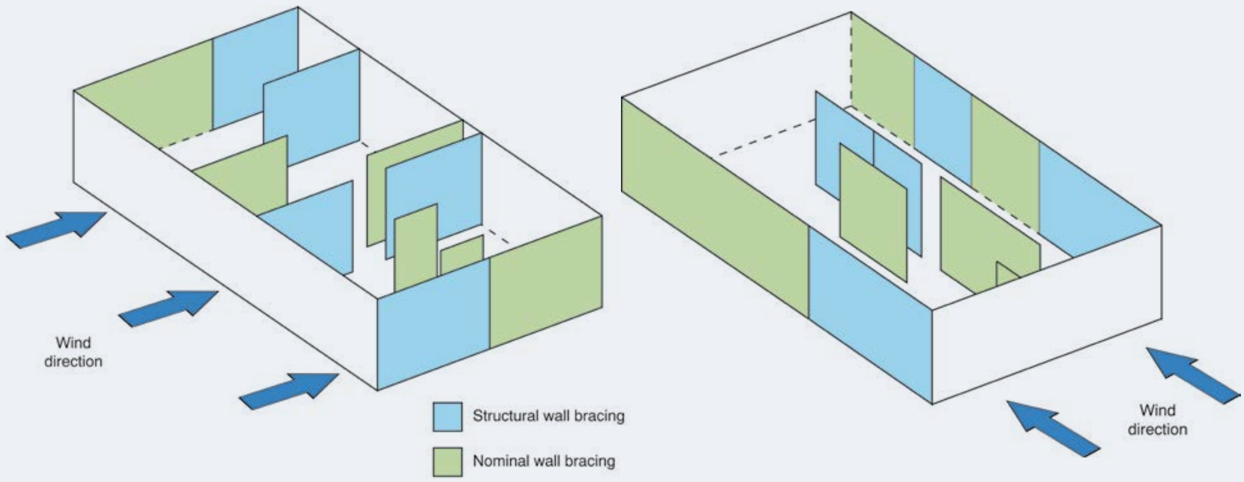
Therefore, for this wind direction, the total bracing capacity of walls perpendicular to the main ridgeline must exceed 38.0 kN. Nominal bracing can only provide half this value, i.e. 19 kN.

6.3 Calculation of Bracing Resistance of the Walls

The total bracing resistance of a house at each floor level is found by summing the capacity of all nominal wall bracing and all structural wall bracing for all walls that are parallel to the wind direction. Figure 6.5 illustrates structural and nominal wall bracing parallel to the wind directions.

The total bracing resistance of a house is found by summing the capacities of all nominal and structural wall bracing parallel to the wind direction.

Each metre length of wall is either nominal or structural bracing. It can’t be both. For example, plasterboard installed over structural bracing is not counted as nominal bracing as that part of the wall is considered structural bracing.



(a) Wind perpendicular to ridge of the roof

Figure 6.5: Potential bracing walls for each wind direction.

6.3.1 Capacity of Nominal Wall Bracing

Nominal wall bracing was defined in Section 6.1.2. Irrespective of the wall cladding material, the nominal wall bracing has a capacity of 0.45 kN/m if the wall frame is sheeted on one side only, or 0.75 kN/m if it is sheeted on both sides (see Table 8.17 in AS 1684.2 or AS 1684.3).

The capacity of nominal wall bracing is calculated by multiplying the total length of each type of nominal wall bracing (i.e. single-sided or double-sided) parallel to the wind by the listed capacity as shown in Equation 3.

Equation 3: Nominal wall bracing (kN) = length (m) of single sided wall x 0.45 kN/m + length (m) of double-sided wall x 0.75 kN/m.

6.3.2 Capacity of Structural Wall Bracing Types

Table 8.18 in AS 1684.2 or AS 1684.3 can be used to determine the bracing capacity in kN/m of different types of bracing for wall heights of up to 2.7 m. Table 8.19 in AS 1684.2 or AS 1684.3 allows these values to be scaled for higher structural bracing walls.

Table 8.18 in AS 1684.2 or AS 1684.3 provides bracing capacities for the following structural wall bracing systems:

- two diagonally opposed metal or timber angle braces
- tensioned metal straps
- timber and metal angle braces
- tensioned metal straps with stud straps
- diagonal timber wall lining or cladding
- structural plywood
- decorative plywood
- hardboard.

Where the frames are of joint strength group JD5 (normally the case for MGP10 studs), then the bracing capacity is reduced, as required in Clause 8.3.6.3 of AS 1684.2 or AS 1684.3.

In open floor plans, there may be a limited length of wall available for structural wall bracing. In these cases, it may be necessary to select the highest capacity systems to provide the required bracing capacity for the house.

The capacity of structural wall bracing is found by multiplying the total length of each type of structural wall bracing parallel to the wind by the listed capacity, as shown in Equation 4.

Equation 4: Structural wall bracing (kN) = Sum [length (m) each type x capacity (kN/m)]

Table 8.18 g), h) and i) in AS1684.2 and AS 1684.3, together with manufacturers' specifications, also infer that sheet bracing can be applied to both sides of wall frames and that this doubles the bracing capacity of these walls. Double-sided bracing walls require increased bottom plate thicknesses and additional tie-down for the walls to match increased capacity for the double-sided walls.

6.3.3 Spacing of Bracing Elements

The best bracing systems have the bracing capacity spread evenly throughout the floor plan of each level. AS 1684.2 or AS 1684.3 prescribes maximum spacing of bracing walls that is a function of the wind classifications.

6.4 Connections for Bracing Walls

As indicated in Section 6.1.1, it is necessary to have appropriate fixing and anchorage details to provide bracing capacity and resistance to overturning.

Details for structural bracing walls are provided in AS 1684.

6.4.1 Fixing of Bracing

Each bracing system uses a fixing system between the bracing and the frame, as illustrated in Figure 6.2(a). The type of connection and the spacing of connections are prescribed in Table 8.18 in AS 1684.2 or AS 1684.3. These are minimum connection requirements.

6.4.2 Connection at the Top of the Bracing Wall

There are a number of different types of connections that can be used between the wall and the ceiling. One type of connection, suitable for walls parallel to the trusses, is illustrated in Figure 6.2(b). A connection must be selected that delivers at least the capacity of the structural wall bracing in the panel it connects (see Table 8.22 in AS 1684.2 or AS 1684.3).

6.4.3 Connection at the Bottom of the Bracing Wall

The connections at the bottom of the structural bracing walls are designed to prevent over turning. These connections also have enough capacity to transmit the bracing loads to the floor system. The bottom part of Figure 6.2(b) illustrates one type of connection suitable for walls parallel to the floor joists.

The first step in selecting the connection at the bottom of a structural bracing wall is to look up the uplift force on the connection from the length of the structural wall bracing and its capacity in kN/m using Table 8.23 in AS 1684.2 or AS 1684.3.

This uplift force is to be delivered by one of the connections selected from Table 8.24 in AS 1684.2 or AS 1684.3. If the Joint Strength Group of the wall frame and floor member are different, use the one with the lowest capacity.

6.5 Temporary Bracing during Construction

Temporary bracing is required to resist the wind forces on the house while it is being constructed. The temporary bracing should have at least 60% of the permanent bracing capacity, and can be used as part of the permanent bracing.

Inadequate temporary bracing during construction may lead to failures under wind actions.

Figure 6.6 shows two houses that failed under wind loads during construction because the temporary bracing could not resist the wind loads applied to the house. In both cases, the houses were two storey.

Figure 6.6: Failures of houses under construction due to inadequate temporary bracing.

7 Construction of New Houses

The Codes and Standards enable designers and builders to address all of the structural requirements that ensure new houses have the capacity to resist wind forces.

7.1 Compliance to BCA

The BCA specifies minimum requirements and assumes that all details will be designed and installed correctly. It makes no allowance for construction errors or omissions. A nation-wide study of houses under construction conducted by the CSIRO (Stewart, 2016) indicated that, in a survey of new houses under construction, almost all houses contained elements or connections that were incorrectly designed and/or installed. These included:

- incorrect wind classification for the site
- incorrect selection of fasteners for batten-to-rafter or truss connections (e.g. nails instead of screws used for batten-to-rafter or truss connections in edge regions of roofs)
- incorrectly installed connectors between trusses and top plates, or between rafters, underpurlins, struts and top plates
- insufficient tie-down in walls on either side of very large openings, or for beams in large outdoor areas under the main roof.

Houses will only perform well in high wind events if all details have been designed and installed correctly.

Damage investigations indicate that wind damage is more likely if there are errors in design or construction. Therefore, it is critical that designers and builders diligently apply the requirements of AS 4055 and AS 1684. If there is any doubt about a design or construction decision, then it is prudent to take a conservative approach.

7.2 Wind Classification

It is crucial that the wind classification for the house is correct. AS 4055 should be followed diligently and the classification checked using a site visit. If the classification is one level too low (e.g. classified as N1 when the site is actually N2), the wind loads on the tie-down connections will be almost double the value expected for the lower classification. This will lead to failure of connections and damage to the house in events with wind speeds lower than the design wind speed (see Section 4).

The correct wind classification for a house is vital to avoid failures in winds near the design wind event.

7.3 Timber Framing

Lightweight timber-framed houses are able to provide resistance to wind forces, provided that designers and builders correctly interpret and apply the tie-down requirements in AS 1684 (see Section 5).

Designers should specify details on the plans that enable tradespeople to install the correct tie-downs for each link in the tie-down chain. (Experience has shown that this approach is more effective than a standard note that refers to compliance with AS 1684.) Education in the industry and supervision during construction are required to focus on the importance of every single tie-down to the performance of the house under wind loads. Even small details matter, as any structural inadequacies have the potential to cause significant damage and compromise the safety of occupants.

Decisions about bracing are made at the design stage. Designating a wall as a structural bracing wall has implications for the adjoining roof and floor structure (see Section 6).

7.4 Internal Pressure

AS 1684.2 and AS 1684.3 span tables and connection loads incorporate assumptions on internal pressures consistent with AS 4055.

7.4.1 Internal pressures in non-cyclonic areas – Wind Regions A and B

Consistent with the internal pressures used in AS 4055 (see Section 3.2), AS 1684.2 bases all member sizes and tie-down loads on the assumption that there are low internal pressures in houses at the time they experience maximum wind loads. This assumption is not valid if there are large openings in the building envelope and the design capacity of members – the capacity of tie-down connections may be exceeded if envelope elements fail under wind loads. The following strategies may improve the resilience of houses in Wind Regions A and B:

- Specify that all windows, doors and garage doors have the same wind classification as the rest of the house.
- Either avoid attaching sail structures or patio roofs directly to the roof structure, or size all tie-down details in the house roof structure to resist the increased uplift loads from the extra roof area attached to the house.

7.4.2 Internal Pressures in Cyclonic Areas – Wind Regions C and D

Consistent with the internal pressures used in AS 4055 (see Section 3.2), AS 1684.3 bases all member sizes and tie-down loads on the assumption that there is full internal pressure in houses at the time they experience maximum wind loads. This assumption means that even if large openings develop in the building envelope, the tie-downs will be sufficient to resist the wind forces.

7.5 Installation of Windows and Doors

Windows and doors should be specified using the wind classification for the site and comply with relevant Australian Standards. If more than 25% of a window or door is within 1200 mm of an external corner, they should be rated for the increased corner pressures.

Windows and doors must be correctly fixed to the building frame. Manufacturers and the Australian Window Association provide guidance.

Wind damage investigations in all Wind Regions have found houses where window and door frames have been removed from the house frame by the wind actions – either blown inwards on the windward wall or sucked outwards on other walls.

The Australian Windows Association (2010) has published guidelines for securing different types of windows and doors to the building frame. Recommended clearances between window frames and wall frames should be used and close fitting packers installed at all fastener locations. Appropriate flashings should also be fitted on all sides, so that water is drained to the outside of the building at the window.

Figure 7.1(a) shows a window where packers had been omitted and, during a wind event, the nails had bent, released from the stud and allowed the window to blow inwards.

(a) Window frame separation from the building frame

(b) Door latch failure on double entrance doors

Figure 7.1: Window and door failures.

The door and window furniture (latches, bolts, hinges) should also be able to transmit the wind loads from the doors or window sashes to the frame. Even in N1 classifications, the net force to be transmitted by all of the furniture on a single entrance door is 1.2 kN. The net force in C1 classifications is 3.6 kN. Double entrance doors will attract twice this load and, if adequate bolts top and bottom are not installed into the frame, even robust door latches will fail, as shown in Figure 7.1(b).

Any failure of a door or window, whether by damage to the door or window, failure of the connection to the frame or failure of the latches, will cause high internal pressures (see Section 7.4), and allow large volumes of water to enter the house.

7.6 Installation of Garage Doors

Garage doors cover large openings in the building envelope and therefore attract significant wind forces. These forces are applied to the guides either side of the opening and are transmitted by the jambs to the floor and ceiling diaphragm. Garage doors installed in Wind Regions C and D must comply with the provisions of AS/NZS 4505 (2012).

The jambs at garage doors require a design check to ensure they can resist the forces generated by the door under wind loads. Manufacturers provide information on the forces the doors will place on the frames.

The garage door Standard requires that design information for the door indicates the wind forces that the building frame needs to resist:

- Inward and outward forces on the total door area are shared equally between the jambs on either side of the door.
- For higher wind classifications, the door may have wind locks. Where these are fitted, the door jambs will also have to resist substantial forces in the plane of the door.

In all Wind Regions, the door jambs should be designed by an engineer to have sufficient strength to resist these forces in two different directions. The connection at the top of the jambs needs to have the capacity to carry the reaction into the roof structure.

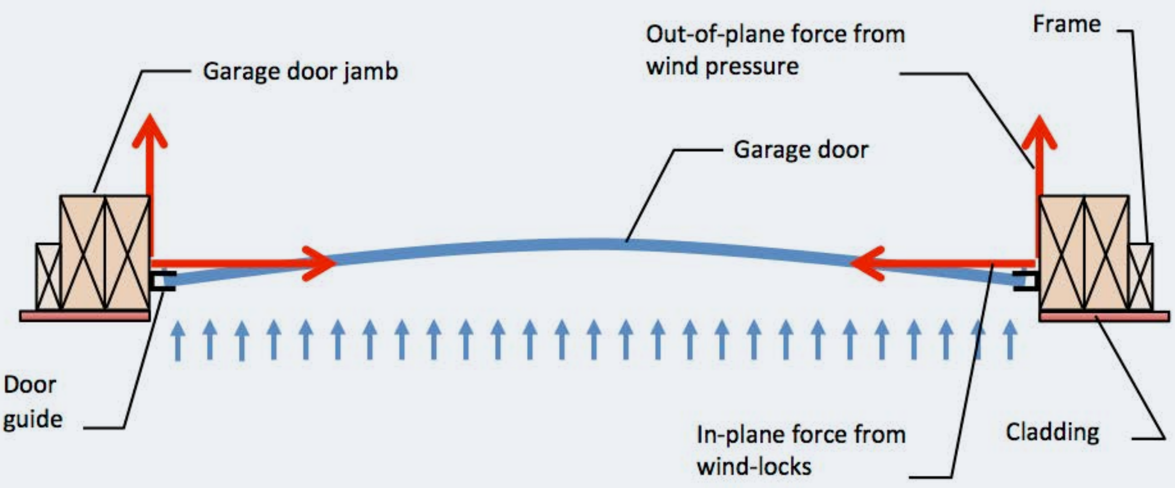


Figure 7.2: Forces on garage door jambs (plan view).

7.7 Tie-down of Stick-built Roofs to Masonry Walls

AS 1684.2 and AS 1684.3 do not provide details on tying timber-framed roofs to masonry walls. Brick veneer construction relies on conventional timber wall frames, so tie-down details in AS 1684.2 or AS 1684.3 apply. The Cyclone Testing Station Technical Report 49 (2003) provides the capacities for some methods of tying trusses to reinforced masonry. The remainder of this section provides information that can be used to tie down timber-framed roofs to unreinforced masonry.

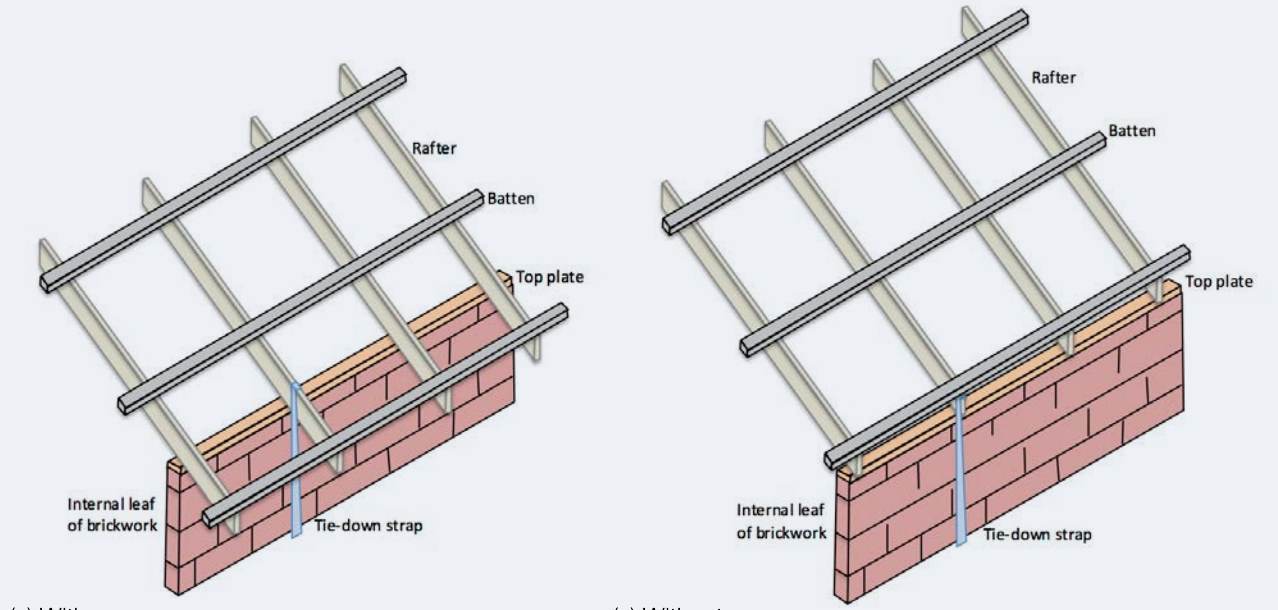
Extra details are required for the connection of timber roofs to masonry walls.

As indicated in Section 5.6.2, in wind classifications N1 and N2, it is possible to tie down coupled roofs to external walls only. This can be achieved using tie-down straps detailed in AS 4773.1.

Figure 7.3, Figure 7.4 and Figure 7.5 illustrate three commonly used configurations of tie-down details used with tie-down straps embedded in the internal leaf of cavity brick walls that comply with Section 3.3.3 of the BCA Volume 2. The connection at the top of the strap in all three details can be designed to resist the tie-down forces calculated for the tie-down spacing using Table 9.13 in AS 1684.2 or AS 1684.3. An appropriate nailed or screwed connection into the strap from Table 9.21 in AS 1684.2 or AS 1684.3 can be selected to carry this force.

Tie-down straps adjacent to large openings carry larger forces and need to be anchored to the base of the wall or directly to the concrete slab. The connectors between the tie-down strap and the timber framing need to have a higher capacity to resist the larger tie-down forces.

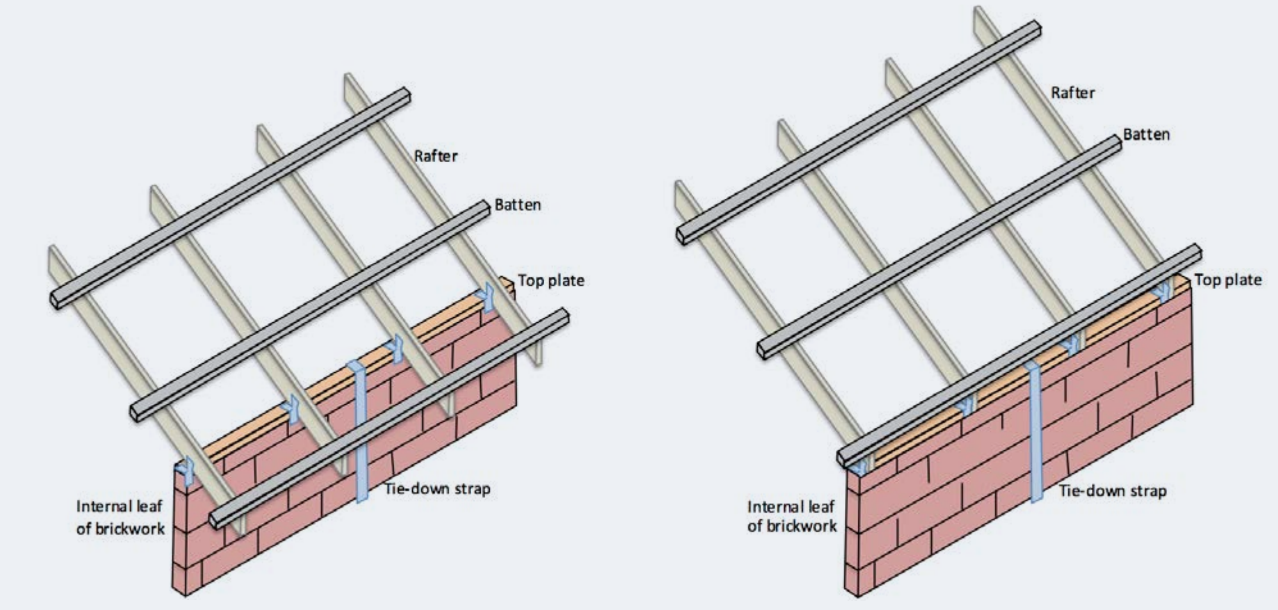
Figure 7.3 shows a tie-down configuration with steel tie-down straps fixed directly to rafters, as illustrated in AS 4773.1. For this configuration, the top plate acts as a packer and has no structural role in the tie-down chain.



(a) With eaves (a) Without eaves

Figure 7.3: Tie-down strap over rafter.

Figure 7.4 shows a tie-down configuration with steel tie-down straps fixed to the masonry top plate over the inner leaf of brickwork in all external walls. This connection is also illustrated in AS 4773.1. The top plate is an integral part of the tie-down system for the roof; uplift loads are transferred from the rafters to the top plate by framing anchors or other connections, in accordance with Table 9.21 in AS 1684.2 and AS 1684.3.

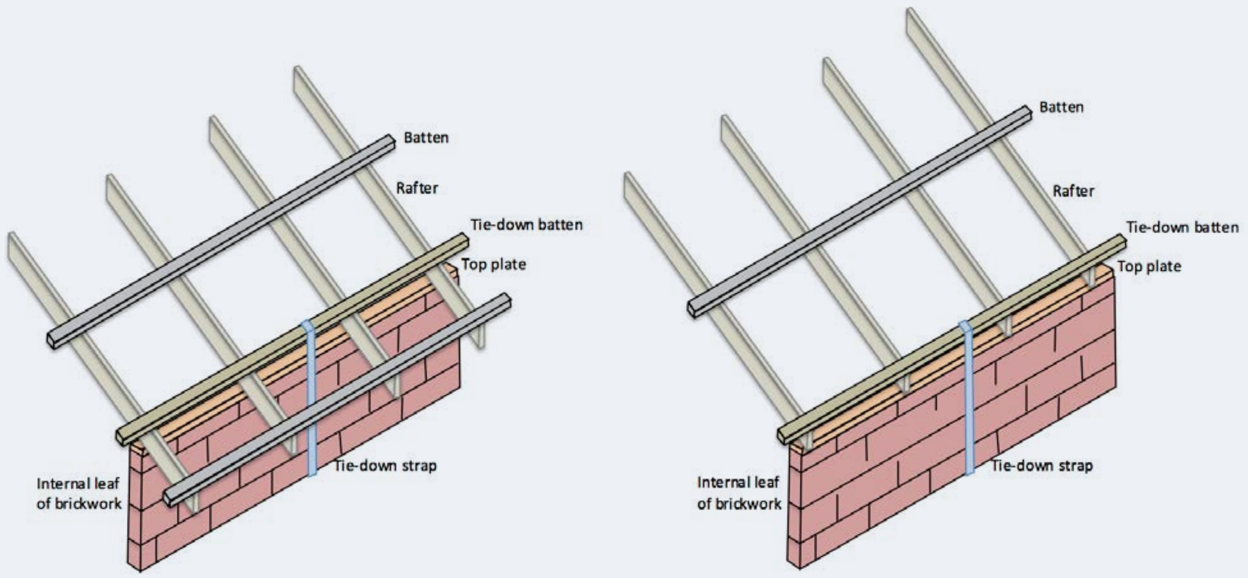


(a) With eaves (a) Without eaves

Figure 7.4: Tie-down strap over top plate with framing anchors on rafters.

Figure 7.5 shows an alternative tie-down configuration that is not illustrated in AS 4773.1. It uses steel tie-down straps fixed directly to a tie-down batten positioned over the inner leaf of brickwork in all external walls. The tie-down batten needs to be positioned directly over the cavity and is the second batten up the slope where the building has eaves, or the edge batten if there are no eaves. For this configuration, the top plate acts as a packer and has no structural role in the tie-down chain.

The tie-down battens should be checked to ensure they have sufficient bending capacity to resist the forces from the tie-down strap and uplift from the roofing.



(a) With eaves

(a) Without eaves

Figure 7.5: Tie-down strap over tie-down batten.

8 Tie-downs in Existing Houses

Investigations following high wind events in all regions of Australia have shown that many older buildings are damaged due to inadequacies in the tie-down system such as:

- deterioration of structural elements (e.g. corrosion of fasteners, rot or termites in timber elements)
- weakening of connections during previous wind events
- unauthorised additions
- construction that would not meet the requirements of current Standards
- damage from people walking on roofs.

Many structural tie-down connections can be difficult to access in completed houses. However, there are opportunities to check and improve the tie-downs when renovating, extending or repairing houses.

Where the roof structure was originally installed as unseasoned hardwood, it will have become seasoned over time and may split if screws are installed without pre-drilling. Seasoned hardwoods should be predrilled before installing all screws, including Type 17 screws. Care should also be used when nailing seasoned hardwoods to avoid splitting.

If existing timber members are split, or damaged by rot or insects, the member will be weakened and should be replaced.

8.1 Replacing Roof Cladding and Other Renovations

Replacement of roof cladding provides an ideal opportunity to check the structural connections between battens and rafters or trusses, and the roof structure to wall connections (see Sections 5.4 and 5.5). If Clause 9 of the current version of AS 1684 shows that the tie-down connections throughout the roof structure do not have the capacity to resist the wind loads for the site, then these connections should be upgraded to give the house sufficient strength to retain its new roofing in a design wind event.

When roof cladding is being replaced, take the opportunity to check and upgrade all tie-downs in the whole roof.

However, replacing a tiled roof with a sheet roof will require some upgrading to the roof tie-downs. The lighter mass of the sheet roofing dramatically increases the net uplift on the structure under wind actions, so the work has to include checks of the entire roof structure to ensure that each connection has the required tie-down capacity for the new cladding. The connections can often be strengthened using screws or straps once the tiles have been removed.

Other renovations that involve removal of at least part of the roof provide an opportunity to check and, if necessary, improve the tie-down chain in the roof structure. If problems are found in the part of the roof that has been inspected, it is advisable to make the same improvements to rest of the roof structure. This will mean that the renovated part of the house will not be endangered by failures in the original structure.

8.2 Additions and Extensions

Additions or extensions increase the size of a house and may mean that the completed house will not meet the geometric limitations in AS 4055 and AS 1684. If this is the case, then AS/NZS 1170:2 must be used to evaluate wind loads and AS 1720.1 used to design members and connections. Otherwise, AS 4055 and AS 1684 can be used as outlined in this Design Guide.

8.2.1 Check Wind Classification

If the wind classification is shown on plans for the existing house, then the wind classification for the site should also be checked (see Section 4) and revised if necessary. If the wind classification is not known, then it needs to be evaluated. Check if the new work requires a building permit. It is advisable that any new structural components comply with AS 1684.2 or AS 1684.3 for the correct wind classification. Although not mandatory, also check the tie-down capacity on the existing part of the house and upgrade if required to ensure the whole house can resist the design wind forces.

8.2.2 Changes to Tie-down Load Paths

An addition or extension, including attaching carports and pergolas, increases the roof area of the house and may change the load paths through the structure. If the tributary area of an existing element increases, then the tie-down force will also increase. It is important to check that all tie-down connections on all load paths have the capacity to resist the new uplift forces created by the additional structure.

All connections in the new addition and those that may change in the older part of the house should be sized and specified to comply with AS 1684.2 or AS 1684.3 (see Section 5).

8.2.3 Removal of Bracing Walls

Some additions may involve the removal of walls in the existing house. If any of these walls were bracing walls, then extra bracing capacity should be provided in the extension so the total structural and nominal bracing in the completed house meets the requirements of AS 1684.2 or AS 1684.3 (see Section 6).

8.3 Repairs following Damage from Wind Events

Check all elements and tie-down connections in the roof structure after significant wind events to ensure that there is no hidden damage. If damage is found, it should be rectified to ensure that the house has the capacity to resist uplift loads in future wind events. Figure 8.1 shows some examples of partial failure of connections in a house in Wind Region A that was exposed to high winds.

Check the roof structure for signs of damage to tie-down connections after severe wind events.

(a) Batten-to-rafter connection

(b) Rafter-to-underpurlin connection

(c) Underpurlin-to-strut connection

(d) Strut-to-top-plate connection

Figure 8.1: Partially damaged tie-down connections in a roof due to wind uplift.

Damage to part of a roof during a wind event is evidence of a structural deficiency, and it needs to be addressed anywhere it occurs in the whole roof, not just in the damaged section, as wind from a different direction may apply loads to connections that weren't damaged in the original event.

If problems in the roof tie-downs are found in any part of the roof, check and upgrade all tie-downs throughout the roof.

Damage to a roof during a wind event could be caused by:

- a local structural deficiency such as a missing or poorly installed tie-down connection. In this case, the damage will be confined to a relatively small area. An inspection will show that other connections comply with AS 1684.2 or AS 1684.3 and do not need to be upgraded. Only the damaged portion of the roof needs to be repaired.
- a systematic weakness in a specific connection through the whole roof, e.g. batten-to-rafter connections with insufficient capacity. In this case, damage will be more widespread. All of the deficient connections, even those in parts of the roof that were not damaged, need to be upgraded to provide resilience for future wind events. It is also advisable to check the tie-down connections further into the structure to ensure they have the capacity to carry loads that will be successfully transferred to them by the upgraded connections. For example, if batten-to-rafter connections have failed, the connections below such as rafter-to-wall connections will not have been subjected to the full wind load during the event. Once the batten-to-rafter connections have been upgraded, then the full wind load can be transmitted to the rafter-to-wall connections in future events. If they haven't been upgraded, they may now be 'the weakest link' in the chain and may fail.
- incorrect wind classification. In this case, other elements in the house such as windows and doors that may have been under-designed may also be damaged. This is particularly evident in houses on hills or in exposed locations where the original wind classification was taken as N1 or C1, but should have been higher. The wind classification should be checked (see Section 4) and the tie-down chain upgraded to match the new classification.
- deterioration of structural elements in the roof. In this case, the whole roof structure should be assessed and upgraded to meet the requirements of AS 1684.2 or AS 1684.3.

A number of useful references for repairing roofing after wind damage are available including:

- *Planning for a stronger, more resilient North Queensland: Part 2 – Wind resistant housing* – published by the Queensland Reconstruction Authority
- *Tie-down details for repair of storm damaged roofs in south east Queensland* – published on the Queensland Building and Construction Commission website: qbcc.qld.gov.au (appropriate for non-cyclonic areas)
- *Repair of sheet metal roofs in cyclonic areas* – published on the Queensland Building and Construction Commission website: qbcc.qld.gov.au (appropriate for cyclonic areas).

References

Australian Window Association (2010). *Installation – An industry guide to the correct installation of windows and doors.* Australian Window Association, Sydney, New South Wales.

Australian Window Association (2010). *Fixing – An industry guide to the correct fixing of windows and doors*. Australian Window Association, Sydney, New South Wales.

Australian Building Codes Board (2016). National Construction Code Series 2016 Volume Two – Building Code of Australia Class 1 and Class 10 Buildings. Australian Building Codes Board, Canberra.

Bureau of Meteorology. *About tropical cyclones.* Available from: http://www.bom.gov.au/cyclone/about/ [23 May 2016]

Boughton, GN (1999). *Tropical Cyclone Vance – Damage to buildings in Exmouth*. Department of Local Government, Western Australia.

Boughton, GN and Falck, DJ (2007). *Tropical Cyclone George – Damage to buildings in the Port Hedland area, TR52.* Cyclone Testing Station, James Cook University, Townsville.

Boughton, GN and Falck DJ (2008). *Shoalwater and Roleystone WA tornadoes – wind damage to buildings, TR54.* Cyclone Testing Station, James Cook University, Townsville.

Boughton GN, Henderson DJ, Ginger JD, Holmes JD, Walker GR, Leitch CJ, Somerville LR, Frye U, Jayasinghe NC, and Kim, PY (2011). *Tropical Cyclone Yasi – Structural damage to buildings, TR57*. Cyclone Testing Station, James Cook University, Townsville.

Boughton, GN and Falck DJ (2015). *Tropical Cyclone Olwyn – Damage to buildings in Exmouth, Western Australia, TR61.* Cyclone Testing Station, James Cook University, Townsville.

Boughton, GN, Falck, DJ, Duong, AVN, Pham, S, and Nguyen, A. (2015). *Static testing of batten connections at University of Western Australia, TR62.* Cyclone Testing Station, James Cook University, Townsville.

Duong, AVN (2015). Analysis of the Failure Behaviour of Screw Batten-to-Rafter Connections in Steel Clad Roof Structures, Thesis. University of Western Australia, Perth.

Henderson, DJ, and Leitch, C (2005). *Damage investigation of buildings at Minjilang, Cape Don and Smith Point in NT following Cyclone Ingrid, TR50*. Cyclone Testing Station, James Cook University, Townsville.

Henderson, DJ, Ginger, J, Leitch, C, Boughton, GN, and Falck, DJ (2006). *Tropical Cyclone Larry – damage to buildings in the Innisfail area, TR51*. Cyclone Testing Station, James Cook University, Townsville.

International Standards Organisation (2016) ISO 12122.5 *Timber structures – determination of characteristic values Part 5 Mechanical connections.* International Standards Organisation, Geneva, Switzerland.

Henderson, DJ, Ginger, J, Kim, P, and Sumant, B (2012). *Investigation of Townsville tornado, 20 March 2012, TR58*. Cyclone Testing Station, James Cook University, Townsville.

Leitch, C, Cummins, W, Henderson, D (2003). *An investigation of bond beam truss hold down connections, TR49*. Cyclone Testing Station, James Cook University, Townsville.

Leitch, C, Ginger, J, Harper, B, Kim, P, Jayasinghe, N, and Somerville, L (2009). *Investigation of Performance of Housing in Brisbane following storms on 16 and 19 November 2008, TR55*. Cyclone Testing Station, James Cook University, Townsville.

Lysaght (2014), *Topspan design and installation guide for building professionals*. Available from: <www.lysaght.com> [5 September, 2014].

Mahendran, M and Mahaarachchi, D (2002). Cyclic pull-out strength of screwed connections in steel roof and wall cladding systems using thin steel battens. *Journal of Structural Engineering* 10.1061 pp771-778.

Nguyen, A. (2015) Analysis of the Failure Behaviour of Screw Batten-to-Rafter Connections in Timber Roof Structures, Thesis. University of Western Australia, Perth.

Parackal, K, Mason, M, Henderson, D, Stark, G, Ginger, J, Somerville, L, Harper, B, Smith, D and Humphreys, M (2015). *Investigation of Damage: Brisbane, 27 November 2014 severe storm event, TR60.* Cyclone Testing Station, James Cook University, Townsville.

Pham, S (2015). *Analysis of the Failure Behaviour of Nail Batten-to-Rafter Connections in Steel Clad Roof Structures*, Thesis. University of Western Australia, Perth.

Prevatt, D, Shreyans, S, Kerr, A & Gurley, K. (2014). *In Situ Nail Withdrawal Strengths in Wood Roof Structures*. Journal of Structural Engineering, Vol. 140, No. 5, pp. 1-8.

Queensland Building and Construction Commission. *Tie-down details for repair of storm damaged roofs in south east Queensland* Available from: http://www.qbcc.qld.gov.au [20 June 2016].

Queensland Building and Construction Commission. *Repair of sheet metal roofs in cyclonic areas* Available from: http://www.qbcc.qld.gov.au [20 June 2016].

Queensland Reconstruction Authority (2011) *Planning for a stronger, more resilient North Queensland Part 2 Wind resistant housing*. Queensland Reconstruction Authority, Queensland Government. Brisbane, Queensland.

Reardon, G. F. (1979a). *The strength of batten-to-rafter joints Part 1 – Test results and derivation of design loads, TR 2.* Cyclone Testing Station, James Cook University, Townsville.

Reardon, GF (1979b) *The strength of batten-to-rafter joints Part 2 – Recommendations for high wind areas, TR 3*. Cyclone Testing Station, James Cook University, Townsville.

Reardon, GF, Henderson, DJ, and Ginger, J (1999). *A structural assessment of the effects of Cyclone Vance on houses in Exmouth WA, TR48.* Cyclone Testing Station, James Cook University, Townsville.

Stewart, M, Wang, X, Bradford, M, Ginger, J, Hao, H, Sanjayan, J, Willgoose, G, and Wilson, J (2016). *Final Report – Climate adaptation engineering for extreme events cluster, CAEx Report 1/2016*. CSIRO, Canberra, Australia, March 2016.

Timber Queensland (2015) *Wind tie-down connections – Getting it right Technical Data Sheet 32.* Timber Queensland Limited. Brisbane, Queensland.

Timber Queensland (2015) *Corrosion resistance of metal connectors Technical Data Sheet 35.* Timber Queensland Limited. Brisbane, Queensland.

Australian Standards

Standards Australia (2002). *AS/NZS 1170.0:2002 Structural design actions Part 0: General principles*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2011). *AS/NZS 1170.2:2011 Structural design actions. Part 2: Wind actions*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2001). *AS 1649 Timber – Methods of test for mechanical fasteners and connectors – Basic working loads and characteristic strengths.* Standards Australia, Sydney, NSW, Australia.

Standards Australia (2010a). *AS 1684.2 Residential timber-framed housing, Part 2 Non-cyclonic areas*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2010b). *AS 1684.3 Residential timber-framed housing, Part 3 Cyclonic areas*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2010c). *AS 1684.2 N3 Supplement 4–2010 Residential timber-framed housing, Part 2 Non-cyclonic areas N3 Supplement 4: Timber framing span tables–Wind classification N3–Seasoned softwood–Stress Grade MGP 10 (Supplement to AS 1684.2–2010).* Standards Australia, Sydney, NSW, Australia.

Standards Australia (2010d). *AS 1720.1 Timber structures Part 1: Design methods*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2015). *AS 1720.5 Timber structures Part 5: Nailplated timber roof trusses.* Standards Australia, Sydney, NSW, Australia.

Standards Australia (2012). *AS 4055 Wind loads for housing.* Standards Australia, Sydney, NSW, Australia.

Standards Australia (2010e). *AS/NZS 4063.2 Characterization of structural timber Part 2: Determination of characteristic values*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2005). *AS 4357.0 Structural laminated veneer lumber Part 0: Specifications*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2004). *AS 4440 Installation of nailplated timber roof trusses*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2012). *AS/NZS 4505: Garage doors and other large access doors*. Standards Australia, Sydney, NSW, Australia.

Standards Australia (2015). *AS 4773.1 Masonry in small buildings Part 1: Design.* Standards Australia, Sydney, NSW, Australia.

WoodSolutions Publications

The WoodSolutions website woodsolutions.com.au provides other User Guides and Technical Design Guides to support the use of AS 1684 and timber-framed construction:

WoodSolutions Technical Design Guide #1: *Timber-framed construction for townhouse buildings Class 1a*. Forest and Wood Products Australia, Melbourne, Australia, 2012.

WoodSolutions Technical Design Guide #5: *Timber Service Life Design.* Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 1 – Nominal vs specific fixings. Forest and Wood Products Australia, Melbourne, Australia.

AS 1684 User Guide 2 – Temporary bracing. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 3 – Simplified tie-down details for coupled roofs. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 4 – External wall heights. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 5 – Fixing of top of bracing walls. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 6 – Roof truss tie-down. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 7 – Ridgeboard and hip rafter tie-down. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 8 – Masonry anchor tie-down. Forest and Wood Products Australia, Melbourne, Australia

AS 1684 User Guide 9 – Fixing of bottom of hardboard bracing walls. Forest and Wood Products Australia, Melbourne, Australia.

AS 1684 User Guide 10 – Distribution of racking forces via diaphragms. Forest and Wood Products Australia, Melbourne, Australia

Additional Resources

The Cyclone Testing Station website (www.cyclonetestingstation.com.au) provides information for builders, designers and home-owners on construction to resist wind loads. The resources include videos on:

- site wind classifications for houses
- a worked example to determine the wind classification for a house
- upgrading an older roof
- re-roofing after storm damage.