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Fire Design

Timber concrete composite floors, timber cassette floors and post-tensioned exposed timber beams

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

978-1-925213-35-5

Acknowledgments

Authors: Prof Keith Crews
A/Prof Christophe Gerber
Prof Stefano Pampanin
A/Prof Alessandro Palermo
Prof Andrew Buchanan

First published: 2013
Revised: October 2016, October 2020

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Introduction

Despite public perceptions to the contrary, timber buildings can be designed to be very safe in fires and other emergencies. Designing for fire safety must enable people to escape in the early stages of a fire, and provide fire resistance to prevent a fire spreading and the building collapsing.[1]

This Guide provides a summary of fire safety design process for a number of EXPAN-developed systems, including timber concrete composite (TCC) floors, timber cassette floors and post-tensioned timber beams without any fire-rated lining or additional protection.

Design for structural stability in fire conditions is covered, as well as worked examples of the fire safety design procedures for each system. To get the best out of this Guide, it should be read in conjunction with AS/NZS 1720.4 and *WoodSolutions Technical Design Guide #17: Alternative Solution Fire Compliance.*

1.1 Fire safety – Fire Resistance Levels

The Building Code of Australia Deemed-to-Satisfy Provisions specifies fire resistance in terms of Fire Resistance Level (FRL). The required fire resistance will depend on the building's height, occupation use, the inclusion of sprinklers, escape pathways and other factors. The fire resistance may differ from the requirements of the prescriptive Deemed-to-Satisfy Provisions in the Building Code if a professional fire engineer provides a specific fire engineering design, leading to a Performance Solution design.

The required fire resistance is expressed in terms of Structural Adequacy, Insulation and Integrity for a specific time period that element is required to protect. Protection times vary from 60 to 120 minutes for various elements and situations in a building.

1.2 Fire Resistance Criteria

The National Construction Code –Building Code of Australia[2] (NCC BCA) and most others around the world specify fire resistance in terms of three criteria: Stability, Insulation and Integrity. The BCA calls this criteria the Fire Resistance Level (FRL). These categories are concerned with preventing the structural collapse of the element, preventing the transmission of fire and smoke through the element, and preventing unacceptable levels of heat being transmitted through the element.

For example, a specified FRL of 60/60/60 requires that all three criteria equate to 60 minutes exposure to a standard test fire, a time and temperature curve specified by AS 1530.4.

Design for Stability is required for structural walls and floors, as well as for isolated beams and columns. Calculations or tests must be used to show that the structural member can carry the applied loads for the full duration of the fire exposure without collapse or excessive deformation.

Design for Insulation and Integrity is required for containment elements, such as walls and floors, to prevent unwanted spread of fire within or between buildings. Design for Integrity and Insulation is not required when considering isolated structural members such as beams or columns.

1.2.1 Determining Fire Resistance Level

The NCC BCA has several methods to determine Fire Resistance Level, detailed in Schedule 5. In essence, there are three methods available for timber. The first method is via a test to AS 1540.4. AS 1540.4 requires a full-scale test that exposes the porotype element to a standard fire test for the relevant time period, measuring the time it takes to obtain failure of Stability, Integrity or Insulation. Another method is based on calculations performed on the test outcome, derived from the full-scale test of the prototype. This is normally done by a Registered Test Authority and generally relates to variations to span or height, or thickness and so on. The third method is also a calculation method, carried out via a reference Standard AS/NZS 1720.4. This method is applicable to certain engineered wood products only and is explained further in Section 1.4.1.

1.2.2 Fire Design for Stability

Design for Stability requires the structure to remain in place after fire exposure and still be able to support a load. The applied load under fire conditions is less than the standard structural load used to check Stability and strength as specified in the AS 1170 series of Australian Standards, as it is not expected that the full load will be in place during a fire. Therefore, fire design load is the full permanent load plus a fraction of the total imposed load, usually 40%.

When an exposed timber element is subjected to fire, a char layer forms that insulates the unburned material and reduces the subsequent rate of charring. For structural fire resistance, the timber element in the buildings may have many hours of fire resistance that can be determined from a full-scale test to AS 1530.4 or calculated using AS/NZS 1720.4 effective depth of charring calculation method. The latter method is based on a notional charring rate that is dependent on the density of the timber element at 12% moisture content. The density of engineered wood products is based on the dominate timber species not the element itself, which may have adhesive included. Design guidance is given in this Guide.

1.2.3 Fire Design for Integrity

Design for Integrity is required for containment elements, such as walls and floors, to prevent unwanted spread of fire between fire compartments or buildings. It is not required when considering isolated structural members such as beams or columns. Design for Integrity can only be determined from a full-scale test to AS 1530.4, as AS/NZS 1720.4 has no calculation method. For many timber mass systems, the joints between panels are the crucial point of failure. Design for Integrity requires that there must be no gaps or openings that allow flames or hot gases to pass through the member during the fire exposure period. Gap size may also be affected by deflection of the element during fire exposure; consequently, this should be considered in the design.

1.2.4 Fire Design for Insulation

Design for Insulation requires that the temperature rise on the unexposed surface of the barrier to be kept to a level that will not ignite objects on the non-fire side and allows for the building occupants to escape past the fire-resisting element without being impaired by high temperatures. Again, this can be determined from a full-scale test to AS 1530.4, where it requires the cold side of the barrier must not exceed the initial temperature by an average of 140°K or a peak of 180°K at the end of the fire exposure time.

Alternative AS/NZS 1720.4 provides a method via empirical data that has shown over many tests that a residual cross-section, after the nominated fire resistance period, is to be a minimum thickness of 30 mm.

1.3 Fire Hazard Properties

Safety in the initial stages of a fire depends on the fire hazard performance of covering materials. The building codes restrict the covering used in building, depending on its occupation, location in the building and whether fire sprinklers are installed. Exposed timber elements may be considered as coverings, and there may be additional limitations in the NCC BCA Deemed-to-Satisfy Provisions. Refer the NCC BCA[2] for further information.

1.4 Fire Resistance: Design of Timber Members by AS/NZS 1720.4

AS/NZS 1720.4 is a primary reference within the NCC BCA and can be used to determine the Fire Resistance Levels of a timber element.

1.4.1 Timber Products Subject to AS/NZS 1720.4

AS/NZS 1720.4 is not applicable to all timber products; it is appropriate for sawn timber, timber in the pole form, plywood, laminated veneer lumber (LVL) and glue-laminated timber. In addition, the timber product must meet the relevant Australian grading Standard for structural purposes; and for engineered timber, use adhesives that are either phenol, resorcinol, phenol-resorcinol or poly-phenolic structural adhesives. Timber products outside this scope can only use the full-scale prototype testing method AS 1530.4 to determining Fire Resistance Levels.

1.4.2 Design Procedure of AS/NZS 1720.4

The purpose of AS/NZS 1720.4 is to calculate the residual cross-section are of a timber element after exposure to a fire for a specific period of time. Failure of structural timber members in fires results from a loss of cross-sectional area rather than a loss of strength due to heating. This behaviour is due to timber's inherent insulating nature, which ensures that the penetration of heat into the timber section and the rate at which it chars is slow. When a timber member is exposed to fire, a char layer forms on its outer perimeter, which inhibits further combustion and heating of the remaining timber section except for a small zone immediately in front of the char layer.

The charring rate of timber varies due to its density, moisture content and grain direction, as well as the intensity of the fire. Higher-density timber chars slower than lower-density timber. The residual section of a timber member under a particular fire loading can be found by subtracting a thickness equal to the charring rate multiplied by the fire resistance period (in minutes) from each side of the member that is exposed to the fire, plus an additional depth to account for the heat-affected zone ahead of the char layer.

The main design steps for all structural timber elements, including beams, columns, floors, frames and walls, are:

1. Calculate the effective depth of char for the required fire resistance period.
2. Add the specified zero-strength layer to allow for heated wood below the char.
3. Calculate the size of the residual cross-section.
4. Design the residual timber section for flexure, axial load and shear strength using the fire load, loading case in AS 1170.0.
5. Include crushing and buckling calculations for any members with axial load.
6. Check the bearing area of elements to prevent bearing failure.

1.4.3 Determining the Structural Stability

The effective residual section is determined by subtracting the calculated *effective depth of charring* from all fire-exposed surfaces of the timber member. When determining the effective residual section, corner charring can be ignored.

The designer must check whether the residual section will withstand the design load for the period of required fire resistance, under the reduced load combinations specified by AS 1170.0, Section 4.2.4 as $G + \psi_l Q$ and any action arising from thermal effects.

In normal Stability and strength design, the timber element used in the horizontal orientation is typically governed by deflection limits, not strength. AS/NZS 1720.4 does not provide any advice on deflection, leaving it the decision of the design engineer. Other than collapse issues, the designer must consider the effect that deflection may have on joints and junction of elements within the timber building. Excessive deflection will cause gaps that will cause integrity failure. As a minimum, the deflection of the elements under fire load conditions should not be less than span/30.

Notional Charring Rate

AS/NZ 1720.4 has three methods to determine the notional charring rate:

- prescriptive notional charring rates for common timber species
- calculation method based on the timber specie's density
- notional charring rate determine from prototype test.

AS/NZS 1720.4 provides the notional charring rate of common timber species (see Table 1).

Table 1 – Notional Charring rates for common timber species.

Timber Species	Notional charring rate mm/minute
Blackbutt	0.50
Cypress	0.56
Douglas fir	0.65
European spruce	0.65
Spotted gum	0.46
Grey Ironbark	0.46
Red Ironbark	0.47
Jarrah	0.52
Merbau (Kwila)	0.51
Radiata pine	0.65
Victorian ash and Tasmanian oak	0.59

Calculation method for Notional Charring rate

AS/NZS 1720.4 provides the calculations method based on the timber species density at a moisture content of 12% in kg/m^3. The notional charring rate by the following equation:

$$C = 0.4 + (280/D)^2 \qquad (1.1)$$

where

C = notional charring rate in mm/min

D = timber density at a moisture content of 12% in kg/m^3

The average timber density of the base timber species at 12% moisture content can be found from AS 1684, AS 1720.1 and the WoodSolutions website. For engineered timber products, the published density (AS 1684, AS 1720,1 and the WoodSolutions website) of the timber species used predominately in the makeup of the element must be used. The actual density of the engineered timber is not used as it contains a component of adhesives that generally increases its density. For plywood and LVL products, the density at 12% moisture content of the timber species used to manufacture the product should be used.

Effective Depth of charring (de) calculated by:

$$d_c = C.t + 7.0 \qquad (1.2)$$

where

d_c = calculated effective depth of charring in mm

C = notional charring rate in mm/min, calculated above

t = period of time, in minutes.

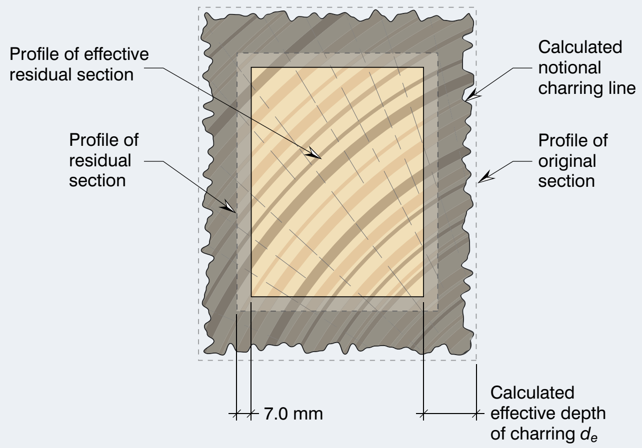

Figure 1.1: Char to timber element.

AS/NZS 1720.4 provides a test method to determine the national charring rate of individual timber species. The test method is based on AS 1530.4 test method and uses a 1,000 x 1,000 mm minimum size furnace that exposes a specimen of at least 75 mm thick to the Standard Fire test curve. The orientation of the specimen is essential as test sample orientated vertically are applicable for elements used in an upright orientation. For horizontally oriented specimens, the results can be used in both horizontal or vertical alignments.

The rate of char is determined by the depth of char divided by the duration of the test. Char depth can be determined by identifying the 300°C isotherm.

Calculation Method

The calculation method under fire load uses the same assumption as for Stability and strength conducted for the ambient temperature, i.e. cold conditions, except for the following minor changes as summarised:

- Design for strength in the ultimate limit state (ULS).
- Use the reduced load combinations specified by AS 1170.0, Section 4.2.4.

  $G + \psi_l Q$ and any action arising from thermal effects.

  ψ_l for Residential, offices, parking and retail $\psi_l = 0.4$. For storage and other $\psi_l = 0.6$.
- k factors as per standard structural design, but the load of duration shall be taken as 5 hours.
- Effective length of the element must consider any reduction of constraint that might occur due to fire exposure.
- ϕ factor as per standard structural design.
- The deflection should be calculated from the effective residual section and that the deflection limits are the responsibility of the design engineer; however, consideration is required to ensure that the deflection does not cause an opening, causing an integrity failure or affect other elements relying on the timber structure for support, such as concrete slab in timber concrete composite design.

1.4.4 Determining the Insulation

AS/NZS 1720.4 provides two methods to determine the insulation capacity of a timber element. The first method is by a test to AS 1530.4, and the procedures of determining Insulation are found in this Standard. Alternatively, AS/NZS 1720.4 provides the minimum thickness of residual timber of 23 mm when the effective depth of char is determined by calculation. This method provided a minimum of 30 mm thick timber.

1.4.5 Determining the Integrity

There is only one method to determine Integrity, and that is by standard fire test to AS 1530.4.

2

Fire Safety of Timber Concrete Composite (TCC) Floors

Timber concrete composite floors are where the timber and concrete work together to act as a floor system. Generally, the timber is connected to the concrete with shear connectors. This can reduce the amount of concrete required, as well as increasing the spanning capacity of the timber. *WoodSolutions Technical Design Guide #30 Timber Concrete Composite Floors* provides a detailed design procedure for this floor system.

2.1 Criteria for Fire Resistance of TCC

The following discusses how fire resistance can be achieved for a timber concrete composite floor system. There are several approaches that can be used to be NCC compliant. The first method is to develop compliance from the timber only components. The second is to ignore the timber and just consider the concrete components. The third is to consider the timber and concrete contribution together. The following discusses the approach required to achieve the Fire Resistance Level's Structural, Insulation and Integrity values.

2.1.1 Design for Stability

Design for Stability requires that the floor must carry the applied loads for the full duration of the fire exposure, without collapse or excessive deformation. The applied load is much less than the standard structural load as specified in AS 1170 and discussed in Section 1.2.2.

The fire design must check whether the residual timber section, after the exposure period, will withstand the design load for the period of required fire resistance. This can be done by calculating the residual timber section by subtracting the effective char depth from all exposed sides of the element. The effective char can be calculated from AS/NZS 1720.4 and discussed in Section 1.4.3.

2.1.2 Design for Insulation

Design for Insulation requires that the temperature rise on the top surface of the floor acting as the fire-resisting barrier, i.e. not floor covering, must not exceed an average of 140°C at the end of the fire exposure. There are a number of approaches that can be used, and each is dependent on the makeup of the TCC floor and fire resistance period required.

The first approach is to consider timber providing all of the necessary Insulation, and this is suitable when permanent wood formwork is used. This approach uses the calculated insulation procedure from AS/NZS 1720.4 and Section 1.2.2. This method requires the timber providing the barrier to be at least the effective depth of char plus 23 mm for the fire resistance period required. If this is satisfied, it is DTS to the NCC BCA[2]. This method is suitable for low Fire Resistance Level periods.

An alternative method is to ignore the contribution of the timber and consider insulation being achieved just by the concrete. Insulation of concrete is achieved by minimum thickness of the concrete, and this thickness of concrete is different for each fire resistance level. The minimum thickness of concrete for the fire resistance levels can be found from the DTS Standard AS 3600 Concrete Structures. This Standard provides the minimum thickness of concrete for various fire resistance levels and is summarised in Table 2.1.

Table 2.1: Minimum thickness of the concrete to achieve various Fire Resistance Levels.

	Fire Resistance Level period (minutes)			
	30	**60**	**90**	**120**
Minimum thickness of the concrete (mm)	60	80	100	120

The third method can be used when there is permanent wood formwork below the concrete. In this situation, both the concrete and timber working together to provide the Insulation. The method is not DTS and will require the development of a performance solution.

The presumption is that the timber in the permanent formwork will take time to char through. This time can be calculated from AS/NZS 1720.4 and Section 1.4.3. The time to char through the permanent formwork can reduce the fire resistance period required for the concrete. This provides for a new fire resistance period for the concrete and the minimum thickness of concrete required can be determined from Table 2.1.

Example: If a TCC floor were used for a residential apartment, it would require 90 minutes of fire resistance. If the TCC floor had 25 mm pine plywood is used as permanent formwork, then for the National Char rate of pine from AS/NZS 1720.4 of 0.65 mm/minute, the plywood would provide 38 minutes, i.e. 25/0.65.

Now the Fire Resistance Level of 90 can be reduced by 38 minutes to 52 minutes fire resistance required by the concrete. From Table 2.1 above, the minimum thickness of concrete for 52 minute fire resistance is 80 mm, a reduction of 20 mm that would be required for 90 minutes of fire resistance.

2.1.3 Design for Integrity

Design for Integrity requires that there must be no gaps or openings that allow the transmission of flames or hot gases through the floor for the fire exposure period. In achieving Integrity, it is dependent on the pathway that has been chosen for compliance. As discussed above, there are three methods, through the timber, the concrete or a combination of both.

For timber, compliance can only be achieved if the joint between panels and other elements is based on a standard fire test, as AS/NZS 1702.4 only has a test to AS 1530.4. For an LVL based system, there is an assessment by Warringtonfire[3] for a number of common methods to join LVL panels together for a variety of FRLs, refer to Figure 2.1.

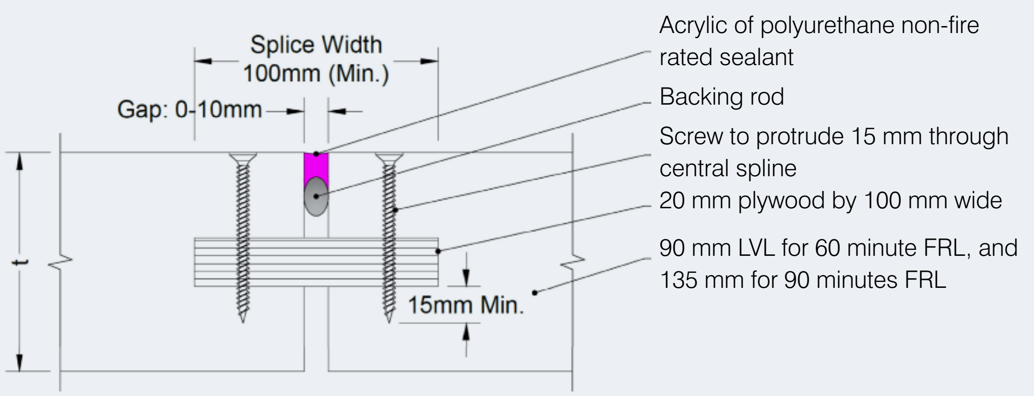

Figure 2.1: Compliant Integrity Joint in LVL Panels.

For concrete, Integrity can be achieved by having the minimum thickness of concrete for the fire-resistant period specified in Table 2.1.

The third method can be used when there is permanent wood formwork below the concrete. In this situation, both the concrete and timber working together to provide the Insulation. The method is not DTS and will require the development of a performance solution.

2.1.4 Shear Connector Fire Resistance

Protection of the shear connector in a TCC floor system can be achieved when the metal connector is fully embedded in the timber and concrete. For timber, the metal connector needs to be at least the minimum depth of effective char calculated for that fire resistance period. Shear connectors are usually provided by placement at the centre of the top edge of the timber joists.

2.2 Fire Design

The following checks the fire resistance capacity of a 400 × 63 mm LVL joist used for a TTC floor for FRL of 90 minutes, using AS/NZS 1720.4 calculation method.

Calculate Notional Charring Rate (C in mm/min):

$$C = 0.4 + (280/D)^2 \quad (2.1)$$

where

D = timber density at a moisture content of 12% in kg/m³

In this example, radiata pine LVL that has a species density of 550 kg/m³ density is used.

$\therefore C = 0.4 + (280/550)^2 = 0.66$ mm/min

Effective Depth of Charring (mm):

$$d_c = C.t + 7.0 \quad (2.2)$$

where

C = notional charring rate in mm/min, calculated above

t = period of time for the fire resistance required i.e. FRL, in minutes 90 mins

$\therefore d_c = 0.66 \times 90 + 7.0$ mm$= 66.3$ mm

This calculates the char protection, but the joist needs to include residual timber section to carry tension and bending, i.e. 67 + 67 + some wood to act in tension.

The char layer has to be applied to all exposed surface; generally, for a beam, this is for three sides, i.e. the two sides and bottom of the beam (see Figure 2.2), while a column requires the char layer applied to all sides. Generally, more economical timber section sizes are found by reducing the exposed surfaces of the timber element.

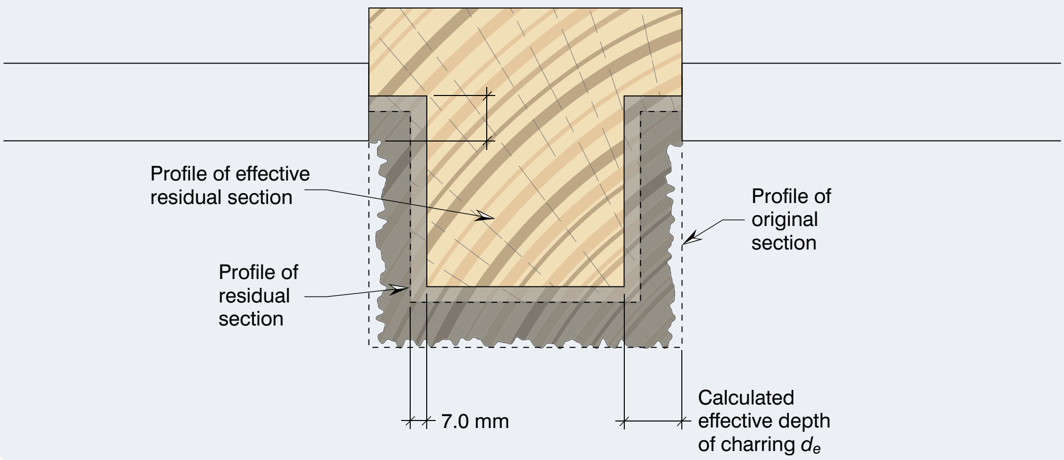

Figure 2.2: Char to timber beam on three sides.

The designer must also check whether the residual timber section will withstand the design load for the period of required fire resistance. Use the reduced load combinations as specified by AS 1170.0, Section 4.2.4 as G + ψ_l Q and any action arising from thermal effects. As the minimum requirement, the deflection should be less to span/30.

2.3 Construction using Double Joists

If the timber joists used for the TCC floors are made from two or more elements laminated together, special precautions must be taken to ensure that the individual elements do not separate during fire exposure. A gap may form between the laminated joists due to shrinkage effects that may expose the internal surfaces of the joist, increasing the number of surfaces that char can occur.

Prevention of this separation can be achieved by factory-gluing the joists or by using screws.

Where an adhesive is used, and the calculation method to determine the AS/NZS 1720.4 effective depth of char is required, only adhesives that are phenol, resorcinol, phenol-resorcinol or poly-phenolic can be used. Where an alternative method to arrive at fire resistance is used, such as a fire resistance ceiling, then the adhesive requirement can be ignored.

Where screws are used, they must be threaded over their full length. Screw spacing should be at 200 mm centres along the joists, preferably staggered in height. For joists greater in-depth than 400 mm, the second row of screws is required. The bottom row of screws must be far enough from the bottom edge of the joist, at least AS/NZS 1720.4 effective depth of char, to avoid charring of the wood reaching the screws during the fire exposure, as shown in Figure 2.3. The screw length must provide for at least 40 mm of penetration into the second joist.

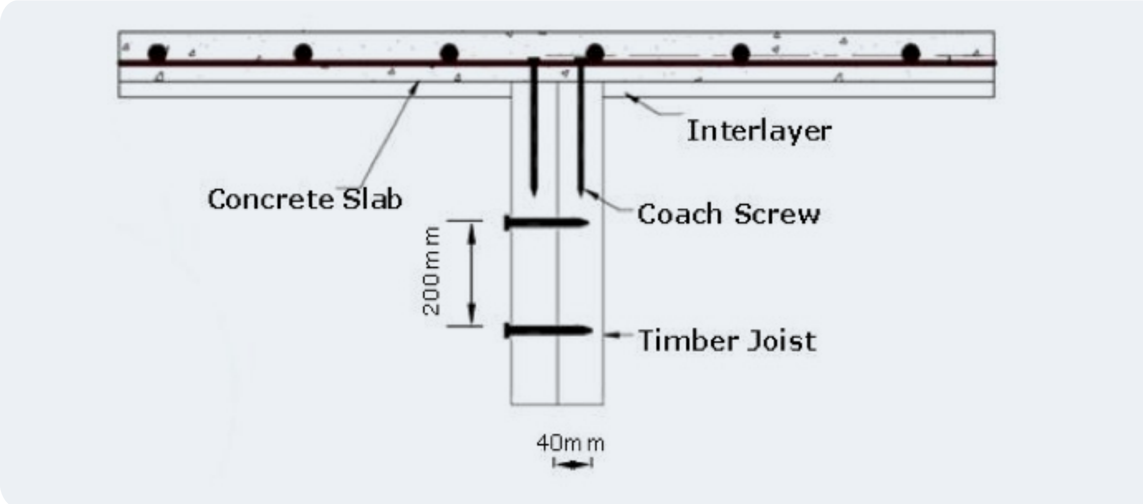

Figure 2.3: Screw arrangement for laminated joists.

2.4 Other Factors

The designer of the floor system must check the fire resistance of any horizontal or vertical penetrations made through the floor. Furthermore, consideration is requirements for surface spread of flame, on exposed wood ceilings. The NCC BCA[2] has specific limits on fire hazard of ceiling linings (Group Number), and limits materials due to the location within the building, occupancy or use and whether sprinklers are installed. It may be necessary to install sprinklers, limit the area of exposed wood surfaces or conduct a Performance Solution, to allow the use of an exposed ceiling in some building type sand locations. *Wood Solutions Technical Design Guide #19: Alternative Solution Fire Compliance, Internal Linings* steps through the process of a Performance Solution for coverings.

2.5 Verification by Test

A part of the Structural Timber Innovation Company research program, full-scale fire resistance tests were conducted at the Building Research Association of New Zealand (BRANZ)[4] laboratories on Timber Concrete composite floors.

3

Fire Safety of Timber Cassette Floors

Timber cassette floors are where the entire floor is made from timber. They consist of a timber joist (LVL or glulam) 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. *WoodSolutions Technical Design Guide #31 Timber Cassette Floors* provides a detail design procedure for this floor system.

3.1 Design for Stability - Timber Cassette Floors

In the case of exposed sections, the same principle of char providing the fire resistance as described in Section 1.4 is used. Calculsations and design process for structural Stability are given below for the floor, without any fire-rated ceiling or additional protection. The calculations are based on AS/NZS 1720.4.

Alternatively, a protective layer of timber can be used, and the thickness must be greater than the effective depth of char calculated from AS/NZS 1720.4. There is no prescriptive guidance on the fixing of the timber coverings to the element, so this must be undertaken as a performance solution.

3.1.1 Design Example – Stability

The calculation method is same as for cold conditions, with minor changes as summarised below:

- Design for strength in the ultimate limit state (ULS).
- Use the reduced load combinations specified by AS 1170.0, Section 4.2.4, i.e. G $+\psi_l$Q and any action arising from thermal effects. Note ψ_l for Residential, offices parking and retail $\psi_l = 0.4$, and for storage and other $\psi_l = 0.6$
- AS/NZS 1720.4 notes that deflection should be calculated from the effective residual section and that the deflection limits are the responsibility of the design engineer; however, consideration is required to ensure that deflection does not cause an opening to occur, leading to an integrity failure.
- *k* factors as per standard structural design but the load of duration shall be taken as 5 hours.
- ϕ factor as per standard structural design AS 1720.1.

Fire design example:

What width of a 400 mm deep LVL joist is required to provide 90 minutes fire resistance?

Assume 35 mm is required to support cassette post-fire.

Step One: Calculate the Notional Charring Rate (C in mm/min):

$$C = 0.4 + (280/D)^2 \qquad (3.1)$$

where

D = timber density at a moisture content of 12% (kg/m^3)

Use 550 kg/m^3 density, a common average density for LVL with a base timber species of radiata pine.

$\therefore C = 0.4 + (280/550)^2 = 0.66$ mm/min

Step Two: Calculate the effective depth of char to give 90 minutes of fire resistance:

Effective Depth of Charring (mm):

$$dc = C.t + 7.0 \qquad (3.2)$$

where

C = notional charring rate in mm/min, calculated above

t = period of time for the fire resistance required, i.e. FRL, in minutes 90 mins

$\therefore dc = 0.66 \times 90 + 7.0\text{ mm} = 66.3\text{ mm}$

The above calculates the char protection, but it needs to include residual timber section to carry tension and bending, assumed before of 35 mm, i.e. 67 + 67 + 35 = 169 mm (see Figure 3.1).

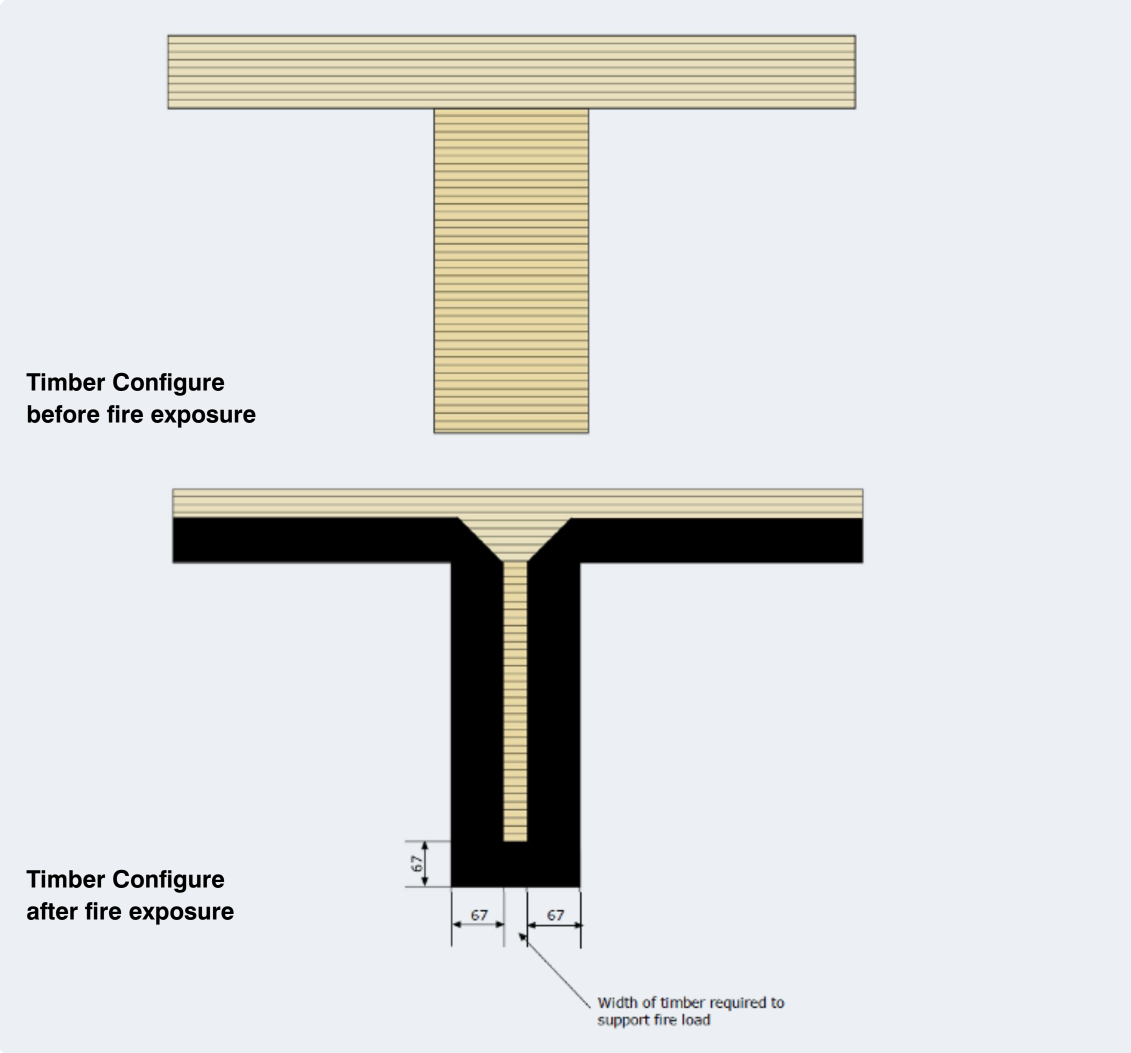

Figure 3.1: Comparison of Timber Cassette floor joists and Residual Section after 90 minutes of fire exposure.

Step Three: Determine the size of Joist

The designer must be mindful of the available timber sizes, as non-standard sizes are not readily available or are expensive. Commonly available LVL widths include 35 mm, 45 mm, 63 mm and 75 mm with depths from 90 mm to 450 mm. Therefore, the design selects the combination of standard LVL widths to make up a size greater than 169 mm.

For example: 63 + 45 + 63 = 171 > 169 mm

3.2 Design for Insulation - Timber Cassette Floors

For an all timber system, the only prescriptive method is to use the calculated insulation procedure from AS/NZS 1720.4, refer to Section 1.2.2. This method requires the timber providing the barrier to be as thick so that after the charring has occurred due to the fire exposure, plus the zero strength layer – 7.0 mm, there remains 23 mm.

Insulation panel thickness = Effective depth of char (including zero strength layer) + 23 mm (3.3)

3.2.1 Design Example – Insulation

AS/NZS 1720.4 requires the effective depth of char plus 23 mm will give the thickness of the floor panel required to provide Insulation for the fire exposure. Form the above design example, the effective depth of char was found in Section 3.1.1, being 67 mm for 90 minutes of fire resistance. Therefore Insulation can be met by adding 23 mm, i.e. 67 + 23 = 90 mm thick LVL floor panel (see Figure 3.4).

The above calculates the char protection, but it needs to include residual timber section to carry tension and bending, assumed before of 35 mm, i.e. 67 + 67 + 35 = 169 mm (see Figure 3.1).

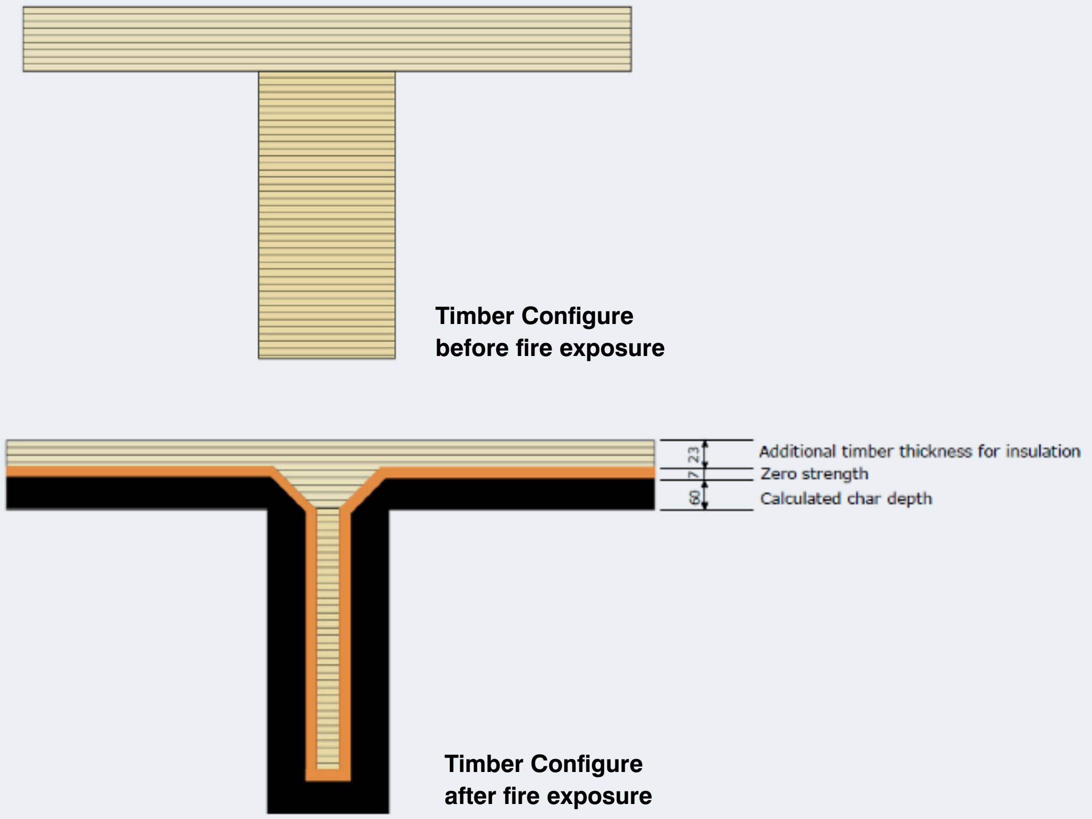

Figure 3.4: Insulation provided by the minimum thickness of the floor panel.

3.3 Other Factors

The designer of the floor system must check the fire resistance of any horizontal or vertical penetrations made through the floor. Also consider surface spread of flame on exposed wood ceilings. The NCC BCA[2] has specific limits on fire hazard of ceiling linings (Group Number), and limits materials due to the location within the building, occupancy or use and whether sprinklers are installed. It may be necessary to install sprinklers, limit the area of exposed wood surfaces or conduct a Performance Solution to allow the use of an exposed ceiling in some building types and locations. *Wood Solutions Technical Design Guide #19: Alternative Solution Fire Compliance, Internal Linings* steps through the process of a Performance solution for coverings.

3.4 Verification by Test

A part of the Structural Timber Innovation Company research program full-scale fire resistance tests was conducted at the Building Research Association of New Zealand (BRANZ)[4] laboratories on Timber Concrete composite floors.

4

Fire Safety of Post-Tensioned Timber Frames

The Post-Tensioned Timber Frames technology uses unbonded steel tendons in ducts in large timber box beams, frames or walls. It creates moment-resisting timber frames, the horizontal steel tendons also pass through the columns, providing moment resistance. In walls, the vertical tendons are anchored to the foundations. The system can also be used with draped tendons in long-span beams over a number of internal supports or with vertical post-tensioning in columns or solid timber shear walls with vertical ducts for the tendons.

Figure 4.1: Post tension Frame Massey University, Wellington, New Zealand. (Image credit: STIC)

4.1 Two Fire Safety Approaches for Post-Tensioning

There are two alternativess for fire design in post-tensioned buildings (see Table 4.1). A decision on which approach to use must be made early in the design process.

Option 1: Protect the post-tensioning

The first option is to protect all the steel components used in the post-tensioning system, i.e. tendons, anchorages, etc. This protection will prevent the temperature of the steel components increasing significantly for the duration of the required fire resistance time. Thus, structural fire safety design can incorporate the contribution of post-tensioning. The timber members must be sized to ensure there is sufficient residual timber to resist the applied loads and the post-tensioning forces for the duration of the fire.

Option 2: No post-tensioning for fire design

The second option relies solely on the residual timber members to resist the applied loads during a fire, so the steel post-tensioning components do not need to be protected. The members must be large enough for the residual timber section to resist the reduced loads in fire conditions. The residual section size is easily calculated by reducing the total section size by the char depth on surfaces exposed to fire.

This option requires careful consideration of the lateral load resistance of the whole building after the fire. This is especially important if the post-tensioning is an integral component of a lateral load resisting frame. For a multi-storey building, the fire will be contained within a single fire compartment at a single floor level, so the residual lateral load-resisting capacity of the structure on the floors unaffected by fire should be able to provide sufficient residual lateral load resistance for the whole building, but this needs to be checked carefully.

Table 4.1: Advantages and disadvantages of each option.

Option post-tensioning	Advantages	Disadvantages
1. Fire-protect all	• No reduction in post-tensioning during a fire • Full lateral-load resistance • during fire	• Specific detailing required • The additional cost to protect the post-tensioning
2. No fire protection of post-tensioning	• No additional design steps for fire design • Faster construction • Reduced cost of protection	• Does not utilise existing • Post-tensioning capacity

Figure 4.2 shows a fire resistance test of a post-tensioned timber beam, during unloading after 60 minutes of fire exposure. Also visible in the photograph are several post-tensioning anchorages that were tested at the same time.[5]

***Figure 4.2: Fire resistance test of a post-tensioned timber beam.**[5]*

4.2 Fire Design of Post-Tensioning, Brackets, Corbels and Dissipaters

Steel is a highly conductive material so any exposed steel components are sensitive to increases in temperature; this is particularly true for exposed steel used in the post-tensioning.

The yield strength of steel is significantly reduced when it exceeds a temperature of about 500°C, so the load-carrying capacity of a post-tensioning system is negligible if the tendons are exposed to fire. Steel tendons inside timber box members will not lose much strength in a fire because they are protected from elevated temperatures by the thermal insulating properties of the timber. The post-tensioning anchorages may need fire protection if they are within the building and likely to be exposed to fire.

4.2.1 Fire Resistance of Post-Tensioning

The design of fire resistance for post-tensioning will depend on the selected strategy. If the post-tensioning requires fire resistance, this must include:

- protection of tendons
- protection of anchorages.

Services or other structural elements, i.e. post-tensioned steel tendons, may be installed in this cavity and serve a particular function. The cavity temperature and the behaviour of these elements must be known to provide adequate information for the fire performance of the floor assembly as a whole.

For example, a post-tensioned steel tendon may be installed in the cavity to aid in resisting loads imposed on the floors, and increases in temperature can result in losses of tensioning force, which can have disastrous effects on these types of systems, as investigated by Spellman et al.[6]

If the tendons are hidden from view in a cavity within the beam or the wall, it is unlikely that additional fire protection is necessary because of the excellent insulating properties of the residual wood below the char layer (see Figure 4.3). However, simple calculations may be necessary to determine the maximum temperature reached inside the cavity.

Figure 4.3: Inside a cavity carrying post-tensioning rods, post-fire.[6]

The cavity surface temperature, and therefore the tendon temperature, can be estimated using an assumed temperature distribution for the timber underneath the char layer. Buchanan1 presents the following parabolic distribution:

$$T(x) = T_i + (T_p - T_i)(1 - \frac{x}{a_{heated}})^2 \quad (4.1)$$

where

$T(x)$ is the temperature of the timber at a distance beneath the char layer.

T_i is the initial or ambient temperature of the timber. Default 20°C.

T_p is the charring or pyrolysis temperature of the timber. Default 300°C.

a_{heated} is the thickness of the heat-affected layer. Default 40 mm.

This relationship is only valid for values of x between zero and a_{heated}. For a greater distance, the temperature should be taken as *Ti*. The value x can also be used as the remaining thickness of the timber. This will return an approximate value for the temperature on the internal surface of the timber cavity. If the thickness of the webs and flanges are different, use the thickness of the thinnest component, which will give a more conservative estimate.

If the post-tensioning is required to be protected from fire, it is essential to ensure that the anchorages also have the same fire resistance. In many cases, these anchorages will be protected by other structures or be outside the building envelope, but in other cases, it will be necessary to apply fire protection to the anchorage using protective material or intumescent paint. Some guidance is given by Spellman.[5] Early discussion with the fire engineer is essential to determine the most appropriate protection requirements corresponding to the fire design strategy for the building.

(a) Anchorages, corbels and dissipaters with no fire protection. Merritt building.

(b) Wall anchorages and UFPs with no fire protection. NMIT building.

Figure 4.4: Some fire-related details in other buildings.

4.2.2 Fire Resistance of Brackets and Corbels

The fire engineer must assess whether exposed steel brackets, joist hangers or corbels require fire resistance, depending on the fire safety strategy. If fire resistance is required, it can usually be provided by protective material or intumescent paint. A corbel protected with intumescent paint is shown in Figure 4.5b.

(a) Exposed tendons, no fire resistance.

(b) Steel corbel protected with intumescent paint.

Figure 4.5: Fire protection details at Massey Building, Wellington, New Zealand.

4.2.3 Fire Resistance of Dissipaters

External dissipaters that are part of the structural system will not normally need to be protected from fire, as shown in Figure 4.3a. If the dissipaters are contributing to the gravity load resistance during normal building operation, then special consideration will need to be given to the dissipaters to determine whether protection is required under reduced fire loading conditions.

4.2.4 Fire Protection of Service Holes

Any service holes through fire-resisting floors or walls must be detailed to prevent fire passing through the assembly, in order to retain at least the same fire resistance as the penetrated element. The effect of charring must be calculated for all exposed surfaces, including those at service holes.

4.3 Fire Design Example of Post-Tensioned Timber Frames

An LVL post-tension timber frame is checked In the following design example.

4.3.1 Design for Stability

The principle of char calculation described in Section 1.4 is used. The calculations are based on AS/NZS 1720.4. A protective layer of timber can be used and can be calculated from AS/NZS 1720.4.

4.3.1.1 Calculation Method

The calculation method is same as for cold conditions, with the following minor changes:

- Design for strength in the ultimate limit state (ULS).
- Use the reduced load combinations specified by AS 1170.0, Section 4.2.4. G + ψ_lQ and any action arising from thermal effects.
- ψ_l for residential, offices, parking and retail $\psi_l = 0.4$. For storage and other $\psi_l = 0.6$.
- AS/NZS 1720.4 notes that deflection should be calculated from the effective residual section and that the deflection limits are the responsibility of the design engineer; however, consideration is required to ensure that there is no failure in the concrete slab or that the deflection does not cause an opening, causing an integrity failure.
- *k* factors as per standard structural design but the load of duration is taken as 5 hours.
  - *k* factors ($k_1 = 1.0$, $k_4 = 1.0$, $k_6 = 1.0$, $k_9 = 1.0$, $k_{12} = 1.0$)
- ϕ factor as per standard structural design.
  - ϕ factor = 1.0

Design Example

Three cases are considered:

- Case 1 considers a simply supported, post-tensioned beam with a straight tendon.
- Case 2 considers a simply supported, post-tensioned beam with a draped tendon.
- Case 3 considers a post-tensioned beam in a frame with a draped tendon.

A design example with calculations is provided for Case 1. Two calculation processes are detailed in Case 1 to consider beams with protected and unprotected post-tensioning systems separately. Detailed design calculations are not provided for Cases 2 and 3; however, the additional considerations needed for the design of a beam with draped tendons or a beam within a frame are discussed. Cross-section of a simply supported post-tensioned beam is shown in Figure 4.4.

The following factors and properties are used in the design example.

- ϕ factor = 1.0
- *k* factors ($k_1 = 1.0$, $k_4 = 1.0$, $k_6 = 1.0$, $k_9 = 1.0$, $k_{12} = 1.0$)
- Characteristic stress values
  - $f_b = 48.0$ MPa
  - $f_s = 6.0$ MPa
  - $f_c = 45.0$ MPa

Case 1: Simply Supported, Post-Tensioned Beam with Straight Tendon

Figure 4.6 illustrates the configuration of the Post-Tension beam.

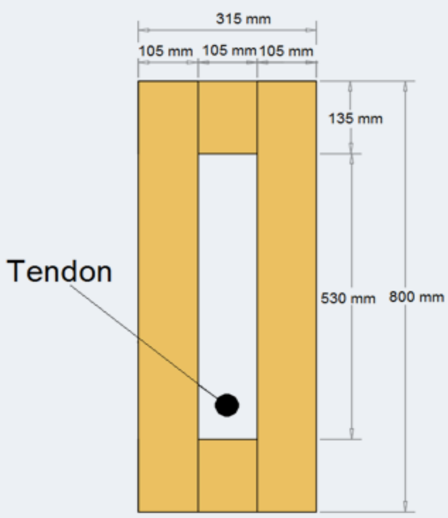

Figure 4.6: Cross-section of a simply supported post-tensioned beam.

The first stage of the fire design is to decide whether the post-tensioning system will be protected from exposure to fire. Calculations detailing the strength checks for beams using both unprotected and protected post-tensioning systems are shown below.

Calculation 1: Unprotected Post-Tensioning System

The FRL will first be checked for a beam where the post-tensioning system is unprotected. Therefore, the post-tensioning system will not contribute to the strength of the beam.

New section properties must be calculated for the residual beam section remaining after the expected duration of fire exposure.

Calculate notional Charring Rate (C in mm/min):

$$C = 0.4 + (280/D)^2 \qquad (4.2)$$

where

D = timber density at a moisture content of 12% (kg/m^3).

Use 550 kg/m^3 density as this is a common average density for LVL with a base timber species of radiate pine.

$\therefore C = 0.4 + (280/550)^2 = 0.66$ mm/min

Effective Depth of Charring (mm):

$$d_c = C.t + 7.0 \qquad (4.3)$$

where

C = notional charring rate in mm/min, calculated above.

t = period of time for the fire resistance required i.e. FRL, in minutes 90 mins.

$\therefore d_c = 0.66 \times 90 + 7.0$ mm $= 66.4$ mm

The char depth after 90 minutes fire exposure is: 67 mm.

Considering the three-sided exposure of the beam, with the top surface concealed from fire by the floor, only the bottom and side thicknesses are reduced by this depth. It is assumed corner rounding has a negligible effect on the performance of the beam so this has not been considered in these calculations. The residual section, with the original section shown by the dashed line, is shown in Figure 4.7.

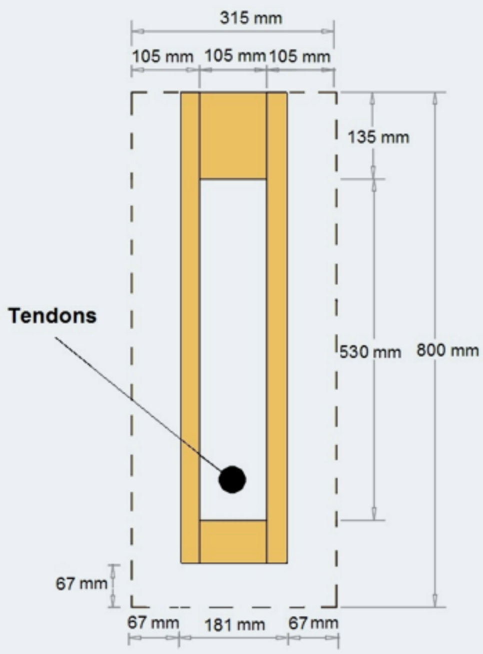

Figure 4.7: Residual cross-section of a simply-supported post-tensioned beam.

The residual section properties are then calculated as:

c =neutral axis depth from bottom=392.3 mm

$A = 77.02 \times 10^3$ mm^2

$Z = 11.60 \times 10^6$ mm^3

$Q_{centroid} = 8.35 \times 106$ mm^3

$I = 4.53 \times 10^9$ mm^4

where

c = neutral axis depth

A = area of timber member

Z = section modulus of timber member

$Q_{centroid}$ = first moment of area of timber member

The external load on the beam is calculated as G + ψ_lQ where the coefficient for the fire case is the same as was calculated for SLS 2, with ψ_l of 0.4. These loads, obtained from previous sections, are:

$$\mathrm{G} = 3.7\frac{\mathrm{kN}}{\mathrm{m}^2} \times 8.0\ \mathrm{m} = 29.6\ \frac{\mathrm{kN}}{\mathrm{m}} \tag{4.5}$$

$$\psi_l\mathrm{Q} = 0.4 \times 3.0\ \frac{\mathrm{kN}}{\mathrm{m}^2} \times 8.0\ \mathrm{m} = 9.6\ \frac{\mathrm{kN}}{\mathrm{m}} \tag{4.6}$$

The uniformly distributed load is:

$$q = \mathrm{G} + \psi_l\mathrm{Q} = 29.6 + 9.6 = 39.2\ \frac{\mathrm{kN}}{\mathrm{m}} \tag{4.7}$$

From this the maximum demands were calculated as:

$$M^* = \frac{qL_B^2}{8} = \frac{39.2 \times 8.6^2}{8} = 362\ \mathrm{kNm} \tag{4.8}$$

$$V^* = \frac{qL_B}{2} = \frac{39.2 \times 8.6^2}{2} = 169\ \mathrm{kNm} \tag{4.9}$$

As the beam chars, the neutral axis of the section changes as the beam is no longer symmetrical about all axes.

The bending strength of the section is:

$$f'_b = f_b \times \left(\frac{300}{h_b}\right)^{0.167} = 48.0 \times \left(\frac{300}{733}\right)^{0.167} = 41.35\ \mathrm{MPa} \tag{4.10}$$

$$\begin{aligned} M_d &= \emptyset k_1 k_4 k_6 k_9 k_{12} f'_b Z \\ &= 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 41.35 \times 11.60 = 479.7\ \mathrm{kNm} \end{aligned} \tag{4.11}$$

Checking the bending:

$$M_d \geq M^* \tag{4.12}$$

$480.6 \geq 362 \rightarrow \mathrm{OK}$

The shear strength of the section is:

$$A_s = 2t_w \times \frac{I}{Q_{centroid}} = 2 \times 38 \times \frac{4.53 \times 10^9}{8.35 \times 10^6} = 41.2 \ \times 10^3 \ \ mm^2 \tag{4.13}$$

$$V_d = \phi k_1 k_4 k_6 f_S^{'} A_s = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 6.0 \times 41.2 \times 10^3 = 247.2 \ \ kN \tag{4.14}$$

Checking the shear:

$$V_d \geq V^* \tag{4.15}$$

247.2 kN ≥ 169 kN → OK

For small residual sections, the maximum stresses in the tension and compression zones should be checked. Crushing failures may occur in the top flange, and failure may also occur when the bottom flange becomes too thin to carry the maximum tensile stresses. The Eurocode 5[7] design checks for tension and compression will be completed.

The mean compressive stress in the top flange is calculated to be:

$$\sigma_{f,c,d} = \frac{M^* \times (c - \frac{t_{f,c}}{2})}{I} = \frac{362.0 \times 10^6 \times (342.3 - \frac{135}{2})}{4.53 \times 10^9} = 22.0 \ \ MPa \tag{4.16}$$

The compression capacity parallel to grain is:

$$f_c^{'} = f_c \times (\frac{95}{h_b})^{0.167} = 45.0 \times (\frac{95}{733})^{0.167} = 32.0 \ \ MPa \tag{4.17}$$

$$\therefore \ k_c f_{c,0,d} = k_c \phi k_1 k_4 k_6 f_c^{'} = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 32.0 = 32.0 \ \ MPa \tag{4.18}$$

Checking the compression capacity against the mean compressive stress:

$$\sigma_{f,c,d} \leq k_c f_{c,0,d} \tag{4.19}$$

22.0 MPa ≤ 32.0 MPa → OK

The mean tensile stress in the bottom flange is:

$$\sigma_{f,t,d} = \frac{M^* \times (h_b - c - \frac{t_{f,t}}{2})}{I} = \frac{362.0 \times 10^6 \times (733.0 - 390.7 - \frac{68}{2})}{4.53 \times 10^9} = 24.6 \ \ MPa \tag{4.20}$$

The tension capacity parallel to grain is:

$$f_t^{'} = f_t \times (\frac{150}{h_b})^{0.167} = 30 \times (\frac{150}{733})^{0.167} = 23.0 \ \ MPa \tag{4.21}$$

$$f_{t,0,d} = \phi k_1 k_4 k_6 f_{t\backslash}^{'} = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 23.0 = 23.0 \ \ MPa \tag{4.22}$$

Checking the tension capacity against the mean tensile stress:

$$\sigma_{f,t,d} \leq f_{t,0,d} \tag{4.23}$$

24.6 MPa ≈ 23.0 MPa → OK

All strength checks are OK. Hence the beam can withstand 90 minutes of fire exposure without failure.

Calculation 2: Protected Post-tensioned System

If the post-tensioning system is fire protected, including all anchorages, the post-tensioning system can be considered in the strength calculations of the beam as shown below.

Second-order effects, including beam end rotations and bowing due to axial forces acting on the deflected member, have a minimal impact on the design and so have not been considered. Also, tendon relaxation due to heating has been ignored as it is assumed the post-tensioning system remains at ambient temperature as it is fully protected from fire exposure, but tendons relaxation overall will need to be included.

The residual section dimensions and properties after 90 minutes of fire exposure are the same as Calculation 1:

c = neutral axis depth from bottom = 390.7 mm

$A = 77.02 \times 10^3$ mm^2

$Z = 11.60 \times 10^6$ mm^3

$Q_{centroid} = 8.35 \times 10^6$ mm^3

$I = 4.53 \times 10^9$ mm^4

The external loads are the same as per Calculation 1. However, the axial compression force caused by the post-tensioning must also be considered. Again SLS2, with ψ_{tl} of 0.85 to account for 15% losses in post-tensioning force, is considered:

$$G = 3.7\frac{\text{kN}}{\text{m}^2} \times 8.0\ \text{m} = 29.6\ \frac{\text{kN}}{\text{m}} \tag{4.24}$$

$$\psi_1 Q = 0.4 \times \frac{3.0\ \text{kN}}{\text{m}^2} \times 8.0\ \text{m} = 9.6\ \text{kN/m} \tag{4.25}$$

$$\psi_{tl}\,\text{PT} = 0.85 \times 1100\ \text{kN} = 935\ \text{kN} \tag{4.26}$$

The uniformly distributed load is:

$$q = \text{G} + \psi_l \text{Q} = 29.6 + 9.6 = 39.2\ \text{kN/m} \tag{4.27}$$

From this, the maximum demands were calculated as:

$$M^* = \frac{\text{q}L_\text{B}^2}{8} = \frac{39.2 \times 8.6^2}{8} = 362.0\ \text{kNm} \tag{4.28}$$

$$V^* = \frac{\text{q}L_\text{B}}{2} = \frac{39.2 \times 8.6}{2} = 169.0\ \text{kN} \tag{4.29}$$

$$N^* = \psi_{\text{tl}}\text{PT} = 935\ \text{kN} \tag{4.30}$$

As the beam chars, the neutral axis of the section changes as the beam is no longer symmetrical about all axes.

The eccentricity of the tendon will create a moment that needs to be considered in the flexural demand on the beam. The change in neutral axis depth, due to loss of cross-section, increases the eccentricity to the post-tensioning.

So the eccentricity is:

$$e = e_0 + \Delta\bar{y} - \delta \quad (4.31)$$

where

e = eccentricity between centroid and tendon of residual section

e_0 = initial eccentricity = 280 mm

$\Delta\bar{y}$ = distance of centroid of original section to centroid of residual section = 60 mm

δ = elastic deflection of residual section at mid-span due to reduced gravity loads

$(\delta_{G+\psi_l Q} = \dfrac{5ql^4}{384EI} = 56.7\ \text{mm})$

minus the hogging deflection of the residual section at midspan due to post-tensioning system

$(\delta_{\psi_{tl}PT} = \dfrac{e \cdot \psi_{tl}PT \cdot l^2}{8EI} = 54.0\ \text{mm})$:

$$\delta = \delta_{G+\psi_1 Q} - \delta_{\psi_{t1}PT} = \frac{5ql^4}{384EI} - \frac{e \cdot \psi_{tl}PT \cdot l^2}{8EI} = 56.7 - 54.0 = 2.7\ \text{MM} \quad (4.32)$$

∴ e = 280+ 60 - (56.7 - 54.0) = 337.3 mm

Note that determining $\delta_{\psi_{tl}PT}$ equires an iterative calculation to be performed.

The post-tensioning moment is:

$$M_{PT} = e \times \psi_{lt}PT = 0.3373 \times 935 = 315.4\ \text{kNm} \quad (4.33)$$

So the new moment demand is:

$$M^* = M_q - M_{PT} = 362 - 315.4 = 46.6\ \text{kNm} \quad (4.34)$$

The bending strength of the section is:

$$f_b' = f_b \times (\frac{300}{h_b})^{0.167} = 48.0 \times (\frac{300}{733})^{0.167} = 41.3\ \text{MPa} \quad (4.35)$$

$$M_d = \phi k_1 k_4 k_6 k_9 k_{12} f_b' Z = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 41.35 \times 11.6 = 479.7\ \text{kNm} \quad (4.36)$$

Checking the bending:

$$M_d \geq M^* \quad (4.37)$$

479.7 kNm ≥ 70 kNm → OK

The shear strength of the section is:

$$A_s = 2t_w \times \frac{I}{Q_{centroid}} = 2 \times 38 \times \frac{4.53 \times 10^9}{8.35 \times 10^6} = 41.2\ \times 10^3\ \text{mm}^2 \quad (4.38)$$

$$V_d = \phi k_1 k_4 k_6 f_S' A_s = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 6.0 \times 41.2 \times 10^3 = 247.2\ \text{kN} \quad (4.39)$$

Checking the shear:

$$V_d \geq V^* \quad (4.40)$$

247.2 kN ≥ 169 kN → OK

The compression strength of the section is:

$$N_d = \varnothing k_1 k_4 k_6 k_9 k_{12} f_c' A = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 45 \times 77.02 \times 10^3 = 3465.9\ \text{kN} \quad (4.41)$$

Checking the compression:

$$N_d \geq N^* \tag{4.42}$$

3476.3 ≥ 935 → OK

Now checking the combined action for bending and compression:

$$\frac{N^*}{N_\mathrm{d}} + \frac{M^*}{M_\mathrm{d}} \leq 1.0 \tag{4.43}$$

where

N^* = design compression force

N_d = design compression capacity

M^* = design bending moment

M_d = wind direction multiplier for the 8 cardinal directions

$$\frac{935}{3465.9} + \frac{70}{479.7} = 0.41 \leq 1.0 \rightarrow \mathrm{OK}$$

Again for small residual sections, the maximum stresses in the tension and compression zones should be checked. This is of paramount importance with high levels of post-tensioning, as crushing failures may occur in the top flanges due to the extra-axial load imposed on the member. Failure may also occur when the bottom flanges become too thin to carry the maximum tensile stresses. The Eurocode 57 design checks for tension and compression will be completed.

The mean compressive stress in the top flange is calculated to be:

$$\sigma_{\mathrm{f,c,d}} = \frac{M^* \cdot (c - t_{\mathrm{f,c}}/2)}{I} + \frac{\Psi_{1\mathrm{t}} PT}{A} = \frac{70 \times 10^6 \times (390.7 - 135/2)}{4.53 \times 10^9} + \frac{935 \times 10^3}{77.02 \times 10^3}$$
$$= 20.1\ \mathrm{MPa}\ \text{(negative compression)} \tag{4.44}$$

The compression capacity parallel to grain is:

$$k_\mathrm{c} f_{\mathrm{c,0,d}} = k_\mathrm{c} \phi k_1 k_4 k_6 f_\mathrm{c}' = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 32.0 = 32.0\ \ \mathrm{MPa} \tag{4.45}$$

Now, checking the compression capacity against the mean compressive stress:

$$\sigma_{\mathrm{f,c,d}} \leq k_\mathrm{c} f_{\mathrm{c,0,d}} \tag{4.46}$$

17.1 MPa ≤ 32.0 MPa → OK

The mean tensile stress in the bottom flange is:

$$\sigma_{f,c,d} = \frac{M^*.(c - t_{f,c}/2)}{I} + \frac{\Psi_{1\mathrm{t}} PT}{A} = \frac{70 \times 10^6 \times (390.7 - 135/2)}{4.53 \times 10^9} + \frac{935 \times 10^3}{77.02 \times 10^3}$$
$$= 7.5\ \mathrm{MPa}\ \ (\text{Positive Tension}) \tag{4.47}$$

The tension capacity parallel to grain is:

$$f_\mathrm{t}' = f_\mathrm{t} \times (\frac{150}{h_\mathrm{b}})^{0.167} = 30 \times (\frac{150}{733})^{0.167} = 23.0\ \ \mathrm{MPa} \tag{4.48}$$

Now apply factor

$$f_{\mathrm{t,0,d}} = \phi k_1 k_4 k_6 f_\mathrm{t}' = 1.0 \times 1.0 \times 1.0 \times 1.0 \times 23.0 = 23.0\ \ \mathrm{MPa} \tag{4.49}$$

Now checking the tension capacity against the mean tensile stress:

$$\sigma_{\mathrm{f,t,d}} \leq f_{\mathrm{t,0,d}} \tag{4.50}$$

7.5 MPa ≤ 23.0 MPa → OK

All strength checks are okay. Hence the beam can withstand 90 minutes of fire exposure without failure.

Back calculations can be made to determine the actual fire resistance of the beam by deriving the section modulus or area required in each strength check and then substituting the charring equation into these. This gives a cubic formulation which, when solved, will give the expected fire resistance time of the member. The above equations can also be put into an excel spreadsheet, and a solver used to find the expected fire resistance.

Case 2: Simply Supported, Post-Tensioned Beam with Draped Tendon

The design of a beam with a draped tendon will follow a similar methodology as to that detailed above; the difference in the design of a beam with a draped tendon will arise when calculating the shear demand.

The draped tendon will result in a vertical component of the post-tensioning force. This will have the effect of decreasing the shear demand on the beam in the areas where the tendon is draped. As such, additional calculations can be completed to include the effect of the draped tendon profile, potentially producing a more efficient design. However, the effect of the draped tendon can be conservatively ignored and the calculations detailed for Case 1 can be followed.

The calculation to determine the eccentricity of the tendon will also change for a draped tendon. As the tendon is fixed in place by deviators, the tendon will deflect with the beam. Therefore the deflection component of the equation to calculate the eccentricity, $\boldsymbol{\delta}$, can be ignored, i.e.:

$$e = e_0 + \Delta\bar{y} \tag{4.51}$$

If the post-tensioning system is not protected there will be no difference in the design of a beam with a draped tendon compared to a straight tendon.

Case 3: Post-Tensioned Beam in a Frame with a Draped Tendon

It has been assumed in the calculations above that the beam is simply supported. As such, the negative moments at the ends of the beams, which would decrease the demand on the beam, are ignored. Therefore, if the beam is shown to be satisfactory for the simply supported case, it will also be satisfactory if the beam is part of a frame system. If the beam is not simply supported, additional calculations can be completed to include the effect of the frame system, which could produce a more efficient design.

4.4 Other Factors

The designer of the beam system must check the fire resistance of any horizontal or vertical penetrations through the beam.

Checks should be made on the size and residual bearing area of corbels and timber seatings, such that adequate bearing remains after a fire event.

The lateral Stability of the entire frame needs to be considered. If a fire event was to completely weaken or destroy a primary beam on one level of the structure, the entire structure should have enough remaining strength to withstand lateral actions.

If it is determined that the post-tensioning will be protected from fire exposure, the entire post-tensioning system, including the anchorages, must also be adequately protected to ensure the tendon is not heated during a fire.

4.5 Verification by Test

The design principles above have been checked by full-scale fire resistance tests at the laboratories of BRANZ (Building Research Association of New Zealand)[6], sponsored by the Structural Timber Innovation Company Ltd.

5

Construction Site Fire Safety

Construction site fire safety is important. Fires in timber buildings are infrequent, but the known examples in Canada, USA and UK have generally occurred in light timber-framed building and while the building is under construction. This is due to the light timber frame not having its protective covering applied until much later in the construction sequence. Massive timber construction is less susceptible than lightweight systems due to the char resistance large timber elements have.

WoodSolutions Technical Design Guide #20: *Fire Precautions During Construction of Large Buildings* provides information to help designers and organisations with responsibilities for fire safety on a construction site to reduce the risk of fire.

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WoodSolutions Technical Design Guides

Guide #3: *Timber-framed Construction for Commercial Buildings Class 5, 6, 9a & 9b*

Guide #17: *Alternative Solution Fire Compliance, Timber Structures.*

Guide #19: *Alternative Solution Fire Compliance, Internal Linings*

Guide #20: *Fire Precautions During Construction of Large Buildings*

Guide #30: *Timber Concrete Composite Floors*

Guide #31: *Timber Cassette Floors*

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AS 3600 *Concrete Structures*, 2009.

A

Appendix A – Notation

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

a_{heated}	thickness of the heat-affected layer. Default 40 mm
A	area of timber member
A_s	shear area of member
c	neutral axis depth
C	notional charring rate in mm/min
D	timber density at a moisture content of 12% in kg/m^3
d_e	calculated effective depth of charring in mm
t	period of time, in minutes
e	eccentricity of post-tensioning
e_0	initial eccentricity of post-tensioning
f_b	bending strength
f'_b	design bending strength (including size factor)
f_c	compression strength of timber member parallel to grain
f'_c	characteristic compression strength parallel to grain
$f_{c,0,d}$	design compression strength parallel to grain (Eurocode 5)
f_s	shear strength of beam
f_t	tension strength of timber member parallel to grain
f'_t	design tension strength (including size factor)
$f_{t,0,d}$	design tension strength parallel to grain (Eurocode 5)
G	permanent action (dead load)
h_b	height of beam
I	moment of inertia for timber member
k_1	duration of load factor specified in AS 1720.1
k_4	partial seasoning factor specified in AS 1720.1
k_6	temperature factor specified in AS 1720.1
k_9	strength sharing between parallel members factor specified in AS 1720.1
k_{12}	stability factor specified in AS 1720.1
k_c	factor that takes into account lateral instability (Eurocode 5)
M_{pt}	bending moment due to post-tensioning
M_q	bending moment due to distributed load
M^*	design bending moment
M_d	wind direction multiplier for the 8 cardinal directions
N_d	design compression capacity
N	design compression force
PT	post-tensioning load

Q	imposed action (live load)
$Q_{centroid}$	first moment of area of timber member
$t_{f,c}$	thickness of compressive flange
t_w	thickness of web
$T(x)$	temperature of the timber at a distance beneath the char layer
T_i	initial or ambient temperature of the timber; default 20°C
T_p	charring or pyrolysis temperature of the timber; default 300°C
V^*	design shear force
δ_d	design shear capacity
$\delta_{\mathbf{G+\psi_1 Q}}$	section modulus of timber member
$\delta_{\mathbf{\psi_{t1} PT}}$	elastic deflection of residual section at mid-span
–	elastic deflection of residual section at mid-span due to reduced gravity loads
δ	elastic deflection of residual section at mid-span due to post-tensioning
Δy	distance of centroid of original section to centroid of residual section
$\sigma_{f,c,d}$	mean flange design compressive stress (Eurocode 5)
$\sigma_{f,t,d}$	mean flange design tensile stress (Eurocode 5)
ϕ	strength reduction factor
ψ_{tl}	factor for tendon force under long-term SLS load
ψ_l	combination factor for long-term imposed action