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Wood Construction Systems

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 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.

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Wood Research and Development

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Introduction

This guide is designed to assist practicing structural engineers and other building design professionals to confidently develop conceptual structural designs for timber-rich buildings and structures. As many design professionals working on commercial-scale structures are typically skilled with concrete and steel structures, this guide takes a whole-of-process approach to the selection of timber-rich structural systems.

It explores available structural systems, connections and material options, and design approaches before providing a concise reference on the technical aspects of wood and timber products. It references other WoodSolutions Design guides and sources that can assist professionals developing their concept designs into detailed structural solutions.

The structural design process

The design of the structural system for a building or other structure usually involves:

1. Design brief formation
2. Information search
3. Conceptual design of suitable structural systems, elements and connections
4. Detailed design
5. Solution documentation.

Design brief formation

Design brief formation involves the designer defining the problems to be addressed in the process. This includes identifying the criteria, performance requirements and constraints that will guide the design and limit possible solutions. These criteria include the structure's functional requirements, site impacts restrictions and the budget.

Information search

Information search is where the designer gathers information specific to the design problem. This includes research into the availability of materials, the site's environmental conditions, regulatory and market requirements, and design loads.

Conceptual design of the structural system

Conceptual design is the process of generating and assessing a range of alternative solutions for the structure that may satisfy its performance requirements. This is an iterative process that includes:

- selecting a number of options for each category of structural element
- establishing basic alternative frameworks for the structure using these options
- determining load paths to carry vertical and lateral loads to the foundations.

In this process, the designer develops and tests options for the solution's structural systems and considers how they can be realised on the site. This involves making realistic estimates of element sizes and solution practicality, often with limited information. Design intent, regulatory requirements, cost, material availability and fabrication capacity are all considered.

Major members may be roughly sized, but not checked for all load combinations or effects. For example, a truss may only be sized on strength with the serviceability limit state not being considered. Important connections may be sketched if it appears that they will contribute substantially to the project's cost. Rough cost estimates are made.

Decisions made in this phase will have a major impact on the cost and ease of construction as well as the structure's visual and architectural character. This phase demands creativity and flexibility from the designer and collaboration with other design professionals active in the project and, occasionally, key material suppliers and installation contractors. Interaction between the engineer and the architect is essential to ensure the implications of option selection are clearly understood and incorporated into the full project solution. Alternative solutions generated during the conceptual design phase are examined in more detail in the design development and structural design phases where changes to the preliminary size of critical structural sections are common.

Detailed design

Detailed design is the process of taking a conceptual design and detailing its components so that they are ready for documentation and implementation. The structure is analysed to determine the load in each member and then the members are designed to provide a satisfactory response under the various design loads.

Design documentation

Design documentation is the preparation of the package of information that communicates the design to the fabricator and builder. This package includes design drawings, specifications, construction drawings and construction notes.

Guide structure

This guide is structured to support practicing structural engineers and other building design professionals develop timber-rich buildings and structures during the conceptual design stage. It covers most regular options for their design and construction. As there are numerous types of wood products and numerous variations on structural solutions; not all options are included.

Many requirements influence the selection of one structural system or material over another. The guide is arranged to support designers address the major performance requirements for an engineered timber structure including its: structural performance; moisture content management; fire resistance; system durability, acoustic separation; thermal performance; and environmental performance.

Design is an iterative and non-linear process. It often requires designers to consider components and key design factors in sequences that change as problems and opportunities in alternative solutions present themselves. To accommodate this, each guide section is presented as a general standalone reference. Figure 1 shows the relationship between stages in the design process and the guide's sections. Table 1 summarises each section's content.

Stage of the design process	Section
Design brief formulation	**Section 1** - Why Wood
Information search	**Section 7** - Material Basics
	Section 8 - Material Properties
	Section 9 - Performance Requirements
	Section 10 - Aspects of *AS1720*
	Section 11 - Worked Examples
	Section 12 - Glossary
Design brief formulation	**Section 2** - System Options
	Section 3 - Connection Options
	Section 4 - Element Options
	Section 5 - Construction Options
	Section 6 - Design Approaches
Detailed design and design documentation	**Other WoodSolutions guides**

Figure 1: The relationship of guide sections to the structural design process

Table 1: Description of guide sections

Section	Key aspects described
1: Why wood	The significant environmental, design and construction edge that timber and wood products, combined with sustainable forestry practices, have over the use of alternative building materials in the design of environmentally responsible buildings.
2: System options	Regular options for the spanning, support and lateral restraint systems used in a project's structure. It includes material options for each system with indicative span tables and span to depth ratios.
3: Connection options	Major options for making reliable structural connections in timber elements in the workshop and on site. Connections, fasteners and connectors are discussed.
4: Element options	Major material options for timber-rich building elements, their applications and standards, and likely means of supply
5: Construction options	Construction process options for timber-rich buildings and structures, including options for element prefabrications.
6: Design approaches	Considerations and design approaches required to address the performance requirement of timber-rich buildings and structures.
7: Material basics	Key differences in practice between wood and other materials, and between different species or types of wood.
8: Material properties	Properties of timber and wood products in relation to the major performance requirements for buildings and structures, such as their weight, strength, fire resistance, and durability.
9: Performance requirements	Regulatory requirements established through the National Construction Code (NCC) and its referenced standards, and the fit-for-purpose requirements that relate to timber and wood products used in building established under the Australian consumer law.
10: Aspects of AS 1720	AS 1720 sets out the limit states design methods for the timber's use in structures.
11: Worked examples	Two worked examples, one seven storeys and the other ten storeys, provide an applicable, step-by-step approach to designing timber components in multi-storey timber buildings.
12: Glossary	Definition of key terms.

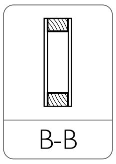
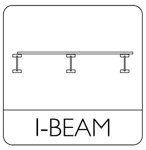
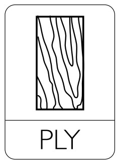
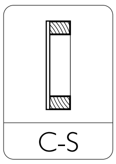
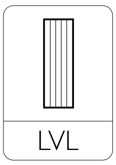
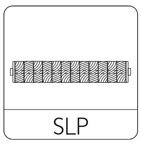
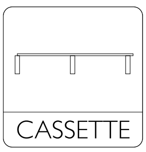
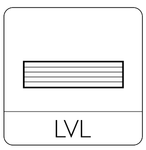
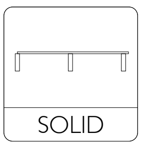
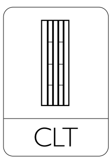
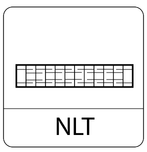
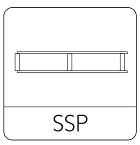
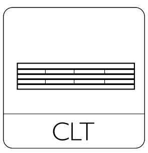
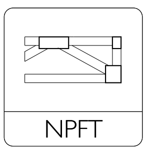
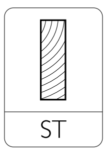
Acronyms and icons

Wood is a highly workable material that can be converted into a broad suite of timber or wood products used directly as building elements or combined with additional wood or other products to form more complex elements or components. To simplify the discussion, this guide uses acronyms and icons to represent readily available options. These are listed below.

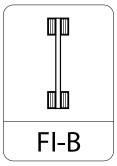
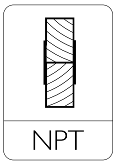
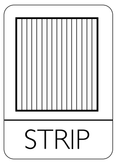
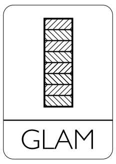
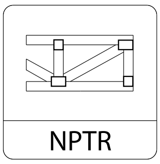
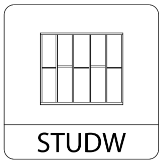
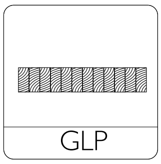
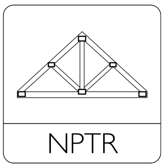
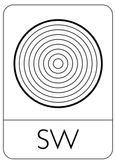
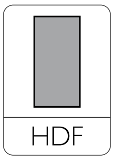
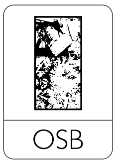
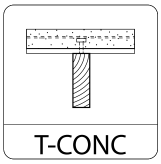
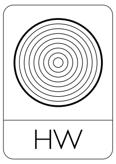
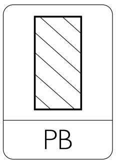
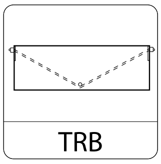
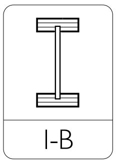
General acronyms

Acronym	Term	Acronym	Term
ABCB	Australian Building Codes Board	**FSP**	Fibre saturation point
AFCS	Australian Forest Certification Scheme	**GBCA**	Green Building Council of Australia
BAL	Bushfire attack level	**kN**	kiloNewtons
BIM	Building information management	**MC**	Moisture content
CoC	Chain-of-custody	**MGP**	Machine grade pine
dl	Design life	**MOE**	Modulus of Elasticity
DTS	Deemed-to-Satisfy Provisions	**NABERS**	National Australian Built Environment Rating System
EMC	Equilibrium moisture content	**NCC**	National Construction Code
EWPs	Engineered wood products	**PEFC**	Program for Endorsement of Forest Certification
FLW	Floor load width	**SED**	Small end diameter
FRL	Fire Resistance Level	**SOU**	Sole-occupancy units
FSC	Forest Stewardship Council		

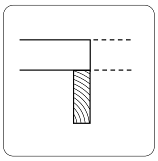
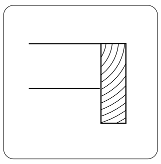
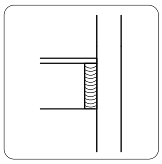
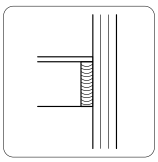
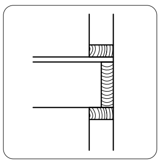
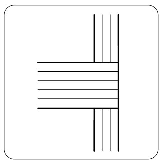
Material icons and acronyms

Icon	Description	Icon	Description	Icon	Description
	**B-B** Box beams		**I-Beam** Cassette floor panel: I-beam joists		**PLY** Plywood.
	**C-S** C-section plywood web beam		**LVL** Laminated veneer lumber		**SLP** Stress laminated timber panel
	**Cassette** Cassette floor panel		**LVL** Laminated veneer lumber panel		**Solid** Cassette floor panel: solid sawn timber
	**CLT** Cross laminated timber: vertical panel		**NLT** Nail laminated timber		**SSP** Stressed skin panel
	**CLT** Cross laminated timber: horizontal panel		**NPFT** Nailplate floor trusses		**ST** Sawn timber

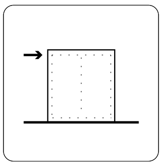
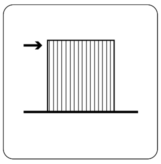
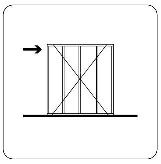
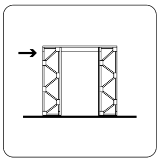
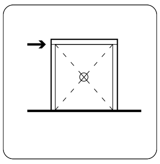
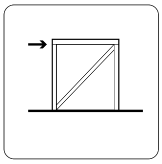
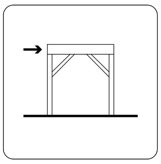
Material icons and acronyms (continued)

Icon	Description	Icon	Description	Icon	Description
	**FI-B** Site fabricated I-beam		**NPT** Nailplate timber		**STRIP** Strip flooring
	**GLAM** Glue laminated timber		**NPTR** Nailplate trusses: parallel chord		**STUDW** Stud wall frame
	**GLP** Glue laminated timber panel		**NPTR** Nailplate trusses: triangular		**SW** Softwood round or pole
	**HDF** High density fibre board		**OSB** Oriented strand board		**TRB** Trussed beam
	**HW** Hardwood round or pole		**PB** Particle board		**ST** Sawn timber
	**I-B** I-beams				

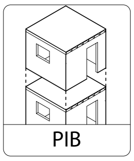
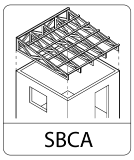
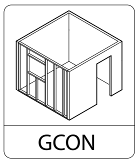
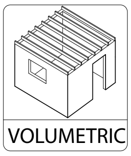
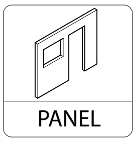
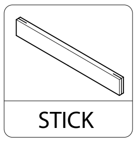
Construction arrangement icons

Icon	Description	Icon	Description	Icon	Description
	Joist or purlin on beam		Joist or purlin fitted between beams		Balloon construction: frame
	Balloon construction: massive timber		Platform construction: frame		Platform construction: massive timber

Bracing systems icons

Icon	Description	Icon	Description	Icon	Description
	Bracing panel: sheet on frame		Bracing panel: massive timber		Bracing panel: strap on frame
	Braced panel: trussed panels		Bracing panel: steel cross brace		Bracing panel: single brace
	Bracing panel: rigid connection				

Construction and prefabrication icons

Icon	Description	Icon	Description	Icon	Description
	**PIB** Prefabrication-intensive building		**SBCA** Site-based component assembly		**GCON** General construction
	**Volumetric** Prefabrication of volumetric modules		**Panel** Prefabrication of panel elements		**Stick** Prefabrication of lineal elements

Key points for effective timber design

Structural design of timber requires a different approach

Designing with timber is different to designing with many other major building materials. Items commonly considered late in the design process with other materials can be important early in the design of a timber solution. For example, connection design and element deflection commonly dictate member sizes rather than strength. Also, the material and properties readily or economically available to use in a solution can often vary significantly by region.

Know the products and interact with the industry supply chain

Timber is a naturally variable and highly workable material. The basic material properties and product ranges can vary between species and suppliers. Also, as innovation and technological developments change the industry, building practices and fabrication capabilities can vary from place to place. To avoid difficulties later, check designs and specifications with local fabricators and suppliers early in the design process.

It is rarely about strength

Strength is often not the governing factor in timber design. Regularly, element stiffness, deflection, vibration, connection detailing, fire, or other considerations dictate design. As timber structures are lightweight, stability and uplift forces may be critical. When checking strength, ensure all the load and material factors are considered, and check different load cases commensurate with different load durations.

Think about connections from the start

There are many ways to connect timber elements. The chosen connection method may dictate structural framing and element sizes as fasteners have specific edge distances and spacing requirement. Connections in timber are also best formed by bearing of one element on another. Joints that rely on fixing withdrawal, or tension and compression applied across grain are best avoided. Also, as timber is relatively soft, many small fasteners rather than fewer large ones. Bringing many members together into a single point or plane may require specific attention.

Keep things spanning one way, parallel frames work best

Timber elements are typically individual straight elements assembled to create frames. In forming the frames, it is preferable to allow members to be as continuous as possible. Often, this means offsetting elements in multiple layers or planes. For example, column to beam connections can work well when a pair of beams are offset to either side of a single supporting column. This allows the elements to continue through the joint. Joists work well if allowed to continue over bearers, rather than framed into the side of them.

Check your deflections

In beams, shear deflection and creep that can make long-term deflections require specific consideration. The size of elements in portal frames is often dictated by limits on lateral sway deflections.

Understand your design criteria

It pays to be clear on non-structural design criteria such as fire, exposure, service life, termite protection early in the design process. These can dictate material selection and detailing.

Apply a hierarchy of design approaches to deal with water.

Water ingress can significantly reduce a timber structure's service life. Use roofs, cladding and membranes to keep water off the wood where possible. If it isn't possible, detail the surfaces with chamfers to shed water and connections with drainage to allow the water to run off and ventilation to allow them to dry out.

If water exposure is expected, specify the appropriate material for the application. Protect it with sealers and coatings. As a last line of defence, treat it.

Don't specify solely by strength grade

Australia is blessed with many different engineered timber products suitable for construction but their material properties, cost and availability can vary widely. Careful species and product selection and specification is necessary, particularly for exposed timber. There are benefits to specifying the species and assumed material properties, including the joint group and durability class. Also, make sure the chosen product is available on the market!

Check for fire

When exposed, timber beams and columns can be designed to char. Provided members are sufficiently wide, it is often found that the fire load case does not govern member capacity. However, it is critical that steel connectors are fire protected to ensure the structural integrity of joints is maintained.

Detail for fabrication and erection

Timber offers great opportunities for prefabrication and rapid construction. The best frame and connection designs are developed with erection in mind. Both require careful design and detailing to achieve success and cost savings.

Seek out resources and advice from industry

The industry offers great resources. Suppliers are very willing to give guidance and support. Use them!

1 Why Wood?

Timber and wood products are renewable and readily accessible materials that provide the base for a versatile range of structural, architectural and envelope solutions for buildings of all types. Wood is a highly workable material, naturally available in logs of a range of species. Processing converts this varied resource into a broad suite of timber or wood products that can be used directly as building elements or combined with additional wood or other products to form more complex building elements or components. Wood is a unique major building material. Readily available and light weight for its strength, timber has been used as a common and trusted building material for centuries.

This section describes the significant environmental and construction edge that timber and wood products have over alternative building materials and fuels.

Timber-rich building solutions present clear benefits to similar solutions assembled from alternative materials, especially non-renewable ones produced through energy-intensive production processes. Timber-rich solutions generally have lower environmental impacts and are easier to build than alternative solutions while delivering similar or enhanced performance. Timber and wood products are natural and renewable, and net stores of atmospheric carbon. In addition to being lighter than alternative solutions, timber-rich approaches are generally more versatile in design and easier to change. Wood components are also highly workable and can be cut and assembled with precision. They are clean to handle and work.

1.1 Environmental advantages

Timber and wood products used in design and construction can support sustainable development and a low carbon economy. All material production generates environmental impacts. Timber and wood products, combined with sustainable forestry practices, provide designers and their clients with a significant environmental edge on the use of alternative building materials and fuels as they are natural, renewable and store atmospheric carbon in the forest and in the building. In contrast to alternative building materials, they also require relatively little energy to make.

Timber is natural

Timber and wood products are natural and not manufactured through a transformative production process. They require additional care in design but can deliver nature-connected design solutions to clients and building users. Nature-connected design brings the physiological and psychological benefits of an association between nature and the built environment. For timber-rich structures and interiors, these include improvements to a person's emotional state and level of self-expression, reduced blood pressure, heart rate and stress levels, and increased occupant comfort as timber's humidity moderation improves air quality.

Timber is renewable

Wood grows in trees, and to grow trees, only a seed, soil, air, water, sunlight and time are needed. Different tree species can be grown to provide timber for particular applications. Parts of the stem not suitable for making building products can be used as fuel for production equipment. In a well-managed process, trees can be grown, harvested and regrown on a continuous basis and provide renewable material and fuels to make and operate the built environment.

Timber stores atmospheric carbon

Timber and wood products and timber-rich building solutions can contribute to a low carbon economy. The wood in forest trees and the timber and wood products in buildings store atmospheric carbon. Also, the use of timber and wood products avoids the use of materials whose production consumes large amounts of fossil fuels.

Forests store atmospheric carbon. About 50% of the wood's dry weight in a tree is atmospheric carbon. This carbon is retained in the timber or wood products recovered from harvested trees and it is stored for at least the product's service life in a building. The volume stored is considerable. A cubic metre of seasoned softwood sequesters about 250 kg of atmospheric carbon or about 0.9 tonnes of carbon dioxide. If enough wood is used in a design and high carbon dioxide emitting materials avoided, the finished building can be a net carbon store.

Using timber avoids the use of alternative materials and their associated carbon emissions. The embodied energy of materials such as steel and cement can be very high when compared to timber and other wood products. As much of this energy is generated from burning fossil fuels, it produces significant carbon dioxide emissions. In contrast, much of the energy used in timber production is generated from burning renewable wood residues. Fossil fuel use is limited. Timber requires less than 5% of the fossil fuel use and less than 6% of the energy required to create a structural element when compared on a load capacity basis to steel or reinforced concrete respectively.

The benefits from avoiding the carbon emissions of alternative materials can equal or exceed the volumes of carbon stored in the wood products of timber-rich solutions.

1.2 Construction advantages

Timber and wood products are diverse, and provide the basis for light, versatile, and creative design solutions.

Timber is light

With nominal densities around 550 kg/m^3 for many softwoods and 700 kg/m^3 for many hardwoods, timber is very light compared to alternative solutions, such as concrete and steel, with densities of about 2,400 kg/m^3 and 7,700 kg/m^3 respectively.

As timber also has a high strength to weight ratio, timber-rich building solutions weigh less than alternative solutions and can touch the ground lightly. This simplifies solutions for substandard or delicate sites or for extensions to existing buildings. Timber solutions can be devised and installed to minimise site disturbance. Heavy foundations and the site disturbance they generate are not needed. This reduces the potential impacts to local biodiversity and environments during construction and use. Similarly, the use of timber in building extensions reduces the additional loads that have to be borne by the existing structures and allows a large extension to be supported. As a light material, timber components are easier to assemble, move and vary. This reduces construction and transport impacts.

Timber has a versatile product suite and versatile construction options

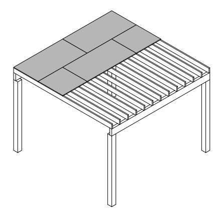
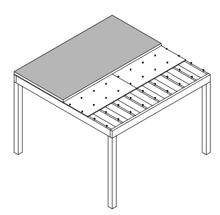
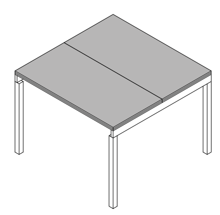
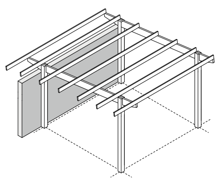
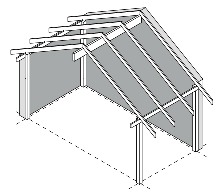
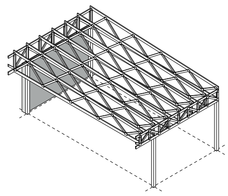
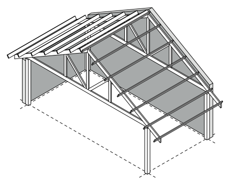
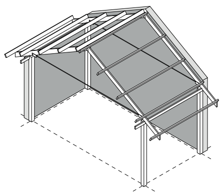
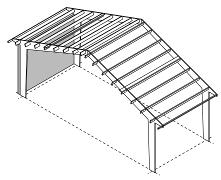
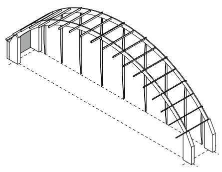
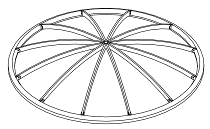
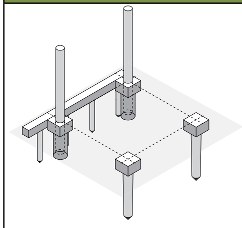
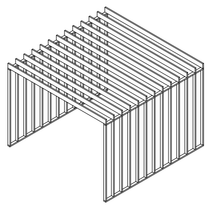
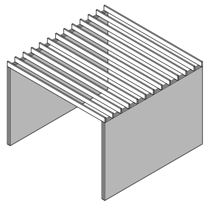
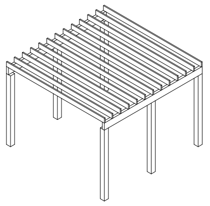
Timber contributes to design flexibility. A versatile product suite is available and these products can be configured into a broad set of construction options. These products are available off the shelf or they can be shaped or assembled to suit a particular project. This is discussed in Section 4.

Timber is clean and easy to work with accuracy

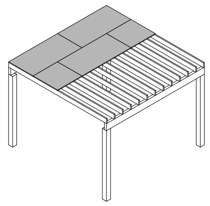
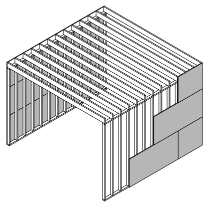
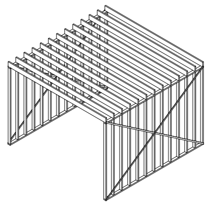
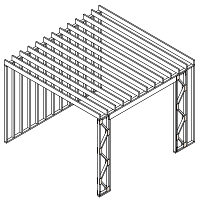
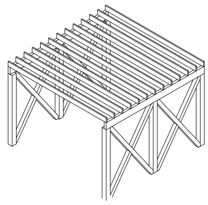
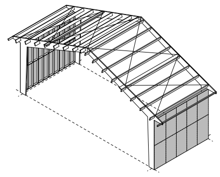
Timber elements are relatively light and clean to handle, and easy to work at either large or small scale. Timber is relatively soft and can be readily worked and shaped with simple hand and power tools or more sophisticated router and computer-controlled robots. It is easy to join. A child can nail pieces of wood together to make a frame. While improved quality and precision come with experience and better tools, the benefits of timber's ease of use remain as solutions become more complex. Timber elements can be joined or prefabricated easily and cleanly.

2 Systems Options

Systems legend

Section 2.2 Spanning systems: floors					
	2.2.1 Floors: Joist and flooring deck		2.2.2 Floors: Massive timber		2.2.3 Floors: Concrete and timber composites
Section 2.3 Spanning systems: roofs					
	2.3.1 Roofs: Beams and purlins		2.3.2 Roofs: Beams and rafters		2.3.3 Roofs: Light nailplate trusses
	2.3.4 Roofs: Heavy trusses		2.3.5 Roofs: Tied frames		2.3.6 Roofs: Portal frames
	2.3.7 Roofs: Arches		2.3.8 Roofs: Dome		
Section 2.5 Support systems: foundations, walls and columns					
	2.5.1 Support systems: Foundations and piles		2.5.2 Support systems: Timber stud frames		2.5.3 Support systems: Massive timber
	2.5.4 Support systems: Columns				

Systems legend (continued)

Section 2.6 Lateral resistance systems					
	2.6.1 Lateral resistance systems: Floor and roof diaphragms		2.6.2 Lateral resistance systems: Wall diaphragms		2.6.3 Lateral resistance systems: Strap braced panels
	2.6.4 Lateral resistance systems: Truss panels		2.6.5 Lateral resistance systems: Timber or steel rod bracing		2.6.6 Lateral resistance systems: Moment-resisting frame

The diversity of available timber-rich elements provides the designer with a broad menu of options to deploy in the alternative structural solutions developed during a project's conceptual design stage.

To assist in generating these alternatives, this section describes regular options for the three component types generally found in structural solutions. These are:

- spanning systems that bridge horizontal distances to carry vertical loads to the support system
- support systems that receive loads from the spanning systems and transfer them to the ground
- lateral resistance systems that resist horizontal forces from the wind, earthquakes or similar sources.

When combined, selections for these systems can generate one of several alternative solutions for review and assessment. The best solutions combine different wood elements and structural forms in a way that economically satisfies the building's functional and regulatory requirements. Opportunities also exist to combine timber elements with other materials in each solution.

The span tables and span-to-depth ratios provided in this section allow designers to estimate during the conceptual design phase the products, grades and sizes of material necessary to resist basic dead and live loads in common structural elements. Larger elements may be needed to accommodate project-specific force combinations or the spacing requirements for fasteners and connections. This guide only covers regularly used structural systems. Information about more specialist solutions can be found in other resources.

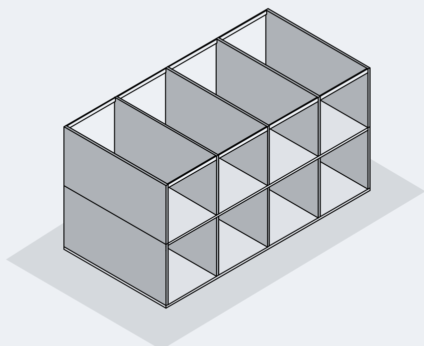
Building typologies

In generating and assessing alternative solutions, the designer can consider the project's similarity to existing building typologies, such as those described below. These typologies have evolved to address projects with similar performance characteristics but can be combined and adapted for new applications.

Closed frame

A closed frame system typically has walls that enclose discrete functional areas and provide lines of support at regular intervals. Floor spans are relatively short at 4-5 metres.

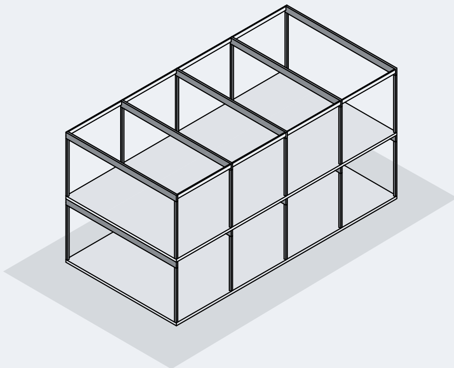
This system is particularly relevant to Class 2, 3 and 4 residential buildings and Class 9 health care buildings. In these building classes, the spanning, support and lateral resistance systems may have to comply with stringent regulatory requirements for fire and sound separation. These can influence the applied load and element configuration.



Open frame

An open frame system typically has columns, beams and floors acting as a frame structure to provide open and flexible functional areas. Columns are regularly spaced on a grid whose spacing is based on efficient beams and floor spans. Most timber floor plate systems are one-way spanning systems.

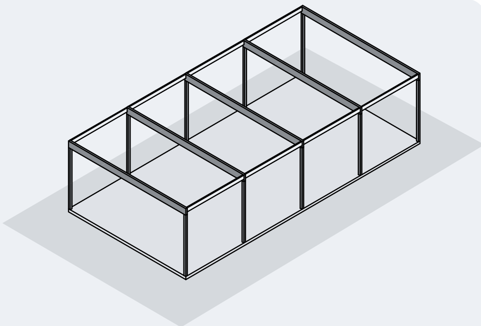
This system is particularly relevant to Class 5 and 6 buildings and Class 9 educational buildings. In these buildings, the columns, beams and floor plates must comply with stringent regulatory requirements for fire resistance while market requirements may dictate sound separation requirements. These can influence the applied load and element arrangement.



Single level system

A single level structural system typically has a roof structure spanning between wall or columns to provide an open and flexible functional area. The roof elements may be exposed or concealed.

This system is particularly relevant to single storey Class 1 domestic, 6 commercial, 7 storage, 8 factory, 9 sports and public assembly buildings and the upper floors of other building classes.



2.1 Spanning systems

Spanning systems include the floor and roof structures that bridge horizontal distances to carry vertical loads to the support systems. In this guide, the spanning system includes the joists, rafters, purlins and panel products that make up the spanning surface and the beams that support them. The walls or columns that support the spanning system are discussed in Section 2.4.

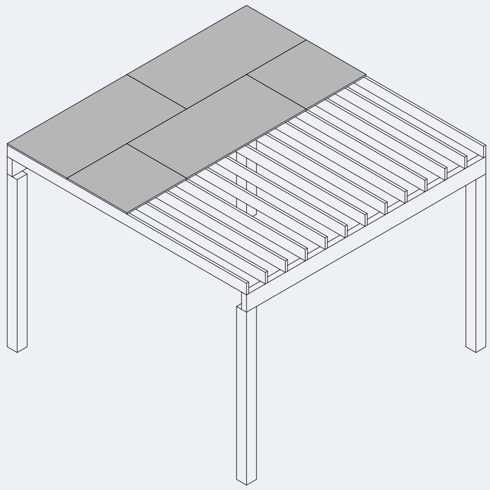
The spanning systems often have a greater impact on the cost and architectural character of a structure than the support or lateral systems. Generally, the critical factors to consider when selecting a spanning system for the span length, the applied loads and the required stiffness. Other important factors include the availability of materials, ease of construction, the potential for prefabrication, and the desired appearance of the final solution.

2.2 Spanning systems: floors

Floor systems characteristically require greater stiffness, carry higher loads, and have to be more compact than roof systems. As a result, they generally have shorter spans. Timber floor systems may combine:

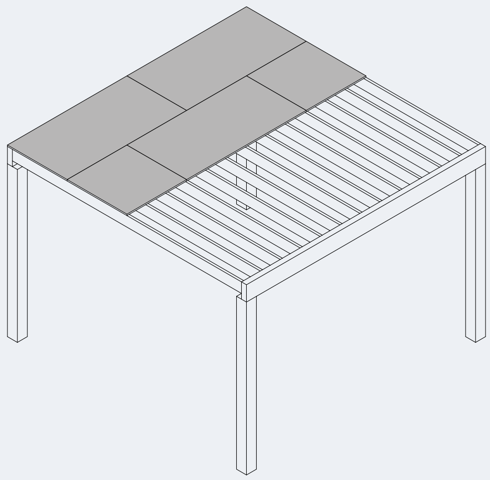
- joists and beams on columns or joists supported on walls and covered with a flooring material
- massive timber plates and beams on columns or plates supported on walls.

Joist and beam spacing is an essential consideration in the selection of the primary spanning system. Spacing effects the loads on the members and their ability to share load, which consequently is the driving force in determining the supporting element sizes. Similarly, the loads that fire resistance and sound separation systems apply to the floor solution should also be considered. Different beam and joist spacings should be tested to establish the most cost efficient option.



Joists onto or continuous over beams

Joist span range: Up to 8 m



Joists fitted between beams

Joist span to depth ratio: 17 to 1

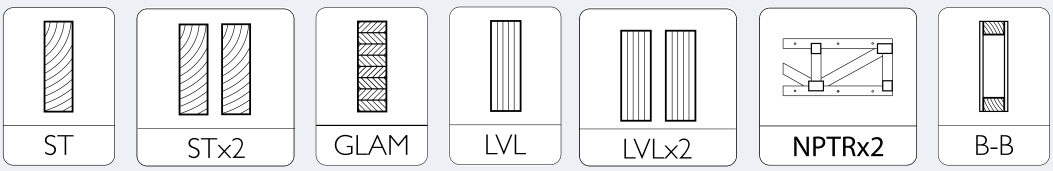
Description

The most common timber floor systems consist of closely spaced joists supported by beams or walls and covered with strip floor boards or sheet material such as plywood. The joists share the imposed load between adjacent elements. Load sharing allows a decrease in element sizes while maintaining a strong, lightweight system. They can be installed on site or prefabricated into cassettes or stressed skin panels and lifted into place.

Joist systems are best suited to relatively light, distributed loads. Where large concentrated loads occur, it is often more efficient to reinforce the system with additional spanning members or support members directly below the load. Joist spacing of 300 to 600 mm is common. With a wider member spacing, the load-sharing effects are minimal so the floor would be considered a beam system rather than a joist system. At spans over 5 or 6 metres, joist systems are likely to be controlled by the serviceability limit state (deflection and vibration) rather than the strength limit state. In these cases, engineered products are likely to be more efficient.

Beams differ from joists in that they are more widely spaced and there is little, if any, load sharing between adjacent members. Depending on the locations, beams may carry distributed loads, concentrated loads, or a combination of both. They must be designed to have adequate strength to carry these loads and enough stiffness to prevent excessive deflection or vibration. Beam design will generally be governed by either bending strength or deflection. The design of timber beams with rectangular cross sections is rarely governed by shear strength unless they have very short spans or large loads, especially point loads concentrated near the supports. More complex cross sections, such as box-beams, are more likely to be shear-governed. Beams can also be cambered to accommodate deflection during construction loading.

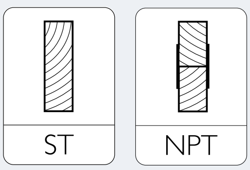
Regular beam options



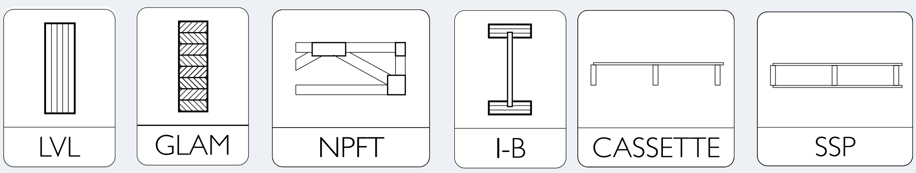
Notes
Glulam is available in wide sections.

Regular joist options by span and configuration

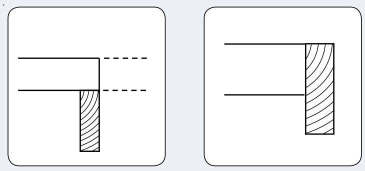
<5 m



5 m+



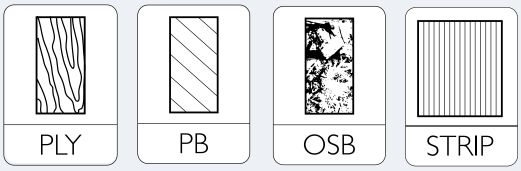
Assembly options



Assembly notes

When joists hang from the beam's sides, metal joist hangers typically secure them. The hangers can also assist resist racking.

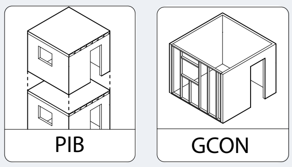
Regular flooring options



Notes

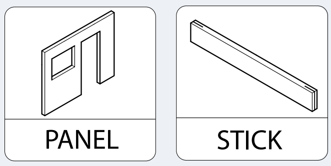
Panel systems can resist shear stress and serve as part of the lateral resistance system. Plywood can generally handle heavier concentrated loads than other floor panels.

Regular construction and prefabrication options



Construction notes

Beams and floors can be fully prefabricated or built from generic products.



Prefabrication notes

Floor cassettes and stressed skin panels can be fully prefabricated while joists and beams can be prepared and optimised.

Indicative span tables – Beams supporting floor joists

Table 2: Indicative span – Beams supporting floor load – FLW 3.6

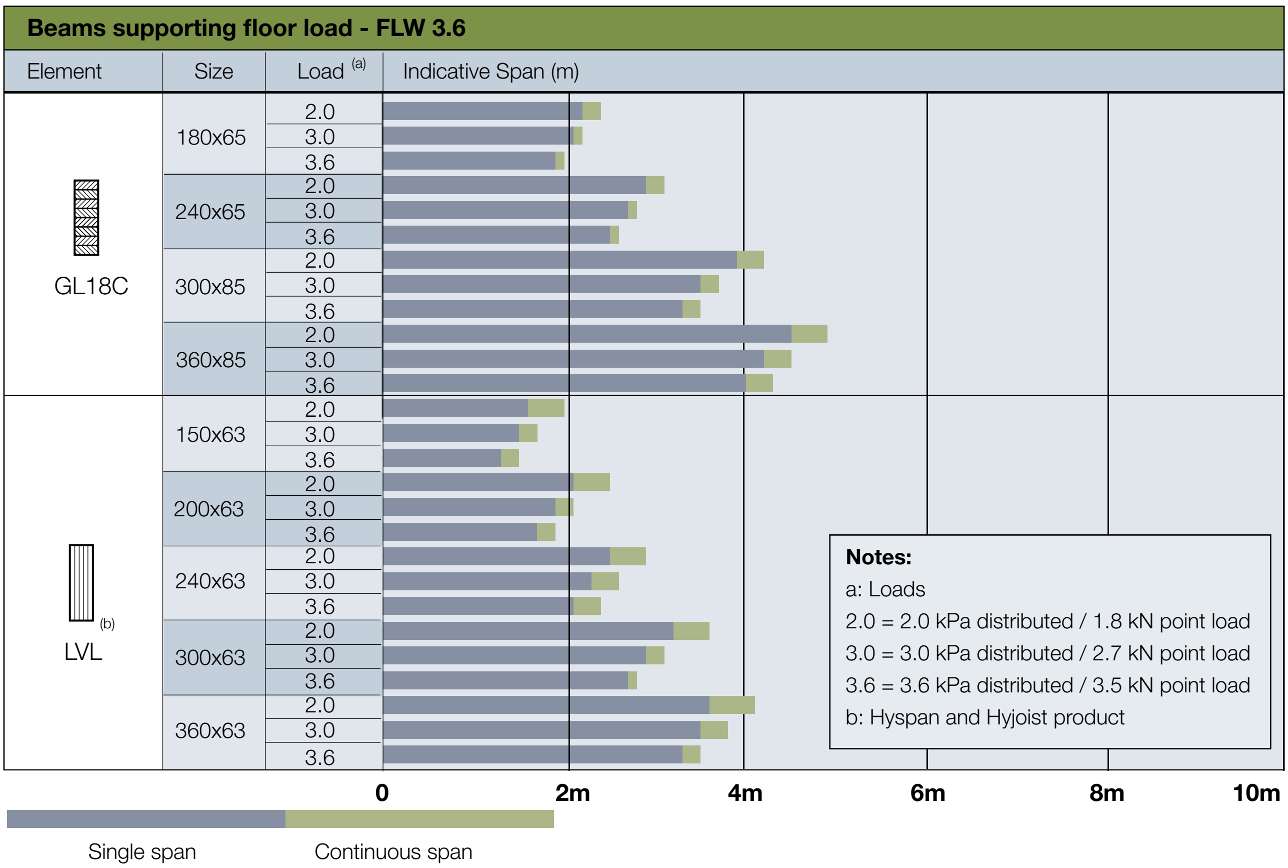


Table 3: Indicative span – Beams supporting floor load – FLW 4.5

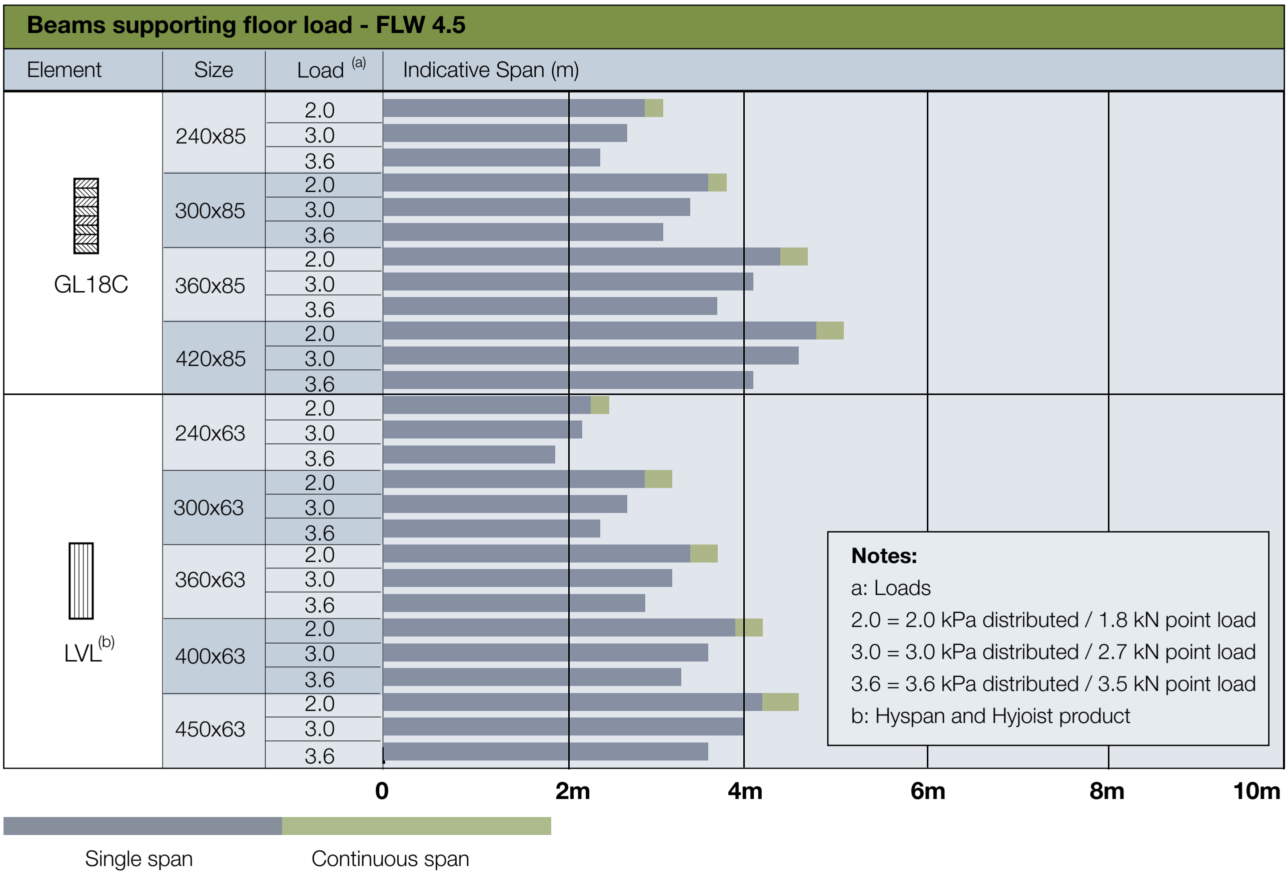


Table 4: Indicative span – Beams supporting floor load – FLW 5.4

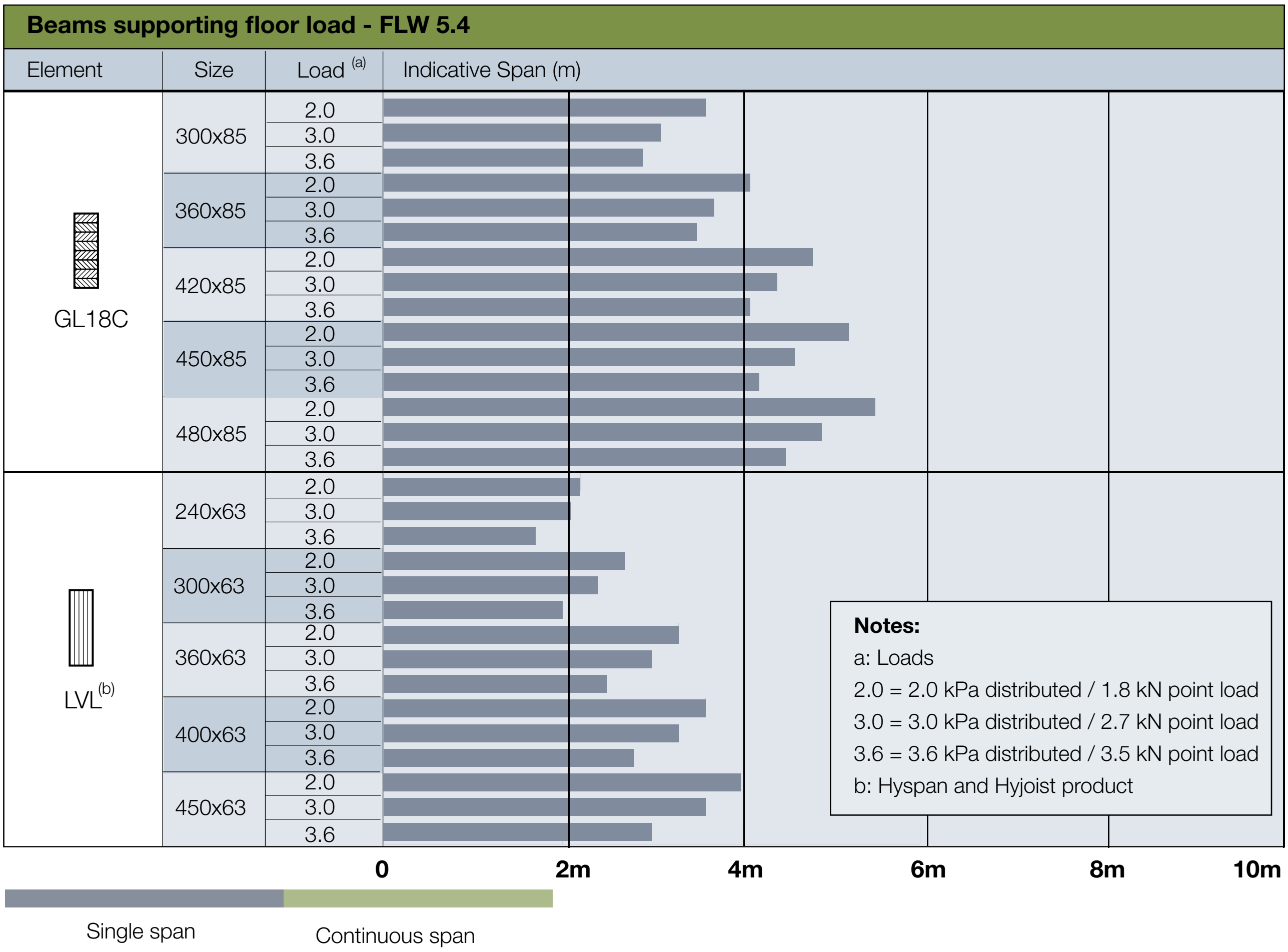


Table 5: Indicative span – Floor joist: 150 nominal depth at 450 centres

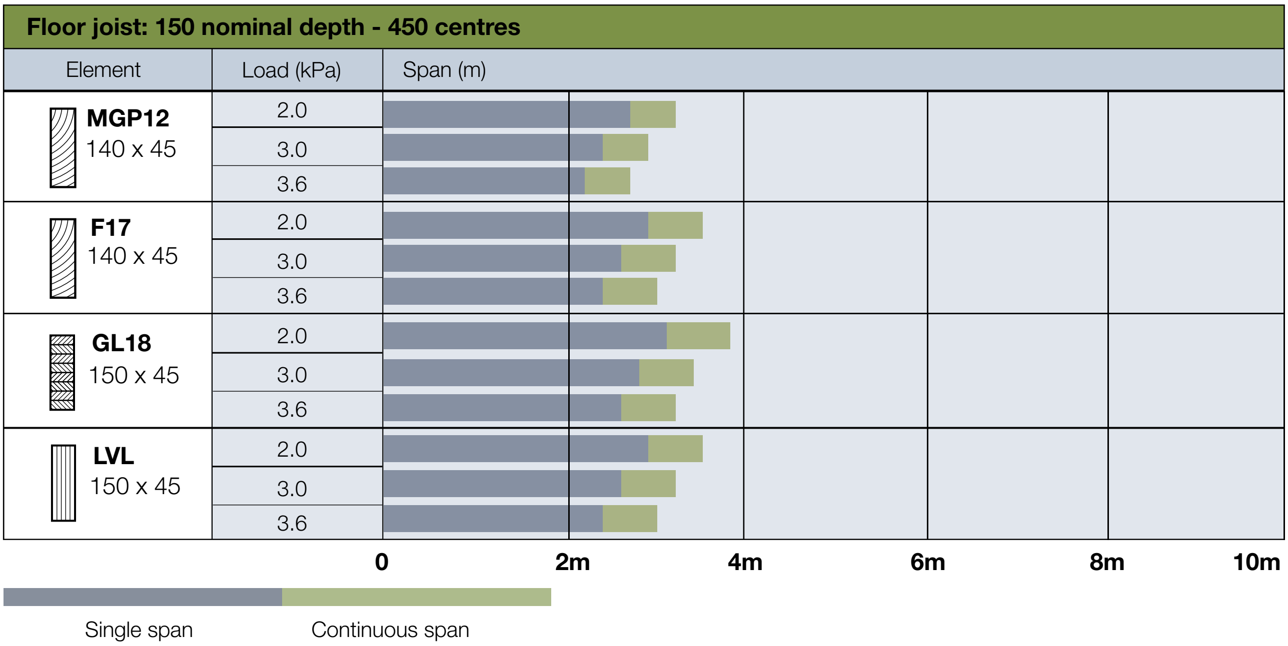


Table 6: Indicative span – Floor joist: 200 nominal depth at 450 centres

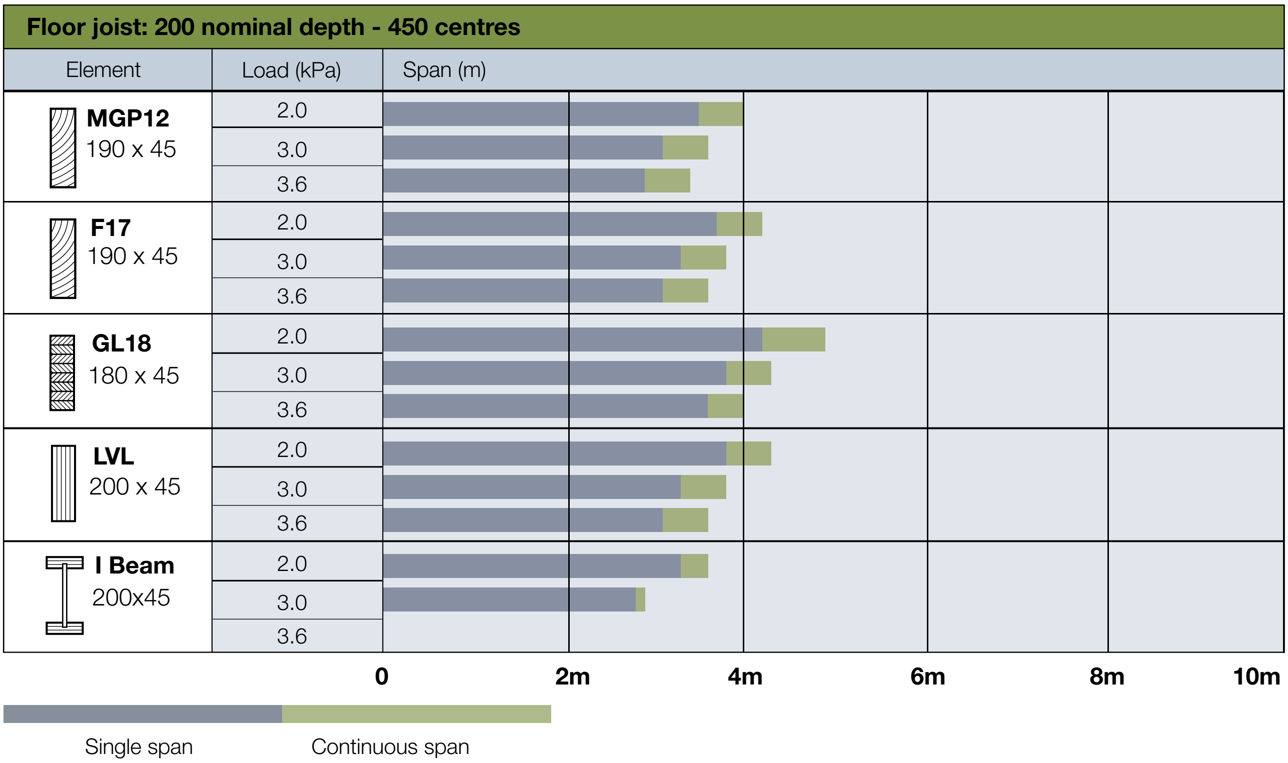


Table 7: Indicative span – Floor joist: 240 nominal depth at 450 centres

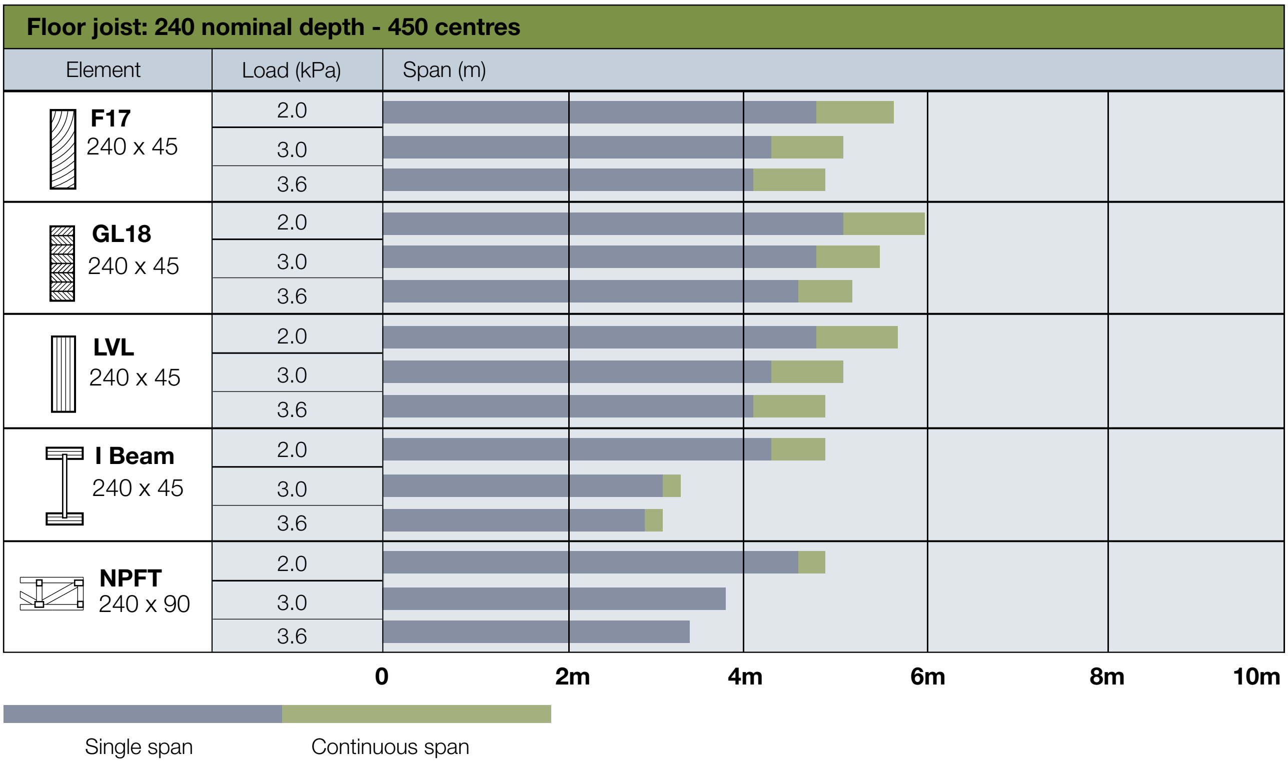


Table 8: Indicative span – Floor joist: 300 nominal depth at 450 centres

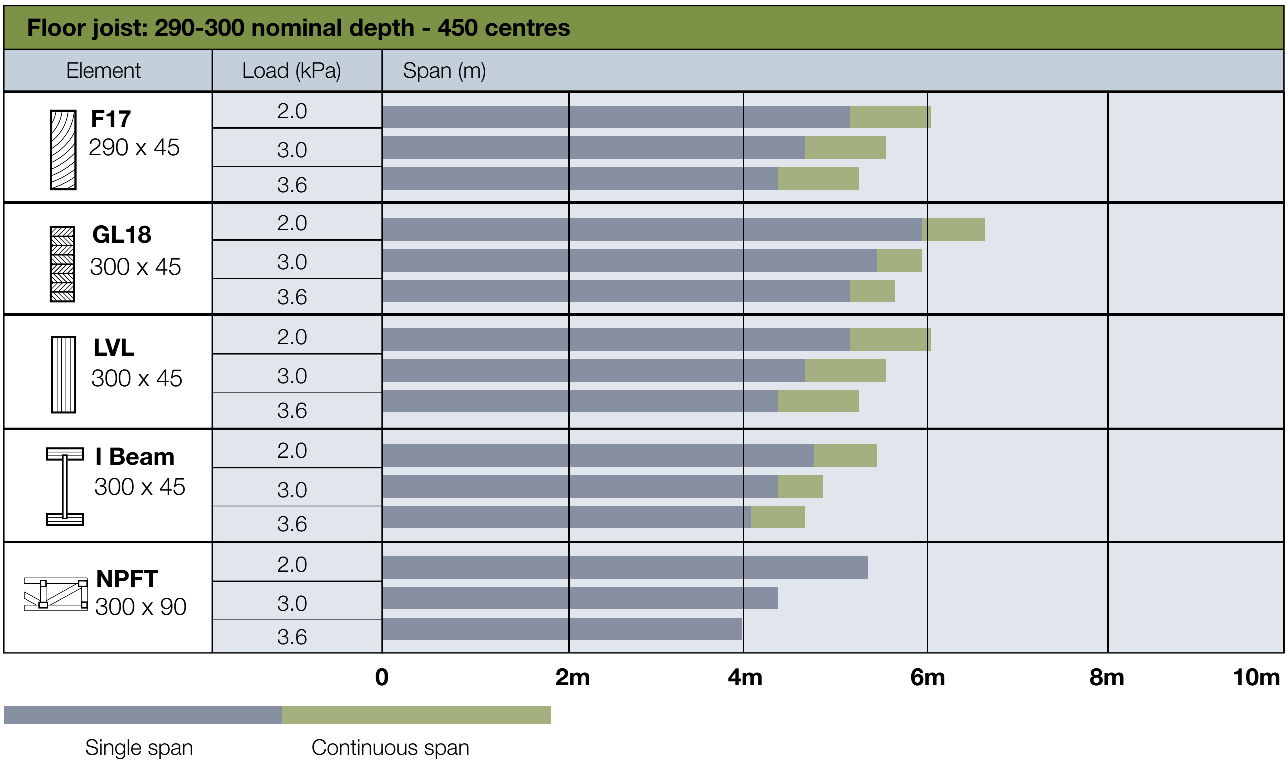


Table 9: Indicative span – Floor joist: 360 nominal depth at 450 centres

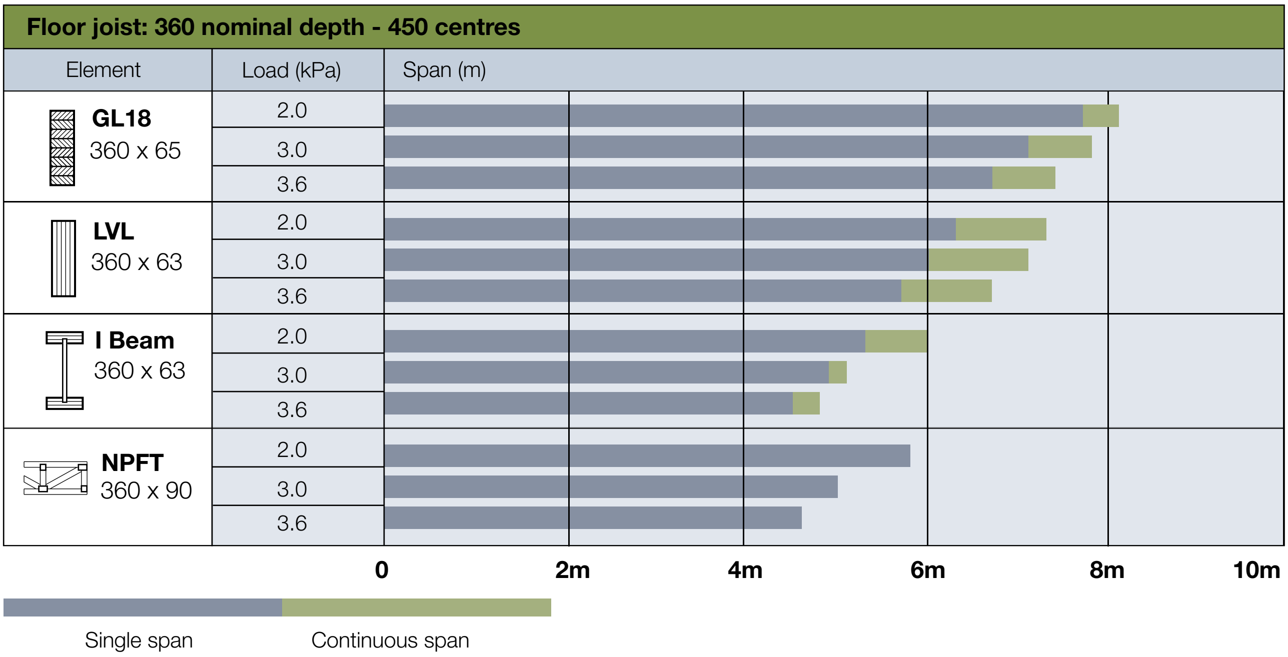


Table 10: Indicative span – Floor joist: 400 nominal depth at 450 centres

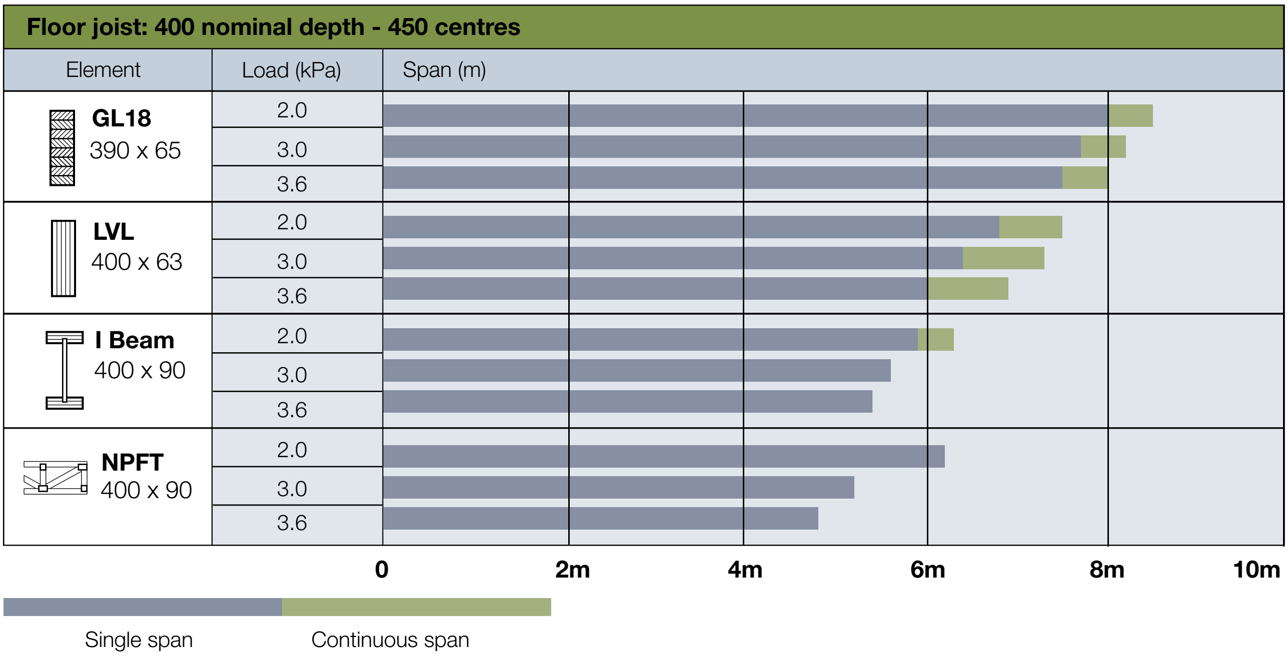


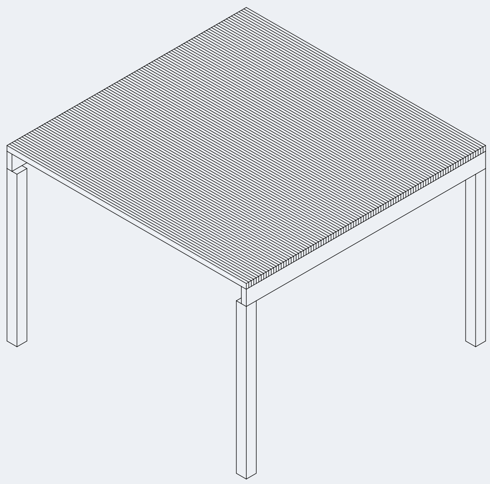
Table 11: Guide to maximum joist spacing for butt-jointed T&G flooring under domestic loads

Material	Grade	Thickness (mm)	Maximum joist spacing (mm)
Australian hardwood	Select	19	680
Australian hardwood	Medium feature – standard	19	620
Cypress	Grade 1	19	580
Cypress	Grade 2	19	580
Radiata pine	Standard	19	450
Radiata pine	Standard	30	920

Note: For butt-joined material only. Source: AS 1684.1 Table 5.3

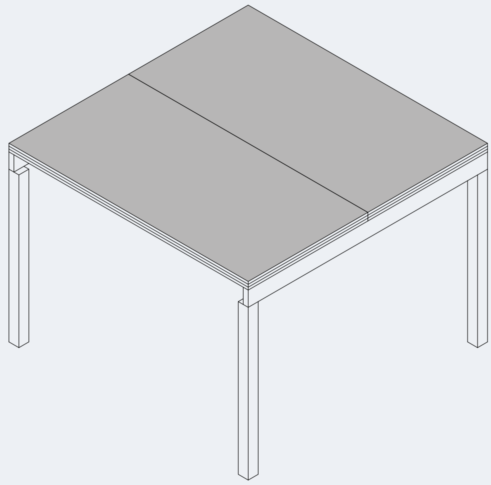
Table 12: Guide to maximum joist spacing for plywood flooring under domestic loads

Maximum joist spacing (mm)	Grade		
Plywood thickness	*F8*	*F11*	*F14*
12	400	420	440
13	430	450	480
14	460	480	510
15	480	520	540
16	510	540	570
17	540	560	600
18	560	590	620
19	590	620	660
20	610	650	680
21	640	670	710
22	660	700	740



LVL or glulam floor panels

Panel span range: Up to 6 m



CLT floor panels

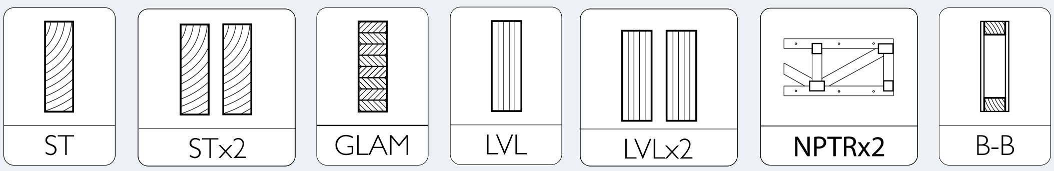
Panel span to depth ratio: 25 to 1

Description

Massive timber construction uses large, solid wood panels as slab elements to span between beams or walls. Panels are prefabricated off-site and lifted into place by crane. The timber panels weigh much less than concrete options and this makes assembly easier, reduces permanent loads on the structure, and improves the system's seismic performance.

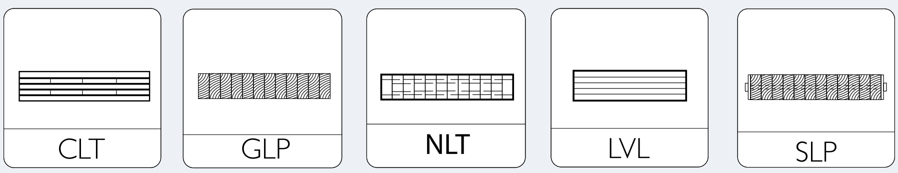
Massive timber systems are well suited to longer spans, heavier loads and larger structures. Compared to traditional joist floors, massive timber floors can have a shallower depth for a given span and provide excellent resistance to concentrated loads. They also have excellent fire-resistance. The panels may be left exposed to provide a finished ceiling. For shorter spans and lower loads, joist framing is typically less expensive. However, savings on fire-protection and finishes and reduced on-site construction time may offset the additional cost of massive systems.

Regular beam options



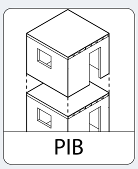
Notes
Glulam is available in wide sections.

Regular panel options

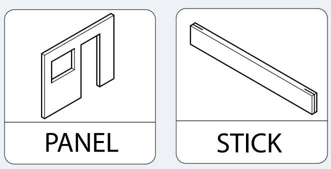


Notes
Stress laminated panels are generally used in civil construction such as bridge decks.

Regular construction and prefabrication options

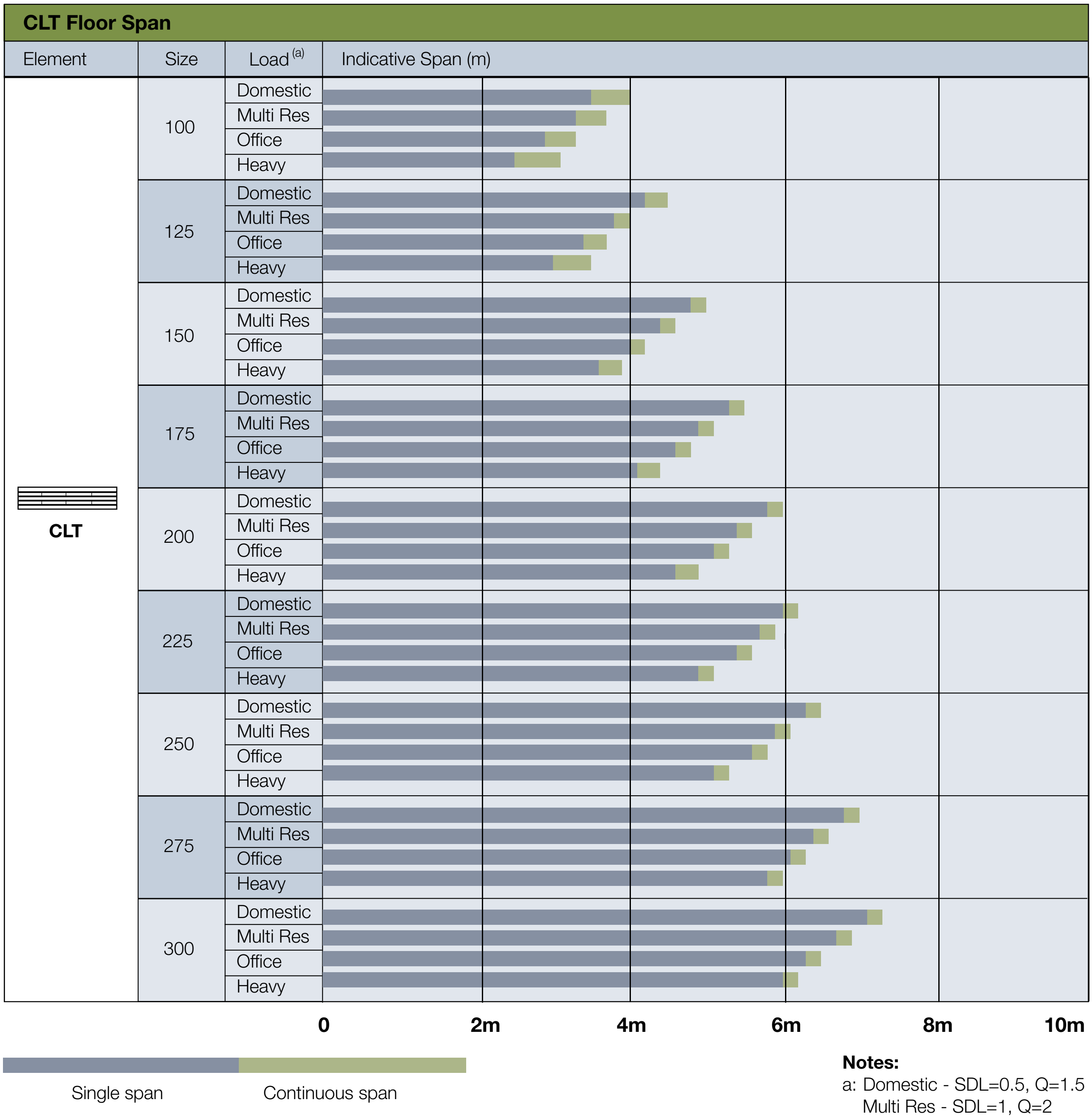


Construction notes
Beams and all floor panel systems can be fully prefabricated.



Prefabrication notes
Panels and beams can be fully prefabricated for the project.

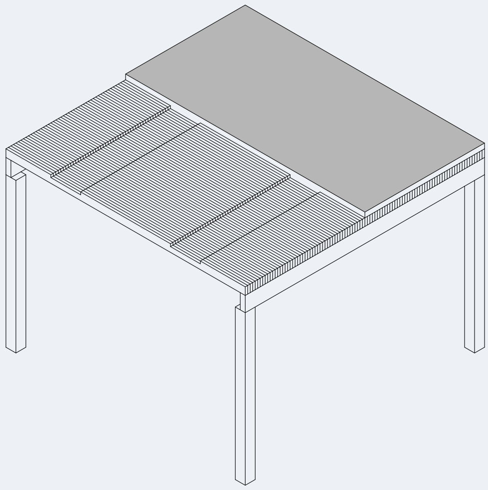
Table 13: Indicative span – CLT floor panels



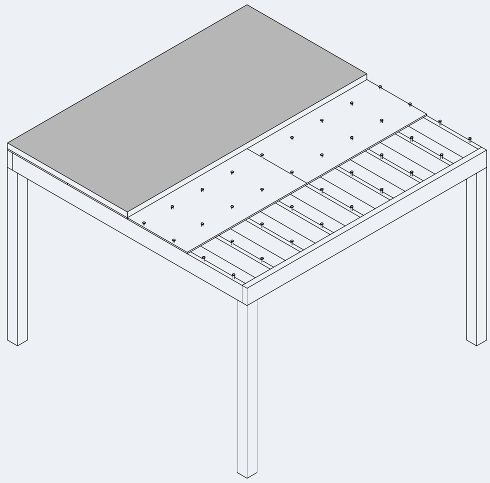
Source: Xlam

Notes:
a: Domestic - SDL=0.5, Q=1.5
Multi Res - SDL=1, Q=2
Office - SDL=1.5, Q=3
Heavy - SDL=2, Q=5

2.2.3 Floors: Concrete and timber composites



Composite massive timber floor



Composite joist system floor

Floor span range: up to 8 m

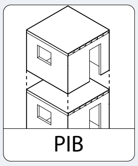
Description

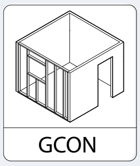
Timber components can be combined with a concrete slab to form a composite floor, with the concrete acting in compression and the timber acting in tension. To achieve composite action, shear connections are required between the timber and concrete. This may be achieved by cutting keyways into the top of the massive timber panel or included beams, installing vertical steel plates or pins to connect the wood to the concrete, or a combination of both. The shear connection system should be considered early as it can have a significant impact on cost. The system also requires coordination of two trades: carpentry and concreting.

The composite floor has increased strength and stiffness compared to simpler timber systems, allowing longer spans or reduced floor depth. Compared to a typical reinforced concrete floor, the composite system has lower dead weight. The wood floor serves as permanent formwork for the concrete and can be left exposed as the finished ceiling.

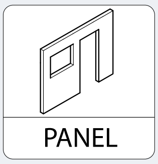
Creating a composite floor system can be an effective way of increasing the load-bearing, thermal or acoustic capacity of an existing wood floor system. The concrete increases the system's mass, reducing vibration and noise transmission, but places higher loads on the support system and foundation.

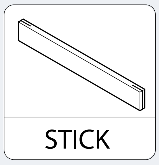
Regular construction and prefabrication options





Construction notes
Beams and floor plates can be prefabricated and installed with or without the concrete layer or built with generic products.





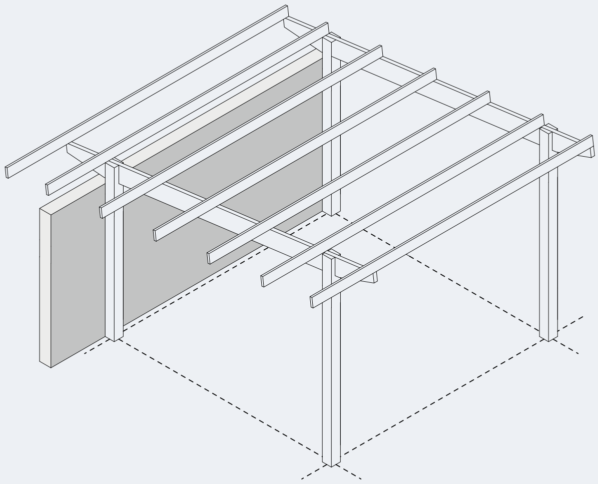
Prefabrication notes
Beams and floor plates can be prefabricated and installed with or without the concrete layer or built with generic products.

2.3 Spanning systems: roofs

The roof is the second spanning system in a building's structure. It spans horizontal distances to carry vertical roof loads to the support system of columns and walls. It also has a greater impact on the structure's architectural character than floors, or the support or lateral resistance systems.

Roofs typically have less stringent performance requirements than floor systems and have fewer constraints in terms of depth and shape. This allows the use of a wider range of structural forms including: beams, trusses, tied frames, portal frames, and arches. Other systems are available but these are not covered in this guide.

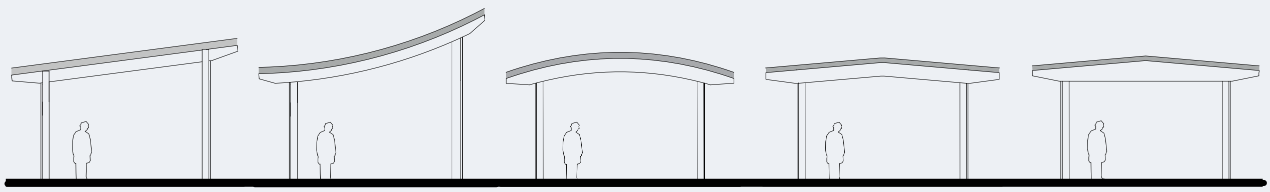
The required span is the primary design consideration for the roof's form and materials while other factors to consider include material availability, fabrication capacity, the potential for prefabrication, and appearance. Often, simple flat or pitched roofs are the most economical choice because they simplify the connections and cladding systems. However, other forms may offer better material efficiency, longer spans or a desired architectural effect.



Beam span range: 30 m+

Beam span to depth ratio: 20 to 1

Form options

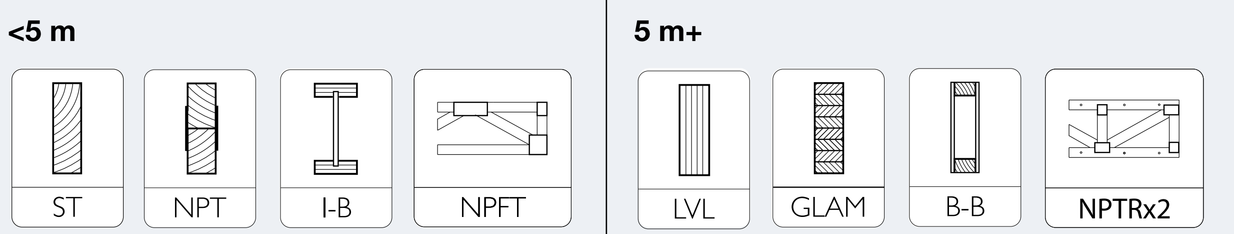


Description

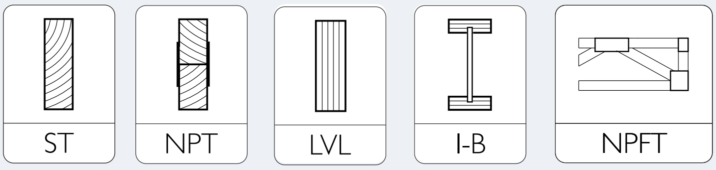
A common roof system consists of beams or primary rafters that span with the roof's pitch and are simply supported on walls or columns. These beams support purlins, insulation and ceiling systems, and the roof material. Numerous options exist for the beam's shape or pitch, depending on the architectural intent and element option selected. Glulam and box beams can be curved. The beams may also taper along their length to add extra capacity at mid-span. Section 2.3.5 discusses options for using trusses instead of beams in similar solutions.

Most timber beams are rectangular in cross section and deep, slender beams are the most efficient at resisting bending loads. However, these may require additional lateral bracing to resist buckling or racking. Purlins set between the beams, instead of running continuously over them, can often provide this bracing. Material efficiency can be improved by varying the element's cross section through the beam's depth and concentrating material in the outer tension and compression zones. Elements with this configuration include I-beams, plywood box beams and nailplate trusses.

Regular beam or rafter options by span

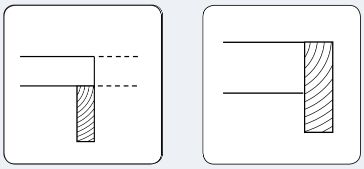


Regular purlin options and configuration



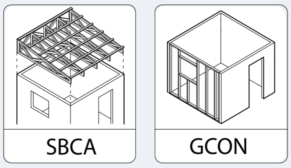
Notes
Purlin span to depth ratio is nominally 24 to 1. Options reduce considerably for spans over 4.8 m. See Table 19.

Assembly options

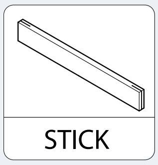


Assembly notes
Purlins set between beams provide them with lateral restraint while continuous spans allow for a smaller purlin section.

Regular construction and prefabrication options

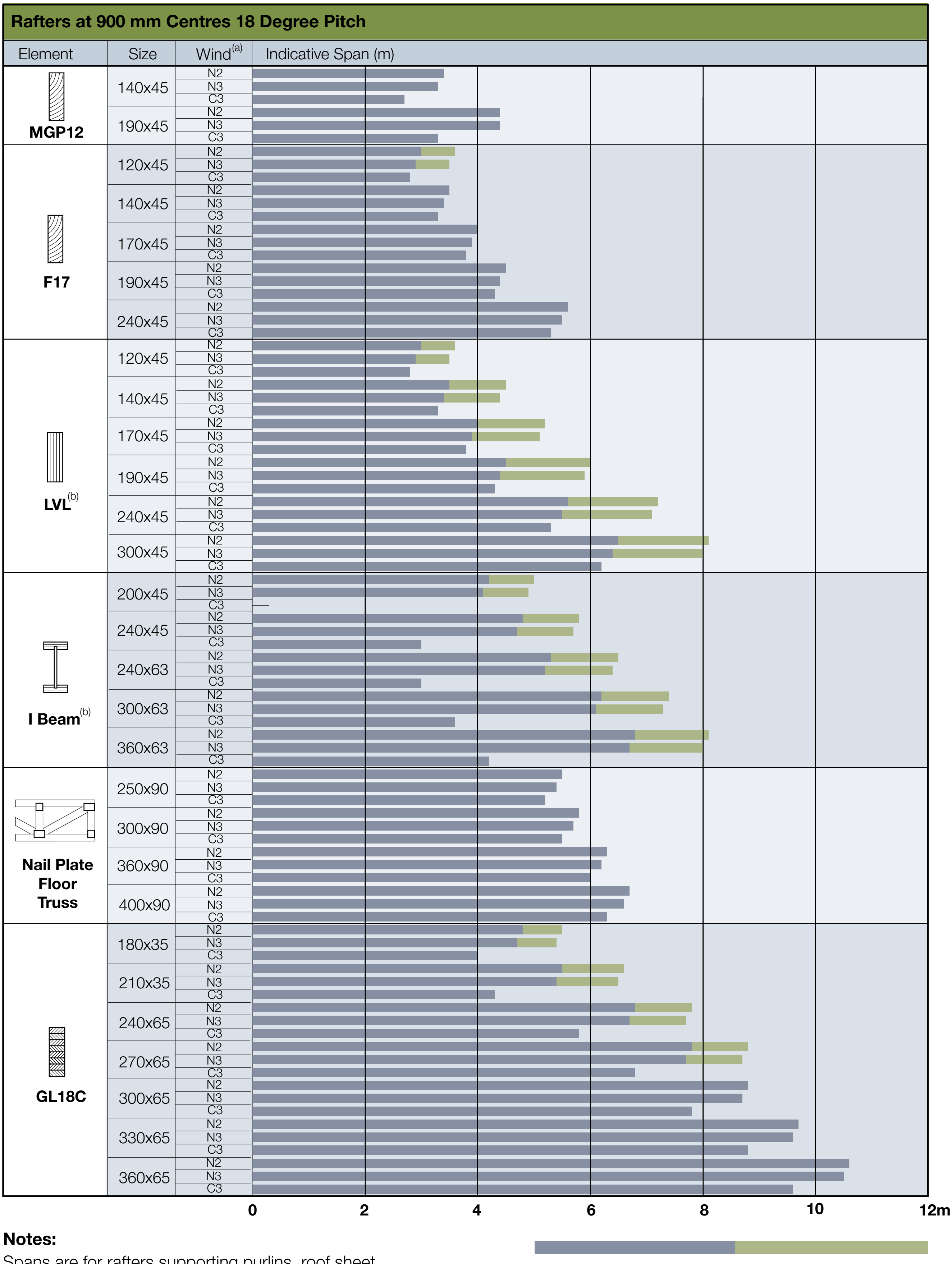


Construction notes
Systems are often site assembled but roof beams and purlins can be assembled as a module on the ground and lifted into place.



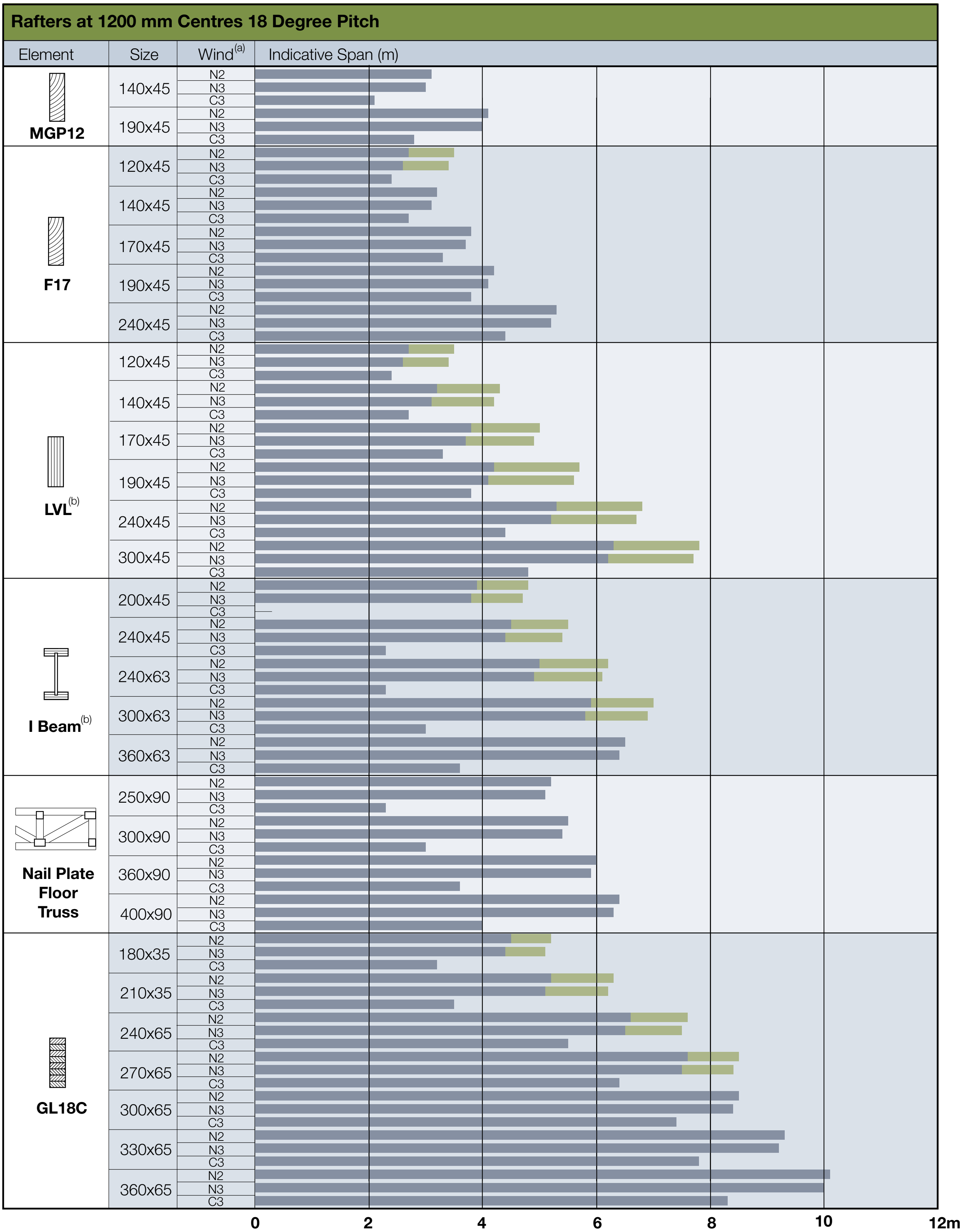
Prefabrication notes
Rafter and purlins can be optimised and shaped.

Table 14: Indicative span – Rafters at 900 centres – pitch 18 deg



Notes:
Spans are for rafters supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services
a. Wind loads are indicative for Class 1 buildings only.
b. Hyspan and Hyjoist products

Table 15: Indicative span – Rafters at 1200 centres – pitch 18 deg



Notes:

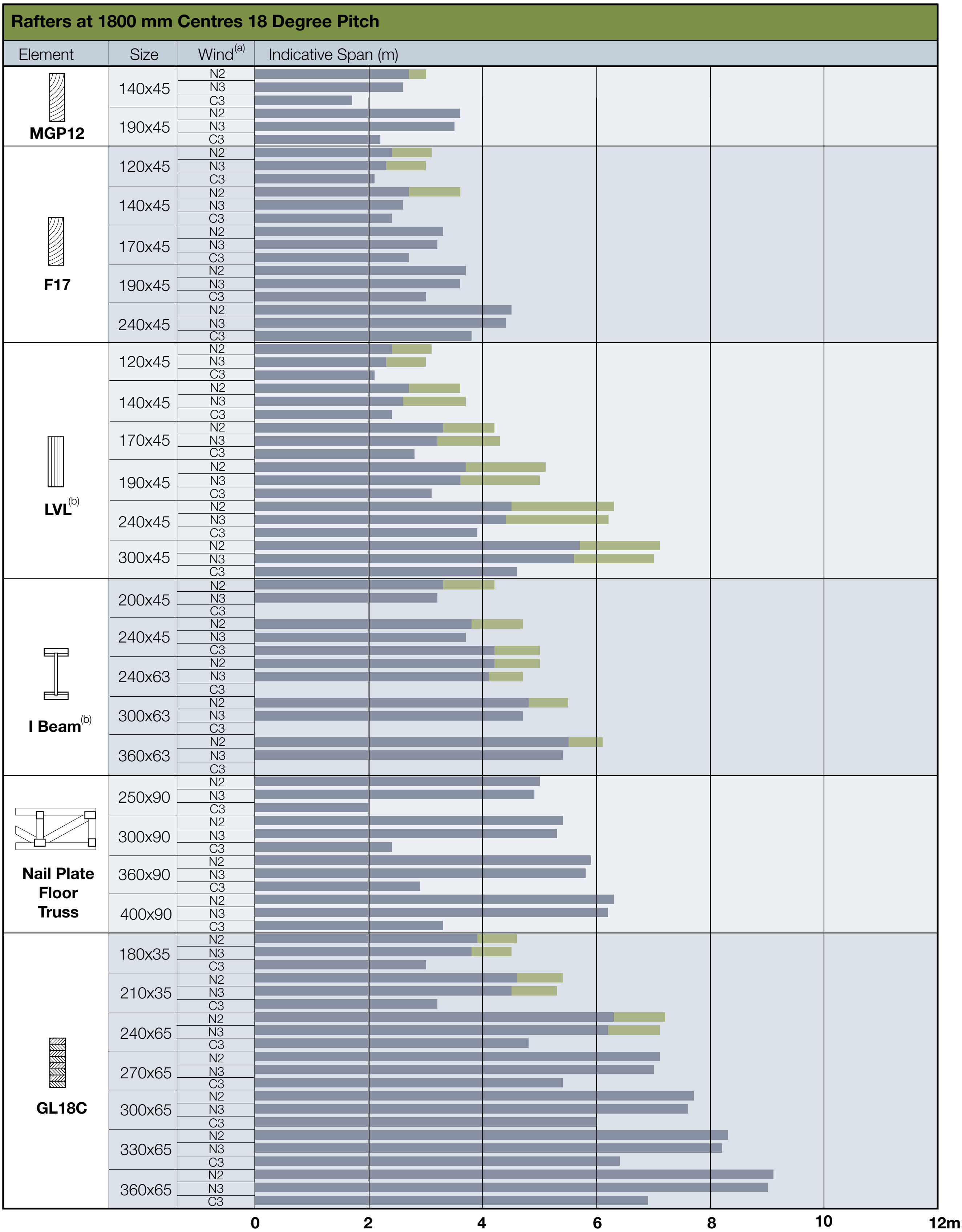
Rafter supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services

a. Wind loads are indicative for Class 1 building only.

b. Hyspan and Hyjoist products

Single span   Continuous span

Table 16: Indicative span – Rafters at 1800 centres – pitch 18 deg



Notes:

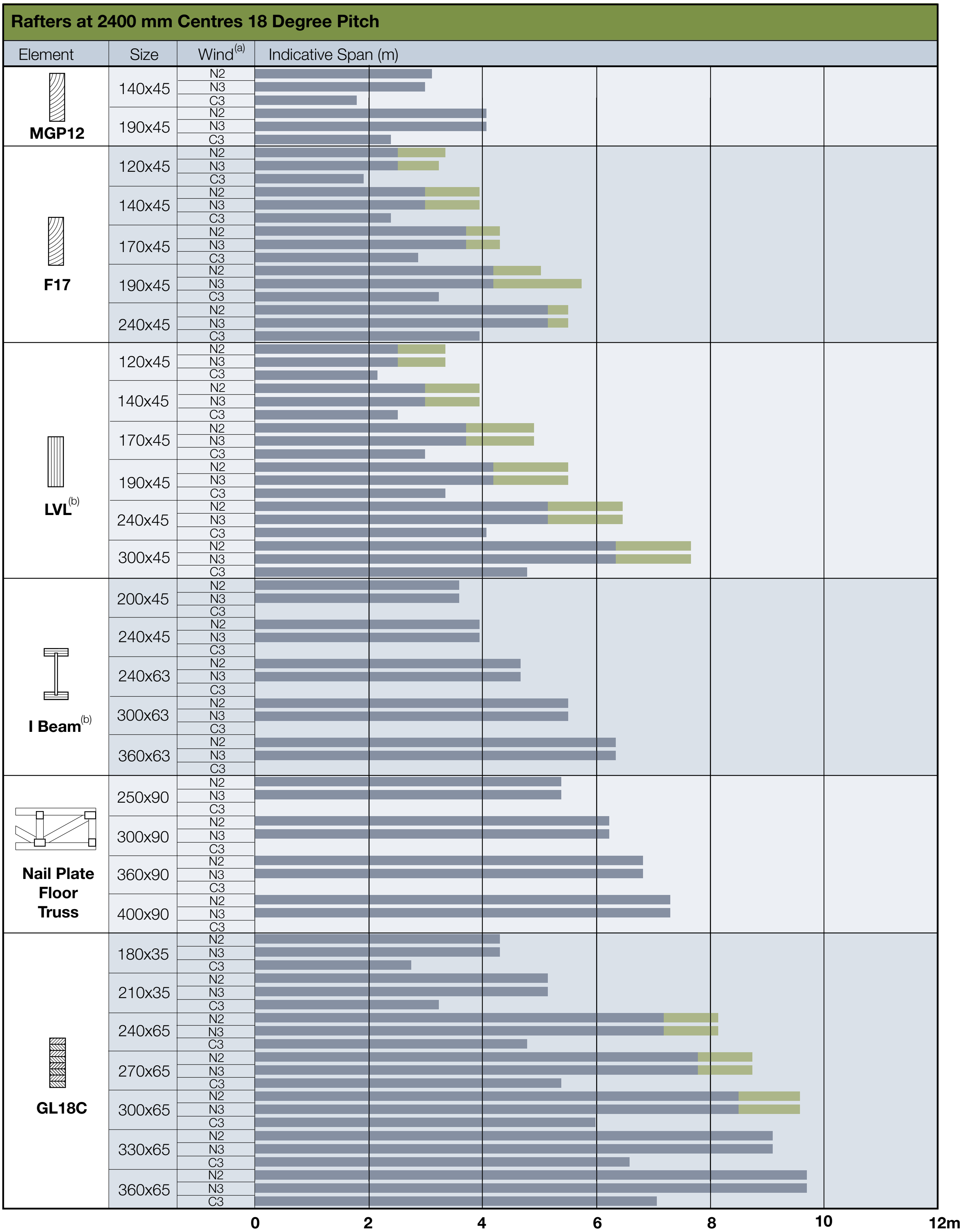
Single span
Continuous span

Rafter supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services

a. Wind loads are indicative for Class 1 building only.

b. Hyspan and Hyjoist products

Table 17: Indicative span – Rafters at 2400 centres – pitch 18 deg



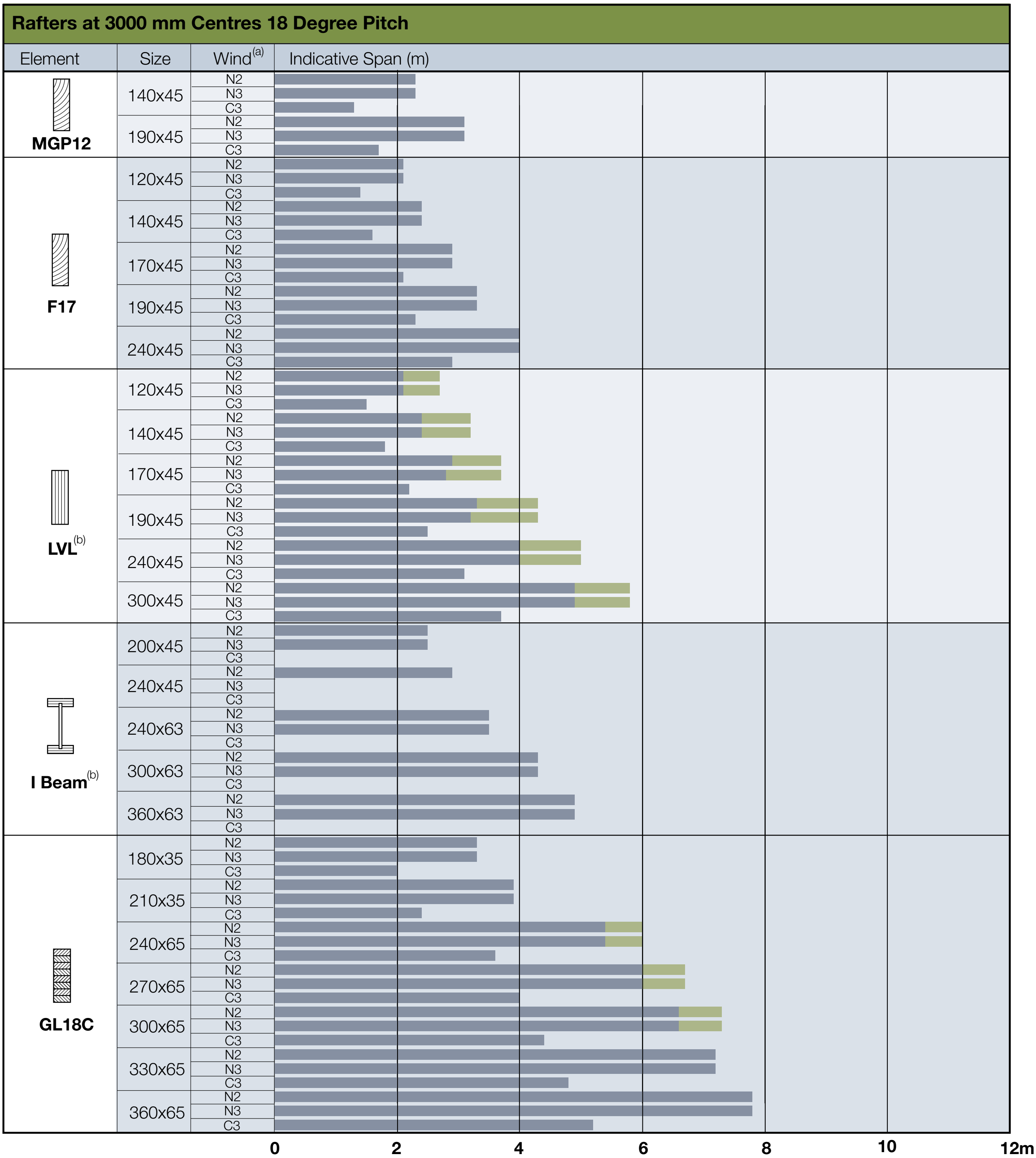
Notes:

Rafter supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services

a. Wind loads are indicative for Class 1 building only.

b. Hyspan and Hyjoist products

Table 18: Indicative span – Rafters at 3000 centres – pitch 18 deg



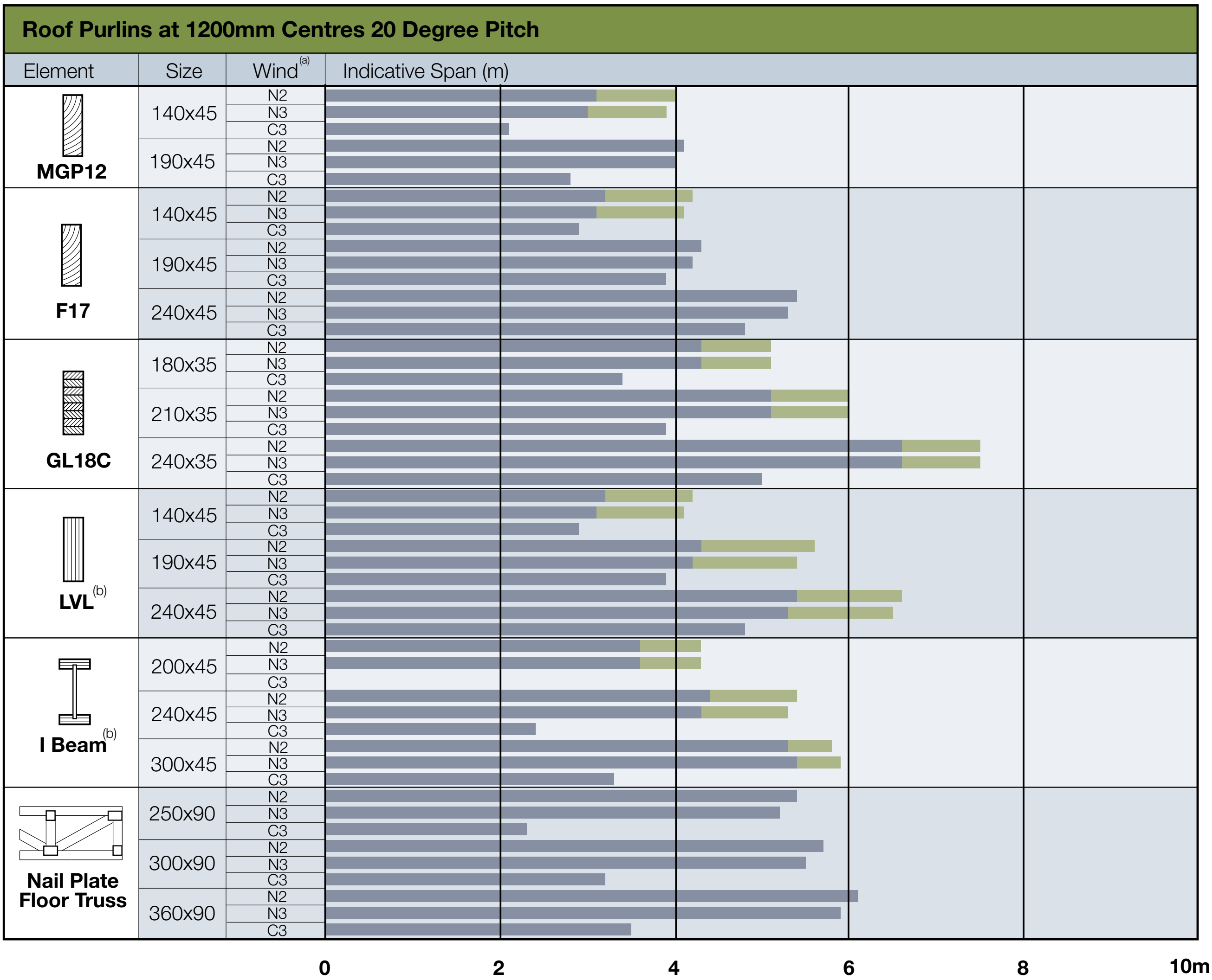
Notes:

Rafter supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services

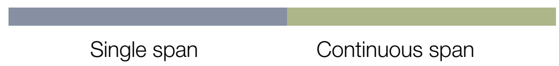
a. Wind loads are indicative for Class 1 building only.

b. Hyspan and Hyjoist products

Table 19: Indicative span – Roof purlins at 1200 centres



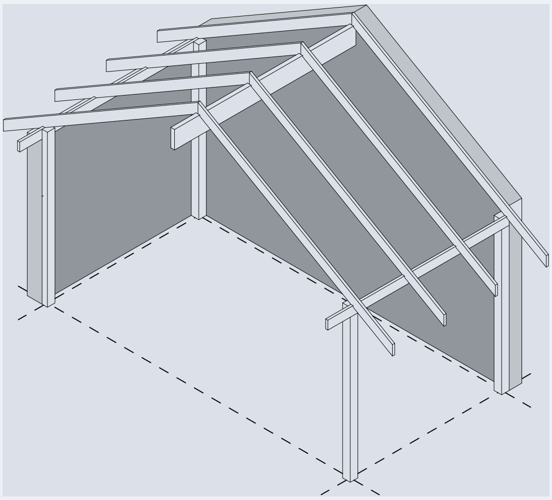
Notes:



Rafter supporting purlins, roof sheet, insulation, plasterboard ceiling and minor services

a. Wind loads are indicative for Class 1 building only.

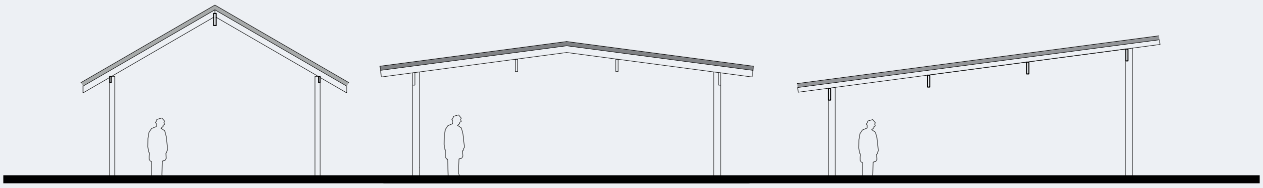
b. Hyspan and Hyjoist products



Rafter span range: Up to 8 m

Rafter span to depth ratio: 24 to 1

Form options

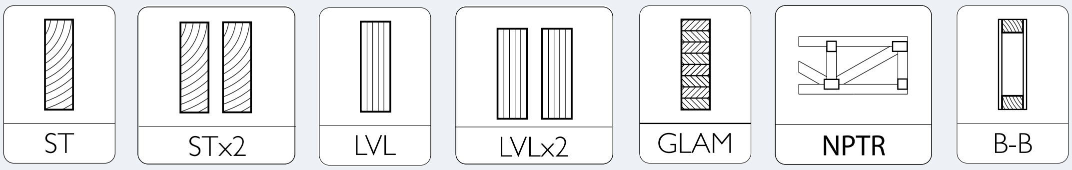


Description

Beams can span between walls or columns at the ridge, springing or intermediate locations and support regularly spaced rafters or massive timber panels. Rafters can span between the beams in single or continuous spans and support purlins or battens, insulation and ceiling systems, and the roof material. Common rafter spacings are between 0.9 and 1.8 m. CLT, other massive timber, cassette or stressed skin panel options can also span between the beams and support insulation and a roofing system.

Numerous options exist for the supporting beams depending on the architectural intent, the element option selected, and the beam spacing. More limited options exist for widely spaced beams as they carry heavy loads and tie-down requirements at the supports can be considerable.

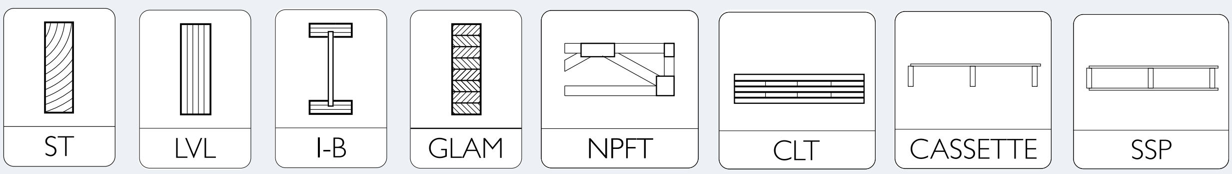
Regular beam options



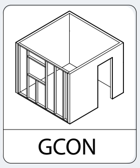
Notes

Glulam is available in wide sections while LVL and box beams do not have the depth limitations of sawn timber.

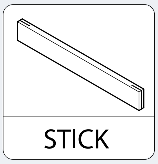
Regular rafter or roof panel options



Regular construction and prefabrication options



Construction notes
Systems are often assembled on site



Prefabrication notes
Beams and rafters can be optimised and shaped. Roof panels can be prefabricated.

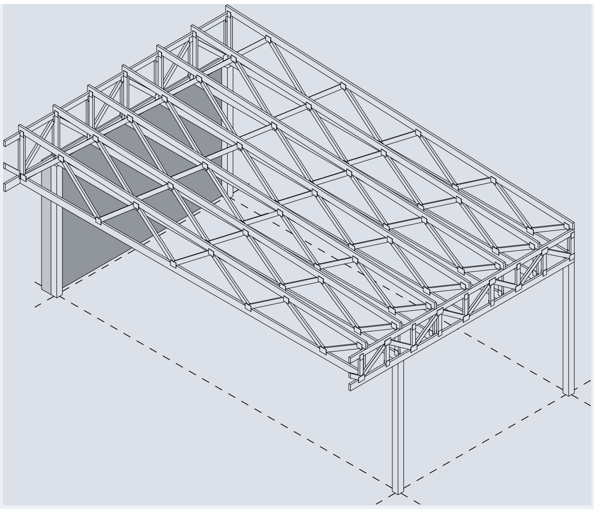
2.3.3 Roofs: Trusses

Trusses perform a similar function to beams but offer a more efficient use of materials and can be much lighter than other systems for equivalent spans. Timber trusses can be made in almost any size and shape and can achieve spans up to 50 m. They may be constructed from almost any combination of solid timber products: sawn timber, LVL or glulam. As individual truss members are generally loaded in axial tension or compression, the tension and compression chords resist the truss's overall bending stresses while the web members maintain the spacing between the chords and resist shear stresses. By spacing the chords farther apart, member stresses are greatly reduced. As they are only loaded axially, members can be much smaller than if stressed in bending.

A controlling factor in timber truss design is often the connections at the truss nodes. The sizes of tension elements are often determined by the space required to make the connection. Design of compression elements will typically be controlled by buckling resistance. Additional web members may be added to reduce the stresses on individual members. This will also reduce the effective unbraced length of the compression chord, providing better buckling resistance.

When loads are applied only at the nodes, the truss elements are only required to resist axial forces. However, when roof loads are distributed across the top chord, they must be designed for combined bending and axial compression loads. Similarly, ceiling loads may create bending stresses in the bottom chord. If a lightweight roofing material is used, wind suction may be enough to cause significant uplift forces and load reversal in the truss elements. These must be sized to adequately resist these loads. A heavy roofing material reduces the potential of load reversal.

2.3.4 Roofs: Light nailplate trusses

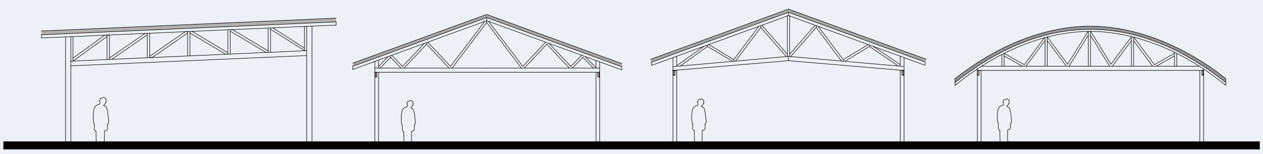


Truss span range: 25 m+

Truss span to depth ratio: 12 to 1 for parallel chord

Truss spacing: 0.6 – 1.2 m

Form options



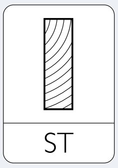
Description

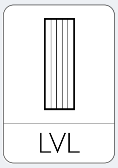
Light nailplate trusses are highly versatile components manufactured for the project by a licensed fabricator using proprietary nailplate connectors to make the truss node joint and generic timber sections for the webs and chords. The trusses and node connections are designed using proprietary software as part of a 'whole of roof' solution. The software can generate designs for highly complex roof forms and produce both production and installation documentation. Due to their economy, light nailplate truss systems have generally replaced heavy trusses in larger commercial and industrial structures. Trusses are usually supported from walls, lintels over windows or openings, and from girder trusses incorporated into the solution. Trusses can be erected as individual elements or assembled into modules on the ground and lifted into place.

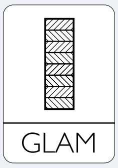
Light nailplate trusses are usually closely spaced and support battens, insulation and ceiling systems and the roofing material. To ensure economy, the trusses are often made from standard sawn timber with LVL or glulam used in high stress elements. Trusses are often 35 to 45 mm thick but can be grouped in multiples to resist high loads. Ply can be applied to the face of the truss or truss group to improve appearance or provide fire protection for the nailplates.

Spans of up to 35 m are possible but transport limitations restrict practical spans to 25 m. It is typically best to keep the truss depth under 3 m, as the web members may become unstable at longer depths. Alternatively, stiffeners can be added to convert a rectangular section in to T-shaped element.

Regular truss chord and web options

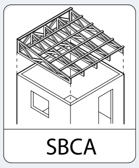


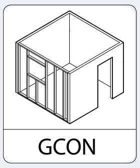




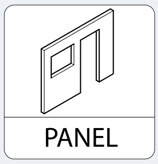
Notes
While economy drives most nailplate truss design, systems allow for the development of architectural solutions using more robust sections, painted nailplates or plywood cladding.

Regular construction and prefabrication options



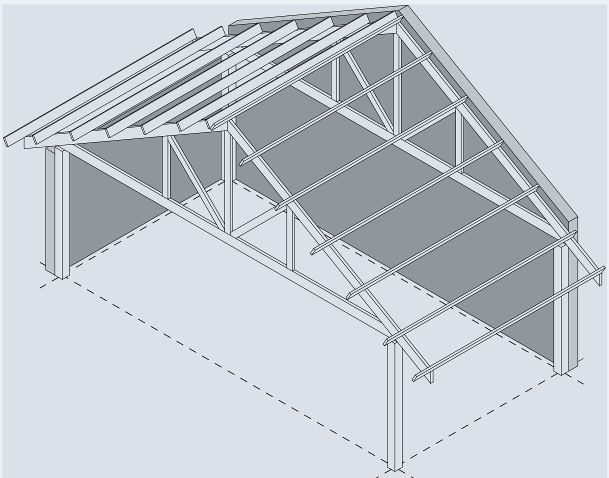


Construction notes
Systems are often assembled on site but trusses can be assembled as modules on the ground and lifted into place.



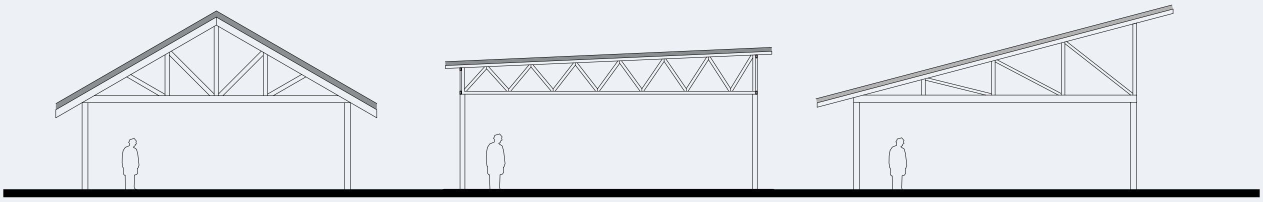
Prefabrication notes
Trusses are prefabricated as panels.

2.3.5 Roofs: Heavy Trusses

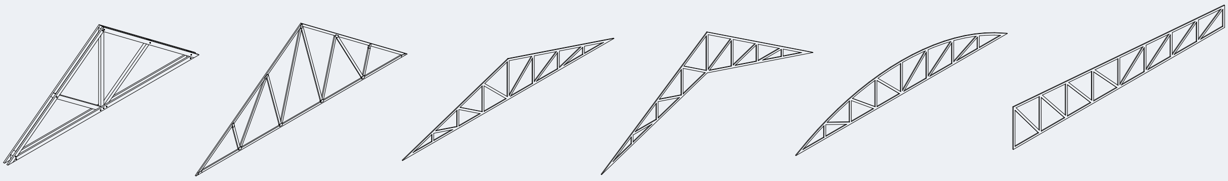


Truss span range: 30 m+
Truss span to depth ratio: 10 to 1 for parallel chord trusses
Truss spacing: 2.4 – 6.0 m

Form options



Additional configuration options

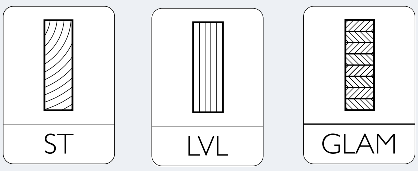


Description

Heavy timber trusses are more common than light nailplate trusses in buildings where heavy loads apply or the trusses form part of the building's architecture. These trusses are usually spaced farther apart than light nailplate trusses and typically support purlins, insulation and ceiling systems, and roofing material. CLT, other massive timber, cassette or stressed skin panel options can also span between the trusses and support insulation and a roofing system. The trusses generally span between columns or reinforced sections of walls. Spacing the trusses more than 4.8 m apart limits purlin options. Like light-plate trusses, pairs of heavy trusses, their purlins and services can be assembled into a module on the ground, before being lifted into place.

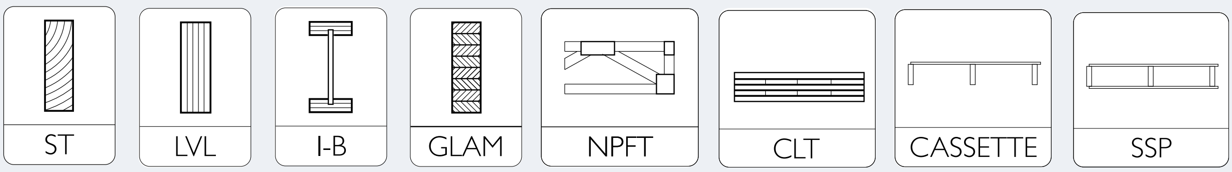
Heavy trusses are made from larger timber elements, which may be sawn timber or, more regularly, glulam. LVL is less common as it is not generally available in appearance grades. Members are most often joined at the nodes with bolted or dowel connections, often using steel gusset or fin plates. Plywood gussets may also be used.

Regular truss chord and web options

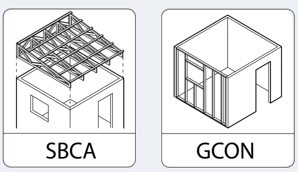


Notes
Architectural considerations can often influence the grade and size of the selected truss material.

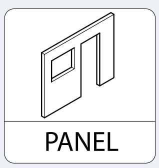
Regular purlin or roof panel options



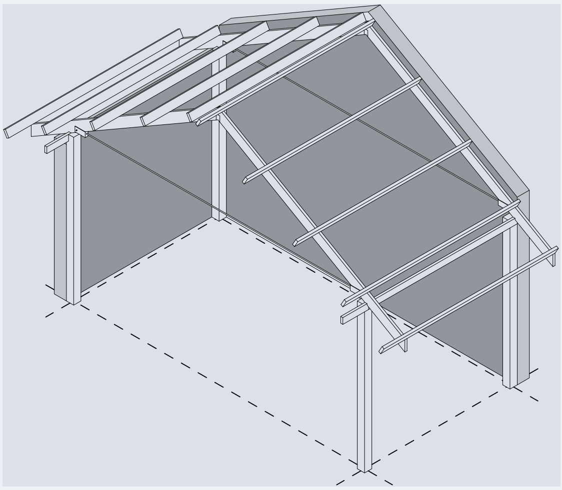
Regular construction and prefabrication options



Construction notes
While systems are often assembled on site, trusses and purlins can be assembled as modules on the ground and lifted into place.



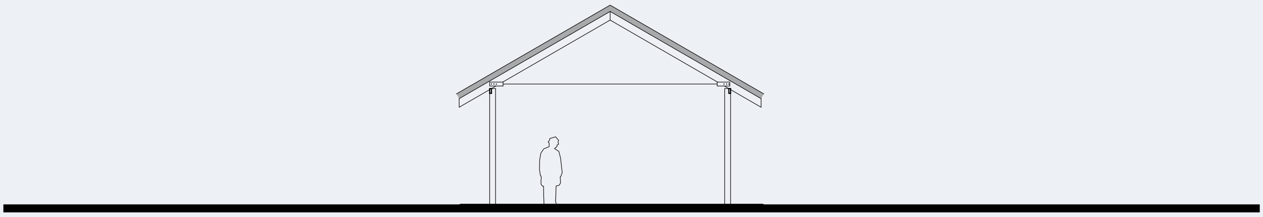
Prefabrication notes
Trusses are prefabricated as panels. Purlins can be optimised for single or multiple spans.



Frame span range: 20 m+

Frame spacing: 2.4 – 4.8 m

Form options

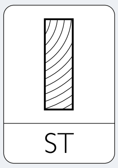


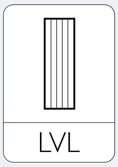
Description

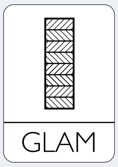
Tied frames are a simplified truss form that spans between walls or columns. Inclined rafters form the component's top chords and steel or timber tension members tie the base of the rafters together to resist horizontal loads and eliminate the need for a beam at the ridge.

These frames are spaced farther apart than nailplate trusses and typically support purlins or roofing panels, insulation and ceiling systems, and the roofing material. The included rafters are made from larger timber elements such as sawn timber, glulam or LVL.

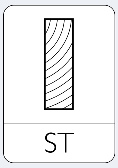
Regular rafter options

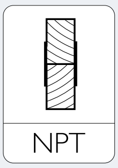


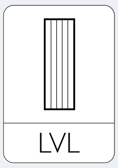


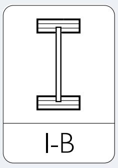


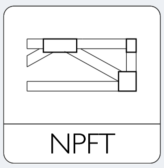
Regular purlin options

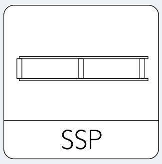




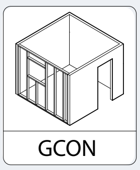




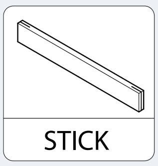




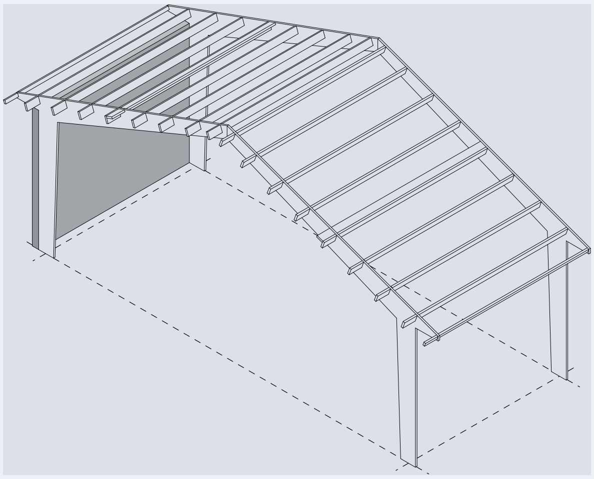
Regular construction and prefabrication options



Construction notes
Systems are generally assembled on site.



Prefabrication notes
Rafters can be shaped and purlins optimised for single or multiple spans.

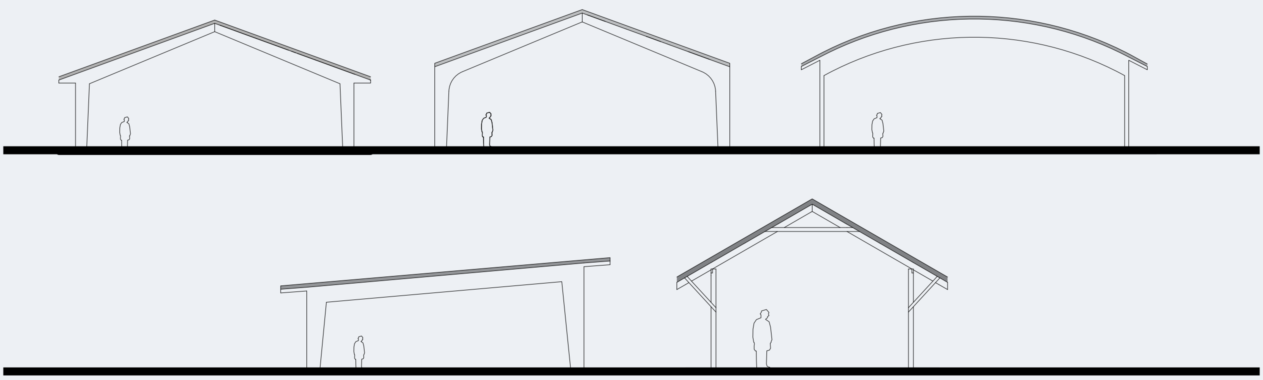


Truss span range: 30 m+

Frame spacing: 3.0 – 4.8 m

Frame span to rafter depth ratio: 35 to 1

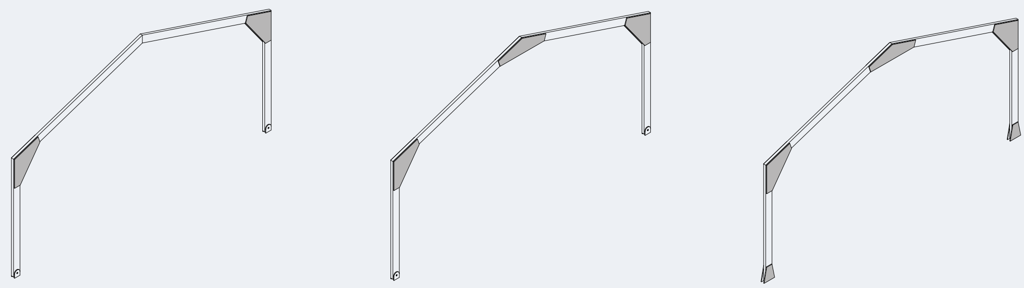
Form options



Description

Portal frames combine the spanning system's rafters and the support system's columns with moment-resisting connections to form a rigid frame. The moment transfer between the rafters and the columns increases the system's spanning capacity and serves as part of the lateral resistance system in the plane of the frame. Portal frames offer material efficiency and allow for rapid construction. They are often used for single storey industrial and recreational buildings or the roof structure of multi-storey offices and shops.

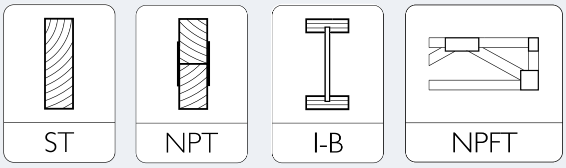
Portals can be 3-pin, 2-pin or rigid frames. 3-pin frames are the most common form for timber portals and have moment-resisting column to rafter connections but pinned connections at the bases and apex. 2-pin frames are where the column to rafter and the apex connections are moment-resisting and bases are pinned. Rigid frames have moment-resisting connections at each joint. This uses less material but constructing moment-resistant connections is more costly than making pinned connections, especially at the foundations.



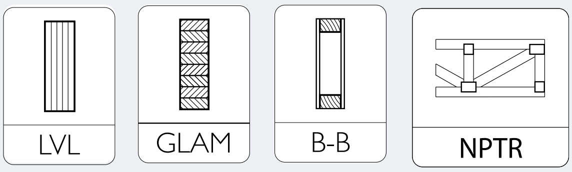
3-pin frames ***2-pin frames*** ***Rigid frames***

Regular column and rafter options by span

<6 m

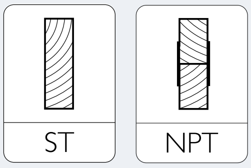


6 m+

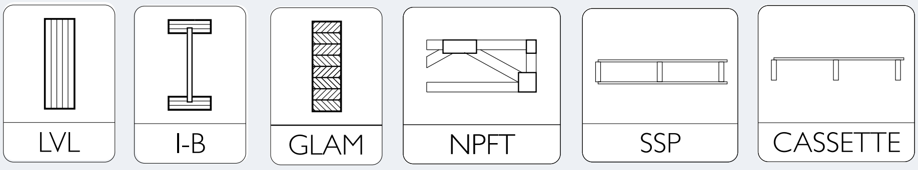


Regular purlin options by span

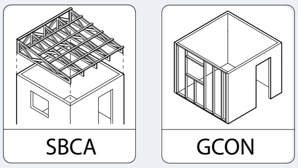
<3.6 m



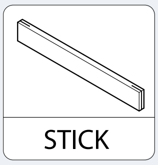
3.6 m+



Regular construction and prefabrication options

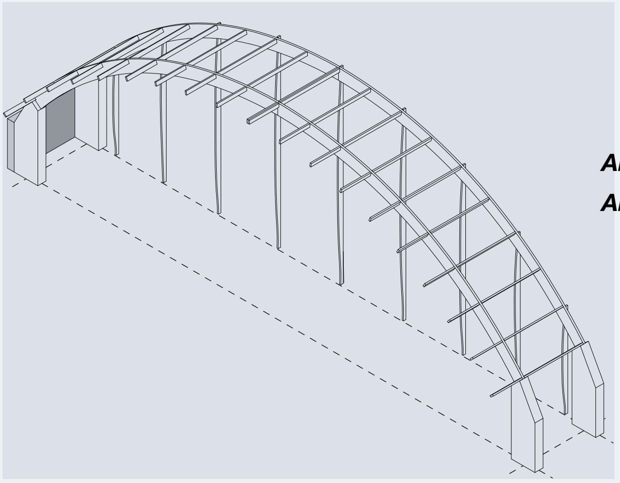


Construction notes
While systems are often assembled on site, rafter sets can be assembled with purlins and roof sheet as modules on the ground and lifted into place.



Prefabrication notes
Columns, rafters and connections can be prefabricated. Purlins can be optimised for single or multiple spans.

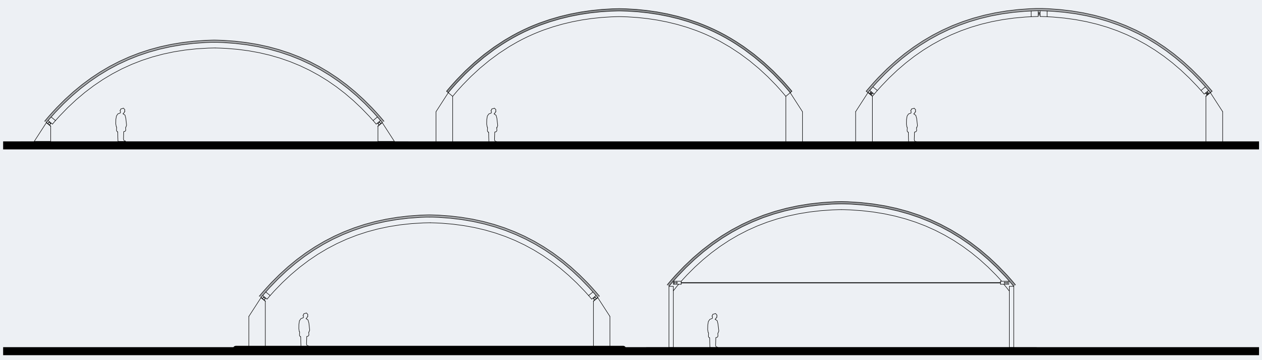
See WoodSolutions Design Guide 33: Quick Connect Moment Connection



Arch span range: 50 m+

Arch span to arch depth ratio: 40 to 1

Form options

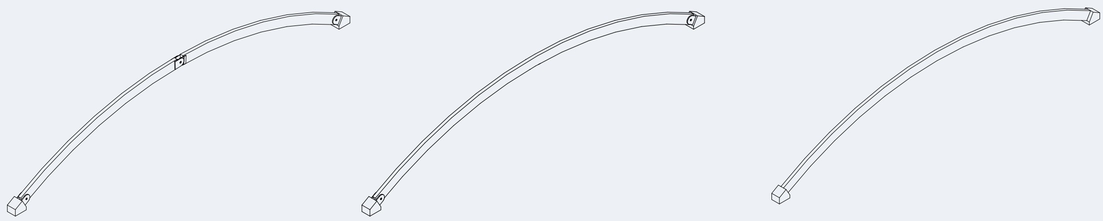


Description

An arch is an efficient form for long span structures as the curved profile resists uniformly distributed vertical loads primarily through compression in the arched element. Localised strength-reducing features such as knots and slope of grain in the timber have a less significant impact on axial compression than elements that resist loads through bending-induced tension parallel to grain. This makes a timber arch an efficient structural element. In practice, loads will not be applied to the arch uniformly and some bending stresses will be induced in the arch member. These will typically control the section size. As the radius of the arch reduces, radial tension stresses increase and can come to govern arch design.

The compressive forces in an arch have to be resisted at the reaction points. This is easiest in foundations at ground level but this reduces the usable head height near the arch's base. Arches raised above the ground require large buttress foundations to resist the thrust or need to include a tension member to tie the bottom of the arches together.

Glulam is the most common material for solid timber arches but arches can also be formed as trusses. Spans of 25 to 50 m are common and spans of 120 m or more can be achieved. While glulam arches can be made to virtually any size, transport restrictions and fabrication capacity limit effective element length.



3-pin arch ***2-pin arch*** ***Rigid arch***

Regular arch options

GLAM

Notes
Glulam fabrication capacity can limit effective depth and length.

Regular purlin options by span

<3.6 m	3.6 m+
ST, NPT	LVL, I-B, GLAM, NPTR, SSP, CASSETTE

Regular construction and prefabrication options

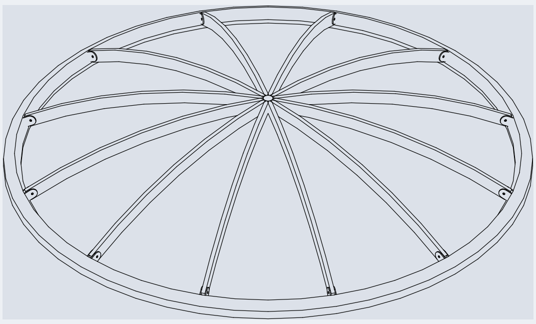
GCON

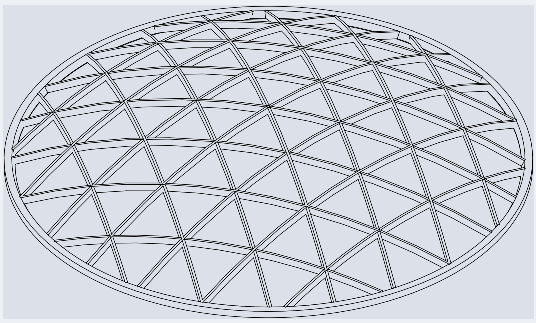
Construction notes
Systems are often assembled on site from prefabricated components.

STICK

Prefabrication notes
Arches and connections can be prefabricated. Purlins can be optimised for single or multiple spans.

2.3.9 Roofs: Dome



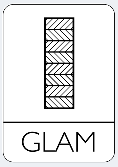


Beam span range: 50 m+

Description

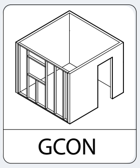
Many domes are essentially a 3-dimensional arrangement of arches that carry loads to a circular foundation. As with arches, domes act primarily in compression with minimal bending stresses in the members but create significant thrust at the foundation, typically resisted by a foundation tension ring beam. Because forces are distributed primarily as axial compression and tension, the members can be quite small, and very large spans can be achieved. The primary members of a 160 m clear span dome may only be 800 mm deep glulam elements. Dome members are usually curved glulam beams. In relatively small domes, the arched members may be continuous from the perimeter to the apex with purlins installed between them to support the roof decking. For larger spans, full length members may be impractical and shorter elements can be joined in a triangulated grid. The connections usually consist of fin plates, dowels and bolts.

Regular arch options

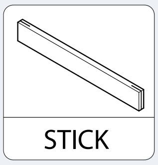


Notes
Glulam fabrication capacity can limit effective depth and length.

Regular construction and prefabrication options



Construction notes
Systems are often assembled on site from prefabricated components.



Prefabrication notes
Arches, arch sections and connections can be prefabricated. Purlins can be optimised for each arch segment.

2.4 Support system

Support systems receive vertical loads carried by the spanning systems and transfer them through the building's lower levels to the ground. In this guide, support systems include walls and columns. The beam portion of a post and beam frame is included in the spanning system. Elements in the support system primarily carry vertical load in compression but they may also be designed to resist wind-generated uplift and contribute to the lateral resistance system.

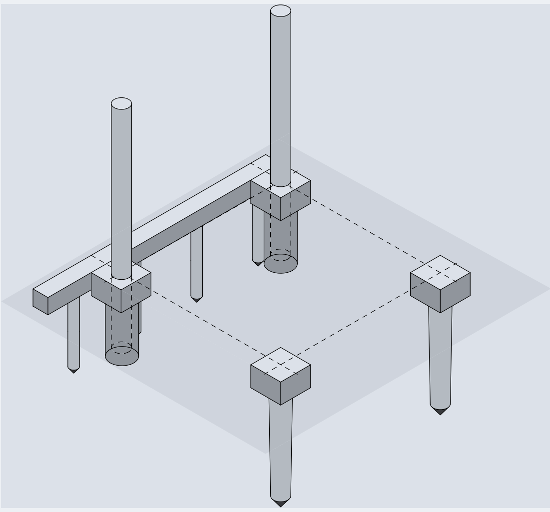
Generally, the critical factors considered when selecting a support system are the elements' required load and buckling resistance and other performance requirements such as fire resistance and acoustic separation. These are discussed in Sections 9.4 and 9.6.

2.5 Support systems: foundations, walls and columns

Support systems must satisfy a range of performance requirements in a design in addition to transferring loads from the roof and floors to the ground. A timber support system may combine:

- timber stud wall frames
- massive timber wall plates
- columns, either as part of a post and beam system or incorporated into wall frames.

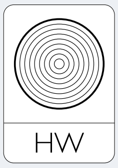
The spacing of the support lines that the walls and columns provide is an important consideration in the selection of the primary spanning and support systems. It affects the spanning members' required span and design, the sizing of support system elements, and often the functional efficiency of the spaces provided in the design. Different spacing arrangements should be tested to establish the most cost efficient option.

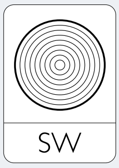


Description

Treated and natural timber rounds used as poles and piles serve a range of foundation and support functions. Poles can be set into the ground to form cantilevered columns. Timber pile systems generally use driven treated hardwood rounds to transfer loads from the building structure, through a concrete pile cap to the pile and then to the ground. Multiple piles can be grouped together under one pile cap to resist higher building loads or positioned under a strip footing to support wall loads. Suitably treated timber piles can be durable, cost effective and reliable. Timber piling in general ground contact should be treated to a minimum of H5. Where tidal salt water and the hazard of marine borer exists, treatment should be to H6. Treatment only effectively penetrates the sapwood, so the heartwood's durability may also be important.

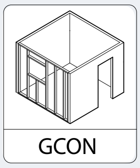
Regular pile options



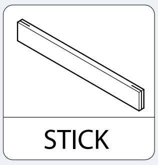


Construction notes
Treatment only effectively penetrates the sapwood. See Section 8.5.2

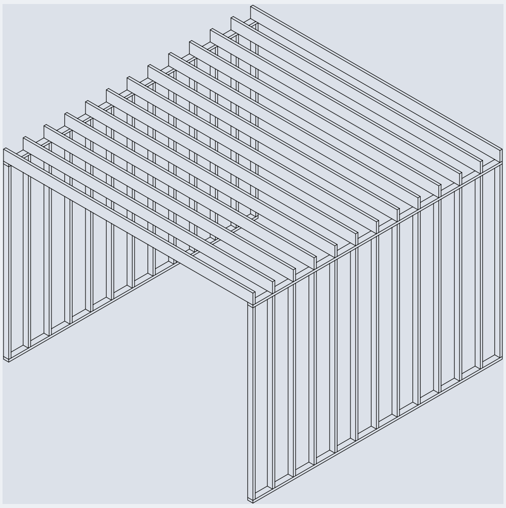
Regular construction and prefabrication options



Construction notes
Systems are often assembled on site from prefabricated components.



Prefabrication notes
Columns and piles can be shaped and cut to length off-site.



Description

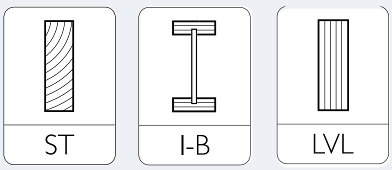
Timber stud frame walls form the primary support system for the majority of timber-framed building. Stud frames are highly versatile and economic components assembled from closely spaced vertical timber studs fixed between horizontal timber plates. The studs share the load applied to the wall between them. Sheathing over the studs or noggins fitted between them reduce the risk of the studs buckling. Steel straps and ties connecting the studs to the plates allow frames to resist uplift loads. The frame can also be clad or otherwise braced to form part of the lateral resistance system.

Timber stud frame walls are simple to construct and widely prefabricated. They are easy to handle and transport and can be installed without special equipment. While stud framing for residential construction is normally 70 or 90 mm thick, wider sawn timber or engineered wood products (EWPs) can be used as studs and allow for thicker frames to be assembled and carry higher loads. Curved walls can be assembled from regular timber studs and curved top and bottom plates of shaped plywood or LVL. To provide acoustic separation across the wall, studs can be staggered on wide plates or pairs of frames provided.

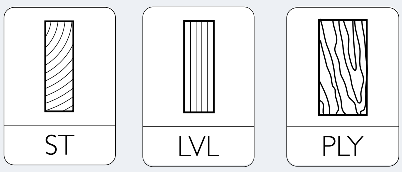
Most modern frame construction is platform construction, where the floor forms a platform on top of the wall frames. The next level of walls is then built off this platform. However, wall framings can be continuous through multiple storeys, with the floor plate hung from the side of the walls. This approach is called balloon framing. Frame design can be optimised to align the studs with the joists, rafters or trusses in the floor and roof and provide a continuous load path. This minimises bending in the plates.

Regular stud and plate options and configuration

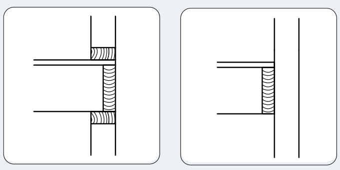
Studs



Plates

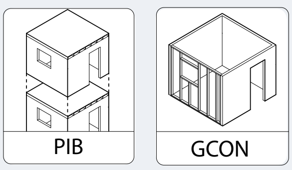


Assembly options

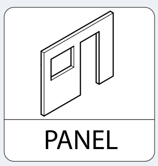


Assembly notes
In platform construction, the floor platform sits on the walls. In balloon construction, the studs run through with the floor supported from a ledger.

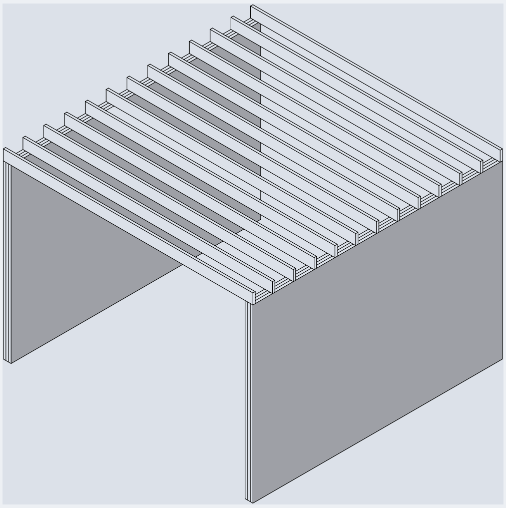
Regular construction and prefabrication options



Construction notes
Wall framing can be fully prefabricated or built on site from generic products.



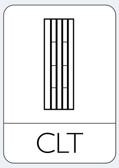
Prefabrication notes
Braced wall frames with or without cladding, lining and insulation can be prefabricated as panels.

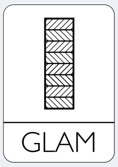


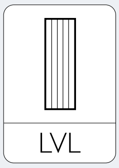
Description

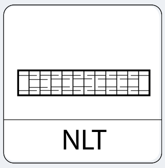
Massive timber walls are solid timber panels that carry forces from the spanning system to the foundation and provide lateral rigidity. Massive timber walls can be constructed from CLT, glulam and, occasionally, LVL. Massive timber walls have similar benefits to massive timber floors as panels are prefabricated off-site and lifted into place. Timber panels are much lighter than concrete, allowing for reduced transportation cost and faster assembly using smaller equipment.

Regular mass wall options and configuration



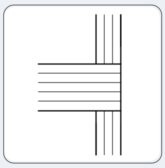
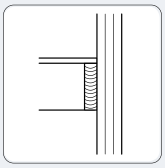






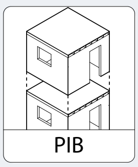
Notes
Systems may combine several materials to provide an efficient solution. For example, glulam and LVL may be used with CLT to increase the load bearing capacity of lintels.

Assembly options

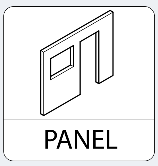



Assembly notes
CLT floor plates regularly sit on the wall frames or panel. However, panels can run through with the floor supported from a ledger.

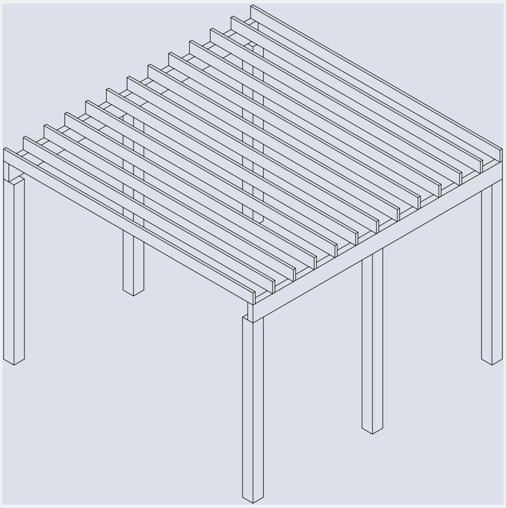
Regular construction and prefabrication options



Construction notes
Mass timber walls are installed as prefabricated panels.



Prefabrication notes
Mass timber wall are generally fully prefabricated.



Description

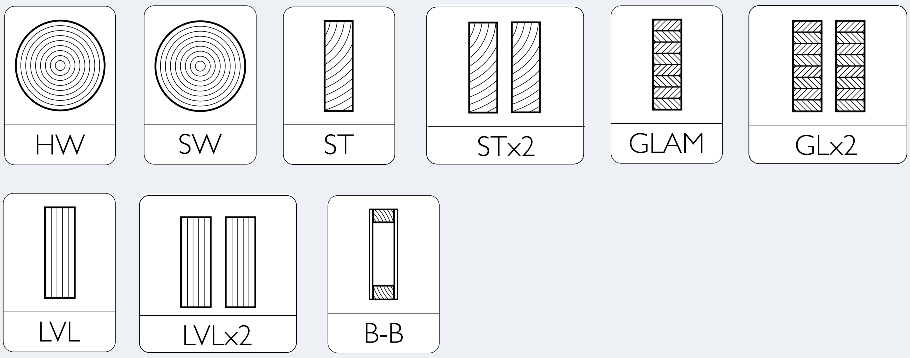
Columns transfer concentrated loads from spanning elements down to the foundations, ideally along a direct load path. Columns can form part of a post and beam system with the beams of the floor spanning system, or receive loads from major roof elements such as heavy trusses or modules of light nailplated trusses.

Load paths through the columns should ideally be direct and along the grain. Concentrated loads from columns not aligned with support points below and that fall onto horizontal spanning elements create very high bending and shear stresses in those elements. Similarly, concentrated load applied across the grain of spanning elements located between upper and lower columns can cause localised compression.

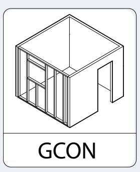
Column sizes must be designed to resist both compression and buckling. Tall slender columns may require bracing to resist buckling. Round, solid square or spaced columns are more efficient than narrow, deep rectangles, as they have equal capacity to resist buckling about both axes. An exception would be columns restrained along their length on one axis, such as when they are incorporated into a wall. These require more buckling resistance in the un-braced plane. In portals and similar forms, the columns provide lateral load resistance in addition to vertical support. These columns need to be designed for combined bending and compression loads and this often requires a larger member. These columns also need moment-resisting connection at the top, bottom or both.

The spacing of the members in the vertical support system is often affected by the spacing of the principal spanning members, but the choice of column system is also affected by the lateral load resistance system selected.

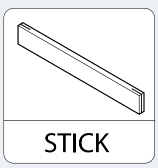
Regular column options



Regular construction and prefabrication options



Construction notes
Systems are often assembled on site from prefabricated components.



Prefabrication notes
Columns can be prepared and optimised

2.6 Lateral resistance systems

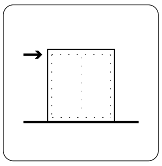
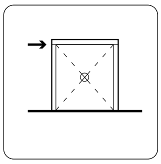
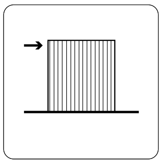
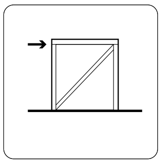
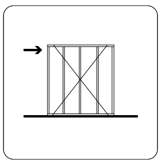
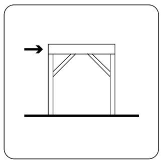
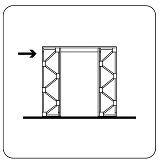
Description

Lateral resistance systems receive horizontal loads from the wind, earthquakes and similar sources and transfer them horizontally and vertically through the building to the ground. These forces are site specific and can vary in direction. A load path for the lateral forces needs to be mapped through the bracing components of the structural system. Different arrangements should be tested to establish the most cost efficient option. A lateral resistance system for a timber structure may combine:

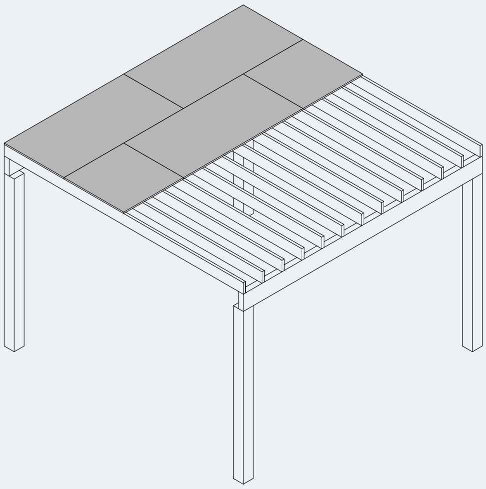
- diaphragms through the floor or roof planes or the walls
- bracing through light steel strap or folded sheet steel
- bracing through truss panels built into the wall, ceiling or roof plane
- bracing through solid timber braces or pairs of steel rod
- moment-resisting connections from columns to beams or rafters, or to foundations
- tie-down mechanisms to secure the ends of the braced panels to the structure and other bracing systems.

Typical bracing systems are summarised in Table 20.

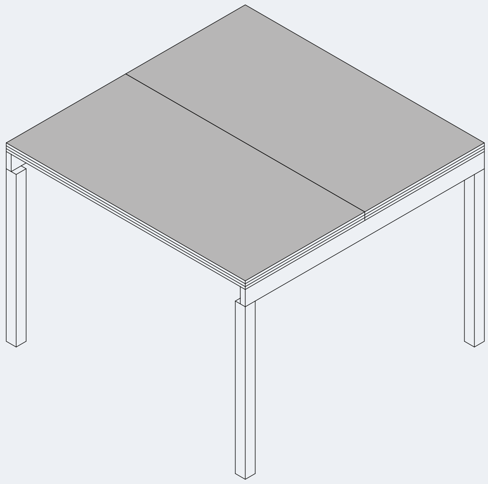
Table 20: Typical bracing systems

Icon	Description	Icon	Description
	Framed components can be clad with a sheet material to provide lateral resistance as a horizontal diaphragm or as a shear wall. Specific fixing requirements of the sheet to the frame apply.		Pairs of tensioned steel rods can be fitted between columns and beams or rafters and purlin sets to provide lateral resistance to vertical or roof elements.
	Massive timber components can be fixed to other elements to provide lateral resistance as horizontal diaphragms or shear walls.		Solid timber as a single or cross-brace pair of elements fitted between columns, beams and rafters can provide lateral resistance to vertical or roof elements.
	Horizontal or vertical framed components can be fitted with cross-brace pairs of steel straps to provide lateral resistance. The strap is fixed at the ends and at intermediate elements and tensioned.		Moment-resisting connections between posts and beams or columns and rafters can provide lateral resistance between spanning and support elements.
	Horizontal or vertical framed components can be fitted with cross-brace pairs of steel straps to provide lateral resistance. The strap is fixed at the ends and at intermediate elements and tensioned.		

AS 1684 provides additional detail and working capacities for several of these systems.



Sheet floor fixed to joists



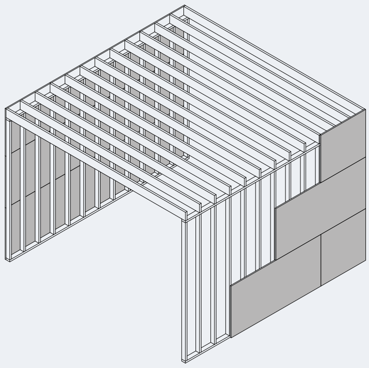
Mass timber fixed to beams

Description

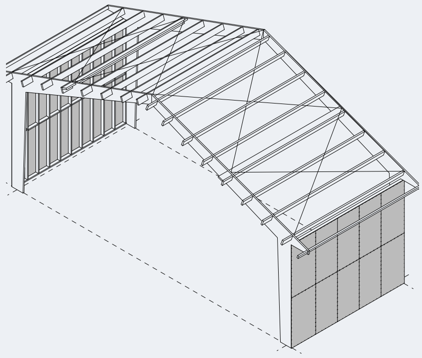
Floors often serve as diaphragms to transfer lateral loads horizontally while sheet bracing or panel systems can establish a diaphragm in the roof plane. Sheet or massive timber panel flooring can form a continuous plate to transfer loads through shear stress in the panels. Specific fixing requirements of the sheet to the joist or the panels to the beams may apply to ensure adequate shear transfer. The roof can also act as a full or partial diaphragm if sheet material, massive timber or stressed skin panels are incorporated into the structural system and fixed to ensure adequate shear transfer. These elements may be installed primarily as part of the structural system in discrete structural bays or along purlin lines, or form part of the envelope system, such as ply or other sheet systems supporting insulation and the roof system.

Regular floor diaphragm options						
PLY	PB	OSB	CLT	GLP	LVL	**Notes** Specific fixing requirements may apply to ensure adequate shear transfer.

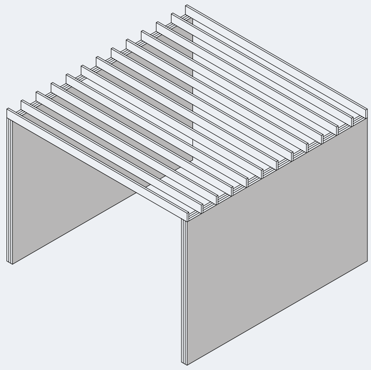
Regular roof diaphragm options					
PLY	OSB	CLT	CASSETTE	SSP	**Notes** Specific fixing requirements may apply to ensure adequate shear transfer.



Sheet material bracing wall



Sheet material bracing wall



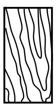
Massive timber wall

Description

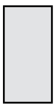
As well as carrying loads vertically to the foundation, stud frame walls clad with ply or similar sheeting material and massive timber walls can serve as bracing diaphragms or shear walls. Effectively anchoring the diaphragm to the surrounding structure or foundations is critical. This is especially the case for stud frame walls where high bracing loads can buckle relatively thin sheet products and distort the wall frame. Specific fixing frequency and spacing requirements also apply for installing the sheet material on the frame.

Window and door openings will affect the design of shear walls, and their size and placement should be considered early in the design process. Large openings, especially near the corners of a structure, can create high stress concentrations and greatly increase the shear wall system's cost.

Regular floor diaphragm options



PLY



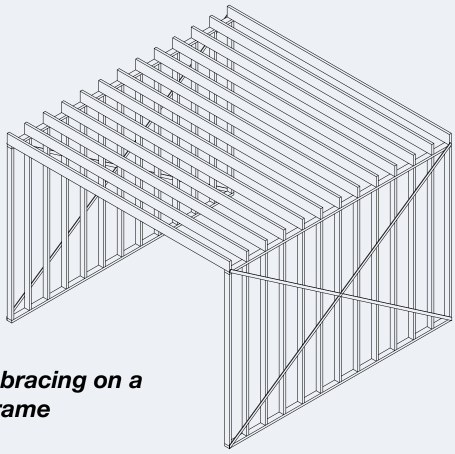
HDF

OSB

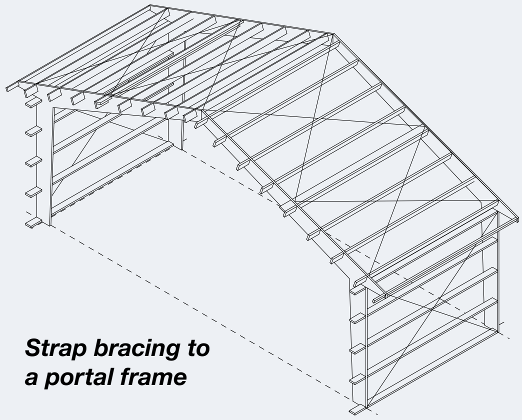
Notes

Bracing capacity from sheet material relates to the sheet's continuity, its thickness, the number and type of fixings that secure it to the frame and the frame's fixing to the surrounding structure.

2.6.3 Lateral resistance systems: Strap braced panels



Strap bracing on a wall frame



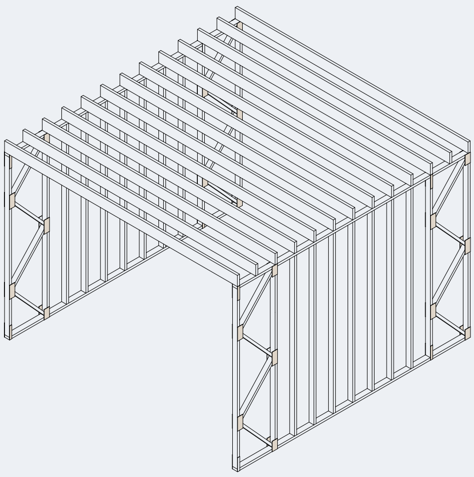
Strap bracing to a portal frame

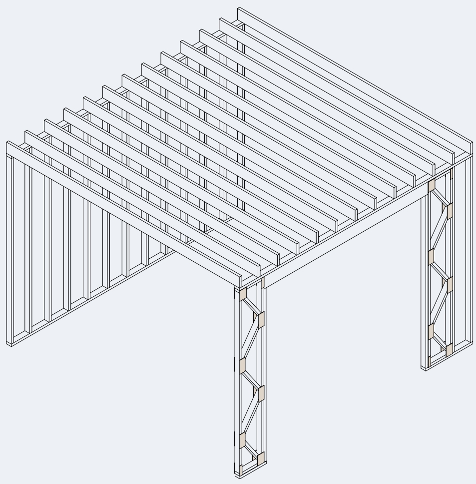
Description

Steel strap bracing can be installed as cross-brace pairs on the face of studs, purlins or girts to provide an effective and economic bracing option. It is an efficient alternative to sheeted diaphragms in walls and roofs. Strap can be wrapped around elements, simply fixed at the ends and to intermediate members and tensioned with proprietary tighteners. It is suitable for concealed or industrial applications where appearance is not a primary performance requirement.

Strap bracing only works in tension. To be effective, it needs to be applied in both directions across a square or rectangular panel of the frame. As it does not generally provide the same bracing capacity as sheet systems for each lineal meter braced, it is often used where ample wall or roof area is available. Specific nailing and tensioning requirements apply to installation of the strap, the fixing of timber elements to each other at the end of the strap, and the frame's fixing to the surrounding structure.

2.6.4 Lateral resistance systems: Truss panels

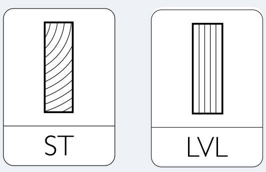




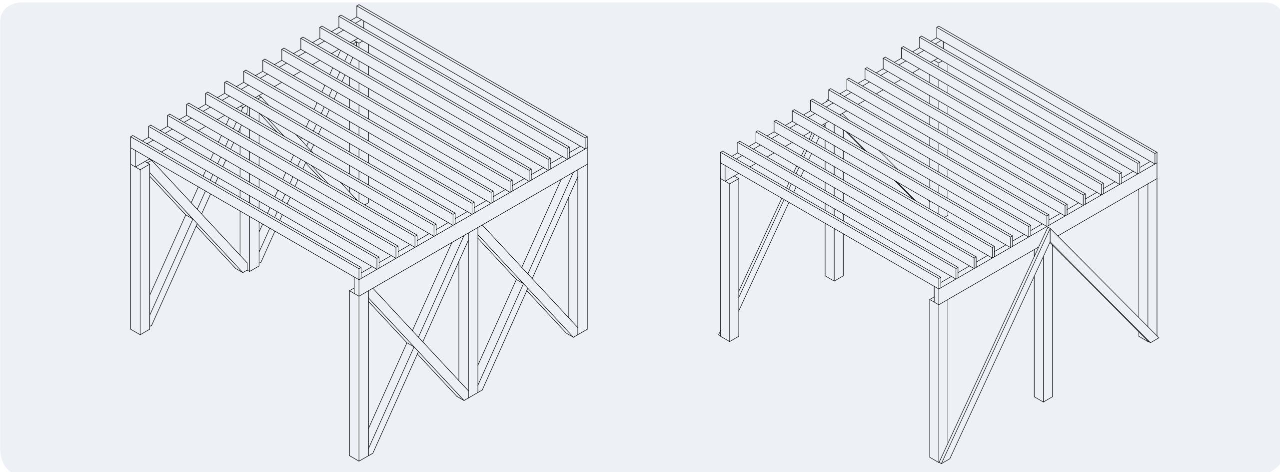
Description

Nailplate trusses can be incorporated into the plane of the wall, roof or ceiling and fixed to other elements to provide an effective bracing option. The bracing resistance achieved relates to the size and configuration of the truss and the efficiency and quality of its fixing to the surrounding structure, particularly the unit's tie-down at the ends of the panel. These should be continuous to the ground. Generally more expensive than strap or sheet bracing, truss panels are most regularly used in domestic structures to provide high bracing capacity in short lengths of wall near large openings or in the roof or ceiling planes when the spacing between bracing walls falls beyond normal practice. Due to the tie-down requirements, these bracing walls are most suited to ground floor applications.

Regular truss bracing options



2.6.5 Lateral resistance systems: Timber or steel rod bracing

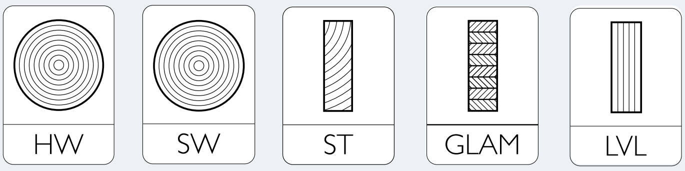


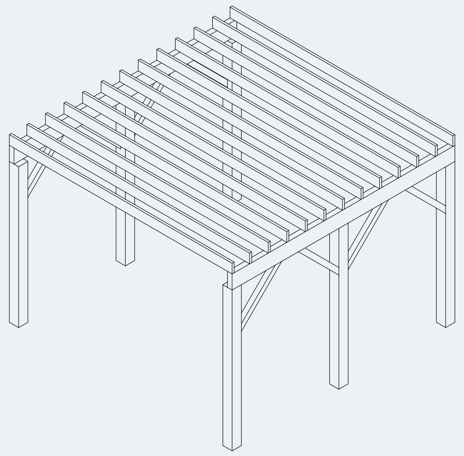
Description

Solid timber and steel rod bracing is common in larger structures, especially where large forces apply or where the support system consists of columns rather than walls. Braces are generally more efficient when they act in tension as buckling does not need to be resisted. However, connections may be simpler in braces working in compression. Often, braces are designed to act in tension or compression, but not both, and are arranged in matched pairs. Timber bracing may be made from timber rounds, sawn timber or engineered products in single or pairs of elements.

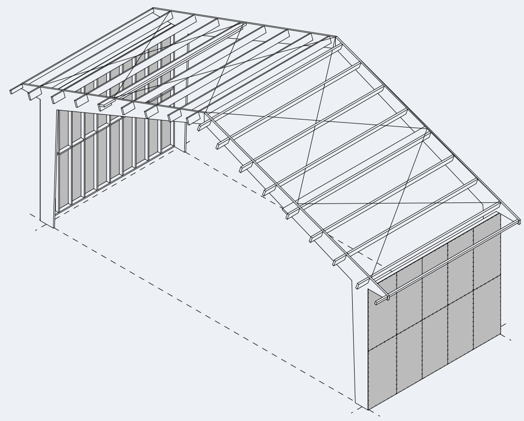
If the bracing only needs to act in tension, steel rods or cables are often used, especially for horizontal bracing in roofs. In these cases, they will be installed as cross-brace pairs to resist lateral loads coming from either direction, with only one half active under a given load condition.

Regular timber bracing options





Knee braces used to create a moment-resisting joint



Moment-resisting joint in a portal

Description

Lateral loads may be resisted by using moment-resisting connections to make a rigid frame of sections of the structure, such as those found in a portal frame. This allows bending action in the spanning and supporting elements to resist lateral loads in the plane of the frame. An additional bracing system is typically used to resist loads acting perpendicular to the frame.

When columns are used to resist lateral loads, the bending stresses generated in the elements will be much higher than if the columns were only used in the support system. This can result in much larger columns. The column must be fixed against rotation at the top, the bottom or both. If both are fixed, the column will act in compound bending, resulting in lower overall moment stresses. The savings on column size must be compared to the increased cost of additional moment-resisting connections. Creating lateral load resisting frames will have less impact on the size of spanning elements, because the spanning elements are already designed to resist bending stresses. The increased demand for lateral resistance may even be offset by the reduced demand that results from designing the beam with fixed end connections and compound bending.

3 Connection Options

Wood is a diverse and highly workable material. This generates a broad suite of timber or wood products and elements and a wide range of methods, connectors and fasteners to join these elements together in timber structures. This variety offers the designer opportunities for creativity in structural assembly and appearance but demands a robust understanding of the principles of good connection design with wood.

Given their importance in timber structures, this section describes the major options for making reliable structural connections in timber elements in the fabrication workshop and building site. Section 3.2 describes fasteners, Section 3.3 describes connectors, and connection arrangements are covered in Section 3.4. Other aspects of making structural connections are discussed in Section 6.1.

A connection in a timber structure encompasses all the components that are used to attach one building component to another: timber to timber or timber to steel or other material. A connection will include fasteners and usually connectors. Fasteners include nails, dowels, screws, bolts and similar fixings that connect the timber and other elements directly or in combination with connectors. Connectors include gussets, metal hangers, brackets, and other proprietary and custom-made items used with fasteners.

3.1 Connection approaches and types

There are two broad classes of timber connection approaches: adhesive-based and mechanical-based. This guide focuses mainly on mechanical-based approaches.

Adhesive-based approaches use glue to connect two wood surfaces mechanically and chemically in a bond whose strength is ideally equal or greater than the strength of the solid wood it joins. High-quality glued joints can be achieved if gluing conditions such as adhesive temperature, surface preparation, clamping pressure and curing times are carefully controlled. However, these conditions are difficult to achieve and maintain on site.

Mechanical-based approaches use contact transfer, fasteners and connectors to transfer load and forces from one piece of wood to another. Joint quality and capacity are usually dependent on the fastener number and size, their spacing relative to the grain in the piece, and the wood's mechanical properties. As these conditions can be controlled in the workshop and on site, mechanical-based connection approaches regularly provide reliable structural connections in construction.

3.1.1 Mechanical connection groups and types

There are hundreds of different ways to configure a mechanical-based timber joint due to: the variety of timber products and connectors available, the connection's potential geometry, and the direction and size of the loads that can be applied through the joint. While it is impossible to cover all of these configurations, it is useful to separate combinations first into broad groups and then into regularly used connection types.

In this guide, mechanical-based timber connections have been divided into three main groups, based on the method by which they achieve primary load transfer between members through the joint. These groups are:

- **Timber-timber contact** connections.
- **Timber-fastener** connections. In these, loads move between timber elements through mechanical fasteners such as nails, screws or bolts.
- **Timber-connector-fastener** connections. In these, loads move from a timber element to a connector such as a gusset through fasteners, and then back from the connector into other timber elements.

As both primary and secondary loads can be applied to a joint, forces may have to be transferred in several ways in a connection. For example, a floor joist sitting on a beam transfers its primary gravity loads through a timber-timber contact connection. However, the joist may also be nailed or strapped to the beam to keep it in place and to resist uplift.
In this case, secondary loads will be resisted by either a **timber-fastener** or a **timber-connector-fastener** connection, or a combination of both.

These three groups can be separated further into eight connection types, described in detail in Section 3.4. Table 21 lists the connection types discussed in this guide, and the primary and secondary load transfer methods common in each type.

Table 21: Load transfer through the joints by connection type

Connection type	Timber-timber contact	Timber-fastener	Timber-connector-fastener
Contact transfer	■	□	□
Fasteners		■	
Gusset plate with fasteners	□		■
Nail and nail-on plates	□		■
Fin plates with fasteners	□		■
Epoxy dowels	□		■
Interlocking housing	■	□	□
Transfer blocks	□	■	

Legend: Primary load transfer method ■ *Regular secondary load transfer method* □

Loads applied through the joint, fasteners and connectors can act in compression, tension or shear. Transfer of tension and shear forces typically requires loads to be carried through fasteners in a timber-fastener connection or a timber-connector-fastener connection. In *AS 1720*, these joints are classified in two types:

- Type 1 where forces are transferred as shear loads in the fastener
- Type 2 where forces are transferred through axial loads (typically tension) in the fastener.

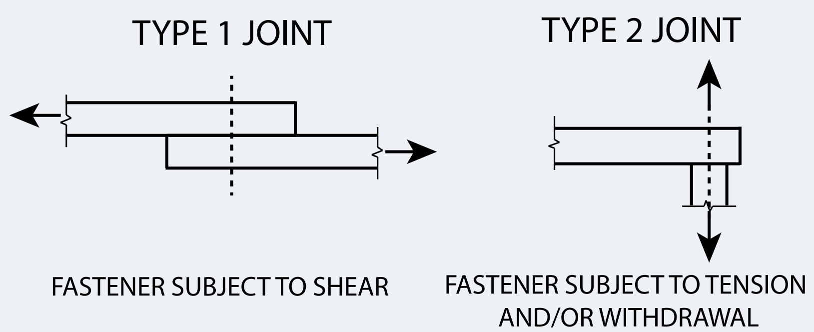


Figure 2: Joint type and direction of forces

3.2 Fasteners

Fasteners are metal, plastic or wooden devices with or without a thread that mechanically join timber with timber, timber with a connector, or timber with another material. This guide covers three types of timber fasteners: dowels, screws and bolts. Other, less common fastener types also exist.

3.2.1 Dowels

Dowels are metal, plastic or wooden pins driven into the wood with or without a pilot hole. These fasteners resist shear or axial withdrawal loads and include nails (thin dowels) and metal or wooden dowels.

Nails

Nails are the most commonly used and economical of all fasteners. Nails are generally thin metal fasteners suitable for Type 1 and Type 2 connections. *AS 1720.1* references *AS 2334 Steel nails – Metric series.*

Nails are generally driven into the timber and the nail's point pushes most of the wood fibres apart. Friction between the nail's shank and the wood fibre provides the nail's withdrawal strength. As most wood fibres remain intact, directly driven nails do not reduce the timber's effectiveness in section. However, they do generate tension across the timber's grain. To avoid the timber splitting, *AS 1720.1* limits nail spacing and their minimum distance from the piece's end and edge, summarised in Table 22. In hard timbers, nails may be driven into holes predrilled to 80% of the nail diameter (D). These are generally required when nailing into timber with a density over 650 kg/m^3. It is not usually required for lighter species.

Nails vary by shank and head type. Standard or plain shank nails are smooth and can be used in temporary or general construction. Nails with annular ring and spiral shanks are also available and provide higher withdrawal strength than plain shank nails.

Table 22: Minimum spacing, edge and end distances for nails (D = nail diameter)

Spacing types	Minimum distance – directly driven	Minimum distance – pre-bored
End distance	20D	10D
Edge distance	5D	5D
Spacing – along the grain	20D	10D
Spacing – across the grain	10D	3D

Source: AS 1720.1 Table 4.4

Dowels

Dowels are generally metal but can be wood. Metal dowels are lengths of round machined material driven in a hole drilled in the timber to the same diameter as the dowel. Similar to bolts but without the thread or head, the dowel performs a similar function in Type 1 joints but has no capacity to resist or transfer axial loads. Dowels are a much less obtrusive connector than bolts and have architectural appeal. They can be fire resistant if covered with plywood or recessed and finished with a timber plug. Dowels can be manufactured in a range of sizes, but 10 mm to 16 mm dowels are the most common. Most are high tensile steel. To ensure that the connected members remain in close contact, dowel joints should be tied together with a number of bolts or screws. Small diameter dowels in close fitting holes are better modelled by nail behaviour.

3.2.2 Screws

Screws are the fastener type subject to most recent technical development. They have become a standard solution for reliably and economically joining massive timber components in multi-storey buildings. Numerous lengths and types of screws are now available for joining timber to timber directly, or surface-mounted or inset connector plates to timber. However, all screws involve cutting a hole in the wood that reduces the timber's effective cross-section and may affect member strength at that point.

Screws are suitable for Type 1 and Type 2 connections and can be self-drilling or set into pre-drilled pilot holes. The fastener strength is significantly greater than that of the same sized nail. *AS 1720.1* references *AS 3566.1 Self-drilling screws for the building and construction industries* and limits the spacing and the minimum distance of screws from the piece's end and edge. These requirements are summarised in Table 23. The nominal diameter of modern wood screws generally relates to the thread's outer diameter, not the shank thickness. In design, it is necessary to specify a 'minimum root diameter' (d_{min}) and base the load capacity on an effective diameter (def) of 1.1 d_{min}. Typically, any pilot hole is 65% of the screw root diameter in outside thread screws and of the outside thread diameter in inside threaded screws in softwoods with density < 650 kg/m^3. It is 90% of the screw root diameter in outside thread screws and of the outside thread diameter in inside threaded screws in hardwoods with density >650 kg/m^3.

Modern self-drilling screws can incorporate: a sharp self-perforating tip; a notch to cut through the grain; one or more lengths of thread that draw the screw into the material and bind it into the wood around the screw; often a cutter which creates the space for the shank's passage, the shank, and a head that generates tension between itself and the thread along the shank when it is driven down onto the wood.

The length and form of the screw, the tip's design, the thread's size, length and placement, the shank's length and placement, and the head's size and shape can all be varied to improve the screw's performance in specific applications. For example, a metal cutting tip can replace the tip and notch and allows the screw to drill through metal plates while the shank can be lengthened so that the screw can act more like a traditional dowel inserted through the wood and metal plate.

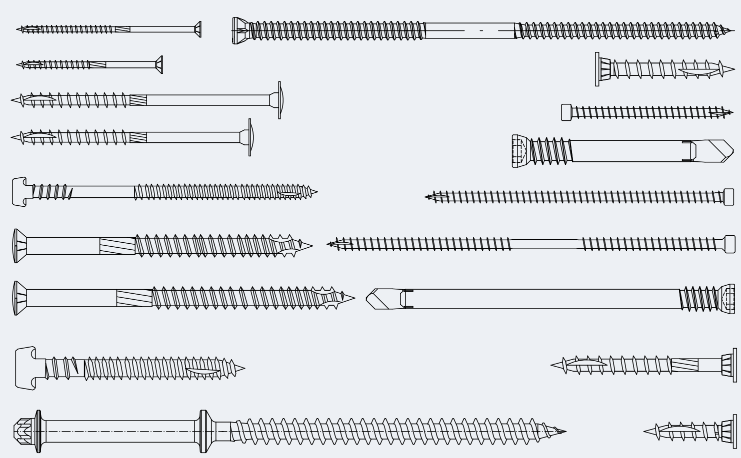


Figure 3: Variety of screw configurations (Courtesy of Rothoblaas SRL)

Table 23: Minimum spacing, edge and end distances for screws (D = screw diameter)

Spacing types	Minimum distance
End distance	10D
Edge distance	5D
Spacing – along the grain	10D
Spacing – across the grain	3D

Source: AS 1720.1 Table 4.8

Coach screws are a more traditional form of screw suitable for Type 1 and 2 connections. *AS 1720.1* references *AS/NZS 1393 Coach screws – Metric series with ISO hexagon heads*. *AS 1720.1* limits their spacing and the minimum distance of screws from the piece's end and edge. These are summarised in Table 25.

Coach screws normally include a threaded section, a shank and a hexagonal head similar to a bolt. See Figure 4. They are commonly used in applications in which a bolt would normally be used, but in which one end of the bolt is not accessible. Coach screws have a much longer threaded portion than the same size of bolts. The plain shank's length is important to enable the correct depth of embedment in the innermost element.

3.2.3 Bolts

Bolts are metal connectors that are installed into pre-drilled pilot holes in the timber. These holes should be larger than the diameter of the bolt shank but not by more than 2 mm. In oversized holes, the frictional forces on the side of the bolt shank can create cleavage stresses that reduce the system's capacity. Similarly, undersized pilot holes can create tension perpendicular to grain in the timber.

A washer under the bolt's head and under the nut ensures that there is adequate bearing on the timber to transfer tension forces from the bolt to the timber without crushing it. Bolts should not be over-tightening. Typically, no more than 35 N-m of torque should be applied to nuts. This ensures the washer is snug to the side of the timber. Only one side of a bolt and nut should have a spring washer. Over-tightening the nut in an attempt to reduce the need for later retightening when unseasoned sections dry and shrink is counterproductive. The timber under the washer can be permanently deformed while the timber will still season and shrink and the fasteners loosen. Also, the connection strength may actually be reduced by overtightening.

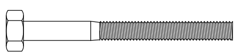
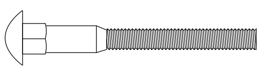
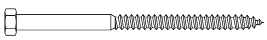
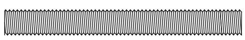
Under shear load, the bolt's shank presses against the timber's end-grain in the pre-drilled holes. In high loads, this can lead to localised crushing. As using bolts involves drilling holes in the wood, the timber's effective cross-section is reduced, impacting the member's strength at the connection. As the pre-drilled bolt hole is marginally larger than the bolt, these connections characteristically have more slip than nail or screw connections. Bolt installation is quite labour intensive and may require closer supervision than the driving of nails.

Bolts are a traditional form of fastener suitable for Type 1 or 2 connections but are mainly used in Type 1 connections. *AS 1720.1* references *AS 1111.1 ISO metric hexagon bolts and screws – Product grade C – Bolts*. It also limits the spacing and the minimum distance of bolts from the piece's end and edge. These requirements are summarised in Table 25.

The types of bolts available and a description of their general application is included in Table 24. Available in a range of sizes and grades, the standard bolt grade for timber structures is Grade 4.6, while Grade 8.8 is more than adequate. The most commonly used sizes for joining structural timber are M10, M12, M16, M20 and M24.

Bolts are ordered by diameter, length and grade. Only a portion of the shank is threaded, so in detailing a connection, care is needed to ensure that after allowing for the thickness of washers, there is enough thread on the bolt to enable the nut to clamp the connected members securely together. If the joint's overall thickness cannot be determined, treaded rod can be used instead of bolts. In applications that use unseasoned timber, the wood will likely shrink as the timber dries after installation and the bolts will require periodic retightening. Sufficient thread should be available to allow for this.

Table 24: Bolt types and application

Type	Diagram	Application
Hexagonal head bolt		General structural purposes
Cup head bolt		Structural purposes where the head must be relatively flush with the surface
Coach screw		Used to replace a bolt where one end of the bolt is not accessible
Threaded rod		Locations where it is difficult to specify the bolt length beforehand, such as in tie-down rods or pole construction

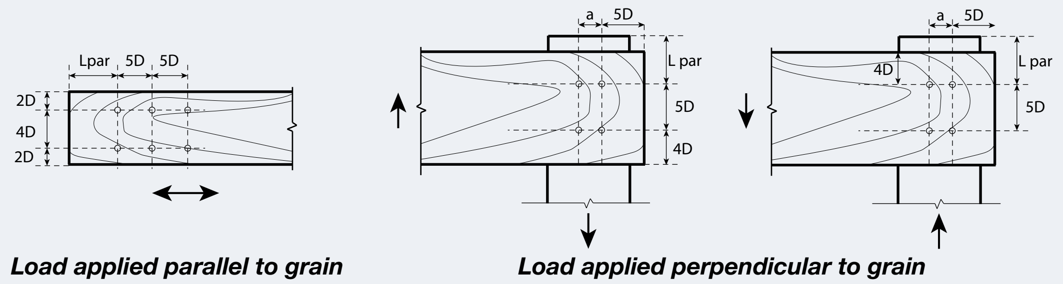


Figure 4: Bolt spacing to AS 1720.1 (Source: AS 1720.1 Fig 4.9 Bolt spacing)

Table 25: Bolt and coach screws – spacing (D = bolt diameter, b = timber width relative to bolt axis)

Spacing types	Minimum distance
Load parallel to the grain	
End distance – tension joint, unseasoned timber	8D[1]
End distance – tension joint, seasoned timber	7D[1]
End distance – compression joint	5D
Edge distance	10D
Spacing – along the grain	5D
Spacing – across the grain	4D
Load perpendicular to the grain	
End distance	5D
Edge distance	4D
Spacing – along the grain[2] – for b/D = 2	2.5D
Spacing – along the grain[2] – for 2 < b/D < 6	1.25 + 0.625b/D
Spacing – along the grain[2]– for b/D > 6	5D
Spacing – across the grain	5D

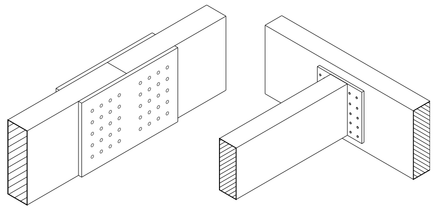
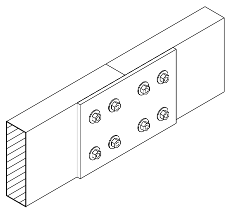
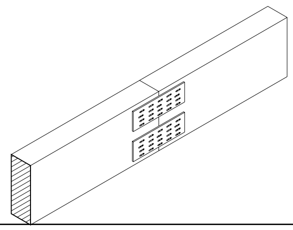
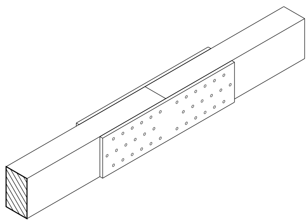
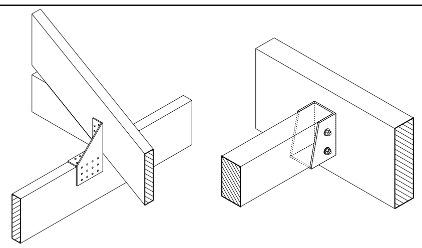
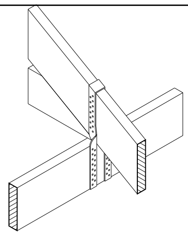
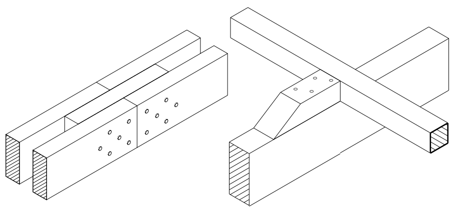
Source: AS 1720.1 Part 4.4.

Notes:
1. End distances may be reduced but capacity must be reduced proportionally. See *AS 1720.1*.
2. Dimension a in Figure 3.
3. For load applied at an angle to the grain, use the *load parallel to grain* distances for grain angles from 00 to 300.

3.3 Connectors

Connectors are metal or wooden devices used in association with fasteners to join timber elements together or connect them to other materials. There are hundreds of different connector options available as proprietary products or custom made items. This guide lists seven major connector types in Table 26. Each has numerous variants.

Table 26: Connector types

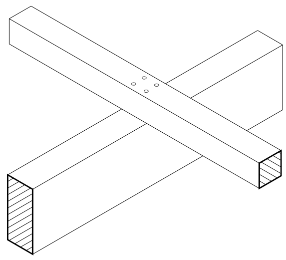
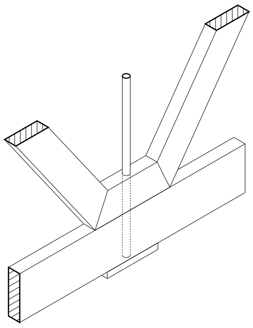
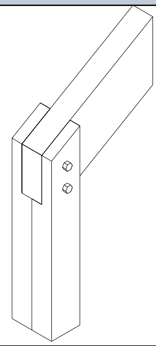
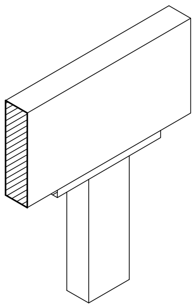
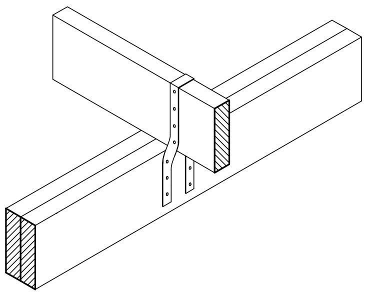
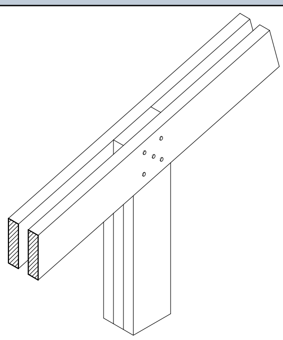
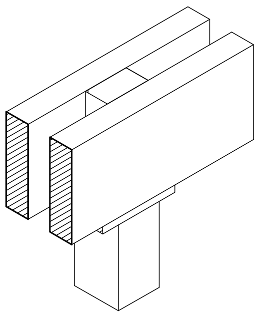
Illustration	Description
	**Wood gussets** Wood gussets are usually sections of plywood or occasionally LVL used with fasteners to join timber elements. Gussets can be applied parallel to the grain or on the end grain of one element and the face of another.
	**Metal gusset plates** Metal gusset plates are generally custom-made steel connectors used with fasteners to join timber elements. Plates can be applied to the face of an element, set into a groove in it, or be sandwiched between elements. Plates can be flat, simply folded, or welded to a steel section to form a connecting surface to other materials.
	**Nailplates** Nailplates are propriety sheet metal connectors stamped so that nails are formed and protrude on one face. These make a reliable connection when pressed into the timber on each side of a joint. Most are made to be installed with specialist presses but some can be hammered in.
	**Nail-on plates and brackets** Nail-on plates are generally propriety sheet metal connector plates with pre-punched holes ready to receive fasteners such as nails. They can also be custom made. Plates can be flat, include a 900 twist, be simply folded or welded to a steel section to form a connecting surface to other materials.
	**Brackets** Brackets are folded or fabricated sheet metal or steel plate items used to connect the end of one element with the side or end of another. Numerous proprietary brackets such as truss boots and joist hangers are available to connect timber elements to the ground or other timber elements. Fabricated brackets can range from a simple boot to complex steel nodes assembled for attaching radial members.
	**Straps and ties** Strap is generally a proprietary sheet metal product used to resist tension in many connection types. Available in various gauges and widths, it is also used in bracing panels. Ties are generally steel rod or similar sections used to resist tension in connections or elements subject to wind and similar loads. They are also used in bracing panels.
	**Timber blocks** Timber blocks can act like a gusset or bracket to join timber elements. They can be shaped to support elements and provide additional area for fasteners.

Other types of connectors exist. These may be specialist items or obsolete historical connectors, such as split ring and shear connectors.

3.4 Connection types

As described above, there are hundreds of different ways to configure a mechanical-based timber joint. However, it is useful to group regularly used connection types based on how they achieve primary load transfer through the joint and their primary connector type. The eight types included in this guide are necessarily broad, due to the variety of timber products and connectors available, the potential connection geometry, and the expected load. Significant overlap can occur between the connection types. Several connectors may be combined to make an effective connection.

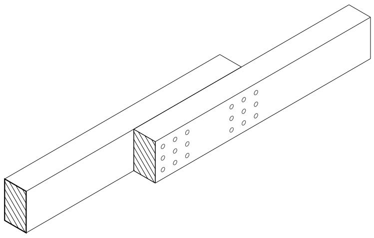
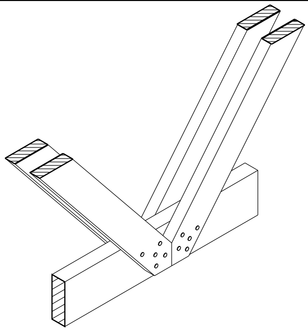
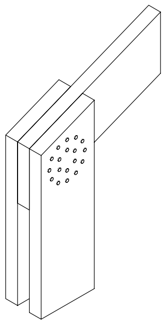
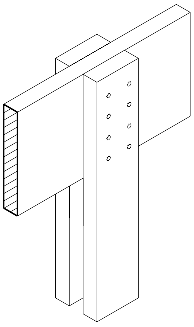
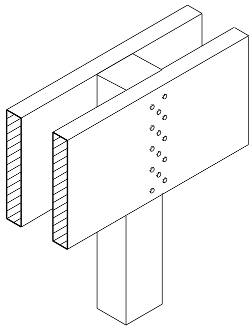
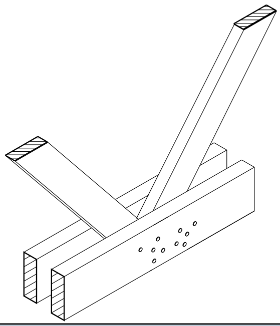
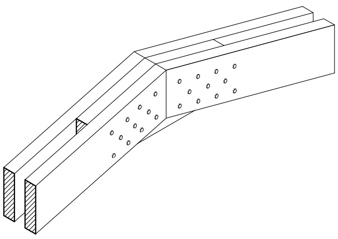
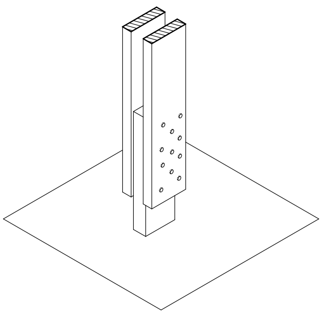
3.4.1 Contact transfer

Purlin or battern to beam	Truss node	Rafter to a column
		
Beam to post	**Joist to bearer**	**Beam to post**
		
Rafter to a column	**Beam to post**	
		

Description

Contact transfer connections transfer compressive load between elements primarily through timber to timber contact. Examples are a stud sitting on a plate or a joist sitting in a bearer. These connections can join pieces in the plane of the primary element, such as in a truss, or across it. In joints with high loads, elements may bear on each other indirectly through a reinforcing plate or corbel. As timber is softer across the grain than along the grain, these spread high compression loads over a larger area of timber and allow room for additional or more accessible fasteners. Joint positioning and secondary load transfer is often through timber-fastener or timber-connector-fastener connections, such as screws or strap.

Contact transfer connections are very common and economic and are used in most types of timber construction systems. They accommodate simple gravity loads easily. In these cases, fasteners and connectors provide joint stability and resist lateral or uplift forces. Connection to other materials is often through a timber plate fixed to steel or concrete, or a fabricated bracket.

In-line join	Truss node	Rafter to a column
		
Beam to post	**Beam to post**	**Truss node**
		
Frame apex	**Post to the ground**	
		

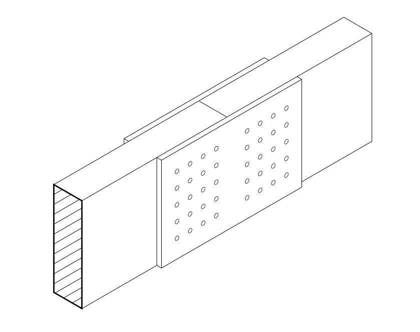
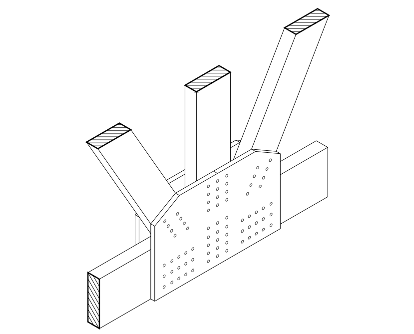
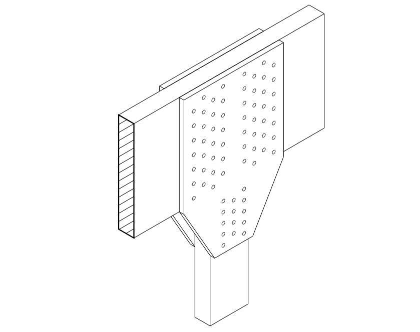
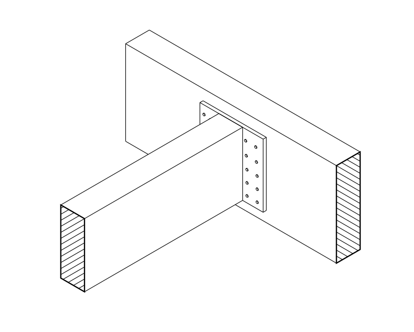
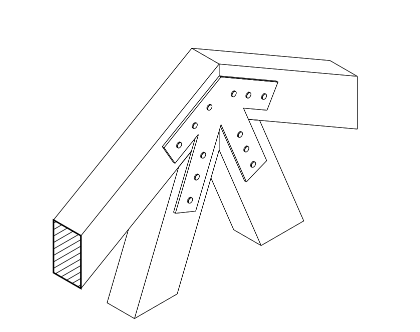
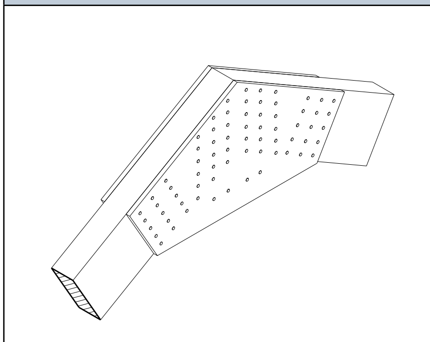
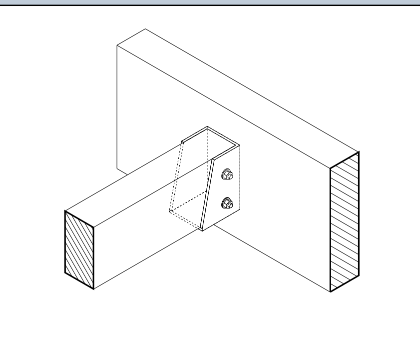
Description

Timber-fastener connections transfer loads from one timber element to the other through shear loads (Type 1) or axial loads (Type 2) in the fasteners. Joint capacity is determined by the size and number of fasteners, their capacity and the timber's joint group. Most fastener connections work with the fastener installed perpendicular to the grain and acting in shear. However, nails and screws can be effective installed into the end grain or sloped across the grain of the piece.

Timber and fastener connections are very common and highly versatile. They can form simple tension and compression connections or multiple fasteners can be installed to form moment-resistant joints. The timber to be joined in a timber-fastener connection must overlap, so pieces must be either offset from the plane of the primary element or housed into each other. As housing is labour intensive and reduces the available timber section significantly, it should only be used with caution.

The economy of timber and fastener connections varies with fastener cost and the required labour time. Nails and screws can generally be installed on site reliably, quickly and economically. Steel dowels require holes that are accurately drilled to the dowel's exact size, and workshop preparation is usually necessary. The accuracy required for bolt connections will vary with the application's quality. However, if tight, low slip connections are required, workshop preparation is required.

Fire resistance can be provided if the fastener head is covered with ply or other sheet material or the timber sections are wide enough to recess the fastener head and cap it with a timber plug.

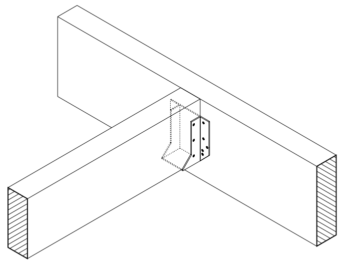
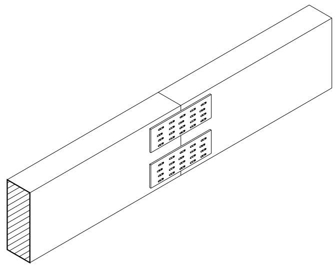
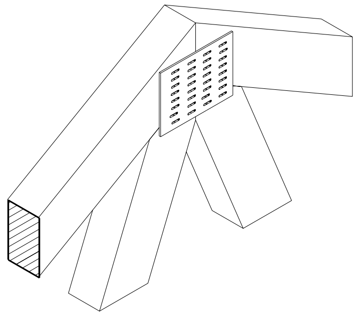
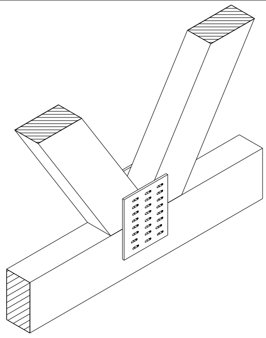
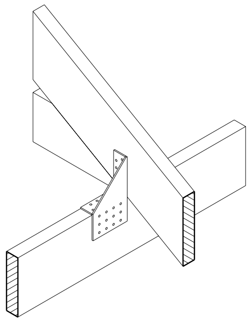
In-line join	Truss node	Rafter to a column
		
Beam to post	**Rafter to beam**	**Truss node**
		
Frame apex	**Rafter to beam**	
		

Description

Gusset plates are shaped sections of plywood, LVL or metal, used in conjunction with fasteners to make a timber-connector-fastener connection. Usually, the elements are sandwiched between gussets on each side of the joint. Load is transferred from the element to the fasteners, into the gusset and then back through the fasteners to other members.

Gusset plate connections are highly versatile and economic means of transferring loads through wood and they are used to form tension, compression and moment-resisting connections. They allow timber elements to be arranged and joined in the same plane, without being offset or housed. Traditional gusset arrangements have steel plates connecting elements using relatively few fasteners: either bolts or coach screws. With these solutions, round washers under the head provide a better appearance than square ones. With the development of nail guns and self-drilling screws, ply and light steel gusset plates are nail or screw fixed, with fasteners in groups sufficient to transfer the required load. High load connections may have thousands of fasteners installed in a predetermined pattern in each gusset. The loads to be resisted, the fastener capacity and their required spacing dictates the fixing pattern and the gusset's minimum size. The fire-resistance of connections can be improved with an additional layer of plywood or timber over the gusset.

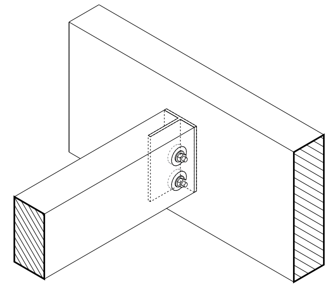
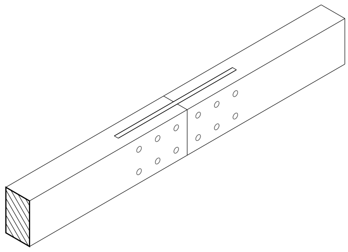
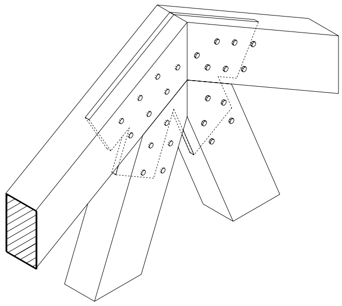
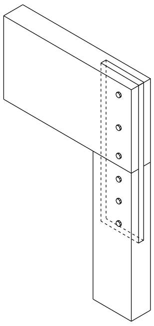
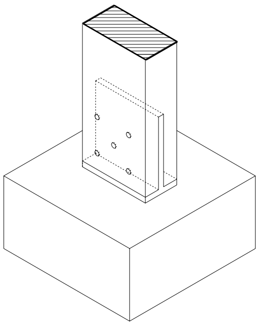
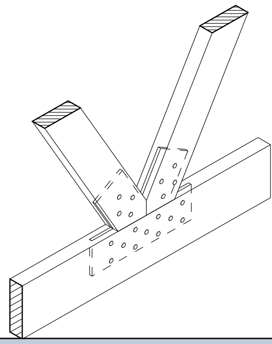
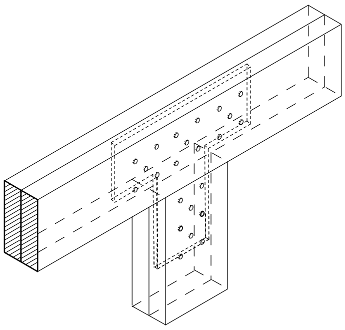
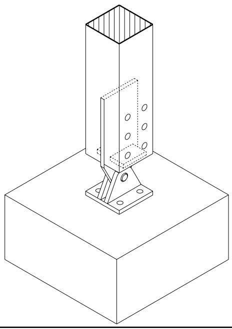
Gussets can join elements at right angles through fabricated steel brackets or a simple gusset applied to a cut end. In this guide, fabricated bracket connections can be grouped with gusset plates, as load transfer can occur in a similar manner. Brackets can act simply as a folded gusset plate or include a seat that transfers load through contact transfer.

Joist to beam	In-line join	Truss node
		
Truss node	**Truss to beam**	**Post to the ground**
		

Description

Nail and nail-on plates are generally proprietary gusset plates generally made from galvanized steel used to make a timber-connector-fastener connection. Nailplate connections are versatile, reliable and widely used to connect timber of the same thickness in the same plane. They can effectively laminate timber and LVL components into longer and wider pieces or form the node connection in various forms of trusses. However, as proprietary software controls the design of nailplated elements and joints, these connections are almost always workshop-installed by licensed fabricators. *AS 1720.5 Timber structures – Nailplated timber roof trusses* controls the design of these connections for roof trusses.

Nail-on plates are also versatile, reliable and widely used to connect timber elements of the same thickness in the same plane. They make use of the shanks of nails or screws to transfer loads from the timber to light gauge steel plates. A wide range of specialist folded nail-on plate connectors are available and are widely used in light timber framed construction. Heavier gauge painted or galvanized plates are also available for fixing massive timber components. Technical design support for these connectors is usually available from the producers. Nail-on plates combined with steel angle can joint timber elements to concrete slabs and other elements.

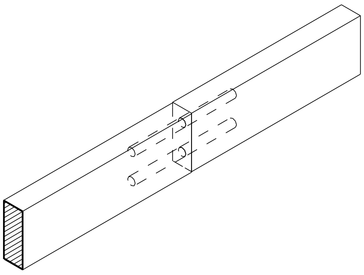
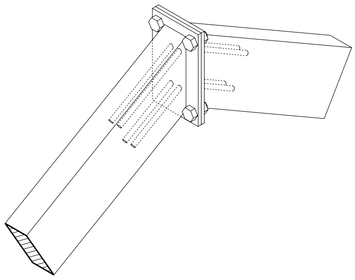
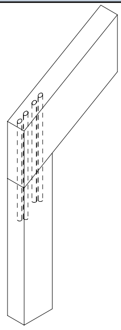
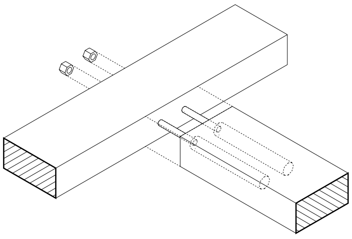
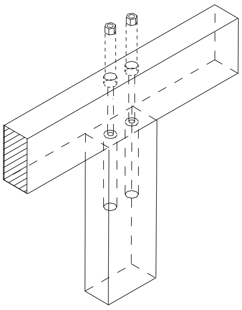
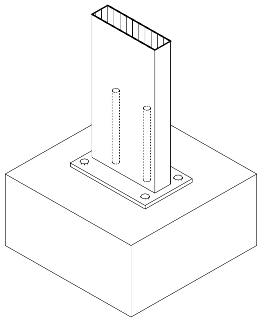
Rafter to beam	**In-line join**	**Truss node**
		
Beam to post	**Column to the ground**	**Truss node**
		
Beam to post	**Column to the ground**	
		

Description

Fin plates are specialist metal gusset plates set into shaped recesses in a timber section or sandwiched between two timber elements to make a timber-connector-fastener connection. Fin plates are usually fixed with steel dowels, screws, nails or occasionally bolts. Fin plate connections can form tension, compression, shear and moment-resisting joints and allow timber elements to be arranged and joined in the same plane. The loads to be resisted, the fastener capacity and required spacing, and the timber's joint group dictate the fastener pattern and the fin plate's minimum size.

Fin plates are discrete and provide architecturally attractive connections. However, they can be expensive. As dowel or bolt holes must be accurately drilled to exact dowel sizes, workshop preparation is usually necessary. To ensure the correct alignment of steel and timber, holes are often drilled when the components are assembled, or the predrilled steel plates are used as templates to drill the timber. If final assembly takes place on site, the plates and timber must be matched to ensure the joints fit exactly. Metal cutting screws reduce the cost of dowel connections as they can be drilled directly through the timber and metal plate. They also produce a very rigid joint as there is very little play between the timber, connector and fastener. A fin plate connected to a fixing plate at right angles can join timber to other materials

3.4.6 Epoxy dowels

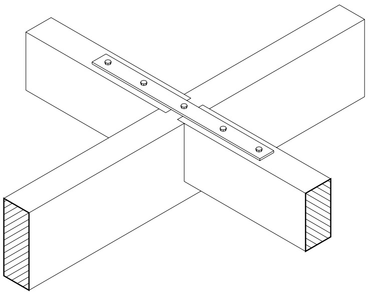
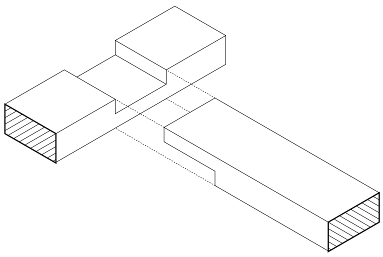
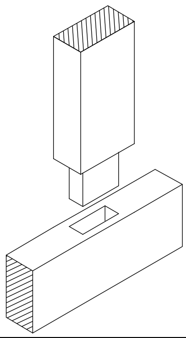
Rafter to beam	In-line join	Truss node
		
Beam to post	**Column to the ground**	**Truss node**
		

Description

Epoxy dowel connections are specialist workshop-assembled joints where steel or wood rods are glued into holes in the timber and used to connect several timber elements together. Some steel rods are finished with ferrules flush with the end of an element to receive a bolt or welded to steel plates before gluing to form a connection plate for site assembly.

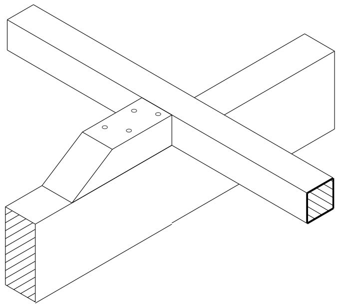
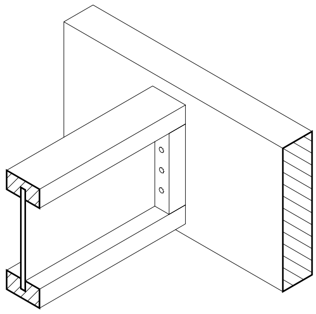
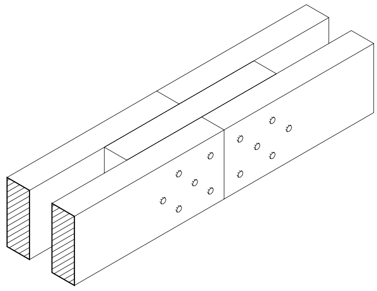
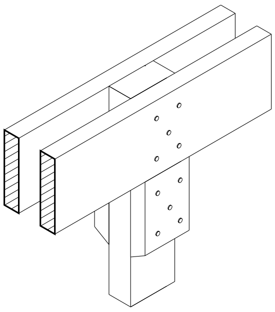
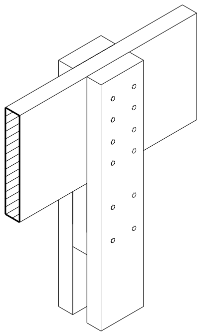
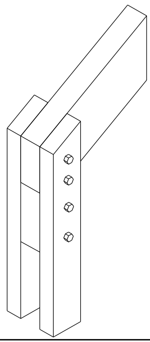
Epoxy dowel connections can form tension, compression and moment-resisting joints and allow timber elements to be arranged and joined in the same plane. The connection relies on the glue forming a full bond between the rod and the timber so workshop assembly is essential.

3.4.7 Interlocking housing

Joists housed into beam	Half housing of plates	Mortice and tenon
		

Description

Interlocking housings are traditional carpentry connections formed from recesses and matching shapes cut into timber to locate and support the joining elements in the desired plane and enable contact load transfer. They are generally poor in tension and have to be held together with fasteners or connectors. While dovetail and similarly wedge-shaped connectors are architecturally attractive, they are expensive and have poor load transfer capacity except in compression. Housings, mortices and similar recesses reduce the element's effective section considerably and generally need to be cut in a workshop to ensure accuracy. However, they can be assembled on site. The MC of elements at fabrication is important. Wet material can shrink considerably and the joint can loosen up in service.

Purlin or batten to beam	Purlin to rafter	In-line join
		
Beam to post	**Beam to post**	**Rafter to post**
		

Description

Transfer blocks are timber or LVL connectors used to join timber elements together in or across the plane of the primary element or to reinforce timber-fastener connections. When loads are transferred through a timber-fastener connection, the number of fasteners required can exceed the number that can be accommodated in the area of timber overlap available in the joint. This can be overcome by increasing the element's size and overlap or by including transfer blocks that receive the additional fasteners. These blocks can be cut to the required size and shape and positioned to locate and support other elements as they are assembled on site. While cutting the blocks and fixing them to elements is more efficient in the workshop, they can be made and fixed fully on site.

3.4.9 Joint summary

Table 27: Summary of joint characteristics

Description	Joins members in plane	Fire-resistant	Workshop manufacture required	Cost range	Method in *AS 1720*
Contact transfer					
Element across element with fasteners and strap	✗	✗	✗	$	✓
Reinforcing plate between column and beam	✓	✗	✗	$	✓
Fasteners					
Nailed or screwed timber to timber	✗ unless housed	✗ unless covered	✗	$	✓
Steel dowelled timber to timber	✗ unless housed	✗ unless recessed and plugged	✓ unless strict site control is provided	$$	✓
Coach screwed or bolted timber to timber	✗ unless housed	✗ unless recessed and covered	✗	$	✓
Gusset plate with fasteners					
Nailed or screwed plywood gusset to timber	✓	✗ unless covered	✗	$	✓
Nailed or screwed external steel gusset to timber	✓	✗	✗	$$	✓
Coach screwed or bolted steel plates to timber	✓	✗ unless recessed and covered	✗	$$$	✓
Nail and nail-on plates					
Nail-on plate to timber	✓	✗ unless covered	✗	$	✓
Nail-on bracket to timber	✓	✗ unless covered	✗	$	✓
Nailplate connector pressed into timber	✓	✗ unless covered	✓	$	✗
Fin plates with fasteners					
Steel dowelled or bolted internal fin plate to timber	✓	✗ unless recessed and plugged	✓ unless strict site control is provided	$$$	
Epoxy dowels					
Bonded in steel rods	✓	✓	✓ unless site made by specialists	$$$	✗
Interlocking housing					
Various carpentry connections	✓	✓	✓ unless site made by specialists	$$$	✗

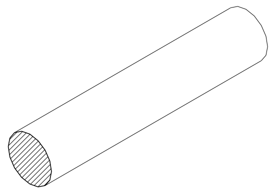
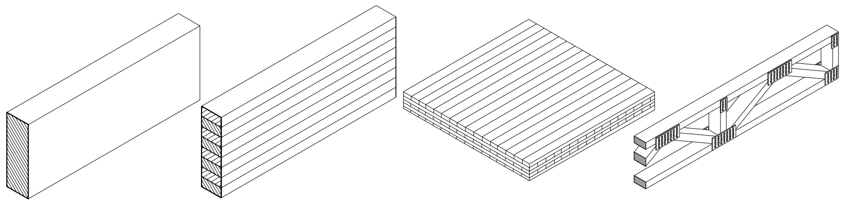
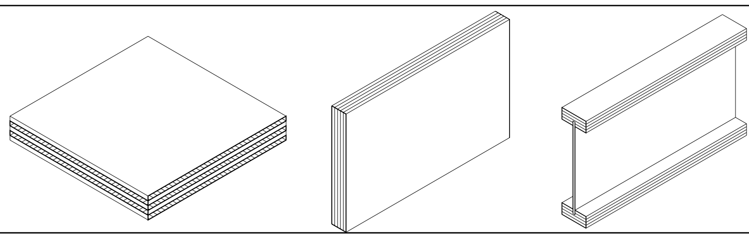
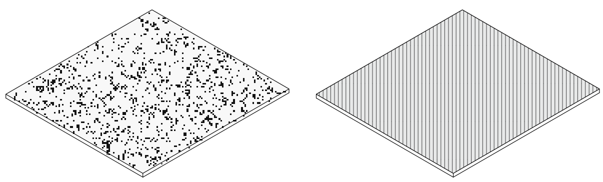
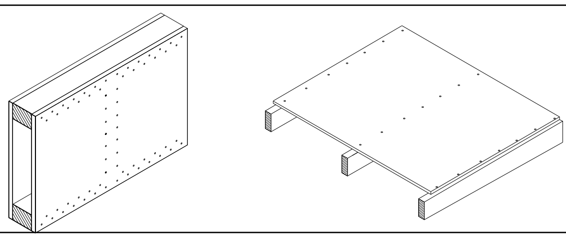
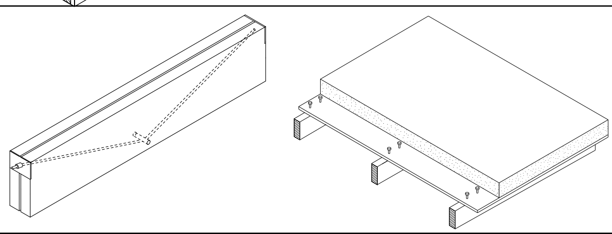
Notes:
Estimated price ranges take into account material and labour costs: $ = low cost, $$ = medium cost, $$$ = high cost

4 Element Options

Wood can be processing into a broad suite of products that can be used directly as building elements or combined with additional wood or other products to form more complex building elements or components.

During the conceptual design phase, the engineering designer should assess the best-suited material for the design by considering multiple alternative element sections and solutions for the spanning, support and lateral restraint systems of the project’s structure. To assist in this, this section describes the major options for timber-rich building elements, their applications and standards, and likely means of supply. Element options are grouped by their primary wood components and then sorted in each group from simple to complex. Table 28 lists the element groups and the guide sections that cover them.

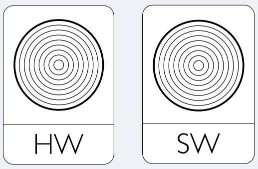
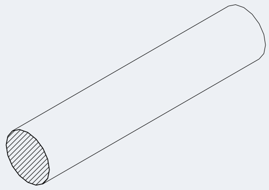
Table 28: Element groups and sections in which they are included

Section	Element group	Diagram
4.1	**Timber rounds,** including logs shaved into rounds or in their natural form	
4.2	**Sawn timber and assembled elements**, such as sawn framing, glulam, cross laminated timber (CLT), and nailplate assembled elements	
4.3	**Veneer-based elements,** such as plywood and laminated veneer lumber (LVL)	
4.4	**Strands, particles and fibre based elements**, such as oriented strand board (OSB) and high density fibre board (HDF)	
4.5	**Combination elements** assembled from several types of wood products	
4.6	**Composite elements** assembled from timber and other materials acting in combination	

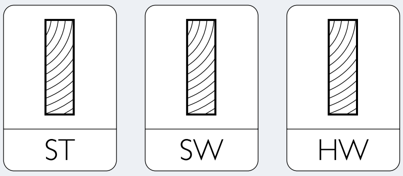
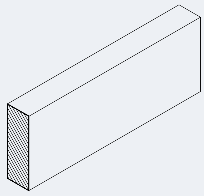
General element supply varies with the types of element. Three general supply options are common:

Supply options	Description
Generic	**Generic elements** are produced to standard sizes and grades and available from multiple suppliers for general use in building.
Fabricated	**Fabricated elements** are usually made for the project by general or specialist fabricators from combinations of generic elements.
Site assembled	**Site assembled elements** are usually constructed on site from combinations of generic and fabricated timber elements and other materials.

4.1 Timber rounds

Icon 	**Diagram** 
Description	Softwood (SW) and hardwood (HW) timber rounds are the simplest form of wood product. They include logs shaved into cylinders or in their natural form, usually with the bark removed. The sapwood of all species has low durability but will accept preservative treatment. Low durability rounds and poles should be treated when they are to be exposed outside, placed in ground contact, or when the sapwood is lyctid susceptible.
Applications	Timber rounds are used in and around buildings and in civil construction. This includes: columns, beams and corbels in buildings and bridges; utility poles and piles; and elements for landscaping, agricultural and similar applications.
Sizes	The size of natural rounds is limited by the tree's size at harvest. The size of generic treated timber rounds varies with species, from 75 to 300 mm diameter in softwoods to larger sizes in hardwoods. Shaved softwood logs are usually cylinders made to standard diameters. Natural logs taper and are specified by the minimum small end diameter (SED).
Standards	*AS 3818.10 Timber – Heavy structural products – Visually graded – Building poles*; *AS 3818.11 Timber – Heavy structural products – Visually graded – Utility poles*; and *AS 1604.1 Specification for preservative treatment – Sawn and round timber.*
Supply	Generic Fabricated

4.2.1 Sawn timber elements

Sawn structural timber	
Icon	 **Diagram** 
Description	Sawn structural timber (ST) is rectangular boards milled from softwood (SW) or hardwood (HW) logs, usually dried and then sorted by structural or appearance grade. It is the most common and widely used timber product, available in a wide range of species and sizes. Sawn structural timber can be treated to improve its durability: surface treated for protection against insect attack, or pressure treated to protect it when exposed outside or in ground contact.
Applications	Sawn structural timber is highly versatile due its structural capacity, workability, and economy. Applications include the floors, wall frames and roof structures of buildings of all sizes; the frames, decking and screens in external decks, pergolas and walkways; and landscaping structures such as fencing and retaining walls.
Grades	F-grades apply to visually graded seasoned and unseasoned hardwood and softwood, including pressure-treated material. MGP-grades define the structural characteristics of seasoned softwood, particularly pine framing.
Sizes	The sizes of generically produced sawn timber are largely standardised. Figure 5 shows the relationship between sawn, nominal and machined dimensions for sawn timber. Table 30 and Table 32 list the nominal sawn size and general availability of softwood and hardwood products respectively. Table 31 and Table 33 list similar information for machined and seasoned softwood and hardwood respectively. Table 29 lists allowable undersize tolerances for solid structural timber.
Standards	*AS 2878 Timber – Classification into strength groups*; *AS 2858 Timber - Softwood – Visually graded for structural purposes*; *AS 2082 Hardwood – Visually stress-graded for structural purposes*; and *AS/NZS 1748 Timber – Solid – Stress-graded for structural purposes.*
Supply	Generic

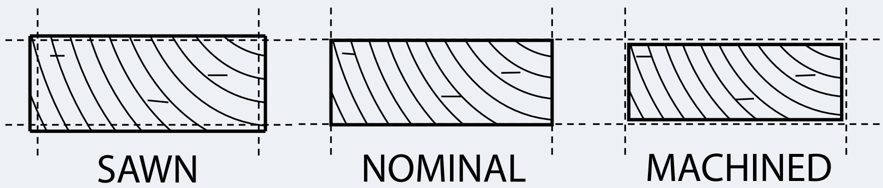


Figure 5: Sawn timber sizes: sawn, nominal and machined.

Table 29: Allowable undersize tolerances for solid structural timber

Element	Allowable undersize tolerance
Unseasoned timber: F 7 and below	**4 mm**
Unseasoned timber: F 8 and above	**3 mm**
Seasoned timber: All grades	**0 mm**

Table 30: Standard softwood sections – Nominal sawn, unseasoned or treated

Width	Depth							
mm	75	100	125	150	175	200	225	250
38	■	■	■	■	□	□		
50	■	■	■	■	■	□	□	□
75	■	■	■	■	■	□	□	□
100	■	■	□	□	□	□	□	□
125			■	◘		◘		
150				■				
200						■		

Legend: Commonly available ■; Available on order ◘; In limited supply □

Table 31: Standard softwood sections – Machined, seasoned

Width	Depth							
mm	42	70	90	120	140	190	240	290
35	◘	■	■	■	■	■	◘	◘
45		■	■	■	■	■	■	■
90			■					

Legend: Commonly available ■; Available on order ◘; In limited supply □

Table 32: Standard hardwood sections – Nominal sawn, unseasoned

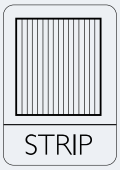
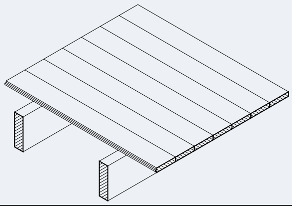
Width	Depth									
mm	50	75	100	125	150	175	200	225	250	300
25	■	■	■	□	■	□				
38	■	■	■	■	■	■	■	■	■	■
50	■	■	■	■	■	□	■	■	■	■
75		◘	■	◘	◘	◘	◘	◘	◘	◘

Legend: Commonly available ■; Available on order ◘; In limited supply □

Table 33: Standard hardwood sections – Machined, seasoned

Width	Depth								
mm	70	90	120	140	170	190	220	240	290
35	■	■	◘	◘	□	◘	□	◘	□
45	■	■	■	■	□	■	□	■	■

Legend: Commonly available ■; Available on order ◘; In limited supply □

Strip flooring	
Icon	
Diagram	
Description	Strip flooring (Strip) is hardwood or softwood boards profiled with a tongue on one edge and matching groove on the other. The boards fit together and are nailed to joists to provide a floor deck. Nailing can be concealed on narrow boards or exposed on the board's face. Board thickness can be varied to increase load resistance, and species selected for hardness to accommodate likely indentation or abrasion. Care is required with wide boards due to their dimension change with moisture content change.
Applications	Strip floors are widely used in appearance applications, sports halls, and in robust industrial applications where floor hardness is important.
Grades	Material is graded to appearance standards, dependent on species.
Sizes	Typical board sizes vary by species and area of production or boards can be milled to order. Board cover widths range from 60 to 140 mm with thickness from 19 to 30 mm for material used as a structural deck.
Standards	*AS 1810 Timber – Seasoned cypress pine – Milled products; AS 2796 Timber – Hardwood – Sawn and milled products;* and *AS 4785 Timber – Softwood – Sawn and milled products.*
Supply	Generic

4.2.2 Glue assembled elements

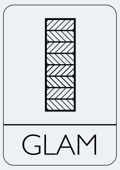
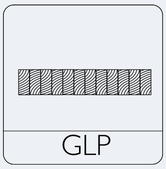
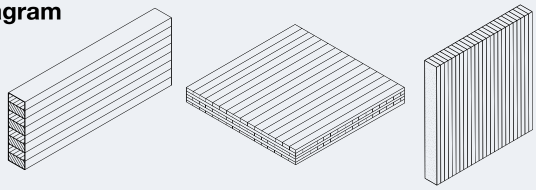
Glue laminated timber	
Icon	 
Diagram	
Description	Glue laminated (GLAM) timber is sections of sawn timber glued together to form larger, more structurally reliable timber elements. Lamination increases the size, structural capacity and reliability of elements and produces a wide range of beams, columns or panel elements. The timber sections are often joined along their length, then laminated into the larger element. Boards can be glued together on their wide face or on their edges. Glulam floor or wall panels (GLP) are vertically laminated on their wide faces. Timber can also be glue laminated into curved shapes, with the curve achieved depending on the laminate's thickness. Glulam beams can be fabricated with a camber to accommodate deflection during construction loading.
Applications	Beam or column elements can be used in architectural structures or in general structural applications in floor, wall or roof framing. Panel elements can be used as part of a massive timber floor system.
Grades	GL grades define the structural characteristics of glulam beams and columns.
Sizes	Sizes vary with producer and species. Typical sizes and availability for hardwood glulam are listed in Table 34. Typical softwood sizes are listed in Table 35. Larger sizes are available on order.
Standards	*AS/NZS 1328.1: Glued laminated structural timber – Performance requirements and minimum production requirements; AS/NZS 1328.2: Glued laminated structural timber – Guidelines for AS/NZS 1328: Part 1 for the selection, production and installation of glued laminated structural timber;* and *AS/NZS 1604.5: Specification for preservative treatment – Glued laminated timber products.*
Supply	Generic Fabricated

Table 34: Typical glulam sections – GL18 Hardwood

Width	Depth													
mm	**90**	**120**	**155**	**185**	**215**	**245**	**270**	**300**	**330**	**360**	**390**	**420**	**450**	**480+**
65	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘
85	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘
135	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘	◘

Legend: Commonly available ■; Available on order ◘; In limited supply ☐

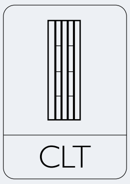
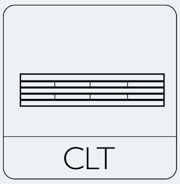
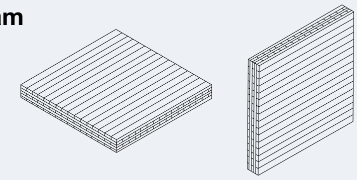
Notes: Hardwood glulam to standard machined, seasoned hardwood section sizes is also available.

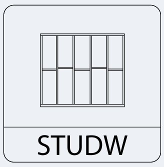
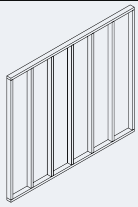
Table 35: Typical glulam sections – GL17 Softwood

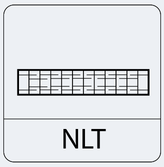
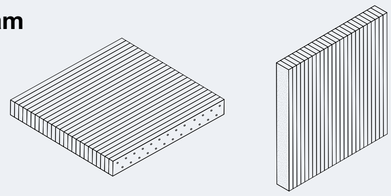
Width	Depth													
mm	**130**	**165**	**195**	**230**	**260**	**295**	**330**	**360**	**395**	**425**	**460**	**495**	**525**	**560+**
65	■	■	■	■	■	■	■	■	■	■	■	■	■	■
85	■	■	■	■	■	■	■	■	■	■	■	■	■	■
130	■	■	■	■	■	■	■	■	■	■	■	■	■	■

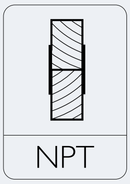
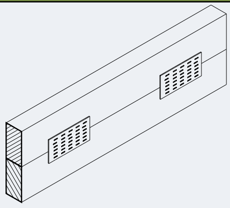
Legend: Commonly available ■; Available on order ◘; In limited supply ☐

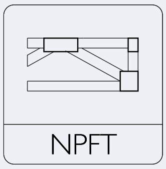
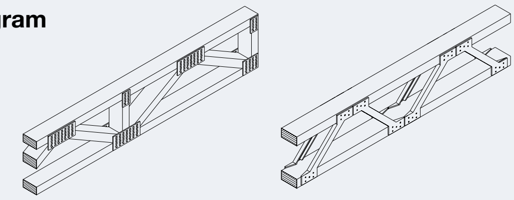
Note: GL21 is available up to 300 x 300 mm, GL18 up to 450 x 450 mm and GL17, GL13 and GL12 up to 600 x 600 mm. Larger sections up to 1200 mm deep are available on order.

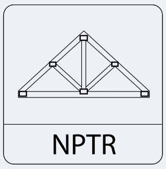
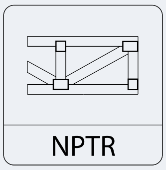
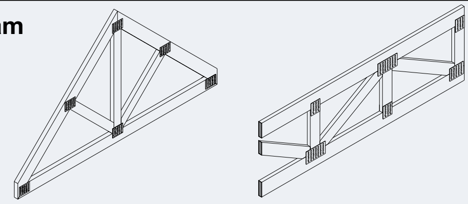
Cross-laminated timber	
Icon	 **Diagram** 
Description	Cross laminated timber (CLT) is engineered wood panels made by joining layers of timber together with the grain direction of alternating layers at right angles. Most CLT is glue laminated but some is assembled with mechanical fasteners, usually nails. In some products, the boards in each layer are glued together into a large sheet initially and then the layers are jointed to form an almost airtight panel. CLT is usually made for the project as part of an integrated building design, fabrication and delivery process. Different timber grades and thicknesses can be used in the panels and they can be made to various layer configurations. Panels are usually assembled so that the outside layers run parallel to the direction of span. However, the panels retain substantial bending strength perpendicular to the primary axis due to the inner layers' spanning capacity.
Applications	CLT panels can be used as structural wall, floor and roof elements in a massive timber construction system.
Grades	Face and concealed panel grades vary with the application and producer.
Sizes	CLT is made to the sizes required for the project, limited only by production and transport constraints.
Standards	No Australian standards currently exist for CLT.
Supply	Fabricated

Stud wall frame	
Icon 	**Diagram** 
Description	Stud wall frames (StudW) are prefabricated or site-assembled planar elements assembled by nailing regularly placed vertical studs between horizontal plates. Multiple plates or groups of studs can accommodate and distribute concentrated loads. Noggins between the studs reduce their potential for buckling. Steel straps and ties connecting the studs to the plates allow the frames to resist uplift loads. Wall frames can resist vertical loads and racking loads if suitably braced with sheet material or steel strap. They are usually sufficiently stiff to be lined and form the basis for internal and envelope wall systems. Stud framing is normally 70 or 90 mm thick, but wider sawn timber or EWPs used as studs and plates allow thicker frames to carry higher loads or span further.
Applications	Wall frames are used widely as the structural components of load-bearing and non-loadbearing wall systems in buildings of all types and scales.
Sizes	Stud wall frames can generally be assembled to the size required for the application but material economy, transport and handling considerations often limit efficient frame size.
Standards	*AS 1720 – Timber Structures* governs the design of wall frames while *AS 1684 – Residential Timber-framed Construction* describes the requirement of wall frames for Class 1 buildings that comply with the *AS 1720* methodology.
Supply	Fabricated Site assembled

Nail laminated timber	
Icon 	**Diagram** 
Description	Nail laminated timber (NLT) is sections of sawn timber nailed together to form larger, more structurally reliable timber elements. Lamination increases the size, structural capacity and reliability of elements while nailing with a nail gun is also a technically simpler process than gluing. The boards in floor or wall panels are generally vertically nail laminated (wide face to wide face) with the timber sections finger-jointed along their length or butt jointed. Nail laminated beams with vertically arranged boards are common but beams with horizontally arranged boards tend to have excessive deflection.
Applications	Column and wall elements can be used in architectural structures or as floor, wall or roof panels in massive timber construction systems.
Sizes	Nail laminated elements can be made to the size required for the project, given the limitations of board size and transportation.
Standards	*AS 1720 – Timber Structures* governs the design of nail laminated elements.
Supply	Fabricated Site assembled

Nailplated timber	
Icon	
Diagram	
Description	Steel nailplates pressed into both sides of a timber joint can laminate sawn timber or LVL sections together and make nailplated timber (NPT) that is longer or wider than the original material. The joints are engineered to ensure that the assembled section has structural properties comparable to a solid section of the same size. This can convert single span sections into continuous span elements, optimising element size and increasing construction efficiency. The nailplates and design process are proprietary and must be assembled by licensed fabricators under factory conditions.
Applications	Nailplated timber elements provide standard solutions for high-load lintels and continuous span elements such as bearers and joists in floors and rafters, purlins and similar elements in a roof.
Grades	The assembled timber sections retain the original timber's structural grade or the LVL's performance.
Sizes	Sizes generally match standard sawn timber sizes.
Standards	Joints are designed in proprietary design software to comply with *AS 1720 – Timber Structures*.
Supply	Fabricated

Nailplate floor trusses	
Icon	
Diagram	
Description	Nailplate floor trusses (NPFT) or floor joists are low-profile elements manufactured for high-load and stiffness applications. The trusses can have solid timber or LVL flanges, and proprietary steel or timber webs. The timber is usually arranged with the wide face of the board horizontal. Floor trusses are custom designed and assembled for the location in the project. The web arrangements can be adjusted to accommodate on-site service installation.
Applications	Floor trusses are versatile and form the core of an efficient solution for floor construction. They are widely used in first floor platforms of domestic and commercial buildings and can be grouped together to form robust floor cassettes. They work well with other timber elements, such as LVL, and with other materials, such as steel. They can also act as long-span purlins between the beams and frames of roof structures.
Sizes	Metal web trusses are limited to standard depths for each manufacturer. Timber web trusses can be fabricated to any depth. Transport considerations limit element length.
Standards	The design and certification of nailplate systems are based on the nailplates' and timber's performance and on proprietary design software that combines the design's requirements with those of *AS 1720 – Timber Structures* to provide a reliable solution.
Supply	Fabricated

Nailplate trusses	
Icon  	**Diagram** 
Description	Nailplate trusses (NPTR) as either roof or girder trusses are varied profile elements for mid-load applications. They can have solid timber and LVL flanges and webs joined with nailplates and sized to suit the intended load. Multiple trusses can be grouped together to resist higher loads. Designed for the project, there are very few limits on their shape and configuration. Spans up to 30 m are possible and spans of 20 m common. Assembled into a three dimensional roofing or floor modules, they can often accommodate complex architectural requirements or services loads. They can also be built with a camber.
Applications	Nailplate roof and girder truss systems are now a standard economic solution for the roof structure of all classes of low to mid-rise buildings. They can be incorporated as beams into wall frames.
Sizes	Nailplate trusses can be fabricated to almost any depth but transport and handling considerations limit element length.
Standards	The design and certification of nailplate systems use proprietary design software that is compliant with *AS 1720.5.- Timber Service Life Design Guide*
Supply	Fabricated

4.3 Veneer-based elements

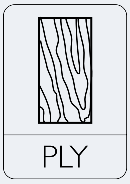
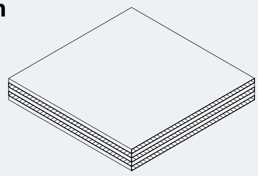
Plywood	
Icon 	**Diagram** 
Description	Plywood (Ply) is a wood panel product manufactured from peeled softwood and hardwood veneers laminated into a sheet with the veneer's grain direction alternating between layers. The grain on the outside veneer runs along the sheet with the grain of alternate inner layers running across the sheet. Plywood is a diverse, versatile and dimensionally stable material, with generally high impact and shear strength.
Applications	Plywood is used in a wide range of applications including form ply in concrete construction, structural bracing and stressed skin panels, flooring, internal lining, furniture, fittings or joinery and external cladding.
Grades	Plywood is graded structurally to F-grades. Glue bond types are listed in Table 37 while Table 38 shows the alignment of veneer-based products and glue bond types. Table 39 describes the quality of veneer face grades.
Standards	Relevant standards include: *AS/NZS 2269.0 Plywood – Structural*; *AS/NZS 2272 Plywood – Marine*, and *AS/NZS 2098 Method of Test for Veneer and Plywood*. Additional standards are listed in Table 38.
Sizes	4, 4.5, 7, 9, 12, 15, 17, 19, 21 and 25 mm are common plywood thicknesses. Panel sizes are given in Table 36.
Supply	Generic

Table 36: Typical structural plywood size range

Width	Length			
mm	**1800**	**2100**	**2400**	**2700**
900	◘	◘	◘	◘
1200	◘	◘	■	■

Legend: Commonly available ■; Available on order ◘; In limited supply ☐

Table 37: Adhesive bond types

Bond types	Description	Key bond test
Type A	Permanent, waterproof	72 hours in boiling water or 6 hours 200 kPa steam
Type B	Semi-permanent, water resistant	6 hours in boiling water
Type C	Suitable for high-humidity areas	3 hours in water @ 70°C
Type D	Suitable for low-humidity areas only	20 hours in cold water

Source: EWPAA

Table 38: Veneer-based products and adhesive bond types

Product	Standard	Bond	Adhesive type	Adhesive colour
Structural Plywood	*AS/NZS 2269.0*	Type A	Phenol Formaldehyde (dark)	Dark
Structural Laminated Veneer Lumber (LVL)	*AS/NZS 4357.0*	Type A	Phenol Formaldehyde (colour-dark)	Dark
Marine Plywood	*AS/NZS 2272*	Type A	Phenol Formaldehyde	Dark
Exterior Plywood	*AS/NZS 2271*	Type A	Phenol Formaldehyde	Dark
		Type B	Melamine Urea Formaldehyde	Light
Interior Plywood	*AS/NZS 2270*	Type C	Urea Formaldehyde	Light
		Type D	Extended Urea Formaldehyde	Light

Source: EWPAA

Table 39: Veneer quality grades and their description

Veneer quality	Description
A	High-quality appearance-grade veneer suitable for clear finishing, suitable where surface decorative appearance is a primary consideration
S	An appearance grade veneer that permits natural characteristics. The type and frequency of the natural characteristics that are acceptable is to be based on an agreed specification.
B	An appearance grade suitable for high quality paint finishing.
C	A non-appearance grade with a solid surface. All open defects such as knot holes or splits are filled.
D	A non-appearance grade with permitted open imperfections. Limited numbers of knots and knot holes up to 75 mm wide are permitted.

Source: EWPAA

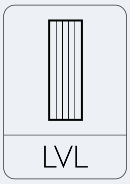
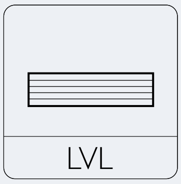
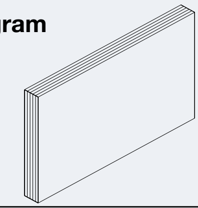
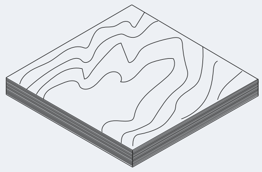
Laminated veneer lumber	
Icon  	**Diagram**  
Description	Laminated veneer lumber (LVL) is a structural wood panel product manufactured from peeled veneers laminated into a panel with the grain of most veneers running parallel to each other along the board. Cross veneers can be included and these increase the section's dimensional stability. These configurations result in a material that has very uniform structural performance as beams or columns because strength reducing characteristics are randomly distributed through the veneers. Due to the longer spans possible, design of LVL beams is often controlled by stiffness, rather than strength.
Applications	LVL is primarily a structural material with applications in floors, wall frames and roof structures of all sizes of buildings, and in formwork. It can also be used as flooring planks in a massive timber system.
Grades	Manufacturers publish the performance characteristics of products independently. Some products have F-grade ratings if they achieve the required performance.
Sizes	LVL panels are manufactured in continuous panels up to 1.2 m wide and 18 m long in standard thicknesses. It is then resawn into market widths. See Table 40. Sections are often available in lengths up to 12 m while larger sizes are available on special order.
Standards	*AS/NZS 4357.0 Structural Laminated Veneer Lumber – Specifications* and *AS/NZS 2098 Method of Test for Veneer and Plywood.*
Supply	Generic

Table 40: Typical LVL sections

Width	Depth												
mm	**95**	**130**	**150**	**170**	**200**	**240**	**300**	**360**	**400**	**450**	**500**	**525**	**600**
36	■	■	■	■	■	■	■						
45	■	■	■	■	■	■	■	■	■				
63	▣	▣	▣	▣	■	■	■	■	■	▣	▣		▣
75			▣	☐	☐	☐	▣	☐	▣	☐		▣	▣

Legend: Commonly available ■; Available on order ▣; In limited supply ☐

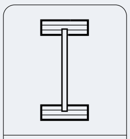
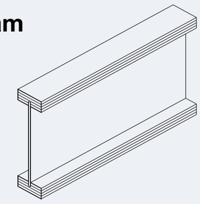
I-Beams			
Icon	 I-B	**Diagram**	
Description	I-beams (I-B) are manufactured timber elements with solid timber or LVL flanges and a vertical web, usually structural plywood or oriented strand board (OSB). This arrangement optimises the element's straightness and its capacity as a beam while minimising its weight. Lightweight compared to conventional timber joists, they are available in long lengths and a range of depths.		
Applications	I-beams are engineered structural timber beams particularly suitable as joists, rafters or purlins in the floor and roof structures in buildings of all sizes. They are also useful in formwork and civil construction.		
Grades	Manufacturers publish the performance characteristics of products independently.		
Sizes	I-beams are produced in typical widths and depths, shown in Table 41, and are available in lengths to 12 m.		
Standards	*AS/NZS 2269.0 Plywood – Structural*; *AS/NZS 4357.0 Structural Laminated Veneer Lumber – Specifications*; and *AS/NZS 2098 Method of Test for Veneer and Plywood.*		
Supply	Generic		

Table 41: Typical I-beams size range

Width	Depth				
mm	**200**	**240**	**300**	**360**	**400**
45	■	■	■		
63		■	■	■	◘
90		■	■	■	■

Legend: Commonly available ■; Available on order ◘; In limited supply ☐

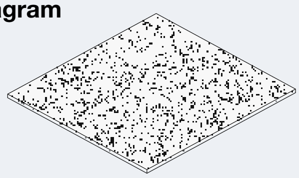
Oriented strand board	
Icon	 OSB
Diagram	
Description	Oriented strand board (OSB) is a reconstituted structural panel product made from thin wood flakes or strands combined with adhesive into a mat and pressed together. A wax may be added to the resin to increase the material's moisture resistance. Controlled orientation of the strands in the matt influences its structural properties and dimensional stability. It is a dimensionally stable material with generally high impact and shear strength.
Applications	OSB can be used for internal and some external applications, particularly as structural bracing panels and stressed skins.
Grades	Manufacturers publish the performance characteristics of products independently.
Standards	No Australian standards currently exist for OSB.
Sizes	OSB is available in a variety of thicknesses including 6, 9, 12, 15, 18 and 25 mm. Sheet sizes are shown in Table 42.
Supply	Generic

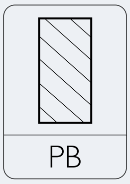
Table 42: Typical OSB size range

Width	Length		
mm	**2440**	**2745**	**3050**
900	◘	◘	◘
1200	■	■	◘

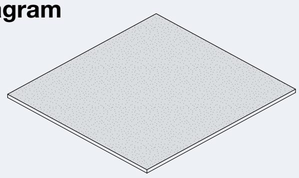
Legend: Commonly available ■; Available on order ◘; In limited supply □

Particleboard

Icon



Diagram



Description	Particleboard (PB) is a reconstituted wood panel product manufactured from wood particles combined with adhesive into a mat and pressed together. As the wood particles are randomly oriented, the finished panel has uniform properties in each direction. Particleboard flooring has a hard layer of particles on the outside faces and enhanced water resistance.
Applications	Particleboard is used structurally mainly as flooring in platform construction with sheet thickness and performance matching the load and span requirements for major building applications.
Grades	Manufacturers publish the performance characteristics of products independently.
Sizes	Particleboard flooring is available in standard 3600 mm lengths and thicknesses of 19, 22 and 25 mm. Typical sheet sizes are shown in Table 43.
Standards	*AS/NZS 1859.1: Reconstituted wood-based panels – Specifications – Particleboard; AS/NZS 1860.1: Particleboard flooring – Specifications;* and *AS 1860.2 Particleboard flooring – Installation*.
Supply	Generic

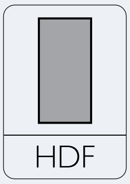
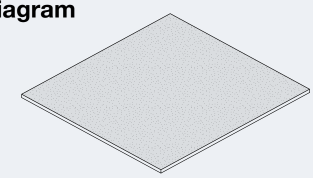
Table 43: Typical particleboard flooring size range

Width	Thickness		
mm	19	22	25
600	■	■	■
900	■	■	
1200	■	■	

Note: Length is 3600 standard

Legend: Commonly available ■; Available on order ◘; In limited supply ☐

Fibreboards

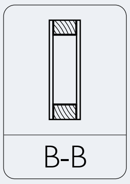
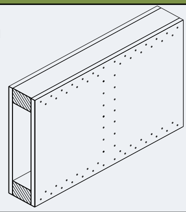
Icon	Diagram
	

Description	Fibreboards are a diverse group of sheet products including low, medium and high-density fibre board manufactured from wood fibres arranged into a mat and pressed. They are generally very stable with uniform thickness and properties. High-density fibre (HDF) boards such as hardboard are typically made without adhesives and are the only product type of the group regularly used structurally.
Applications	High-density fibreboard is used for bracing, lining and cladding.
Grades	Manufacturers publish the performance characteristics of products independently.
Sizes	High-density fibreboard used as bracing is available in thicknesses of 4.8 and 6.4 mm. Typical sheet sizes are shown in Table 44.
Standards	*AS/NZS 1859.2 Reconstituted wood-based panels – Specifications – Dry-processed fibreboard*; and *AS/NZS 1859.4 Reconstituted wood-based panels – Specifications – Wet-processed fibreboard.*
Supply	Generic

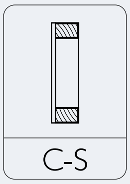
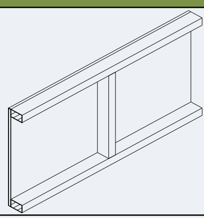
Table 44: Typical high-density fibreboard size range

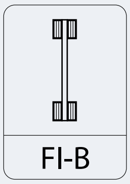
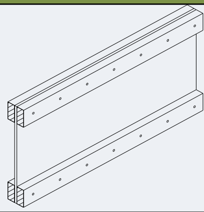
Width	Length		
mm	**2440**	**2745**	**3050**
460	■	■	◘
610	◘	◘	◘
900	■	■	◘
1200	■	■	◘
1350	◘	◘	◘

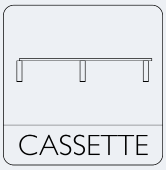
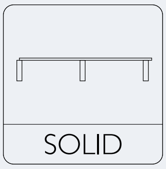
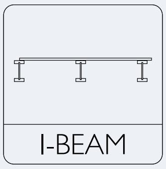
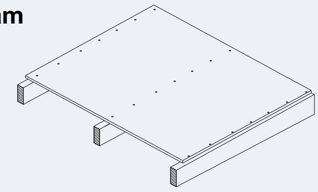
Legend: Commonly available ■; Available on order ◘; In limited supply ☐

Plywood box beams	
Icon 	**Diagram** 
Description	Box beams (B-B) are lightweight, structurally efficient, engineered elements fabricated with solid timber, glulam or LVL flanges and frames, and plywood, OSB or similar sheet material as webs. The webs connect to the frame with a determined nail or screw pattern. Glue may also be used. The beams can be discrete roof or floor elements, or form part of a wall frame. Lighter and more versatile than solid timber or glulam, they can be made to any curve or shape. This is only limited by the flanges' flexibility. To minimise waste, the flanges are typically timber to standard sizes while the webs' width is a convenient division of standard sheet sizes.
Applications	Box beams are often used in roof structures, as the columns and rafters of portal frames or as lintels as part of wall frames.
Sizes	Box beams can be made to the sizes that suit the application. In practice, beam depth is often set at the width or half the width of a plywood sheet: 450, 600, 900 or 1,200 mm. Transport considerations often limit element length.
Standards	*AS 1720 – Timber Structures* governs the design of plywood box.
Supply	Fabricated Site assembled

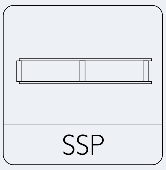
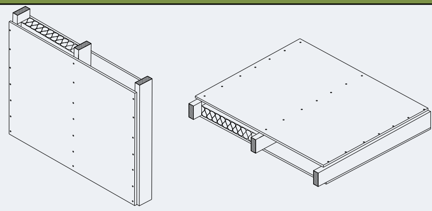
See WoodSolutions Design Guide 7: Plywood box beam construction

C-section beams	
Icon 	**Diagram** 
Description	C-section beams (C-S) are lightweight, structurally efficient and engineered elements fabricated with solid timber or LVL flanges and frames, and plywood, OSB or similar sheet material as the web. The ply connects to the timber frame with a determined pattern of nails or screws. Glue may also be used. The beams can be discrete roof elements or form part of a wall frame.
Applications	C-section beams are often used in roof structures or as a beam in wall frames.
Sizes	C-section beams can be made to the sizes that suit the application. In practice, beam depth is often set at the width or half the width of a plywood sheet. Transport considerations often limit element length.
Standards	*AS 1720 – Timber Structures* governs the design of plywood box and C-section beams.
Supply	Fabricated Site assembled

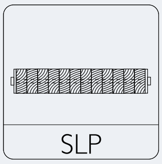
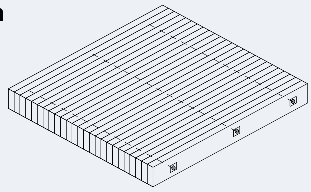
I-section beams	
Icon	 **Diagram** 
Description	Fabricated I-section beams (FI-B) are lightweight, structurally efficient and engineered elements fabricated with solid timber or LVL flanges, and plywood, OSB or similar sheet material as the central web. While I-beams are available as generic off-the-shelf products, I-section beams can also be fabricated to satisfy specific design conditions.
Applications	I-section beams are used in roof structures or in floor frames.
Sizes	Like C-section beams, I-beams can be made to the sizes that suit the application. In practice, beam depth is often set at the width or half the width of a plywood sheet. Transport considerations often limit element length.
Standards	*AS 1720 – Timber Structures* governs the design of I-section beams.
Supply	Fabricated Site assembled

Cassette floor panels	
Icon	   **Diagram** 
Description	Floor cassettes (Cassette) are prefabricated, engineered modules made with solid timber, LVL, I-beams or floor trusses as joists, and ply, OSB or particleboard as flooring. Workshop fabrication allows for precise assembly, increased quality control, and reduction in site waste and construction time. The intended installation process determines the maximum cassette size. Small units can be installed by hand with larger units installed by crane.
Applications	Cassette floors provide versatile and efficient solutions for flooring systems in medium-scale residential and commercial buildings. Similar units can be adapted for roof construction.
Sizes	Floor cassettes are made to the sizes required for the project, often in widths that use the sheet flooring efficiently. Transport considerations and crane capacity limit element width and length.
Standards	*AS 1720 – Timber Structures* governs the design of cassette floors.
Supply	Fabricated

See WoodSolutions Design Guide 31: Timber Cassette Floors

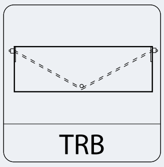
Stressed skin panels	
Icon 	**Diagram** 
Description	Stressed skin panels (SSP) are prefabricated, engineered modules used as either wall or floor elements. To increase module strength and stiffness, joists or studs of solid timber, LVL, I-beams or floor trusses are fixed or bonded to structural panels of ply, OSB, HDF or particleboard on the top and bottom faces. The assembly acts as a composite system, with the panels acting as tension or compression chords and the joists acting as web-members. To ensure adequate shear transfer between the joists and panels, the joints are usually fixed and glued with structural adhesives. Stressed skin panels are workshop fabricated to ensure module quality.
Applications	Stressed skin panels provide versatile and efficient solutions for the floor, roof and wall systems in small to medium-scale residential, commercial and industrial buildings.
Sizes	Stressed skin panels are made to the sizes required for the project, often in widths that use the sheet material efficiently. Transport considerations limit element length.
Standards	*AS 1720 – Timber Structures* governs the design of cassette floors.
Supply	Fabricated

4.6 Composite elements

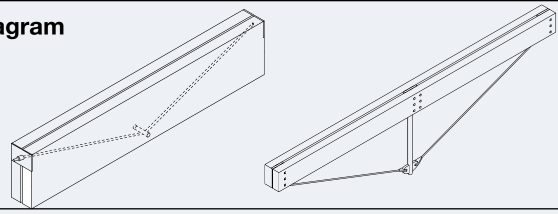
Stress laminated panels	
Icon 	**Diagram** 
Description	Stress laminated panels (SLP) are vertically laminated boards joined together by high-tension rods or long screws. The rods or screws run through the boards and join edge beams. When tensioned, the rods compress the boards between the beams and the friction between the boards converts it in a solid and rugged panel. The panels can be produced in a workshop or assembled on site. Panels stress laminated from unseasoned timber need to be regularly retightened as the boards in the panel dry and shrink to be in equilibrium with the surrounding environmental conditions.
Applications	Stress laminated panels can be used in mass timber construction or as deck elements in industrial and civil construction, particularly bridge decks.
Sizes	Stress laminated panels can be made to the sizes required for the project. Transport and handling considerations limit the length and width of prefabricated elements.
Standards	*AS 1720 – Timber Structures* governs the design of stress laminated panels.
Supply	Fabricated Site assembled

Trussed beams

Icon



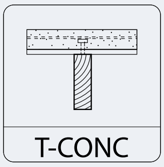
Diagram



Description	Trussed beams (TRB) are composite elements with steel rod, cable or tendons acting in tension and timber sections working in compression. The steel is trussed off under the middle of the span or between deep elements in a shallow V-shape. Traditionally used because the depth of available sawn elements limited their effective span, this arrangement is used in modern construction with deep LVL or glulam beams and columns to limit deflection, increases load carry capacity and improves the system's resilience under seismic loading. The tendons can also be incorporated into the connection systems. As the steel cable only works in tension, care is needed with elements subject to load reversal, such as roof beams. Also, as steel's thermal expansion is high compared to timber, the temperature at which the steel rod is tensioned can be important. Unexpected changes in the steel's temperature can cause unanticipated impacts on the system.
Applications	Beams in floor and roof systems and as wind posts.
Sizes	Trussed beams can be made to the size required for the project, limited by the size of available timber elements and transport considerations.
Standards	*AS 1720 – Timber Structures* governs the design of trussed beams.
Supply	**Fabricated** **Site assembled**

Composite timber-concrete floor or decks

Icon



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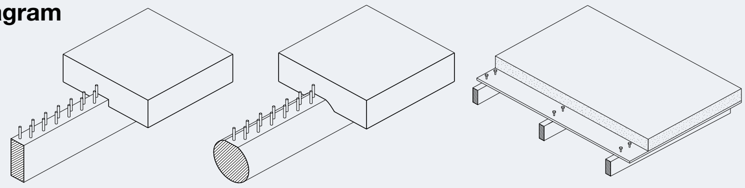


Description	Composite timber-concrete (T-Conc) floors or decks combine a timber structure acting in tension with a concrete slab acting in compression. To achieve composite action, shear connections are required between the timber and concrete. These can be concrete keys set into the top of a timber beam or mass timber floor panel, steel pins or plates set into the beams or joists, or a combination of these methods. The composite floor has increased strength and stiffness compared to a traditional timber system, allowing longer spans or reduced floor depth. The floor can be cast in place or be made as a prefabricated component. If a composite system is used externally in a bridge or large deck, the concrete should be finished and the surrounds flashed to protect the timber from water. If the concrete channels moisture to the wood, it can accelerate decay.
Applications	Floor systems in residential or commercial building or the decks in bridges and landscaping elements.
Sizes	Composite timber-concrete floors can be made to the size required for the project. Transport and handling considerations limit the length and width of prefabricated elements.
Standards	*AS 1720 – Timber Structures* governs the design of composite timber-concrete floors.
Supply	**Fabricated** **Site assembled**

See WoodSolutions Design Guide 30: Timber Concrete Composite Floors

5 Construction Process Options

Wood's workability and product range create assembly and prefabrication opportunities that are often impracticable for other materials.

During the conceptual design phase, the designer needs to consider how the structure's parts can be assembled on site or prefabricated elsewhere and supplied for installation during the construction process. These factors can influence the solution's economy and the configuration, spans and loads on its various components. To develop economically viable solutions, early collaboration with other design professionals and key systems suppliers and installation contractors should be considered.

5.1 Construction process options

There are three major construction process options for timber-rich structures. Based on the intended level of prefabrication, these are: prefabrication-intensive building; site-based component assembly; and general construction. These options are described in detail below while approaches to prefabrication are discussed in Section 5.2.

In practice, these options are often blended into a solution that satisfies the project's design requirements and budget, and the likely site conditions. Factors that influence the selected construction process include:

- The option's construction cost relative to other options.
- Site access and infrastructure such as cranes, accommodation, and loading and storage areas. The site's size and location influence the efficiency and cost of this infrastructure.
- Required project quality and the potential for quality control given the site conditions.
- Required construction speed.
- Availability of suitably skilled suppliers and subcontractors.
- Worksite safety.
- Environmental impacts and loss of amenity on the site and surrounding properties and businesses.
- Likely exposure to adverse weather events, such as extended rain periods.

In general, prefabricated timber-rich construction should be preferred if the site is sensitive, exposed, constrained, remote or has poor soil conditions.

5.1.1 Prefabrication-intensive building

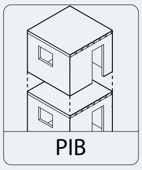
Icon	
Description	Prefabrication-intensive building (PIB) is the building's assembly from elements and components predominantly prefabricated off site and transported to it before being lifted into place. In this type of construction, most elements are designed and documented as volumetric modules or panels and made in controlled conditions using workshop-based and increasingly computer-controlled equipment. This type of prefabrication exploits the output of building information management (BIM) systems-based design, improves build accuracy, limits the potential for weather delays, reduces worker risk, and significantly increases the potential for effective quality control. However, size limits may apply to elements and components due to transportation costs for wide loads. As work is moved from the site to the workshop, the site's labour force, accommodation and storage areas can be reduced and the build time considerably shortened. Crane access is essential. Prefabrication-intensive building can deliver precise and economic built solutions but relies on early and complete building documentation and effective construction programming. Design decisions have to made prior to prefabrication and the sequencing of component assembly and delivery is critical to an efficient outcome.
Applications	Prefabrication-intensive building is suited for most types and sizes of building projects but has particular benefits for construction on constrained urban sites, sites remote from an available labour pool, and where the project involves multiples of similar components or elements, such as hotel rooms or regular post, beam and floor units.

Figure 6: Prefabrication-intensive construction – panel construction.

5.1.2 Site-based component assembly

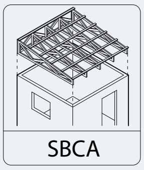
Icon	
Description	Site-based component assembly (SBCA) is the separate site assembly of significant building components from generic and fabricated elements. Once assembled, they are lifted into place in the structure. This approach overcomes the transport limitations of prefabrication-intensive building but retains many of its benefits. Large elements can be assembled in an area that is more accessible or safer to work in than their final location. The potential to control quality and limit the impacts of adverse weather are higher than in general construction but lower than in fully prefabricated options. Site-based component assembly requires the site to be large enough for the components to be assembled, moved and then lifted into place, and for skilled labour to be available on site. Crane access is essential during the component lift.
Applications	Site-based component assembly can be used for columns and floor modules but is particularly suited to roof structures. Large modules can be assembled, clad and potentially serviced at ground level, often near their supports and then lifted and fixed into place. In some single storey projects, the whole roof can be assembled on the ground with its supports hinged in place. As the roof is raised, the supports swing into position and are fixed in place.

Figure 7: Site-based component assembly – roof modules

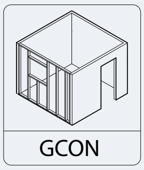
Icon	
Description	General construction (GCON) is a building's assembly on site from a combination of generic and readily available fabricated elements, such as prefabricated wall frames and nailplate floor and roof trusses. Generally, the work site is exposed to the elements and construction relies on a relatively large site labour force using mainly portable equipment. While cranes may be used, element weight is often limited to a safe one or two person lift. Suitable accommodation and storage areas also need to be provided. This construction option is highly flexible as the element's length and final location is often determined on site. The availability of a skilled workforce reduces the requirements for accurate documentation and the flexibility of material supply reduces the need for detailed construction programming. However, construction tolerances are higher than in other options, quality is harder to control, and competent site supervision is essential. Site safety issues generally increase with the building's size.
Applications	This is a very common construction process and is used extensively for residential and commercial buildings up to three storeys. It is particularly suited to renovation projects where a new structure must interconnect with an existing one, for customised solutions where the client's final requirement may vary, and where an experienced site labour force is readily available.

Figure 8: General timber-rich construction

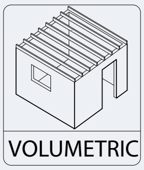
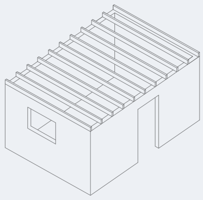
Icon	 **Diagram** 
Description	Prefabricated volumetric modules (Volumetric) are three-dimensional components such as rooms assembled from wall, floor and other elements. They can be designed and assembled as whole buildings, as components to be stacked one on another, or as serviced or structural cores in buildings otherwise made from prefabricated panels. In their simplest form, volumetric modules can be simple structural frames. In more advanced applications, the modules can be lined, clad, fully serviced and effectively complete spaces. They can be timber framed or incorporate massive timber panels in the wall, floors or ceilings. As the modules have to be lifted, care is required in detailing lifting locations and in general module robustness. As transport regulations limit and influence the cost of moving modules to site, practical limits apply to module size. These vary with transport regulations and the intended path of travel.
Applications	Volumetric modules are suited to any project with multiples of similar serviced rooms such as a hotel or apartment building, or where compact, highly serviced spaces have to be provided. This may be an office building's amenity units or the service core in houses or apartments.

Figure 9: Prefabricated modules in installation and assembly

5.2.2 Panels

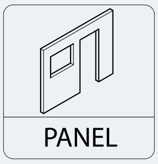
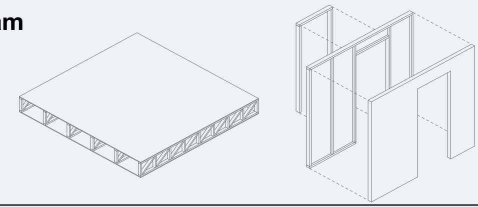
Icon	 **Diagram** 
Description	Prefabricated panels include preassembled wall frames, nailplate and other floor and roof trusses, floor cassettes, stressed skin panels, assembled post and beam units, and massive timber panels. In their simplest form, they are open frame items that are installed and fixed to other panel elements to make a structural frame. As the level of preplanning increases, wall assemblies can include factory-fitted doors and windows and lining on one side or be fully insulated, serviced and lined both sides before being lifted into place. Floor cassettes can be assembled from individual floor trusses or joists and interconnected with edge beams, braces and flooring. Massive timber components are generally made for the required span and delivered to site as panels cut to size for location. While transport regulations enforce some size limits, large timber panels can usually be transported economically and installed with standard lifting equipment.
Applications	Panels can be used in timber-rich buildings of any size. Prefabricated wall frames and nailplate trusses are very common in low to mid-rise residential and commercial buildings. Floor cassettes and clad wall frames are well suited for similar sizes and classes of buildings. Massive timber panel systems are being used in buildings to ten storeys and beyond.

Figure 10: Prefabricated wall frames and floor cassette

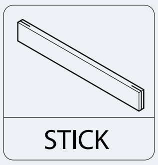
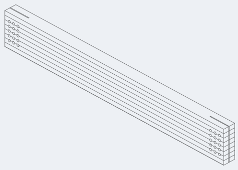
Icon	 **Diagram** 
Description	Prefabricated sticks are timber elements made or cut to size and prepared for projects. In their simplest form, these are simply cut to length, shaped and delivered to site as part of optimised, knock together component systems. They can also be generic products nailplated together to form continuous span joists, rafters or purlins for the project. In more complex forms, they can be fabricated structural elements such as high-performance, prefinished glulam beams, cut to length and shaped to accept connectors and fasteners.
Applications	Optimised and nailplate elements are widely used in the spanning floor and roof elements of low to mid-rise buildings. They limit site work, reduce material wastage and improve economy. More complex elements can form the rafter, beam, post or arch elements in major roof and wall structures.

Figure 11: Prefabricated roof beams

6 Design Approaches for Performance

In addition to considering the element, construction, systems and connection options available, the designer has to ensure that the alternative structural solutions developed during a project's conceptual design stage can satisfy its performance requirements. To assist designers, this section discusses the considerations and design approaches required to address performance requirements that regularly apply to timber-rich buildings and structures. These include structural performance, fire resistance, acoustic separation, system durability and other requirements.

As conceptual design is an iterative process where the designer develops and tests options, the approaches covered below are discussed in broad principle. They deal with the key considerations required to efficiently ensure a satisfactory solution given conditions regularly addressed in practice. During design development and in specialist applications, other WoodSolutions Guides and resources should be consulted in detail.

6.1 Structural performance

Support Information	Section
Element options	4
Indicative span tables and span-depth ratios	5
Wood's anisotropic strength characteristics	7.2.2
Wood's basic properties and joint group	8.1
Wood's structural performance and characteristic properties	8.2
Performance requirements - structure	9.2
Worked example	10

See SA HB 108 Timber design handbook

Standards and handbooks cover required and best-practice engineering approaches to satisfy the requirements of NCC Section B for the structural performance of timber buildings and structures. Resources that provide working structural solutions for timber-rich systems in many regular applications Include:

- *AS 1684 Residential timber-framed construction for the structural timber frame of Class 1 buildings*
- Industry-distributed design software covering the performance of generic engineered wood products in most regular applications.
- Proprietary design software used by licensed operators to generate solutions for single and systems of fabricated elements.

During design development, these standards, guides and resources should be consulted in detail. To inform conceptual design, this guide summarises considerations and design approaches necessary to efficiently ensure a satisfactory structural performance for timber elements. These include the requirements for element and connection design.

Key considerations during conceptual design

- Timber and wood products perform satisfactorily as structural elements in most types of buildings and structures. Solid structural timber is economic and performs well but element sizes are limited. EWPs have highly regular and reliable structural performance and are available in a wider range of sizes.
- The NCC's DTS provisions list *AS 1720* and *AS 1684* as compliant means of determining the structural resistance of materials and forms of construction for timber-rich building. *AS 1684* provides solutions useful for both conceptual and detailed design for Class 1 residential structures.
- Unless a performance-based solution is developed, structures outside the scope of *AS 1684* are designed to the requirements of *AS 1720*. Available proprietary and industry-proved design software is generally based on *AS 1720*.

- The environmental conditions in and around the structure can strongly influence the applied load and the material's performance and durability. These conditions need to be determined in the brief and information search stage and considered during conceptual design.
- Key design actions and limit states regularly govern element design and these should inform option selection during conceptual design.
- Connections are often the most complex and cost-sensitive part of the design of a timber structure. Connection design can often govern the required timber section size. Connectors may contribute up to 25% of the total construction cost of the timber members they connect.
- The quality of connection design can affect on the structure's serviceability and durability. Due to wood's anisotropic and hygroscopic nature, poorly designed connections can split the wood, potentially accelerating decay and risking premature failure.

Design actions and limit states

A design load effect is the action in a member or element that is induced by the loads on a structure. Design load effects include: axial tension, axial compression, bending moment, shear force and reaction forces. These effects have to be addressed in element design. Limit states and load conditions relevant to element design to *AS 1720* are discussed in general in Section 10.

AS 1720 models timber member behaviour in a consistent manner. For the strength limit state, it gives the capacity of a structural member as the product of:

- A geometric parameter – the member's net cross-sectional area
- An appropriate material strength – the timber's tensile, compressive, or bending strength
- A series of modification factors that allow for the effect of the environment and the setting on the strength of the element. These are the *k* factors, listed in Table 45.
- A capacity factor ϕ.

Key limit state conditions regularly influence the performance of different types of elements.

Design of tension members

The most common use for tension members is in trusses. Studs in framed construction can also be used to transmit tensile force from roof tie-down or overturning restraints from roof level to the ground.

Tension members are most often designed for the strength limit state. In some critical truss elements where deflection is important, the members that have been designed for the strength limit state may also be checked against the serviceability limit state. In many cases, the size of a tension member is dictated by the room needed to affect the connections at each end.

Design of compression members

Compression members are most commonly used in the studs in framed construction, columns in all types of structures, and in trusses. Many compression members must also resist bending actions. Load-bearing wall studs carry primarily vertical compressive loads but may also have to resist horizontal load from wind pressure and occasional impact.

Compression members are mostly designed for the strength limit state and, in rare cases, are checked for the serviceability limits afterwards. The two major failure modes are the material's compression failure and member buckling.

Compression failure occurs in stocky members and is where individual fibres are pushed end to end into each other, buckle and push out the side of the member. This maintains the load path's continuity throughout the failure sequence and gives the process very ductile characteristics. Failure of slender members is by member buckling. This a geometric failure and can happen well below material failure loads. Most practical timber compression members are slender, so buckling failure tends to limit the strength limit state capacity.

Design of bending members

Bending members are structural elements subjected to loads perpendicular to their length. The most common bending members are simply supported beams, such as single span joists, lintels, purlins and rafters. Bending members are usually, but not always, horizontal and are often loaded on the narrow face, about the beam's major axis. This is generally the most efficient configuration for resisting bending actions. Forces applied to a bending member result in bending moments (flexure), shear, bearing, and deformations (or deflections). Lateral stability is considered in determining the capacity in flexure. In general, the governing limit states which apply to bending members are:

- For medium loads and span, strength limit state with bending or flexural strength often governing the design
- For heavy loads with very short spans, strength limit state with shear strength usually governing the design
- For light loads and long spans such as roof beams supporting a lightweight steel roof or long span floor beams, serviceability limit state with deflection often governing the design.

Table 45: Modification factors (k_{mod}) for strength

Modification factor	Description	Modification factor	Description
k_1	Factor for load duration	k_{19}	Factor for moisture content of plywood
k_4	Factor for in-service absorption or desorption of moisture by timber	k_{20}	Factor for timber immaturity
k_6	Factor for temperature/humidity effect	k_{21}	Factor for effect of shaving
k_7	Factor for bearing length	k_{26}	Factor related to design load duration
k_9	Factor for load sharing in grid systems	k_{27}	Factor for duration of test
k_{11}	Factor for effect of volume, in tension perpendicular to grain	k_2	Factor for prototype testing
k_{12}	Factor for stability	k_{28}	Factor for effect of sample size and coefficient of variation
k_{13}	Factor for end grain effects	k_{tg}	Factor for grain orientation for single-tapered straight beams
k_{14}	Factor for effect of double shear	k_{tb}	Factor for taper angle for single-tapered straight beams
k_{15}	Factor for effect of seasoning of timber	k_{sh}	Factor for depth and curvature of curved and pitched cambered beams
k_{16}	Factor for plywood or metal side plates	k_r	Factor for radius of curvature
k_{17}	Factor for multiple fastener effects	k_v	Factor for volume/size
k_{18}	Factor for effect of tension loads	k_{tp}	Factor for radial stress

Source: AS 1720.1 Table F2

6.1.1 Connection and joint design

Connection design is often the most complex part of the design of a timber-rich structure. The connections can often govern the required timber section size while ease of construction, aesthetics, cost and product availability must also be considered.

The load-carrying capacity of connections is a function of the timber's characteristics, the behaviour of connectors and fasteners and the joint's configuration. The interactions between these properties is complex and accurate modelling of a connection's behaviour can be very difficult. *AS 1720* provides simplified models for assessing the capacity of various common fastener types. Most connections in timber are designed for the strength limit state and may not even be checked for performance under the serviceability limit state.

This section looks at the interaction between the load direction and timber's capacity and the joint's basic configuration and fixing effects. A joint's performance in fire is discussed in Section 6.3, while its durability is discussed in Section 6.4.

The timber's characteristic and load direction

Timber has anisotropic properties and is stronger in compression and tension parallel to the grain than perpendicular to it. It is also stronger in compression perpendicular to the grain than in tension perpendicular to it.

While the preferable way of designing wood connections is for the wood to be loaded parallel to the grain, this is not always practical or possible. The most common loads found in connections are gravity loads and these are most commonly transferred through contact transfer (or simple bearing). This generates compression perpendicular to the grain at the contact surface and any resulting deformation limits the joint's capacity, rather than strength. Stresses have to be limited to a level that creates a maximum 2 to 3 mm deformation at the point of contact. Overstressing elements in the joint in compression perpendicular to the grain can lead to excessive localised deformation and loss of overall 'plumb and square' in the structure. In turn, these can cause additional eccentric loading and overstressing in other elements. These conditions can be avoided through additional or reinforced bearing in the connection.

Care is still required when applying concentrated vertical load through members loaded perpendicular to grain as localised crushing can occur. Wood is weakest when tension is applied perpendicular to the grain. This condition should be avoided where possible and particular care taken when it is essential.

Joint configuration and fixing effects

Timber is a relatively soft material and tends to crush under concentrated loads. Given this, it is generally preferable to use more small fixings in a timber connection that fewer large fixings. Smaller fixings distribute the load over a larger effective surface area than larger ones. This limits the potential for localised crushing and produces a connection that is generally stronger and with less slip. Spreading the load also builds in a degree of redundancy which is useful in irregular loading events. Generally, applying large concentrated loads though a few large fixings should be avoided unless the connection is designed not to exceed the wood's strength capabilities.

Most mechanical connections include fasteners that penetrate the wood and cut wood fibres. This reduces the member's capacity at that point. While the amount of cutting varies with the fastener type, calculation of the member capacity must be based on the reduced cross-section size. Fastener penetrations can also create localised stress concentrations when forces are redirected around discontinuities in the wood. These stress concentrations can result in tension perpendicular to the grain as the force changes direction.

6.2 Moisture content control

Support Information	Section
Wood's moisture content and dimensional change	7.3
Standard-based MC requirements Species shrinkage and expansion rates	8.3
Performance requirements – moisture	9.3

See WoodSolutions Design Guide 9: Timber flooring

Timber and wood products are hygroscopic materials and will shrink and expand with changes in environmental moisture. If high MCs are maintained, timber products may decay. Design has to limit the potential for adverse MC levels and MC change, constrain the potential for decay, and accommodate likely movement.

Other WoodSolutions Design Guides cover designing timber structures to comply with the requirements of NCC Section F for subfloor ventilation and bathroom design. During design development, these guides should be consulted in detail.

To inform conceptual design, this guide summarises considerations and design approaches to address moisture-induced movement in elements.

Key considerations during conceptual design

- When installed at the correct MC and not unduly restrained, timber and wood products will perform satisfactorily, while expanding and contracting marginally in response to normal changes in environmental conditions.
- In any application, specification of a suitable MC for the timber or wood products is critical to their satisfactory performance.
- In applications with large timber sections or surfaces such as timber floors, design needs to accommodate and manage the potential total movement that may occur through changes in environmental moisture.
- Timber used unseasoned will shrink and may distort as it dries to its equilibrium moisture content (EMC) in service. Design needs to accommodate element shrinkage and its potential impacts on joint performance.
- MC variation has implications for the selection of materials used externally and for the design of joints using multiple rigid fasteners.

MC specification

The MC of specified products needs to correspond with the expected EMC of the material in service. Australian standards define acceptable MC ranges for specific products, such as floor or decking. However, these ranges allow for the variation in MCs expected between regions and seasons across Australia. They are generally too broad for particular projects. Best practice suppliers usually match product MC to likely conditions in their region. In most applications, these MCs should be adequate.

In applications where dimensional stability is critical, the service EMC should be estimated, the timber's MC specified to a target range, and compliance requirements established. The specified MC may be outside the Australian standard range. Critical applications may include large floor installations, major architectural structures, and high performance rigid connections.

Accommodating movement and expansion joints

Expansion joints may be needed to accommodate expected element movement in large or very long elements. Concurrently, ventilation and other provisions may be used to manage MC variation. Section 5.4 of AS 1684.2 details requirements for expansion joints in timber strip flooring on joists in Class 1 buildings. These requirements imply a similar level of provision in other building classes with significant areas of timber strip flooring. In these larger scale applications, particular care is needed in selecting the size of board, its MC at delivery and the conditions under the flooring. This can include controlling the MC of supporting substrates such as slabs, or their isolation from the timber; providing additional ventilation and drainage in subfloor spaces; and, where possible, balancing of EMC conditions above and below the floor. Similarly, the cumulative expansion and contraction of floor joists in multiple-storey timber buildings may require expansion gaskets and joints in elements that connect or pass through multiple levels.

Dimensional change in timber used unseasoned

Element and joint design must accommodate the expected dimension change of unseasoned timber used in a structure as it will shrink and possibly distort as it dries to the surrounding environment's EMC. Cell collapse in some unseasoned eucalypts such as Messmate may compound the overall reduction of the element's size in section.

While longitudinal shrinkage rates are low and rarely reported, longitudinal shrinkage can be significant in unseasoned columns or poles made from certain species. In drying from fibre saturation point (FSP) to 12% MC, low-shrinkage species may shrink 0.01% longitudinally. Anecdotally, high-shrinkage eucalypts can shrink longitudinally by 0.6% or more, or 15-20 mm in a 3 m tall, unseasoned column.

Shrinkage during drying effectively loosens fasteners. Nails may need to be re-punched, and screws and bolts retightened. With large unseasoned elements such as poles, bolts may need to be tightened at regular intervals. Shrinkage across the grain during drying may also split wide timber joined with multiple rigid fasteners.

Moisture-induced breakdown of materials and joints

Some timber products and joints are susceptible to moisture-induced breakdown. Care is required when using glue laminated timber externally, especially in exposed locations. Periodic or seasonal variations in EMC can generate internal stresses in a section as the MC changes between its core and surface. In solid timber sections, this may cause surface checks to form on the board but these are not considered strength reducing. However, in glulam, MC changes combined with the varying grain orientations in laminates can cause differential shrinkage between them. This establishes stress concentrations at the glue-lines and the risk of delamination. This risk increases with the level of exposure. Ideally, glulam used externally should be protected under an eave and coated with an appropriate paint or stain.

Multiple rigid fasteners in a connection exposed to adverse MC change can cause cross-grain stress and element splitting. This is unlikely with nail connections as timber shrinkage or expansion may result in the fastener bending rather than in the timber splitting. However, it can occur with dowel or bolted connections. Cross-grain shrinkage problems can be minimised by:

- Connection design that permits unrestricted movement of bolts across the grain. In the case of steel side plates, use separate plates for each bolt row or slotted holes.
- In timber to timber connections in exposed locations where members frame at right angles, use a single high capacity fastener in preference to multiple fasteners. If multiple fasteners cannot be avoided, placing them close together will help limit differential movement between them.

Support Information	Section
Wood's fire performance	8.4
Performance requirements – fire	9.4

See:
WoodSolutions Design Guides 2 and 3: Timber-framed construction
WoodSolutions Design Guide 4: Timber in bushfire-prone areas
WoodSolutions Design Guide 6: Sacrificial timber construction joints
WoodSolutions Design Guide 15: Fire design
WoodSolutions Design Guide 37: Mid-rise timber building

Fire resistance needs to be considered at conceptual design phase because it may affect the size of members, the loads imposed on the structure, and the type of construction and connections used. Other WoodSolutions Design Guides cover designing timber structures to comply with the requirements of NCC Section C for the building elements' fire resistance and of Section G5 for the design of buildings in designated bushfire-prone areas. During design development, these guides should be consulted in detail and other professional advice sought.

To inform conceptual design, this guide summarises the major methods for ensuring satisfactory fire-resistance with massive and framed timber structures on general sites and those declared prone to bushfire attack.

Key considerations during conceptual design

- Timber and wood products perform satisfactorily in structural elements that do not require a Fire Resistance Level (FRL).
- The approaches regularly used to ensure timber elements have the required FRL are: covering the wood with insulating linings such as fire-resistant plasterboard and using sacrificial timber construction. These approaches are often used in combination.
- The inclusion of sprinklers and similar devices can significantly increase the fire safety in buildings and influence the required FRL of elements.
- Timber elements clad in fire-resistant linings can regularly achieve FRLs of 120 minutes.
- Sacrificial timber construction relies on the timber element's charring and the retention of an adequate structural section during the required fire period. The protection of metal fasteners and connectors can be a critical aspect of this approach.
- Intumescent coatings and fire resistance impregnating chemical treatments can be used on the timber elements.
- Bushfire protection is required for particular external elements in buildings in areas declared bushfire prone, especially windows, doors, and elements close to surfaces where wind-blown embers can collect. Restrictions do not apply to the enclosed internal structure.

Covering with fire-resistant linings

NCC-required FRLs for timber elements can be achieved by covering them with insulating or fire-resistant linings, such as plasterboard. The linings insulate the timber from flame and heat, delay the onset of charring, and allow the element to maintain adequate strength and stiffness for the required period. The level of protection achieved is directly related to the linings' thickness, the covering's continuity, and the regularity of support.

Fire resistant linings are effective in framed and massive timber walls and ceilings. FRLs to 120 minutes are available for load bearing timber stud framed wall and framed floor systems. See Figure 12. These linings are also effective on massive timber components. See Figure 13. As fires burn upwards, the upper floor surface does not need additional protection.

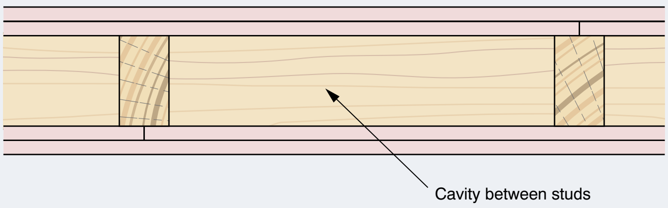


Figure 12: Plan – Fire-resistant lining on a stud framed wall

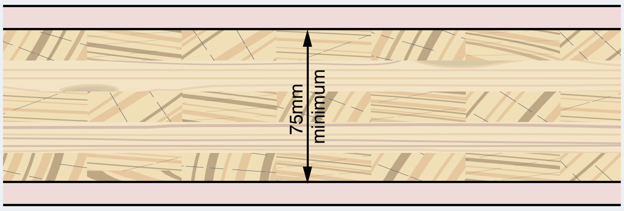


Figure 13: Plan – Fire-resistant lining on a massive timber wall

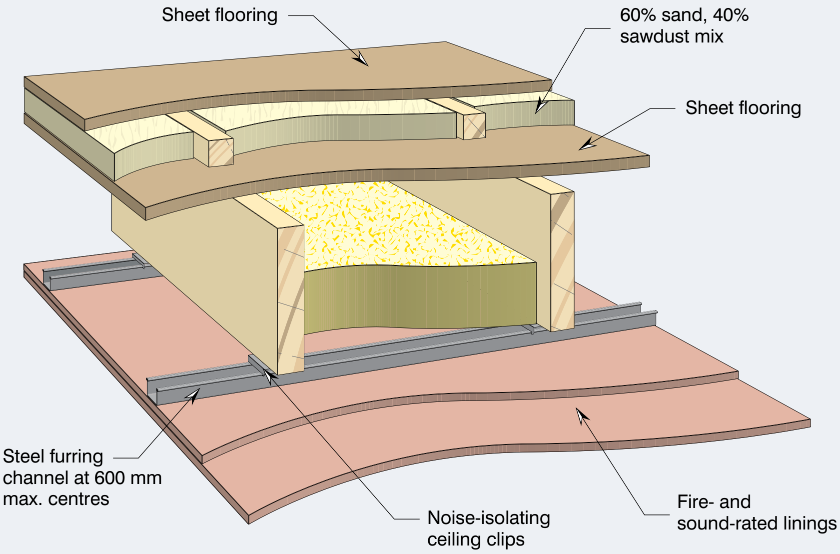


Figure 14: Section – Fire-resistant lining on a framed floor

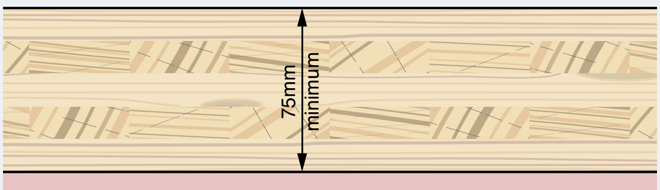


Figure 15: Section – Fire-resistant lining on a massive timber floor

Sacrificial timber construction

NCC-required FRLs for timber elements can be achieved by using the predictability of timber charring. 'Sacrificial' timber can be added to a section or construction assembly and this can protect the inner material from fire damage. This approach is regularly used to complement plasterboard-based fire protection systems in light timber framed construction and provide fire protection for more open roof and floor spanning and support systems. It can also protect connectors in nailplate elements.

AS 1720.4 Timber Structures: Fire-resistance of structural timber members provides a method for calculating the fire resistance level for solid timber and glued timber to timber joints. The depth of charring after a time (t) is:

$D_e = C.t + 7.5$

Where: d_e = calculated depth of charring in mm; C = the species' notional charring rate in mm/min; and t = period of time in minutes

The notional charring rate and other fire hazard properties for readily available species are listed in Table 58. Figure 16 illustrates the profile of the effective residual section after charring.

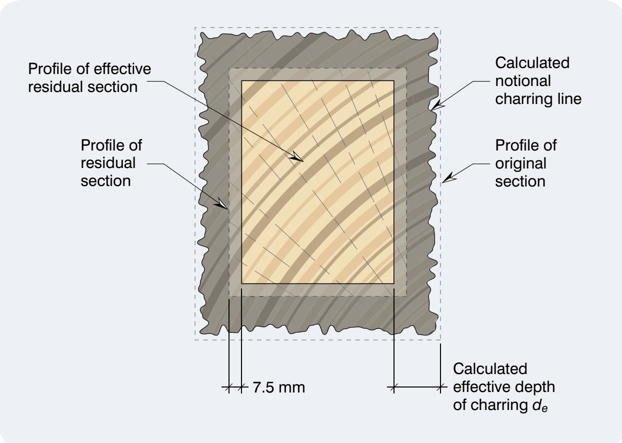


Figure 16: Charring rate and the effective section.

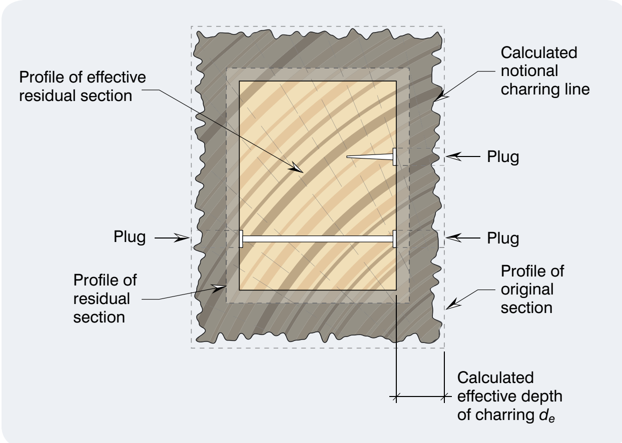


Figure 17: Embedded connectors in the timber.

Protection for fasteners and connectors

Fasteners and connectors such as nailplates, gussets and bolts also need to be protected if the necessary FRLs are to be achieved. Insulating fire-resistant linings, intumescent paint and sacrificial timber plates or plugs can reduce the rate of temperature increase in these metal fixings and allow joints to maintain adequate capacity for the required period. For example, plywood can be applied over nailplate truss connectors in trusses to give them a suitable fire rating.

Bushfire resistant construction

Designing for bushfire is a three step process:

1. Determining the site's Bushfire Attack Level (BAL). A trained assessor is often required for this.
2. Noting the restrictions on building envelope elements for the assessed BAL.
3. Choosing the correct timbers and detailing for the required element performance.

AS 3959 Construction of buildings in bushfire-prone areas sets fire resistance requirements for particular external building elements and this limits the use of untreated timber of specific species. Table 46 provides examples of envelope requirements for each BAL. The standard also classifies the fire resistance of timber species into four groups:

- Bushfire-resisting (listed in Appendix H)
- Density >750 kg/m^3 at 12% MC (listed in Table E1, Appendix E)
- Density >650 kg/m^3 at 12% MC (listed in Table E2, Appendix E)
- Other species (not listed)

Table 59 lists the fire resistance grouping of species to this standard. Intumescent and other fire-resistant treatment or coating systems may be used to improve the timber bushfire resistance.

Table 46: Examples of envelope requirements for each BAL level

BAL	Examples of building materials required externally
BAL – Low	No specific construction requirements. All material allowed.
BAL – 12.5, BAL – 19, BAL - 29	Use specific timbers for doors and frames, windows, cladding and decks.
BAL – 40, BAL - FZ	Fire resistant lining materials, treated glass, special shutters and special building systems

Support Information	Section
Bio-degradation mechanisms	7.4
Species and product durability	8.5
Performance requirements – durability	9.5

See:
WoodSolutions Design Guide 5: Timber service life design
WoodSolutions Design Guide 13: Finishing timber externally

System durability needs to be considered at conceptual design phase because it may affect the type, size and treatment of members, the type and arrangement of connections used, and the provision of eaves, ventilation spaces, and other design aspects. Other WoodSolutions Design Guides cover best practice for the design of durable timber structures and for finishing timber used externally. During design development, these guides should be consulted in detail.

To inform conceptual design, this guide summarises the key considerations and design approaches necessary to ensure a satisfactory service life for timber elements.

Key considerations during conceptual design

- When protected from moisture and termites, timber-rich structural solutions can have long service lives (100 years+).
- Timber and wood products exposed to the elements in bridges, landscape structures, decks and external building envelopes can have effective service lives if suitably designed.
- The project's location and climate and an element's specific arrangement and exposure strongly influence its likely service life and the requirements of effective design.
- Decay and termite attack are the primary hazards to a timber-rich structure's service life on land. In marine conditions, the primary hazards are decay and marine borers.
- Decay can be managed by ensuring the material's MC is kept below 20% or the material is unavailable as a food source. It can be unpalatable to fungi due to the timber's natural decay resistance or its treatment with fungicides or other preservatives.
- Where possible, design should aim to keep external timber elements dry, shed water and allow any retained moisture to dry off.
- Termites can be managed by excluding them from the buildings and making the components unavailable as a food source. It can be unpalatable due to the timber's natural termite resistance or its treatment with insecticides.
- Fasteners and connectors have to be as least as durable as the timber elements they join. Since timber largely resists chemical breakdown, the fasteners and connectors need to resist likely corrosion.
- Consider element inspection, maintenance and replacement during conceptual design.

Service expectations and conditions

Since the durability of timber-rich elements is directly related to their exposure to biological hazards, the service expectations and conditions need to be clearly defined at the beginning of any design process. The service expectations include the required service life and element reliability. Service conditions include the element's exposure to biological hazards and the severity of that exposure due to local environmental and climatic conditions.

It is advisable to use preservative-treated wood or naturally durable wood for all exterior situations and where highly humid conditions are present inside buildings, such as in textile mills, cold-storage plants and swimming pools.

Managing decay

Decay is a primary hazard to a timber-rich structure's service life when it is exposed outside or subject to regular moisture inside. Key approaches in decay control include design to reduce the availability of moisture and keep the timber dry, and specification of timber naturally resistant to the expected decay hazard or unpalatable due to adequate fungicide treatment. Table 61 lists the durability rating of the heartwood of readily available species while Table 62 lists treatment classes and their suitability for applications.

Timber with a MC below 20% does not decay. So, design and detailing to limit decay focuses on keeping the timber dry by shielding it, allowing moisture to run off exposed surfaces, and providing adequate ventilation to dry out moisture retained on surfaces or in connections. Following these approaches is critical for untreated timber and desirable for treated timber.

Trapped water can accelerate degradation, particularly where the timber's end-grain has direct access to the trapped water. When it is impossible to separate the timber from a regular source of moisture, the wood should be preservative treated or be a species with high natural durable.

Shielding the timber from moisture

- Untreated timber and wood products should be protected from direct sun and rain by roofs, eaves and well-designed gutters, isolated from damp surfaces by flashings and damp-proof courses, and placed clear of ponding or splashed water.
- A protective vapour barrier such as paint or stain can exclude moisture from exposed untreated timber. Paint, metal flashings or paraffin-emulsion sealants can be used to protect the element's moisture permeable end-grain.
- A flashing can be provided on the top and end of a beam exposed to the elements. This should be fixed on the element's side and include a ventilated air-space between the flashing and the wood. This allows condensation under the flashing to dry out. Avoid putting fasteners through the top of flashings as this will allow water to penetrate the flashing and the wood.

Allow moisture to run off

- The upper surfaces of exposed timber members should be pitched or sloped to shed water.
- Details, especially connections, should be arranged to prevent moisture being trapped. Joints should be free draining.
- Subfloor spaces and the surrounding ground should be graded to drain water away from the building and foundation walls.
- Avoid installing fasteners through the top of members because this creates a path for moisture to travel into the heart of the wood. Moisture on the surface may dry out. Moisture in the core will collect and foster decay.

Providing adequate ventilation

- Where possible, allow for ventilation around timber members and connections to reduce their MC.
- Support openings in masonry walls for timber beams or joists should have a damp-proof course on the base and an air space around the top, sides and ends of the wood members. If the members are below the outside soil level, a flashing should be provided on the end of members.
- Provide adequate ventilation between the ground and timber floors or decks. Avoid placing timber next to unventilated and inaccessible spaces.
- Provide adequate air flow under metal flashings. Avoid poorly ventilated flashings that can trap moisture in the timber and accelerate decay.

Managing termites

Termites are a primary hazard to a timber-rich structure's service life. Key approaches in termite control include design to exclude the insects from edible timber and other cellulose in the building, specification of timber naturally resistant to the expected termite hazard or unpalatable due to adequate insecticide treatment, and regular inspection of the timber and paths from the ground to the timber. Galleries indicate termites are active. Table 61 lists the termite resistance of the heartwood of readily available species while Table 62 lists treatment classes and their suitability for applications.

Physical or chemical termite exclusion systems are available. A suitable system should be included in the design and maintained. In addition, treated timber can be used in areas where the risk of severe termite attack exists.

Adequate connection service life

The quality and reliability of fasteners and connectors should match the timber elements' expected service life. Particular care is needed in marine structures, swimming pool enclosures or industrial environments. In these cases, high grade stainless or galvanized steel should be specified. Avoid contact between dissimilar metals in fasteners, connectors, flashing or the wood. Some wood treatments can react with the metal in the connectors and salts in the air of corrosive environments, accelerating corrosion or wood breakdown.

Where high strength connections restrain the timber's movement, stresses can be generated and split the timber. Sealing the timber to prevent moisture ingress, and the use of single fasteners or detailing connections to allow movement can help prevent this.

Inspection and maintenance

Inspection and maintenance are essential to ensuring a timber-rich structure's service life. While detailing to replace exposed elements is part of detailed design, providing access to critical elements for inspection and maintenance is a necessary part of conceptual design. Periodic inspection is necessary to reveal:

- early indications of moisture penetration, condensation or decay
- incursions by termites
- the integrity or breakdown of protective barriers or coatings.

Support Information	Section
Acoustic performance	8.6
Performance requirements – acoustic separation	9.6

See:
WoodSolutions Design Guides 2 and 3: Timber-framed construction
WoodSolutions Design Guide 11: Systems for external noise
WoodSolutions Design Guide 37: Mid-rise timber building

Acoustic separation needs to be considered at conceptual design phase because compliance may affect wall configuration, flooring system selection, and the floor structure's mass. Other WoodSolutions Design Guides cover designing timber structures to comply with the requirements of NCC Section F for limiting sound transmission and ensuring acoustic insulation between parts of Class 2, 3 and 9c buildings. They also cover satisfying the fit-for-purpose or market-driven requirements for more rigorous acoustic performance. During design development, these guides should be consulted in detail.

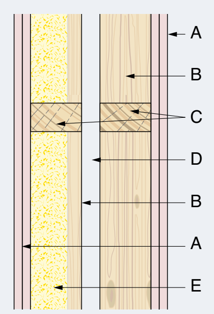
To inform conceptual design, this guide summarises the major methods for ensuring the structural system selected provides satisfactory acoustic separation in apartment buildings and between floors in office and similar buildings.

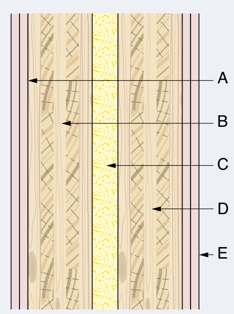
Key considerations during conceptual design

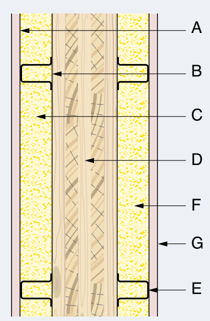
- In buildings without sound separation requirements, timber-rich solutions generally provide acoustic performance superior to systems based on harder materials. As timber absorbs part of any sound that hits it, timber solutions are quieter and have noticeably less echo.
- Acoustic separation can have a greater impact on sole occupancy unit (SOU) bounding wall configuration than fire separation. Achieving adequate sound separation in walls generally requires a double stud or an adapted single stud system and insulation.
- Achieving adequate sound separation in the floors between SOUs generally requires only insulation and noise-isolating furring connections.
- Upgrading the acoustic performance of floors in SOU or meeting market requirements for sound separation between floors in office and other buildings often requires the inclusion of additional floor layers and mass, such as concrete toppings, mixed sand and sawdust layers, or similar systems. These increase floor loads.
- With careful design, timber, plywood and similar materials provide excellent sound-moderating and reflecting surfaces in public spaces and special purpose buildings.

Walls separating SOUs

Adequate acoustic separation in the walls between adjacent SOUs usually requires a double stud system: two wall frames separated by insulation and an air-gap. A similar approach is used with massive timber systems with the space between the mass timber walls filled with insulation. See Figure 18. Connection between the frames is kept to the minimum necessary for structural adequacy.







Acoustic performance: $R_w + C_{tr} \geq 50$	Acoustic performance: $R_w \geq 50$	Acoustic performance: $R_w \geq 50$
Configuration: A - 2 x 13 mm plasterboard B - Timber studs typically 90 x 45 C - Noggings at mid-height D - Cavity min 20 mm E - 75 mm mineral wool insulation	**Configuration:** A - 2 x 13 mm plasterboard B - 95 mm CLT C - 60 sound absorbing insulation D - 95 mm CLT E - 2 x 13 mm plasterboard	**Configuration:** A - 16 mm 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 plasterboard

Figure 18: Acoustic performance of regular SOU separating wall system

Separation between floors

Effective impact sound insulation is the main challenge in timber floor design. The most practical approaches are to reduce impact effects with carpet or a floating floor on underlay at the point of contact and to deploy a high mass material in the floor assembly to deal with low-frequency sounds.

As a result, upgrading the acoustic performance of floors in SOU or meeting market requirements for sound separation between floors in office and other buildings often requires multiple layer systems. These can include: a floating timber floor on an acoustic mat; high mass components such as a concrete topping; dry concrete tiles or sand and wood shaving mixes contained between battens; insulation; resilient ceiling mounting systems and fire rated plasterboard ceilings; or structural independent ceiling support systems. These approaches can be deployed on joist or massive timber floor systems.

Whichever methods are selected, the additional layers required to improve sound separation between floors will increase **loads on the spanning and support systems.**

Acoustic moderation

With careful design, wood can provide excellent sound-moderating and reflecting surfaces in public spaces and sound-sensitive buildings. Timber acoustic paneling increases sound absorption, and breaks reflected sound, reducing echo.

6.6 Thermal performance

Support Information	Section
Thermal performance	8.7
Performance requirements – Thermal performance	9.7

See WoodSolutions Design Guide 22: Thermal Performance

The building's thermal performance needs to be considered at the conceptual design phase because compliance may affect the thickness and configuration of wall and roof support systems. WoodSolutions Design Guides cover best practice in designing timber structures to comply with the requirements of NCC Section J for the thermal performance of the fabric of all classes of buildings. During design development, these guides should be consulted in detail.

To inform conceptual design, this guide summarises considerations and design approaches necessary to efficiently ensure that the required level of insulation can be incorporated and potential condensation problems are avoided.

Key considerations during conceptual design

- Timber-rich solutions generally provide better thermal performance than steel, concrete or masonry-based systems. Warm to the touch and a useful natural insulator, timber does not provide a ready thermal bridge through insulation in the building's envelope.
- Well-designed timber-rich solutions can provide effective levels of thermal comfort in a wide range of climates.
- Regulated insulation levels in the building envelope can establish effective minimum depth for rafters, purlins or external wall frames. These may be larger than the sizes required for structural adequacy.
- Market-required insulation levels can significantly exceed regulated levels. The depth and spacing of structural elements may need to accommodate the required insulation.
- The increased use of insulation and building wraps can lead to condensation in buildings where significant temperature differences are likely to occur between the internal and external environments.

Accommodating insulation

Regulatory compliance and market expectations require an effectively continuous layer of insulation around the conditioned spaces in buildings. The required insulation level varies with the local climate and its location in the building, with the highest level required in the roof. While many forms of insulation are available, bulk insulation is the most economic and the required thickness of insulation has to be installed between or over the structural elements.

In practice, this can mean that: required insulation thicknesses may govern rafter, purlin and external stud depths rather than simple structural adequacy; upstands are needed at the support points of gable-shaped roof trusses; and I-beams may be preferred over solid timber sections for some elements. I-beams are less of a thermal bridge than a solid timber section.

Support Information	Section
Timber's environmental edge	1.1
Environmental performance	8.8
Performance requirements – Environmental	9.8

See WoodSolutions Design Guide 22: Thermal Performance

Enhancing the structure's environmental performance is a key consideration at conceptual design phase as it is much more efficient to apply environmentally-responsible design approaches from the beginning of the design process than attempt to implement them after initial decision have been made. Other WoodSolutions Design Guides cover best practice for designing timber structures to provide thermally comfortable buildings and limit the energy used to heat and cool structures.

To inform conceptual design, this guide summarises design approaches necessary to optimise the environmental characteristics of the building's structure.

Key considerations during conceptual design

- Timber-rich solutions provide superior environmental performance when compared to steel, concrete or masonry-based systems. Wood is a renewable material and a net store of atmospheric carbon.
- With timber-rich construction, the carbon sequestered in the timber can offset the emissions in complementary materials such as concrete in foundations and steel in roof sheeting and connectors.
- In most countries, forestry practice operates in a strong regulatory environment. Forest certification exists for many local and imported products.

Low carbon construction

Many environmental design guides focus on the carbon emissions generated through a building's operational life without addressing the carbon emissions embodied in the materials used to make the building. Advanced timber construction options are forcing this to be revised. Recent LCA studies have shown that with improved building thermal efficiency, the carbon embodied in the materials is a significant proportion of total lifecycle impacts. As 50% of wood's mass is atmospheric carbon and net carbon storage remains after production emissions are deducted, timber elements can offset the carbon impacts of other materials used in the building. With good thermal design, a viable renewable energy strategy, and sufficient timber in the building's structure, a design's total lifecycle embodied and operational carbon emission can be zero.

Strong regulatory environment for forestry

A strong regulatory environment exists for forestry operations in most developed and many developing countries. This can assure designers that the vast majority of timber in the market is legally and responsibly harvested. Forest and CoC certification reinforce this assurance. Certification is usually available for most material from the forest to the last major wholesaler in the supply chain.

Forest and CoC certification are voluntary, market-based systems. They are also expensive and organisationally difficult processes to establish and maintain, especially for small community-based processors in developing countries. Care is needed in specifying material from developing countries. While illegal logging may occur, community forestry and milling in these countries provide necessary local employment and income.

Support Information	Section
Construction and prefabrication options	5

Component and material procurement need to be considered during the conceptual design phase because the availability of skilled fabricators and suitable product can influence which structural options can be economically achieved in the project. As support guides rarely cover day-to-day material supply and contractor skills, designers need to develop an awareness of local supply capacity. To inform conceptual design, this guide summarises approaches to ensure efficient procurement of timber-rich solutions.

Key considerations during conceptual design

- A developed material supply and element fabrications network exist to support timber-rich solutions, particularly for Class 1 buildings.
- As timber fabrication skills increase in response to increased demand, early collaboration between designers and potential component and material suppliers is highly desirable.
- Given the ease of working with timber-based systems, domestic carpenters and existing ‘frame and truss’ fabricators often have the skills to assemble more complex commercial building solutions.
- Supply of material from fabricators based overseas is practical if the intended construction program allows for product delivery lead-times.
- In complex buildings, fabricators used to making complex steel elements may be better structured to supply a full systems solution than those more used to relatively simple timber components.

Understand and use the local skill base

Designers need to develop an awareness of the supply capacity near the project. Early engagement and collaboration with the local fabricator and material supply base can be a necessary part of developing an efficient design solution.

A developed fabricator and material supply base exists for delivering timber-rich solutions. Fabricators exist in most major centres, have access to sophisticated proprietary software for timber component design, can prefabricate components, and are capable of providing effective and economical solutions for most small to medium scale buildings. Similarly, a significant labour pool exists among carpentry subcontractors skilled in assembling timber components. However, this supply chain is currently focused on providing solutions for Class 1 buildings and may be inexperienced with the requirements of other building types. Also, generic product information may not indicate a timber supplier’s full capacity or product range. For example, many glulam manufacturers produce large elements on order.

International supply

As timber fabrication and product options can be more developed overseas than in Australia, designers may want to incorporate imported components or connectors in their designs. As suppliers of these products often have local affiliates, this can be an efficient way to achieve an innovative design solution. However, allowance needs to be made in the construction program for these products to be made and delivered.

7 Material Basics

Wood is fundamentally different in its production and properties to the majority of materials regularly used to construct building and civil structures. To design with wood efficiently, engineers and other design professionals must understand and appreciate key differences between wood and other materials, and then between different species or types of wood.

Timber and wood products are natural, made from the wood recovered from the trunks of trees in a relatively low energy and often low capital process. The recovered material is not transformed and is used effectively as the tree made it. The species' characteristics and the wood's form and growth in the tree over time strongly influence the characteristics and properties of the products recovered. As a result, wood is:

- **Variable**. Its properties vary between species, between trees in each species and across the recovered log.
- **Strong but its properties are anisotropic**. They vary with the grain direction.
- **Hygroscopic**. It gains and loses moisture to be in equilibrium with the surrounding atmosphere.
- **Biodegradable**. It is susceptible to mechanical damage, fire and organisms that use it as food.
- **Easy to produce and use.** It can be processed with relatively simple equipment and is available from a range of suppliers.

To manage this natural variability, timber and wood products are sorted, or graded, into broadly consistent groups before being used with confidence in applications.

7.1 Wood is variable

Timber and wood products are recovered from trees of different species and age, and from different sections of the log. This generates variability in the material's properties and a need to sort the material into groups with more consistent properties.

7.1.1 Variability between species and trees

Trees are living organisms and their growth varies according to their genetic characteristics, the local environmental conditions and their age. Like most organisms, tree stems include cells that serve particular functions in the tree and have different properties. These variations affect the material's character and behaviour in production and use.

Softwood and hardwood

There are two major botanical groups of tree species available for wood production: softwoods and hardwoods. Softwood trees generally have seeds in cones and needle-like leaves and the wood is usually light in colour and less dense than most hardwoods. Common Australian softwoods include Radiata Pine, Cypress Pine and Hoop Pine. Hardwood trees have seeds from flowers and broad leaves. Their wood is generally darker in colour and denser than softwoods. Common Australian hardwoods include all *Eucalyptus* species trees, such as Jarrah, Messmate and Blackbutt, and all *Corymbia* species trees, such as Spotted Gum.

There are significant genetic and anatomical differences between softwood and hardwood trees and the timber from them. The cells in softwood timber are more open and regularly arranged and have thinner walls than those found in hardwoods. They are often aligned in parallel straight radial rows. Hardwoods are more complex organisms and their wood has more cell types. Several types are often smaller than those found in softwoods and have thicker walls. They are also arranged in a more complex wood matrix around large vessel cells.

Age effects

Many wood properties can vary with the tree's age at harvest. Like many organisms, trees have an adolescent period and during this stage, the properties of the wood produced from one year to another can change. Wood density and related properties such as stiffness and strength generally increase. As trees mature, the rate of change in wood properties per year slows until the properties of the wood established each year are more consistent.

Given these age effects, the properties of timber recovered from young trees of a particular species will differ from the timber from older trees of the same species. Also, the properties of timber recovered from the outside section of a log from a young tree will differ from those in the inner part of the same log.

7.1.2 Variability across the log

Trees grow to capture the available sunlight and resources. In good growing conditions, the trunk is straight and its diameter is largest at its base. As it grows, the tree makes new wood and alters the properties of existing wood to maximise its chances for survival and growth. Trees grow both outwardly as new wood cells develop at the plant's growing tips and radially from a special layer of growth cells, the *cambium*, directly under the bark. This layer generates new wood cells inwardly towards the tree's centre and outwardly to form protective bark. So, the youngest wood is on the outside of the stem and the oldest wood is in the centre.

The availability of light, nutrients and water influence cell development. As seasons change and growing conditions become less favourable, cell development slows and *latewood* forms. Latewood is higher density wood that has relatively smaller cells with thick walls. When conditions improve, the rate of cell growth increases, and the cambium produces *earlywood*. This is lower density wood with relatively bigger cells with thin walls. This cycle of latewood and earlywood production creates the growth rings visible on the surface and the end grain of most timber.

Figure 19: Native forest hardwood log

Figure 20: Plantation hardwood log

Figure 21: Plantation softwood log

The tree alters the properties of discrete zones of its wood cells to serve specific functions. Working toward the centre of the stem from the cambium layer, the zones are:

- **Sapwood**. This is the newest wood on the outside areas of the stem that transports and stores nutrients between the root and the leaves. Its cell walls increase in thickness as the tree grows. As it contains starches and lacks protective extractives, the sapwood is attractive to insects and fungi and is always classed as low durability.
- **Heartwood**. This is older wood towards the stem's centre. Often filled with extractives and other material that increase its durability, its primary role is to support the tree. In hardwoods, it is often significantly darker than the paler sapwood. As new wood forms each year, some of the inner sapwood isn't needed to transport nutrients and is converted into heartwood.
- **Pith** or **heart** in the stem's centre. This is low-quality wood from the original sapling or growing tip. It generally has low strength and durability, and high shrinkage rates.

When logs are processed, wood from each of these zones can be incorporated in the products.

7.2 Wood is strong with anisotropic properties

The tree's needs and growth influence the strength and uniformity of properties in timber products. The tree stem has to resist gravity loads and bending stresses generated by wind and other forces on the stem or canopy. The tree's cell structure has evolved to resist these loads. Also, the stem's growth is both progressive, as its circumference increases with age from the older centre to the younger outside of the stem, and intermittent throughout the seasons, as growing conditions improve and then worsen, particularly in temperate and colder climates.

As a result, wood has anisotropic properties, particular its strength, stiffness and related structural characteristics.

7.2.1 Anisotropic wood properties

The tree's growth pattern establishes three distinct directions in the wood cells and grain. Shown in Figure 22, these are: radially or perpendicular to the growth rings; tangentially or around the growth rings; and longitudinally or along the grain and the axis of its cells. Wood's properties differ in each of these directions. For example, the timber hardness longitudinally is higher than hardness measured radially or tangentially. Also, dimensional change in each grain direction with changes in moisture content is significantly different.

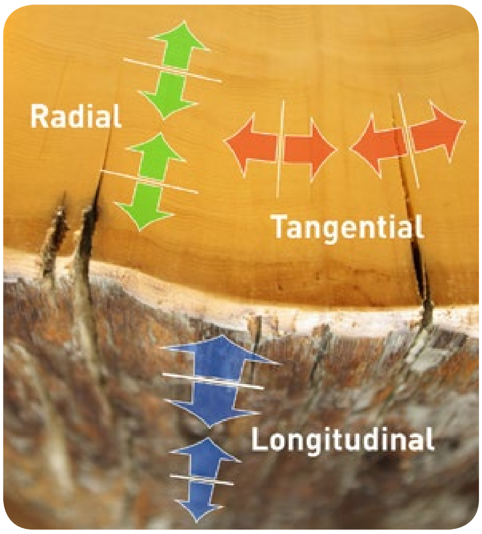


Figure 22: The three principal directions of wood grain

7.2.2 Anisotropic strength characteristics

Timber's strength, stiffness and related characteristics are anisotropic. They relate directly to: the wood's cell structure - their size and arrangement; the cell direction relative to the applied load; and its moisture content (MC).

Timber's cells are arranged like a group of tubes joined together. As shown in Figure 23, the cells are strong and stiff if loads are applied along the line of cells, parallel to the tubes. If the cell walls are relatively thick, they will be stronger parallel to the tubes than when the cell walls are relatively thin. As species characteristics largely determine cell structure and arrangement, wood from a single species tends to have similar structural properties, and the properties of each species will differ to a greater or lesser degree to the properties of other species.

Figure 23: Strength along the grain

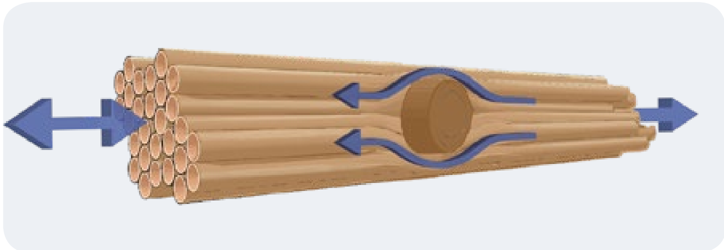


Figure 24: Strength and grain impact of knots

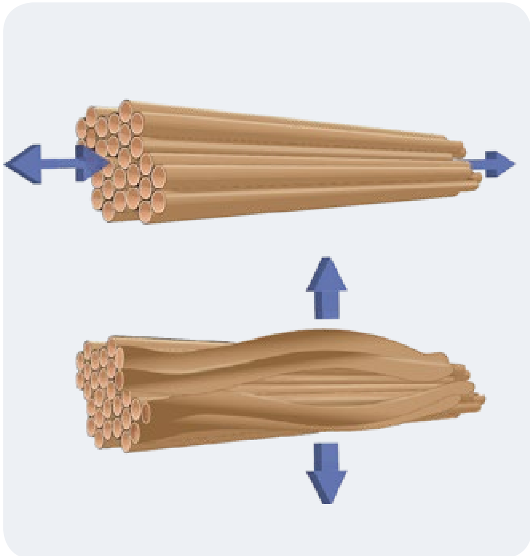


Figure 25: Strength perpendicular to grain

The cells are much weaker when load is applied across the tubes. As shown in Figure 25, the cells can pull apart or crush relatively easily. The reduction in strength with the applied load's direction is significant.

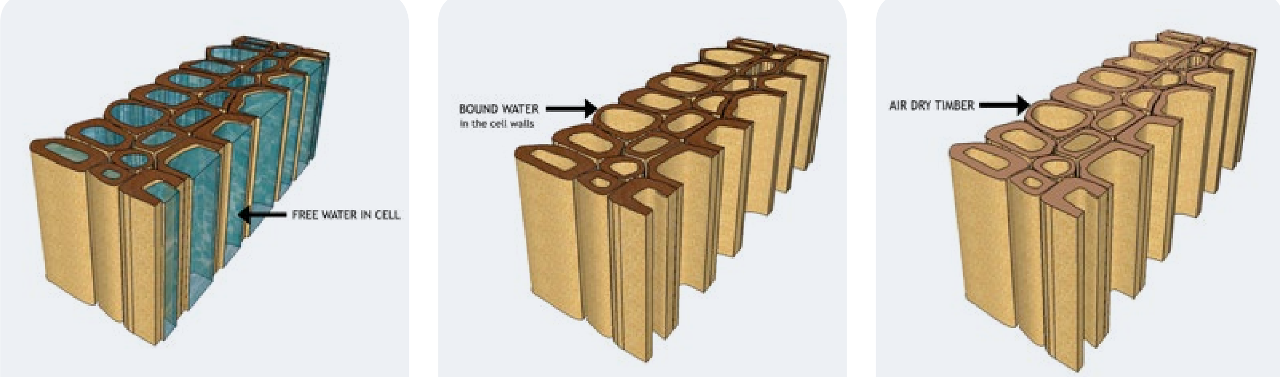
Obviously, as soon as the direction of loads moves away from running directly along the cells, the timber's capacity to resist them reduces quickly. This can occur naturally. As shown in Figure 24, when the cells sweep around an obstruction in the wood, like a knot, the load applied along the piece has to be resisted across the cells near the knot. This significantly weakens the piece's strength. When combined with weakness that results from the knot itself – there can be a significant reduction in the piece's structural capacity.

Lastly, the wood's MC influences its structural capacity. The cell wall's rigidity is related to the amount of water in the wood matrix. As the cell dries, its walls harden and can resist a higher load than cells with a higher MC. The effect is significant. Seasoned timber's structural properties are between 10% and 30% higher than those of unseasoned material of the same species and grade.

7.3 Wood is hygroscopic

Timber in production or service contains water. The amount of water contained in wood at a particular time is known as its *moisture content* (MC). Timber's MC affects many of its performance characteristics such as strength, shrinkage, and durability. Wood's MC is defined as the weight of water in the piece as a percentage of the weight of its dry wood fibre.

The MC of wood in the tree or buried in wet ground can be greater than 100%. This water can occur in the cell wall, in the cell's lumen and in the cavities between the cells. The water inside the cavities or lumens is called free water. The water weakly chemically bound in the cell walls is called *bound water.* When timber is dried after processing, the free water dries off first. Fibre saturation point (FSP) is the point when the cell walls are saturated with bound water but the cell cavities are free of water. This occurs when the timber is about 25 - 30% MC. Timber's MC in service in the open air under cover varies with the local climate and conditions but generally ranges from 10% to 14%.



Unseasoned timber ***Timber at fibre saturation point*** ***Seasoned timber***

Figure 26: Types of water in timber

Equilibrium moisture content (EMC)

Timber is a hygroscopic material. It gives off and absorbs moisture to remain in equilibrium with the surrounding atmosphere. This moisture exchange is dynamic. As environmental humidity increases, moisture enters the timber until the two are balanced. When environmental humidity decreases, moisture leaves the timber. After timber is initially processed and its MC is high, it loses moisture until it reaches its EMC. This is the MC level at which timber neither gains nor loses moisture from the surrounding atmosphere. There is a direct relationship between the relative humidity and temperature of an environment or space and the EMC of wood in it.

Change in wood's MC occurs gradually through the piece. Moisture is gained or lost first on the exposed faces. If external conditions are damp, moisture migrates into the wood and eventually into the piece's core until it reaches equilibrium. If external conditions are dry, the process is reversed and moisture is gradually lost. This moisture movement and change in the piece's MC occurs at different rates with different species. Generally, softwoods have a relatively porous cell structure and their MC can change relatively quickly. Hardwoods have a non-porous cell structure and their MC changes relatively slowly.

Moisture and dimensional change

As water in the timber forms part of the wood matrix, timber's dimension changes with MC changes below FSP. Wood below FSP will shrink when it loses moisture and expand as it absorbs moisture. This shrinkage or expansion occurs at different rates in the three principle grain directions: radially – across the growth rings, tangentially – around the growth rings, and longitudinally – along the grain, and at different rates for each species.

Timber shrinks and expands most in the tangential direction and less radially. Longitudinal shrinkage is usually low. Local eucalypts shrink and expand more than most softwoods. The difference between tangential and radial shrinkage rates can result in the distortion in a board's cross-section as the timber dries. Effectively, the growth rings tend to straighten. This is most noticeable in back sawn boards as they tend to cup across their width. See Figure 27.

Given this relationship between MC and the piece's size, managing timber's MC is a critical part of its production, its use in design and construction, and its management in buildings in service.

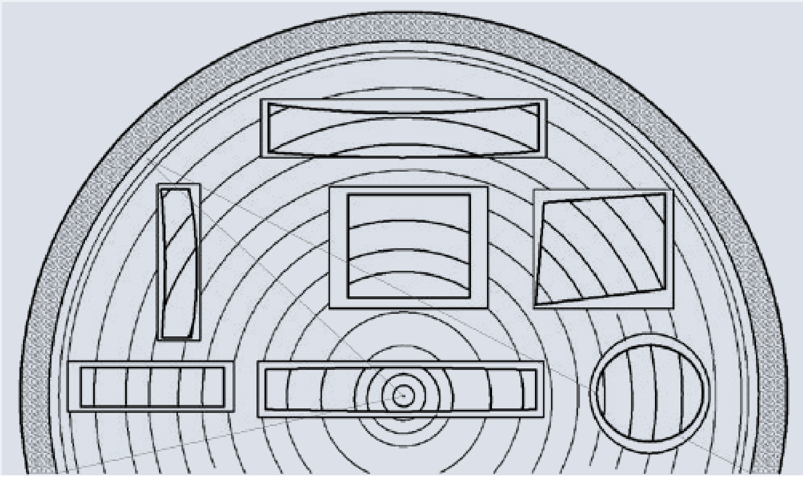


Figure 27: Characteristic shrinkage of boards with different growth ring patterns as the wood dries.

7.4 Wood is biodegradable

As a natural material, wood is biodegradable. Organisms have evolved that use wood as food. Light can break its surface down chemically and it is susceptible to mechanical damage and fire. As the impact of these mechanisms varies with the timber's exposure to the hazard and the wood's nature, they can be managed through careful design and specification.

7.4.1 Natural breakdown mechanisms and hazards

Natural breakdown by decay

Decay is the softening, weakening or total decomposition of wood by fungi: microorganisms that eat organic material. To survive and grow, wood-eating fungi need: wood available as food; a MC in the wood above 20% and below 60%; oxygen and a temperature between 5° and 60°C. Temperatures between 25° and 40°C are ideal for fungal growth. If any of these conditions is absent, decay cannot start or continue if it is present.

In practice, decay rates in timber vary according to its MC, the ambient temperature and the natural durability of the wood. Its durability varies with its species and the type of wood: sapwood, heartwood or pith.

Decay occurs most readily in timber kept regularly moist and particularly on timber end-grain where moisture can enter the piece, in starch-rich sapwood and in the central pith in hardwoods that have low natural resistance to attack.

Natural breakdown by termite, and other insect and organism attack

Insects and similar organisms can break timber down naturally. Termites are the most significant insect pests for timber and they occur in most parts of Australia. Several species cause commercial damage by eating the cellulose in buildings, structural and lining timber, paper off plasterboard, and even plastics. There are two types of wood-eating termites:

- **Subterranean termites.** These have colonies in the ground and need a constant source of moisture. They cannot live in sunlight and build tunnels or galleries through the earth and around obstructions to get to a food supply.
- **Drywood termites.** These are found in damp tropical climates, mostly in north Queensland. As these termites rely on moisture in the atmosphere, they don't require contact with the ground and are harder to detect than subterranean termites. Attack is usually slow and mainly confined to sapwood.

Other insects and similar organisms that can break timber down naturally include beetles and their larvae on land, and molluscs, worms and other organisms in water. The larvae of lyctid beetles, also known as the lyctid borer or the powder post borer, eat the sapwood of susceptible hardwood species. Susceptible sapwood can be attacked at any time, even after years in service. Because of this, Australian standards and state marketing laws limit the inclusion of susceptible sapwood in timber elements. A wide range of molluscs, crustaceans and other marine organisms can also eat untreated timber. The severity of attack in any location is influenced by several factors, including the ambient temperature and water salinity. In any location, the greatest attack will generally be in the tidal zone. Attack is also more severe in tropical and subtropical waters.

Natural breakdown through weathering

Weathering is the greying and minor cracking of unprotected timber as a result of mechanical or chemical breakdown of the wood surface. This can be due to: the scratching action of dust, sand and other material carried by the wind; the shrinking and swelling of surface fibres due to variations in MC; freezing and thawing of moisture in the timber; or exposure to oxygen and UV sunlight, which break down the structure of wood molecules.

Generally, the sequence of weathering of unpainted timber exposed to sunlight starts with the surface discolouring. It will then fade and bleach to silvery grey as the surface roughens, and may check, crack or splinter. Weathering may also cause cupping and warping of boards, surface staining, and eventually growth of surface moulds. Weathering does not include decay and preservative treatments do not reduce it. Treated timber weathers at the same rate as untreated timber of the same species.

The rate of weathering in timber depends on the level and intensity of the element's exposure. Shaded wood may take decades to weather while exposed wood may weather noticeably in less than a year. The effects of weathering are often limited to the surface and usually occur slowly.

Natural breakdown by fire

Very high temperatures can break timber down into flammable components that can burn away. The rate of thermal breakdown and burning is directly related to the timber's density. The denser the timber, the slower it is consumed. From 100° to 200°C, timber dries out, creating water vapour and other non-combustible gases. From 200° to 300°C, some parts of the wood begin to undergo significant *pyrolysis*. This is the thermochemical breakdown of wood that occurs when oxygen is absent. From 300° to 450°C, flammable gases are created that can ignite into flames if mixed with air. Decayed wood has a lower ignition point and burns more quickly.

Wood burns from the surface towards the core. As it burns, charred wood is left behind, creating a layer of charcoal over a zone of partially degraded wood. This char layer acts as an insulator protecting the timber's inner core, keeping its temperature relatively low and slowing combustion. This unaffected wood retains its structural capacity. If high temperatures continue, the charred layer burns away and the partially degraded zone extends further into the piece, until the section is completely consumed.

7.4.2 Natural and treated resistance to breakdown

The impact of these mechanisms or hazards varies with the timber's exposure to the hazard and the wood's nature. Timber and wood products can resist these hazards through their *natural durability,* their resistance to fungal and insect attack, or by the use of applied preservative treatments. Timber's natural durability varies with its species. Timber's *treated durability* varies with the type and retention of preservative chemicals in the wood. These are described in greater detail in Section 8.5.

7.5 Wood is easy to produce and use

Timber and wood products can be easy to produce and are always relatively easy to use. While sophisticated processing systems are used to manufacture timber and wood products industrially, timber rounds and boards can be effectively produced with very simple equipment, such as axes or Lucas mills. Large and robust timber structures with spans over 50 metres have been built in Australia from packs of green timber, bags of nails and with a tool kit containing no more that hammers, handsaws, bevels, and block and tackle. In modern construction, timber systems offer construction speed and efficiency that other major material systems find difficult to match. Timber's characteristic ease of construction presents designers with a range of opportunities in material selection, construction approach and structural form.

7.6 Grades and grading

Grading is a process of sorting a naturally variable material against established limits into groups of products with similar properties and characteristics. This can be by visual assessment, machine assessment or a combination of the two.

A *grade* is the name or term used to describe the sorted group of material. A *grade definition* is the list of criteria that establishes the properties and characteristics acceptable in the group and the limits set for each. Grade definitions for timber and wood products usually bundle their criteria and limits into two groups, dealing with: the form of the piece – its size, shape, straightness, and similar characteristics; and the wood's quality in the piece – its appearance or structural capacity.

Customer demand and regulatory requirements are the primary drivers for grades and the criteria in grade definitions. Given this, grades and grade definitions vary with the product's intended use in applications, and with the capacity or perceived material quality. Specific grade structures exist for: structural and appearance boards; glue laminated products; structural and appearance plywood and laminated veneer lumber (LVL); and for most other types of wood products.

There are also progressions of grades for products of the same type. For example: a series of grades exist for sawn timber with differing levels of structural capacity, such as MGP 10, 12 and 15.

Grades and grade definitions are critical to effective timber supply and use as they establish confidence along the supply chain. They allow designers to define what they want from producers or suppliers, and producers to define what they can provide.

8 Material Properties

This section describes the properties of the timber and wood products suite in relation to the major performance requirements for buildings and structures, such as structural, thermal and environmental performance.

8.1 Basic properties

Density

Wood's density is largely determined by the tree's species, growing conditions and age at harvest. It is a useful and easy-to-measure indicator for other key timber properties such as strength and stiffness, fire resistance and ease of machining.

Wood's density (D) is defined as the mass (M) of wood substance and moisture enclosed within a piece divided by its volume (V). As a piece's mass varies with the moisture in the piece, timber's density is often expressed at a specified moisture content (W), usually 12%.

$$D = \frac{M}{V} \cdot \frac{100}{(100 + W)}$$

Equation 1: Wood's density

Density measurement is complicated because timber below fibre saturation point shrinks as it loses moisture or expands as it takes it up. This changes the piece's size, its weight and its calculated density. So, separate terms are used for density measured at particular times. Density at 12% MC is called air-dry density.

Joint group

Density is used as a proxy for the nail holding and joint capacity of timber species. For joint design, species are classified into one of six joint groups for unseasoned timber, J1 to J6, and for seasoned timber, JD1 to JD6, based on the average species density. Table 47 lists the relationship between density and joint group while Table 48 lists the densities of readily available species.

Table 47: Density and joint group relationship

Unseasoned timber						
Joint Group	J1	J2	J3	J4	J5	J6
Basic Density kg/m^3	750	600	475	380	300	240
Seasoned timber						
Joint Group	JD1	JD2	JD3	JD4	JD5	JD6
Air-dry Density (kg/m^3 at 12% MC)	940	750	600	475	380	300

Source: AS 1684.2-2010_Table 9.15.

Hardness

A species' hardness is a strong indicator of the timber's ability to resist abrasion and indentation in use. Hardness is typically indicated by the board's Janka hardness given in kiloNewtons (kN). A high value indicates a hard timber.

Table 48: Density and hardness of readily available species

Species	Colour	Density (kg/m^3 at 12% MC)	Hardness	Janka rating (kN)	Strength group (seasoned)
Hardwoods					
Blackbutt	Pale straw to light brown	900	Hard	9.1	SD2
Jarrah	Dark red brown	800	Hard	8.5	SD4
Messmate	Pale straw to light brown, pink	750	Moderately hard	7.1	SD3
Mountain Ash	White, yellow, pale straw to light brown	650	Firm	4.9	SD3
Southern Blue Gum	White, yellow, pale straw to light brown	1000	Hard	12	SD2
Spotted Gum	Brown, dark brown, light sapwood	1100	Very hard	11	SD2
Softwoods					
Cypress Pine	Pale straw sapwood, dark brown heartwood	700	Moderately hard	6.5	SD5
Douglas Fir	White, yellow, pale straw to light brown	560	Soft	3.1	SD5
Hoop Pine	White to straw	550	Soft	3.4	SD5
Radiata Pine	Straw	550	Soft – firm	3.3	SD6
Slash Pine	Straw	530	Soft – firm	3.4	SD5
Western Red Cedar	White, yellow, pale straw to light brown	350	Very soft	1.5	SD8

Source: AS 1684.2-2010_Table 9.15.

Resistance to impacts from chemicals

While many common chemicals can cause building materials to deteriorate, timber is resistant to all but the strongest alkalis and acids (pH>10 and pH<2) and can be used with confidence for structural members in corrosive environments, such as industrial buildings and near salt water spray. In general, heartwood is more resistant to chemical attack than sapwood, due to its limited permeability. Softwoods resist chemical attack more successfully than hardwoods due to their different chemical composition.

Strong acids and alkalis can break down and destroy timber elements but this takes time. In strongly acid and alkali environments, the rate of degradation is dependent on factors such as species, chemicals and exposure. The adhesives used to bond EWPs such as glulam and plywood are typically robust enough to resist chemical degradation in the majority of common applications. However, specialist advice is required if the timber is to be used in highly corrosive ambient conditions.

Fatigue effects

Timber resists cyclic loading for high winds and other sources well. Sustained loading produces a time-dependent deformation and increase in timber's deflection known as *creep* and allowance for this effect is made in the relevant standards.

8.2 Structural performance

The primary factors that influence the strength and stiffness of a piece of timber include: its cell structure, broadly a function of its species; its physical characteristics, such as knots and sloping grain in the piece; and its MC. Moisture content is covered in more detail in Section 8.3.

Timber elements are held to provide the structural performance required by engineering design standards if they: comply with the requirement of the relevant structural grading standards; or have independent engineering certification of their characteristic structural properties. Most sawn timber products satisfy the requirement of the structural grading standards. Independent engineering certification is often used for LVL and similar EWPs.

8.2.1 Species strength groups

Wood from an individual species has similar structural properties. These properties have been determined and grouped with other species with similar structural performance into strength groups defined in *AS 2878 Timber – Classification into strength groups*. There are seven strength groups for unseasoned timber: S1 to S7, with S1 being the strongest timber. There are also eight strength groups for seasoned timber: SD1 to SD8, with SD1 being the strongest timber. Table 48 lists the strength groups of readily available species.

8.2.2 Structural grading and grades

Structural grading is based on correlations between the piece's actual structural properties and the measurement of one or more accepted grading parameters. During grading, the parameters are assessed either visually or by machine and a stress grade is assigned to the board based on that assessment.

A stress grade is the name given to a collection of structural properties defined in *AS 1720 Timber Structures* or other standards. When the timber is assigned a stress grade, it is held to have these structural properties. Major stress grade systems include: F-grades, MGP grades and GL grades. With each of these, the higher the number assigned to the timber in the grade, the higher its assigned structural properties. The grade term describes the minimum performance assigned to the piece.

Grade terms are species and product independent. A piece of F 17 softwood LVL has the same assigned structural properties as a piece of F 17 sawn hardwood. Grade schemes are not interchangeable. The grade and performance definitions for F and MGP systems are not the same.

F and A-grades

F-grades apply to both seasoned and unseasoned hardwood and softwood. Common stress grade terms include F 5, F 17, F 27, and F 34. A-grades such as A 17 also exist for timber that satisfies a lower density limit than F-grade requirements. The key standards covering F and A graded material are: *AS 2082 Timber – Hardwood – Visually stress-graded for structural purposes*; *AS 2858 Timber – Softwood – Visually graded for structural purposes*; and *AS 2878 Timber – Classification into strength groups.*

Trained graders visually grade timber to the F-grade system. The species of timber is determined and a strength group assigned. The grader then assesses the characteristics in the piece, looking for potentially strength-reducing characteristics such as knots and sloping grain. Taking these assessments into consideration, the grader assigns a *strength grade* to the board. The species' strength group and the board's strength grade combine to determine a stress grade for the piece. The relationship of strength group, visual grade and stress grade are shown in Table 49.

Most Australian native and imported structural hardwoods, unseasoned and pressure-treated softwood products, and seasoned softwoods thicker than 45 mm are visually graded to the F-grade system.

Table 49: The relationship of strength group, visual grade and stress grade

Strength Group	Stress Grade				
	Structural No. 1	Structural No. 2	Structural No. 3	Structural No. 4	Structural No. 5
Unseasoned Timber					
S1	F22	F22	F17	F14	F11
S2	F27	F17	F14	F11	F8
S3	F17	F14	F11	F8	F7
S4	F14	F11	F8	F7	F5
S5	F11	F8	F7	F5	F4
S6	F8	F7	F5	F4	
S7	F7	F5	F4		
Seasoned Timber					
SD1	F34	F34	F27	F22	F17
SD2	F34	F27	F22	F17	F14
SD3	F27	F22	F17	F14	F11
SD4	F22	F17	F14	F11	F8
SD5	F17	F14	F11	F8	F7
SD6	F14	F11	F8	F7	F5
SD7	F11	F8	F7	F5	F4
SD8	F8	F7	F5	F4	

Note: Structural grade No. 5 applies to softwood only, as specified in AS 2825

MGP Grades

Machine grade pine or MGP grades are the predominant grading system used for seasoned structural pine products. Common MGP grade terms include MGP 10, MGP 12 and MGP 15. The key standard covering MGP-graded material is *AS/NZS 1748 Timber – Solid - Stress-graded for structural purposes.* During grading, the pieces go through a dynamic bending machine at high speed and are subject to a non-destructive bending load generally on their flat face along most of their length. A stress grade is assigned based on its assessed deflection.

GL Grades

Glue laminated or GL grades apply to glue laminated timber sections. Common grade terms include GL 8, GL 12 and GL 18. The key standards covering GL-graded material are: *AS 1328 Glued laminated structural timber* and *AS 5086 Timber – Finger joints in structural products.*

Plywood grades

Structural plywood products are manufactured to produce F-graded material from F 8 to F 34. The key standards covering plywood products include: *AS/NZS 2269.0 Plywood – Structural*; *AS/NZS 4357.0 Structural Laminated Veneer Lumber – Specifications*; *AS/NZS 2272 Plywood – Marine*; and *AS/NZS 2271 Plywood and Blockboard for Exterior use*.

Plywood is produced with face veneers suitable for the intended application to grades shown in Table 39. For example, AD grade plywood has one face grade A and one grade D. Structural plywood normally has C and D grade faces.

8.2.3 Characteristic properties of standard-defined stress grades

Table 50: Characteristic values of F-graded timber for design

Characteristic values, MPa	Stress Grade									
	F34	F27	F22	F17	F14	F11	F8	F7	F5	F4
Bending (f'_b)	84	67	55	42	36	31	22	18	14	12
Tension parallel to grain (f'_t) - Hardwood	51	42	34	25	22	18	13	11	9	7
Tension parallel to grain (f'_t) – Softwood	42	34	29	22	19	15	12	8.9	7.3	5.8
Shear in beam (f'_s)	6.1	5.1	4.2	3.6	3.3	2.8	2.2	1.9	1.6	1.3
Compression parallel to grain (f'_c)	63	51	42	34	27	22	18	13	11	8.6
Short duration average modulus of elasticity parallel to the grain (E)	21,500	18,500	16,000	14,000	12,000	10,500	9,100	7,900	6,900	6,100
Short duration average modulus of rigidity (G)	1,430	1,230	1,070	930	800	700	610	530	460	410

Source: AS 1720.1 Table H2.1

Table 51: Characteristic properties of common MGP-graded timber

Characteristic values, MPa	Stress Grade											
	MGP10				MGP12				MGP15			
Section Size	70-140	190	240	290	70-140	190	240	290	70-140	190	240	290
Bending (f'_b)	17	16	15	14	28	25	24	22	39	36	33	31
Tension parallel to grain (f'_t) - Softwood	7.7	7.1	6.6	6.1	12	12	11	9.9	18	17	16	14
Shear in beam (f'_s)	2.6	2.5	2.4	2.3	3.5	3.3	3.2	3.1	4.3	4.1	4.0	3.8
Compression parallel to grain (f'_c)	18	18	17	16	24	23	22	22	30	29	28	27
Short duration average modulus of elasticity parallel to the grain (E)	10,000				12,700				15,200			
Short duration average modulus of rigidity (G)	670				850				1010			

Source: AS 1720.1 Table H3.1 MGP-Grades

Table 52: Characteristic properties of GL-graded timber

Characteristic values, MPa	Stress Grade					
	GL18	GL17	GL13	GL12	GL10	GL8
Bending (f'_b)	50	42	33	25	22	19
Tension parallel to grain (f'_t)	25	21	16	12	11	10
Shear in beam (f'_s)	5.0	3.7	3.7	3.7	3.7	3.7
Compression parallel to grain (f'_c)	50	35	33	25	22	16
Short duration average modulus of elasticity parallel to the grain (E)	18,500	16,700	13,300	11,500	10,000	8,000
Short duration average modulus of rigidity for beams (G)	1,230	1,110	900	770	670	530

Source: GLTAA Technical Data Sheet No. 4.

Table 53: Characteristic properties of structural plywood

Characteristic values, MPa	Stress Grade							
	F34	F27	F22	F17	F14	F11	F8	F7
Bending (f'_b)	90	70	60	45	36	31	25	20
Tension (f'_t)	54	45	36	27	22	18	15	12
Panel shear (f'_s)	6.0	6.0	5.5	5.1	4.8	4.5	4.2	3.9
Compression in the plane of the sheet (f'_c)	68	55	45	36	27	22	20	15
Bearing normal to the plane of the sheet (f'_p)	31	27	23	20	15	12	9.7	7.7
Short duration average modulus of elasticity (E)	21,500	18,500	16,000	14,000	12,000	10,500	9,100	7,900
Short duration average modulus of rigidity (G)	1 075	925	800	700	625	525	455	395

Source: AS 1720.1 Table 5.1

8.2.4 Certified characteristic properties

Independent engineering certification of a product's characteristic properties is often used for LVL and similar EWPs, as the structure of established stress grades may not provide a useful representation of their performance. Indicative characteristics are listed in the tables below but need to be confirmed with likely suppliers during final design.

Table 54: Characteristic properties of common LVL products

Characteristic values, MPa	Hyspan	Hyspan F17
Bending (f'_b)	$50 \times (95/d)^{0.154}$	$50 \times (95/d)^{0.154}$
Tension parallel to grain (f'_t)	25	25
Shear in beam (f'_s)	4.6	4.6
Compression parallel to grain (f'_c)	41	12
Short duration average modulus of elasticity parallel to the grain (E)	13,200	14,000
Short duration average modulus of rigidity for beams (G)	660	700

Source: CHH Woodproducts. See additional information on the producer's site.

Table 55: Characteristic properties of OSB products

Characteristic values	Sheet thickness (mm)		
	6-10	>10 to <18	18-25
Density (kg/m^3 ± 15%)	600	590	570
Strenght – longitudinal direction (MPa)	≥22	≥20	≥18
Strenght – transverse direction (MPa)	≥11	≥10	≥9
MOE – longitudinal direction (MPa)	≥3,500	≥3,500	≥3,500
MOE – transverse direction (MPa)	≥1,400	≥1,400	≥1,400

Source: Kronospan. For additional information contact material suppliers.

Table 56: Indicative properties of structural high-density fibreboard

Property	Standards	Results
Density	AS/NZS 4266.2	1,000 kg/m^3
Bending Strength	AS/NZS 4266.4	32 MPa
Modulus of Elasticity	AS/NZS 4266.5	4,500 MPa
Equilibrium Moisture Content	AS/NZS 4266.5	7.5%
Moisture Resistance	AS/NZS 4457.5 24 hour Submission	<2% Swell <6% Absorption

Source: Weathertex

8.3 Moisture content

The MC of timber products when supplied will vary with the products. Logs and poles are usually supplied unseasoned. Sawn timber may be available either unseasoned or seasoned while engineered wood and fibre products are generally only available seasoned. Unseasoned or green timber is any piece with a MC greater than 25%. Unseasoned timber will dry to its EMC, shrink and possibly distort in service. Seasoned timber or wood products have a MC below 15%. More stable than unseasoned material, seasoned timber is lighter, stronger, more durable, holds fastening better and remains at a more constant size.

Standards-based MC requirements

As MC influences structural performance and dimensional stability, Australian standards define the required MC for compliant material. Requirements for sawn timber varies with product type. Generally, the more valuable the material and demanding the application for sawn board, the lower and tighter the MC requirements are. For structural hardwood products, 90% of the pieces being graded to *AS 2082* must have a MC not more than 15% with no piece having a MC of more than 18%. Seasoned softwood graded to *AS 2858* is supplied at a MC not exceeding 15%. See Figure 28.

The required MC for appearance hardwoods graded to *AS 2796* varies with each of three product types, with the lowest MC required for parquetry and furniture products. The required MCs for appearance softwood graded to *AS 4785* also vary with product types, with lower and tighter limits set for material used internally. Higher broader limits apply for material used externally.

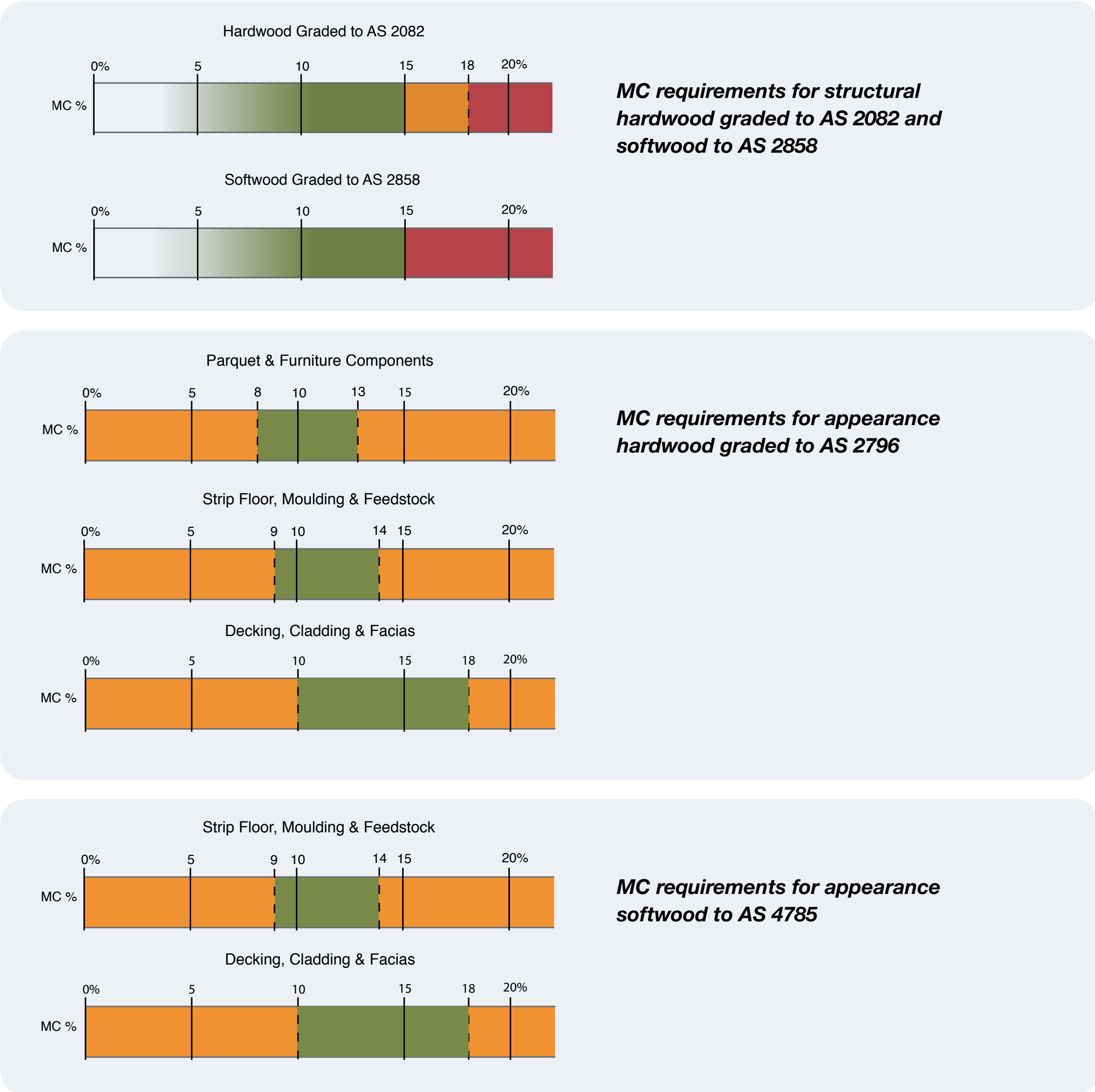


Figure 28: MC requirements in key Australian Standards

The allowable MC ranges for plywood specified in *AS/NZS 2269* for structural plywood are 10–15% MC for sheets up to 7.5 mm thick, and 8–15% for sheets more than 7.5 mm thick. Standard MC ranges for LVL are 8–15%. Particleboard is available at an MC of approximately 10% with a usual range of ±2%.

Shrinkage and expansion rates

Shrinkage and expansion rates for sawn board vary by species and grain direction: tangentially, radially and longitudinally. The radial and tangential shrinkage rates for a 1% change in MC (or unit shrinkage rate) for readily available species are listed in Table 57. Unit longitudinal shrinkage is usually low and not significant with seasoned timber. However, total longitudinal shrinkage can be significant for unseasoned timber dried to 12% MC in service. Unit shrinkage and expansion rates for other species are available on the WoodSolutions internet site.

Table 57: Unit shrinkage and expansion of readily available species

Species	Radial%	Tang.%
Hardwoods		
Blackbutt	0.26	0.37
Jarrah	0.24	0.30
Messmate	0.23	0.36
Mountain Ash	0.23	0.36
Southern Blue Gum	0.49	0.61
Spotted Gum	0.32	0.38
Softwoods		
Cypress Pine	0.26	0.22
Douglas Fir	0.23	0.23
Hoop Pine	0.18	0.23
Radiata Pine	0.19	0.27
Slash Pine	0.2	0.29
Western Red Cedar	-	-

Source: www.daf.qld.gov.au/forestry/using-wood-and-its-benefits/wood-properties-of-timber-trees

Shrinkage and expansion rates in plywood are low as cross lamination restricts veneer movement across the grain relative to movement along the grain. The percentage movement of structural plywood for a 1% change in MC when averaged between 5% MC and FSP varies with sheet thickness and direction between 0.011 and 0.014. For example, the expansion of a 1200 mm wide sheet installed at 10% MC that becomes fully saturated (nominally 28%) is about 3.0 mm.

Particleboard flooring shrinks and expands in width, length and thickness. Linear dimensions will change about 0.03-0.06% for each 1% MC change while thickness change will be about 0.3-0.5% for each 1% MC change.

8.4 Fire performance

Timber's performance in a fire can be defined through its charring rate, its fire hazard properties for use in internal spaces, and its resistance to flame and radiant heat during a bushfire.

8.4.1 Charring rate

A timber element loses its effective structural section gradually during a fire at a rate directly related to the species' density. As the timber burns, charred wood is left behind and this acts as an insulator protecting the section's inner core. This unaffected wood retains its structural capacity. *AS 1720.4 Timber Structures: Fire-resistance of structural timber members* provides a method of calculating the notional depth of charring and from this the residual section of unaffected timber after exposure to fire for a given time. This formula is:

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

Where: C is notional charring rate in mm/minutes (min), and D = timber density at 12% MC in kg/m^3. The notional charring rates for readily available species are listed in Table 58.

In addition to charring, timber and wood products have fire hazard properties relevant to their use in the internal spaces of buildings of particular types. These are discussed further in Section 9.4. Fire hazard properties for readily available species are listed in Table 58.

Table 58: Notional charring rate to AS 1720.4 and fire indices for readily available species

Species	Density kg/m^3 at 12% MC	Notional char rate, mm/min.	Material group number	Average specific extinction area	Critical radiant flux	Smoke-developed index	Smoke development rate	Spread-of-flame index
Hardwoods								
Blackbutt	900	0.496	3	<250	>2.2 and <4.5	3	<750	7
Jarrah	820	0.516	3	<250	>4.5	3	<750	6
Messmate	780	0.528	3	<250	>2.2 and <4.5	3	<750	5
Mountain Ash	680	0.569	3	<250	>2.2 and <4.5	3	<750	8
Southern Blue Gum	900	0.496	3	<250	<4.5		<750	
Spotted Gum	950	0.486	3	<250	<4.5	3	<750	3
Softwoods								
Cypress Pine	680	0.569	3	<250	<4.5	3	<750	8
Douglas Fir	550	0.65	3	<250	-	3		9
Hoop Pine	550	0.65	3	<250	>2.2 and <4.5*	2	<750	7
Radiata Pine	500	0.713	3	<250	>2.2 and <4.5	3	<750	8
Slash Pine	530	0.68	3	<250	>2.2 and <4.5	3	<750	8
Western Red Cedar	350	1.04	3	<250	-	4		10

Source: www.woodsolutions.com.au
*Notes: Critical radiant flux result is for 19 mm thick board or greater. * For 15 mm plywood*

8.4.3 Resistance to bushfire

AS 3959 groups timber species for their resistance to flame and radiant heat during a bushfire. After tests, seven species have been identified as naturally bushfire-resisting species while the remaining species are grouped by density. Table 59 lists the grouping in the standard and some of the key species included in each.

Table 59: Fire resistance grouping of species to AS 3959

Fire resistance grouping	AS 3959 listing	Species included
Naturally bushfire-resisting species	Appendix H	Blackbutt, Kwila (Merbau), Red Ironbark, River Red Gum, Silvertop Ash, Spotted Gum and Turpentine
Timber with a density of >750 kg/m^3 at 12% MC	Appendix E, Table E1	Bushfire-resisting timbers listed in Appendix H, and additional species such as Grey Box, Grey Gum, Grey Ironbark, Jarrah, Manna Gum, Messmate, Mountain Grey Gum, Stringybark, Sydney Blue Gum and Tasmanian Blue Gum
Timber with a density of >650 kg/m^3 at 12% MC	Appendix E, Table E2	Bushfire-resisting timbers listed in Appendix H, species listed in Appendix E1, and additional species including Alpine Ash, Slash Pine, Mountain Ash, Shining Gum and Cypress
Other species		Low-density species not listed in Appendix E or H, such as Radiata Pine, Western Red Cedar.

Source: AS 3959 Construction of buildings in bushfire-prone areas

8.5 Durability

Timber can resist hazards by its *natural durability* – its resistance to fungal and insect attack, or by the *use of applied treatments*. Timber heartwood has a natural resistance to breakdown and its natural durability is classified by species. Timber's *treated durability* varies with the type and retention of preservative chemicals in the wood and its treatment class determines its resistance to breakdown. Expected performance of species heartwood in various hazard classes is listed in *AS 5604 Timber - Natural durability ratings*.

8.5.1 Wood's natural resistance to breakdown

The natural resistance of the heartwood of many species to fungal and insect attack has been assessed and categorised into durability classes for both in-ground contact and exposed above-ground use. These are listed in *AS 5604 Timber - Natural durability ratings*. Durability classes are rated on a 1-4 scale with Class 1 being highly durable and Class 4 being low durability. Lacking protective extractives, the sapwood of all species is Durability Class 4. The durability classes and their relationship to service life expectancy of heartwood in characteristic levels of hazard are shown in Table 60.

Table 60: Durability Class and probable life expectancy of heartwood in years

Natural durability class for heartwood	Probable protected above-ground (H1) life expectancy	Probable exposed above-ground (H3) life expectancy	Probable in-ground (H5) life expectancy	Probable marine-borer-resistance life expectancy in southern waters
Class 1 Highly Durable	Greater than 50	Greater than 40	Greater than 25	Greater than 60
Class 2 Durable	Greater than 50	15 to 40	15 to 25	41 to 60
Class 3 Moderately Durable	Greater than 50	7 to 15	5 to 15	21 to 40
Class 4 Non-durable	Greater than 50	0 to 7	0 to 5	0 to 20, usually less than 5

Source: General guide to probable life expectancy according to AS 5604

Note: Marine borer resistance is based on natural round piles containing 350 mm diameter of heartwood in southern seas reaching from Perth in the west to Batemans Bay in the east. Only class 1 timbers can be expected to give reasonable service life (12 to 30 years) in northern waters.

AS 5604 rates heartwood resistance to termite attack and sapwood susceptibility to lyctid attack. Heartwood resistance to termite attack is rated as R for heartwood resistant or NR for heartwood not resistant. All sapwood is not resistant to termites. Lyctid beetles attack only susceptible sapwood in hardwoods. The heartwood for hardwoods and all softwoods are not susceptible. Hardwood sapwood susceptibility to lyctid attack is rated S for sapwood susceptible, or NS for sapwood not susceptible. Table 61 lists the durability rating, termite resistance and lyctid susceptibility of the sapwood of readily available species.

Table 61: Durability properties of the heartwood of readily available species

Species name	Durability class exposed above-ground (H3)	Durability class in-ground contact (H5)	Termite resistance	Sapwood lyctid susceptibility
Hardwoods				
Blackbutt	1	2	R	NS
Jarrah	2	2	R	S
Messmate	3	3	NR	S
Mountain Ash	3	4	NR	NS
Southern Blue Gum	2	3	NR	S
Spotted Gum	1	2	R	S
Softwoods				
Cypress Pine	1	2	R	NS
Douglas Fir	4	4	NR	NS
Hoop Pine	4	4	NR	NS
Radiata Pine	4	4	NR	NS
Western Red Cedar	2	3	R	NS

Legend: R = heartwood resistant, NR = heartwood not resistant, S = sapwood susceptible, NS = sapwood not susceptible
Source: AS 5604

8.5.2 Wood's treated resistance to breakdown

Preservative chemicals such as insecticides and fungicides introduced into or onto the wood or added to the glue lines in EWPs improve its ability to resist fungi, insects and other biological agents by making the wood unpalatable as food. This extends the range of wood products suitable for many high-durability applications. The type and quantity of preservatives introduced determine the level of protection.

AS 1604 Specification for preservative treatment defines the suitability of each treatment class for the level of hazard expected in particular applications. It sets out the compliance requirements for the type and amount of chemical that must be retained in or on the wood and the depth of chemicals' penetration. Treatment classes, their suitability for applications, and the biological hazards they are designed to resist are listed in Table 62.

Table 62: Treatment classes and suitability for applications

Treatment class	Suitability	Biological hazard
H1	Indoors applications, protected and ventilated	Borers only
H2	All internal applications	Borers and termites
H3	Applications up to above ground outside	Decay, borers and termites
H4	Applications up to in contact with dry ground	Severe decay, borers and termites
H5	Applications up to all in-ground uses	Very severe decay, borers and termites
H6	Applications up to marine uses	Marine wood borers and decay

Source: www.daf.qld.gov.au/forestry/using-wood-and-its-benefits/wood-properties-of-timber-trees

Wood's permeability limits the penetration and retention of chemical treatments in the wood. Heartwood, especially in hardwoods, is very hard to treat reliably. The sapwood of most species can be treated because the cells' structure is relatively open. Figure 29 illustrates likely preservative penetration in hardwood and softwood poles and sawn elements.

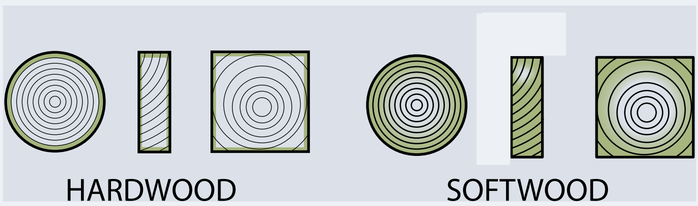


Figure 29: Likely preservative penetration after treatment for hardwood and softwood

8.6 Acoustic performance

Timber's relative softness and cellular structure mean that wood has a significantly different acoustic performance than harder materials such as steel, concrete and masonry. The cells in timber vibrate with sound but absorb some of the sound energy as they do, converting it into heat. This dampens the sound and reduces the magnitude of resonant vibrations. Since the wood's cell structure varies with species, some timber species absorb sound energy or vibrate with it in different or highly useful ways.

8.7 Thermal performance

Thermal conductivity (or *u-value*) is the rate that heat will conduct through one meter of a material. A material's insulation value (or R-value) is the inverse of its u-value, adjusted for the material's thickness. R-values are regularly used in specifying insulation products, such as an R2.5 or R4 product. Timber's organic and cellular structure means that hardwood and softwood have useful R-values, especially when compared to other major building materials.

Specific heat capacity is the amount of heat energy (in joules) needed to raise the temperature of a kilogram of the material by one degree Kelvin. In thermal performance design, the specific heat capacity is directly related to the building's thermal mass, which is the amount of heat that it can store. Hardwoods and softwoods have useful thermal capacity.

Table 63: Indicative insulation performance of readily available building materials

Material	U-Value (W/mK)	R-Value (100 mm)	Specific heat capacity (kj/kgK)
Rock wool insulation	0.045	2.22	0.75
Softwood	0.12	0.83	2.5
Hardwood	0.16	0.63	2.5
Concrete	1.45	0.05	0.85

8.8 Environmental performance

Timber's environmental performance can be defined in terms of its carbon storage and the forestry practices used to acquire the resource.

8.8.1 Carbon storage

Approximately 50% of the wood's dry weight in a tree is atmospheric carbon and this is sequestered in wood products for at least the product's service life. To determine the amount of carbon effectively stored in wood products in building, the carbon emitted during forestry, processing and transport has to be set against the carbon sequestered in the wood. Table 64 lists characteristic values for the effective carbon dioxide storage in seasoned Australia-produced softwood and hardwood.

Table 64: Carbon sequestration and storage for 1 m^3 dressed, kiln dried Australian timber

Species	Nom. Density (kg/m^3)	Carbon sequestration (kg $C0_2$ e)	Production emissions (kg $C0_2$ e)	Net carbon stored (kg $C0_2$ e)
Softwood	560	900	330	570
Hardwood	800	1289	678	611

Source: Environmental Product Declaration: Softwood Timber and
Environmental Product Declaration: Hardwood Timber

8.8.2 Certification of forest practices and chain-of-custody

The quality of forestry practices effects the overall environmental credentials of timber and wood products. As the assessment of forestry practice that is environmentally responsible for a particular location is a highly specialised field, international processes for *forest certification* exist that enable building design professionals to know that the forests from which the resource for timber products are recovered are legally harvested and sustainably managed to an agreed and acceptable standard. *Chain-of-custody (CoC) certification* assures customers that the supplied timber comes from these certified forests.

Two major international forest certification schemes operate in Australia: the Forest Stewardship Council (FSC) and Program for Endorsement of Forest Certification (PEFC) schemes. FSC operates internationally and establishes national organisations that observe FSC processes and principles. FSC Australia certifies to an interim forest management standard and to an FSC-published CoC standard. FSC-certified companies in Australia and internationally use and display the FSC logo.

PEFC endorses national schemes that satisfy its requirements and provides a mechanism for mutual recognitions between them. PEFC has endorsed the Australian Forest Certification Scheme (AFCS) which uses *AS 4708 Australian Forestry Standard* as its forest management standard and *AS 4707 Chain of custody for certified wood and forest products*, as its CoC standard. Timber and wood products certified under these standards can display the PEFC or AFS logos. International products from certified companies can enter Australia and display the PEFC logo or their national scheme's logo.

9 Performance requirements

Engineers and other building design professionals must ensure that the timber-rich structures and building components that they design satisfy the performance requirements required by legislation, regulation, client contract and other mechanisms.

This section deals primarily with regulatory requirements established through the National Construction Code (NCC) and its referenced standards, and the fit-for-purpose requirements established under the Australian Consumer Law that relate to timber and wood products used in building. Coverage of these and other requirements is not comprehensive and reference should be made to the NCC, the relevant WoodSolutions technical guides, professional advice and other sources.

9.1 Introduction

Regulatory requirements

As buildings are critical to human health, community wellbeing and safety, most aspects of building design and construction are regulated to ensure acceptable minimum standards of performance. Prepared by the Australian Building Codes Board (ABCB), the NCC is a national code, with some state variations, that applies to the design and construction of all buildings. The NCC incorporates the Building Code of Australia. NCC Volume 1 covers Class 2 to Class 9 Buildings (commercial and multi-residential buildings) while Volume 2 covers Class 1 and Class 10 Buildings (residential buildings). The NCC's goal is to achieve nationally consistent, minimum necessary standards for health, safety (including structural safety and safety from fire), amenity, and sustainability.

The NCC is a performance-based code, setting out the performance that a building or building element needs to achieve, not necessarily how it is to be achieved. Compliance with the NCC's *Performance Requirements* can be achieved three ways:

- Satisfying the *Deemed-to-Satisfy (DTS)* provisions.
- Developing a *Performance solution*. This must comply with the NCC's Performance Requirements or be shown to be at least equivalent to the Deemed-to-Satisfy Provisions.
- Combining both these methods, following the DTS provisions in some areas and developing a suitable performance solution for others.

Table 65 lists the sections of NCC Volume 1 that contain key performance requirements for timber construction and the section of this guide in which each is covered. Additional requirements may exist in other sections.

Table 65: NCC Section, key performance requirements and relevant guide section

NCC Section	Key performance requirements for timber structures	Guide section
A	This classifies buildings by their function. These building classes are then used to define particular requirements.	Table 66
B	This includes the structural requirements for buildings and structure and calls up standards used to determine structural integrity and acceptable design processes.	9.2
C	This deals with fire resistance and aims to protect people from fire and as they evacuate, avoid the spread of fire between buildings, and to protect nearby property from structural failure. It defines the required levels of fire resistance for elements.	9.4
F	This deals with building amenity over time. It sets out requirements for water exclusion, control of damp, and related aspects and establishes requirements for sound transmission and insulation between apartments and other areas.	9.3, 9.5 and 9.6
G5	This sets out requirements for the design of residential buildings in bushfire-prone areas.	9.4
J	This sets out requirements for thermal performance. It deals with the thermal performance of a building's services, fabric, external glazing, sealing, and related aspects.	9.7

Generally, the requirements in each NCC section vary with the building's function and factors such as its height. The NCC's classification of buildings by function is summarised in Table 66.

Table 66: NCC Building classes

Building Class	Building function summary
1	Single dwelling units, either detached or attached, and other accommodation that is not one above another.
2	A residential building with two or more sole-occupancy units (SOU). Units are often one above another.
3	Hotels, motels, boarding houses, and similar buildings
4	A single dwelling unit in a Class 5, 6, 7, 8 or 9 building
5	Office building
6	Shops, including display rooms, restaurants and showrooms
7	Car parks and store buildings
8	Factory or laboratory
9	Public buildings: Class 9a covers health-care buildings; Class 9b is for assembly buildings, such as theatres or educational buildings; and Class 9c is for aged-care buildings.
10	An ancillary building, such as a shed, carport, or other outbuildings

Source: NCC Volume 1

Fit-for-purpose requirements

The NCC only establishes minimum necessary standards for health, safety, amenity and sustainability. There are aspects of component design where customers or clients can expect or require performance above these minimums and in areas not regarded as necessary under the NCC. Under Australian consumer law, buildings must provide performance that is fit for a client or customer's intended purpose or application. A product like a building is fit-for-purpose if it does the job that the consumer wants or was told that it would do for an expected or reasonable period of time.

9.2 Structural performance

The NCC defines the minimum structural performance of elements that resist imposed loads in a building or structure while fit-for-purpose or client requirements may establish more rigorous requirements. *NCC Section B* includes the structural requirements for buildings and structures and requires that a building or structure, during construction and use, with appropriate degrees of reliability, must, by resisting the actions to which it may reasonably expect to be subjected: perform adequately under all reasonably expected design actions; withstand extreme or frequently repeated design actions; 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 avoid causing damage to other properties.

The DTS provisions require that the resistance of a building or structure must be greater than the most critical action effect resulting from different combinations of actions, where the most critical action effect on a building or structure is determined in accordance with *Section B1.2* and the general design procedures contained in *AS/NZS 1170.0 Structural design actions*; and the resistance of a building or structure is determined in accordance with Section B1.4.

Section B1.2 defines means for determining the permanent, imposed and wind, snow and ice, earthquake and other actions while *Section B1.4* lists the means that must be used for determining the structural resistance of materials and forms of construction. For timber construction, its lists *AS 1720.1 Timber structures - Design methods for the design of timber structures*, Parts 2, 3 or 4 of *AS 1684 Residential timber-framed construction*, and *AS 1720.5 Timber structures - Nailplated timber roof trusses* for nailplated timber roof trusses.

9.3 Moisture content

The NCC and fit-for-purpose requirements establish the need to consider the initial and likely service MC of timber and wood products. Timber's strength is related to its MC. Also, health and amenity in building is related to the absence of decay in biodegradable materials such as wood. As a result, *NCC Section F* establishes requirements for water exclusion, control of damp, and aspects such as subfloor ventilation and bathroom design.

The DTS provisions in Section F1.7 establish requirements for waterproofing of wet areas in buildings and specify the locations where waterproof and water-resistant surfaces are required. This affects the use of timber flooring materials in these areas, and generally requires them to be made waterproof.

To allow subfloor spaces to dry out, limit the likelihood of decay in timber components and allow for termite inspection if required, the DTS provision in Section F1.12 establish minimum aggregate areas for subfloor ventilation openings in enclosed subfloors and minimum ground clearance height for three climate zones. These zones are based on likely relative humidity in each zone.

This section also requires that the subfloor space be clear of debris and vegetation and be graded to prevent ponding, and other action be taken to provide effective ventilation in the subfloor. It also requires that where the ground or subfloor space is likely to be excessively damp or subject to frequent flooding, the level of subfloor ventilation required be increased and subfloor timber be Durability Class 1 or 2 timbers or preservative treated to H3.

Section 5.4 of *AS 1684.2* requires that cut-in strip floors include a minimum 10 mm expansion gap between the board and surrounding obstructions. For floors over 6 m wide, measured at right angles to the boards, it required intermediate expansion gaps equivalent to 10 mm for each 6 m of floor.

Fit-for-purpose requirements for timber's MC relate to the timber's overall shrinkage or expansion in service, and the system's durability. These imply that the design recognises and accommodates the expected dimensional change in timber elements as they reach or maintain EMC with their service environment. Two conditions need to be considered:

- If the timber's MC at installation is appropriate for the expected service conditions, provisions should be made to accommodate expected movement as EMC fluctuates. The acceptability of large timber surfaces such as timber floors in gymnasia and similar spaces is particularly sensitive to MC variations as small changes in the timber's EMC can result in large total changes in the surface's size.
- If the material's MC at installation is likely to be considerably different to its service EMC, provisions should be made to accommodate the section's expected movement and potential distortion as it reaches its EMC. Unseasoned material will shrink and potentially distort as it seasons in a dry environment and the dimensions of seasoned material will change and thin sections can distort in exposed or damp conditions. Excessive shrinkage or expansion of elements can compromise structural performance and the effectiveness of connections, damage surrounding surfaces, and be unsightly.

The timber's MC in service affects its durability. The likelihood and rate of decay and termites attack both relate directly to the timber's EMC and the level of moisture in surrounding elements and surfaces. See Section 9.5 for more detail.

9.4 Fire resistance

The NCC establishes fire-related provisions that influence design with wood. The major provisions are included in *Section C: Fire resistance* and *Section G5: Construction in Bushfire Prone Areas.*

9.4.1 Section C requirements

NCC Section C deals with fire resistance of building elements and aims to protect people from fire and as they evacuate, avoid the spread of fire between buildings, and protect nearby property from structural failure. The section's DTS provisions for the fire resistance of elements defines:

- The type of construction required for buildings of different classes and heights.
- In Specification C1.1, the required levels of fire resistance for structural and other elements in each of those construction types.

A Fire Resistance Level (FRL) is the required resistance of an element to fire, expressed in minutes, for each of three categories: structural adequacy/integrity/insulation. For example, a wall system may have a FRL requirement of 60/60/60.

Specification C1:10 establishes requirements for the fire hazard properties of linings, materials and assemblies in buildings of different classes. Fire hazard properties indicate a material's likely behaviour in generating heat or smoke in a fire. Separate requirements exist for the *critical radiant flux* and *smoke development rate* of floor linings and coverings, for *smoke growth rate index, average specific extinction area* and *material group number* for wall and ceiling linings, and for other components.

9.4.2 Section G5 requirements

NCC Section G5 includes requirements to reduce the risk of occupant harm and building loss due to bushfires in designated bushfire-prone areas. The DTS provisions state that residential buildings in these areas must be designed to comply with AS 3959 *Construction of buildings in bushfire-prone area*. This standard requires that a design must enable a building to:

- withstand ember attacks prior to the fire front – with assistance from occupants
- provide a safe refuge while the fire front passes
- allow occupants extinguish any elements that are still burning after the fire front passes.

It then sets fire resistance requirements for particular external building elements such as building subfloors, external wall surfaces and included windows and doors. These restrictions increase with the site's expected bushfire attack level (BAL).

9.5 System durability

Durability is the capability of a building, structure or component to perform its function over a specified period of time under the influence of the hazards and agents anticipated in service. The period of time after installation during which the building or its parts meets or exceeds its performance requirements is its service life.

Design service lives are usually established by fit-for-purpose provisions and a client's contractual requirements. The design service life can be short (1-5 years), medium (about 50 years) and long (over 100 years). The ABCB recommends minimum design lives for building components based on the building's expected design life and the ease and economy of the component's maintenance. See Table 67.

Table 67: Design life (dl) of buildings and their components

Building Design Life Category	Building design life (years)	Design life for components readily accessible and economical to replace or repair (years)	Design life for components with moderate ease of access but difficult or costly to replace or repair (years)	Design life for components not accessible or not economical to replace or repair (years)
Short	1 < dl < 15	5 or dl (if dl<5)	dl	dl
Normal	50	5	15	50
Long	100 or more	10	25	100

Source: ABCB 2015: Handbook: Durability in Buildings Including Plumbing Installations, *Table 3-1*

9.5.1 Hazards and agents anticipated in service

The hazards and agents likely to affect the service life of timber components and their connections vary in relation to: the component's location in the building, particularly exposure to sources of moisture and insect attack; and the climatic and other conditions associated with the building's location.

A timber component's exposure to biological hazards in an application is defined in hazard classes. Hazard classes are rated on a 1-6 scale: Hazard Class H1 represents the lowest level of hazard and H6 represents the highest level. Table 68 lists the expected exposure conditions and biological exposure for each Hazard Class.

Table 68: Hazard Classes and expected exposure conditions

Hazard Class	Exposure	Service Conditions	Biological Hazard
H1	Inside above ground	Fully protected, well ventilated	Borers only
H2	Inside above ground	Protected from wetting, nil leaching	Borers and termites
H3	Outside above ground	Moderate wetting and leaching	Decay, borers and termites
H4	Outside in ground	Severe wetting and leaching	Severe decay, borers and termites
H5	Ground contact	Extreme wetting, leaching and/or critical use	Very severe decay, borers and termites
H6	Marine waters North and South	Prolonged immersion in sea water	Marine wood borers and decay

Climatic and other conditions associated with the building's location influence the potential for its timber components to decay or be attacked by termites and for any metal connectors to corrode. The potential for the decay of timber components installed in-ground contact (H4 and H5) and outside above ground (H3) varies with local climatic and other conditions throughout Australia. Generally, the hotter and more humid a location, the higher the potential decay hazard. Decay hazard in hot, dry areas is generally low. *WoodSolutions Design Guide No 5: Timber Service Life Design* identifies four decay hazard zones across Australia for in-ground decay hazard, shown in Figure 30, and for above-ground decay hazard, shown in Figure 31.

The potential for termites to attack and damage timber components also varies with local climatic and other conditions. Figure 32 shows termite hazard zones for Australia. The zones for each of these range from Zone A with the least potential for decay to Zone D with the greatest potential.

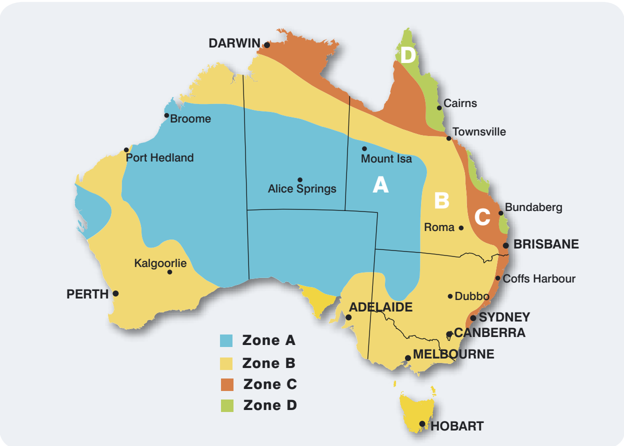


Figure 30: In-ground decay hazard zones for Australia

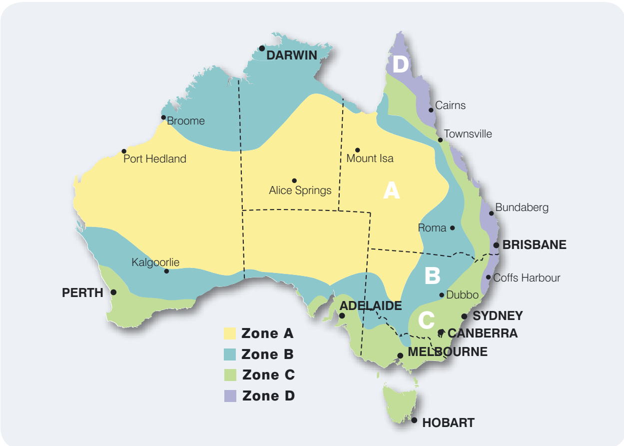


Figure 31: Above-ground decay hazard zones for Australia

The hazard of corrosion for embedded metal fixing is shown in Figure 33 with Zone A representing the least potential for corrosion and Zone C the greatest potential.

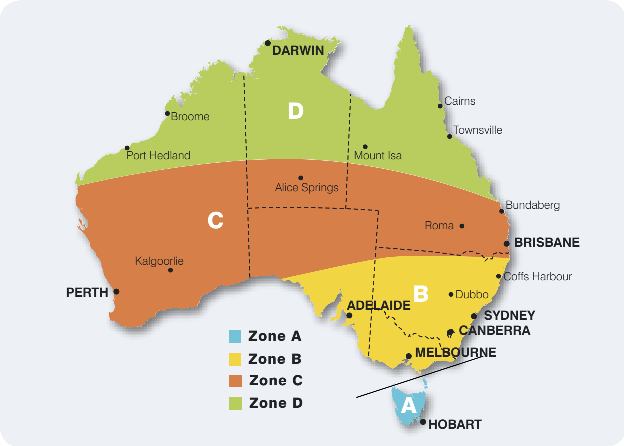


Figure 32: Termite hazard zones for Australia

Figure 33: Hazard zone for embedded corrosion

9.6 Acoustic separation

NCC Section F defines minimum requirements for sound transmission and insulation between parts of Class 2, 3 and 9c buildings. Fit-for-purpose, client or market-driven requirements may establish more rigorous performance requirements for these and other building classes. For NCC DTS compliance, forms of construction between spaces are required to have specific airborne sound and impact sound insulation ratings. Table 69 lists the sound insulation requirements for walls between Class 2 and 3 buildings while Table 70 lists similar requirements for floors.

Airborne sound insulation ratings are expressed as either a *weighted sound reduction index* (R_w) or *weighted sound reduction index with spectrum adaptation term* ($R_w + C_{tr}$) determined in accordance with the relevant standards. Impact sound insulation ratings are expressed as either the *weighted normalised impact sound pressure level* ($L_{n,w}$) determined in accordance with the relevant standard or compliance with NCC Specification F5.2.

Market acceptance and client requirements may demand a level of performance higher than the regulatory minimum, especially for floors between apartments and between office spaces.

Table 69: Deemed-to-satisfy sound insulation requirements for walls in Class 2 and 3 buildings

Situation			Wall Rating	Entry Door Rating
First Space	**Action**	**Adjoining space**		
All spaces except those noted below	Separates	SOU – generally all spaces except those noted below	≥ 50 $R_w + C_{tr}$	N/A
Bathroom, sanitary compartment, laundry or kitchen	Separates	SOU – habitable room[1] (except kitchen)	≥ 50 $R_w + C_{tr}$, and of discontinuous[2] construction	N/A
Bathroom, sanitary compartment, laundry or kitchen	Separates	SOU – non-habitable[3] room (including kitchen)	≥ 50 $R_w + C_{tr}$	N/A
Plant and lift shaft	Separates	SOU – all spaces	≥ 50 R_w and of discontinuous construction	N/A
Stairway, public corridor, public lobby or the like or part of a different NCC building classification	Separates	SOU – all spaces	≥ R_w 50	≥ R_w 30 (except a part of a different BCA Building classification)

Source: WoodSolutions Design Guide No 2, Table 1

Notes:
1. Habitable room means a room used for normal domestic activities including a bedroom, living, lounge or family room, music room, television room, kitchen, dining room, sewing room, study, playroom, 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.

Table 70: Deemed-to-satisfy sound insulation requirements for floors in Class 2 and 3 buildings

Situation			Wall Rating
First Space	**Action**	**Adjoining space**	
SOU – all spaces	Separates	SOU – all spaces	$R_w + C_{tr\ (airborne)} \geq 50$, and $L_{n,w} + C_{I\ (impact)} \leq 62$
Public corridor or lobby or the like	Separates	SOU – all spaces	$R_w + C_{tr\ (airborne)} \geq 50$, and $L_{n,w} + C_{I\ (impact)} \leq 62$
Stair and lift shaft	Separates	SOU – all spaces	$R_w + C_{tr\ (airborne)} \geq 50$, and $L_{n,w} + C_{I\ (impact)} \leq 62$
Plant rooms	Separates	SOU – all spaces	$R_w + C_{tr\ (airborne)} \geq 50$, and $L_{n,w} + C_{I\ (impact)} \leq 62$
Different NCC building classification	Separates	SOU – all spaces	$R_w + C_{tr\ (airborne)} \geq 50$, and $L_{n,w} + C_{I\ (impact)} \leq 62$

Source: WoodSolutions Design Guide No 2, Table 2

9.7 Thermal performance

To ensure efficient energy use, *NCC Section J* establishes thermal performance requirements for the fabric of all classes of buildings. This affects: the insulation of the building envelope and other components; the size, type, performance and placement of windows and any associated shading; and the control of ventilation and air infiltration through sealing the building. DTS requirements are available for Class 1, 2 and 4 buildings while the compliance of Class 3, 5, 6, 7, 8 or 9 buildings is verified by modelling the building's annual energy consumption against the performance of a reference building.

Clients may demand levels of thermal performance higher than Australia's regulatory minimum and benchmark this against other national or international performance standards.

9.8 Environmental performance

NCC Section J establishes requirements connected to environmental sustainability through regulating factors that influence a building's operational energy. However, it does not consider other aspects of environmental sustainability, such as the energy embodied in the material used to make the building. Given this, clients may demand levels of environmental performance in the buildings they procure or lease and benchmark these against rating schemes, such as the government-based National Australian Built Environment Rating System (NABERS), the Green Building Council of Australia's (GBCA) Green Star rating system, or international benchmarking programs.

NABERS is a national system that rates the environmental performance of Australian buildings based on their actual operational performance when measured over a year using energy or water bills, or waste audits. As such, its influence on the use of timber and wood products in the building's fabric is indirect.

Green Star rating tools use a point-based system to score buildings against a range of criteria designed to ensure they have been built with a low environmental impact, are or can be operated sustainably, and provide a healthy indoor space for their occupants. Green Star rating tools award points for the use of responsible building materials in design, including timber. Green Star's Mat-7 outlines the points available for the use of reused timber, legally sourced timber, and timber sourced from forests whose conservation values are not degraded. Effectively, one point is available where at least 95% (by cost) of timber used is certified to an acceptable forest certification scheme, from a reused source, or a combination of both. The GBCA recognises FSC and PEFC-endorsed schemes such as AFCS as compliant with their criteria.

10 Aspects of *AS 1720*

10.1 Limit states and loads to *AS 1720*

AS 1720 sets out the limit states design methods for the timber's use in structures. It provides design values for various timber products, including solid sawn and engineered products, as well as fasteners, and provides the methods for determining the capacity of members and connections. The design of a structural member involves:

- Examination of all possible loading combinations
- Selection of the critical loading combination
- Consideration of the environment of the structural elements over their lifetime
- Use of design techniques to select a member for the critical design load combination
- Detailing of connections or restraints assumed in the design
- Checking the member's performance for other important load combinations and relevant limit states.

While *AS 1720.1* can be used for the design of generic proprietary products such as I-beams, it is most often used for the design of sawn timber members, or glulam or LVL elements. Manufacturers may often supply design information for generic proprietary products.

10.1.1 Limit States

Most Australian structural design codes are now in limit states format, ensuring the designed product must be serviceable (serviceability limit state), stable (stability limit state), and safe (strength limit state). If the performance of any of these criteria is unsatisfactory, the 'limits' have been exceeded in one or more of the performance states. Other limit states include:

- the fire limit state, where loads must be carried by a building partially damaged by fire in order to enable safe evacuation and fire control operations
- the fatigue limit state that applies to a structure that is loaded and unloaded repeatedly throughout its service life.

Each performance state represents a different loading scenario and loads must be combined in an appropriate manner for each limit state. These are given in *AS 1170.1*.

Serviceability Limit State

Under the prescribed loads and load combinations, the structure must be serviceable, and satisfactorily achieve the tasks for which it was designed. Limits will exist on deflection, cracking in concrete and composite structures, and vibration. These limits may not be prescribed in codes, but left to the designer or the client to determine the conditions that constitute fit-for-purpose performance.

Stability Limit State

Under the prescribed loads and load combinations, the parts or whole of a structure must be stable and not overturn. In light timber structures, this limit state requires particular attention as timber's dead load is less than other conventional materials. Due to the catastrophic nature of instability, the structure's stability must be checked for all possible loading conditions, including very rare loading events, and for a reasonable service life. Applied load factors should give the target probability of a failure load being exceeded of less than 5% in a 50 year service life.

Strength Limit State

Under the prescribed loads and load combinations, the structure must not fail. Just as violations of the stability limit state could be catastrophic, violations of the strength limit state and element failure are serious and design must ensure a very low probability that a structure will fail in its lifetime. Failure can be due to overloading, deterioration, damage, or under performance. Decisions made in the conceptual design stage can reduce the probability of structural deterioration and damage. Structures that use repetitive systems and load sharing are less susceptible to the underperformance of a single element.

10.1.2 Loads and loading

Structural design aims to select a structural system and members that perform satisfactorily under probable loading conditions during the structure's life. The actual loads on a structure are associated with events over its life. In the conceptual design stage, it is important to understand the local environment such as topography, vegetation, and surrounding development effect the loads applied to a structure. For example, wind loads may be substantially larger in one direction due to these parameters.

When determining the appropriate loads, the load's origin, distribution, certainty, and duration need to be considered.

Load origin

The load's origin is where the load originates or what causes the loading. Knowing this provides information that assists in resisting the load. Loads from different origins can be broadly categorised as dead loads or live loads.

Dead load is the gravitational force due to the structural elements, cladding, and permanent fixtures. In *AS 1170.1*, dead loads are known as permanent actions (G). Some dead loads, such as interior partitions and fixed equipment, may be altered or removed throughout the structure's life, and this possibility must be considered in the design.

Live loads are transient loads that act on the building. Live loads include the gravitational force on the contents, wind, snow, earthquake, and possible water flow. In *AS 1170*, live loads from a number of origins are considered:

- **Imposed Actions (Q)** – Actions resulting from the structure's intended use or occupancy. Minimum imposed actions for various building occupancies are defined in *AS 1170.1* Table 3.1.
- **Liquid Pressure (F_{lp})** – Static liquid pressure acting on the structure
- **Ground Water (F_{gw})** – The hydrostatic pressure from water acting on surfaces below ground level
- **Rainwater Ponding (F_{pnd})** – The result of rainwater collecting on a structure. This must account for the possibility of malfunctioning drainage as well as sagging of the structure allowing for greater ponding. It is best to design so that ponding is not possible, but if water can accumulate it must be designed for.
- **Earth Pressure ($F_{e,u}$)** – Lateral actions imposed on earth-retaining structures. Proper drainage around foundations is important for reducing earth pressure.
- **Wind Actions (W)** – The result of wind acting on the structure. Determination of wind actions is governed by *AS 1170.2* and depends on many factors, including the regional climate and local terrain and site conditions. The ultimate limit state (Wu) considers a wind speed associated with rare events, for example a 0.1% annual probability of occurrence. The serviceability state (Ws) considers a more common, lower wind speed, for example a 5% annual probability.
- **Snow and Ice Actions (F_{sn} and F_{ice})** – Actions resulting from the build-up of snow or ice on a structure. These are governed by *AS 1170.3*. Like wind actions, these are highly variable depending on the region, local terrain and building geometry.
- **Earthquake Actions (E)** – Actions induced by the inertia of the structure and its contents in response to a seismic event. These are governed by *AS 1170.4*.

There are also loads caused by other scenarios such as differential settlement, temperature, or changes in MC.

Load distribution

Load distribution is the type of load effect caused. Concentrated loads are localised and are associated with loading events or structural systems that put loads in very specific areas. Distributed loads are associated with loading events in which the forces are applied over much larger areas. Wind loads tend to be spread over large surfaces. Earthquake loads are associated with the mass in the structure and tend to be concentrated at floor levels. Live loads can be spread out, especially where the loading comes from crowds, or very localised under machine support, vehicle wheels or jacking points.

Load certainty

Load certainty is how precisely the load level can be predicted and reflects the accuracy and confidence with which future loading events in the structure's life can be predicted. They can be categorised as either known or estimated loads.

Known loads are well defined or can be found from a supplier or manufacturer, and can be estimated with reasonable confidence. Known loads are not usually included in the loading code and are generally considered as live loads. They include machinery, filing cabinets, and shelving, or hoisting or jacking loads generated in construction. A known load may be determined by referring to manufacturer's data for equipment, or the depth and density of material stored.

Estimated loads are calculated on the basis of possible future events and are generally associated with environmental factors or building occupancy. They include wind loads, earthquake loads, and occupancy live loads.

Load duration

Load duration is the length of time the load will exist and is an important consideration with composite materials that may creep, like timber. For most load combinations, estimates are needed for the magnitude of longer duration and shorter duration loads. Shorter duration loads cause only elastic deformations with minimal influence of creep and generally cause little reduction in strength with time. Generally, shorter duration loads will exist in combination with the longer duration loads. *AS 1720.1* uses the term "short-term" loads to refer to those of five hours' duration.

Longer duration loads may cause larger deflections than short duration loads in timber structures as creep adds to the elastic deformation. The timber's strength characteristics can also change after being subjected to long duration loads. These include the weight of the structure itself and any permanent or semi-permanent items in the structure, such as machinery and partitions. In *AS 1720.1*, 'long-term' loads refer to those of five months' duration.

11 Worked examples

Building use, construction approach, loading requirements and element configuration are essential considerations in the design of both sample buildings; at the very least, the design of timber structure systems are contingent upon these parameters.

This section presents worked examples for the design of the timber structure systems for two different, multi-storey, mixed-use buildings; the first example is seven storeys high, and the second example is ten storeys high. Both examples include a brief discussion of the conceptual design of the structural system, a summary of the design criteria, and detailed calculations of the design of various individual timber elements in the building. The timber elements include glue laminated timber (glulam) and cross laminated timber (CLT). Although many of the buildings' elements are not included in workings, the examples demonstrate the steps that would need to be taken for their design. Loading scenarios are estimated and should be calculated on a case by case basis.

11.1 Worked Example 1: Seven Storey Mixed-Use Building

11.1.1 Conceptual Design

In this example, the initial concept is for a seven storey, mixed-use structure that includes retail, office, residential and restaurant accommodation. All of the main structural elements are timber or wood products. The building has a ground plan of 18m x 40m and a height of approximately 25m.

The building's ground level is for retail use, with an open floor plate and a 4.5m ceiling height. Half of the ground floor has a 9m x 4.5m grid of posts supporting the floor and ceiling above. These posts form part of a portal-frame to resist lateral forces. A 6m x 4.5m grid is used in the other half of the ground floor space. Open-web timber joists span in the 6m direction.

The first and second levels house offices and the third through fifth levels house residential apartments. These levels all have 2.5m floor-to-ceiling heights. Because the office spaces have open floor plates to provide flexibility for multiple uses, the column grid in the office levels matches the ground floor grid. This provides continuity of load through the line of the columns. The residential spaces are divided into a smaller grid to reduce member sizes. A 4.5m x 4.5m post grid is used for the fourth and fifth levels. As a result, the fourth floor beams with 9m spans must carry point loads at mid-span from the columns above.

The general design concept is shown in Figure 34.

The required fire separation between the office and residential spaces is an FRL of 240/240/240. The third floor beams are designed so the residual section provides the demand capacity after four hours of fire exposure.

To take advantage of the upper storey views, one half of the top floor features a large open space with a circular floor plan. A timber dome spanning 18m forms the roof to this space. The dome's vertical rise is 3.6m and it sits on wall 1.88m high. Based on this span length, a ribbed dome system has been selected. A gridded dome would typically allow smaller member sizes but on this relatively short span, the cost savings are offset by the additional connection costs.

The other end of the top level houses a conference centre and auditorium. This area is 18m square in plan and has a clear-span roof. A glulam Tudor arch spans this space and resists both gravity and wind loads.

In the area between these two spaces, there is an outdoor roof deck, designed as an occupied roof, with 5kPa live load.

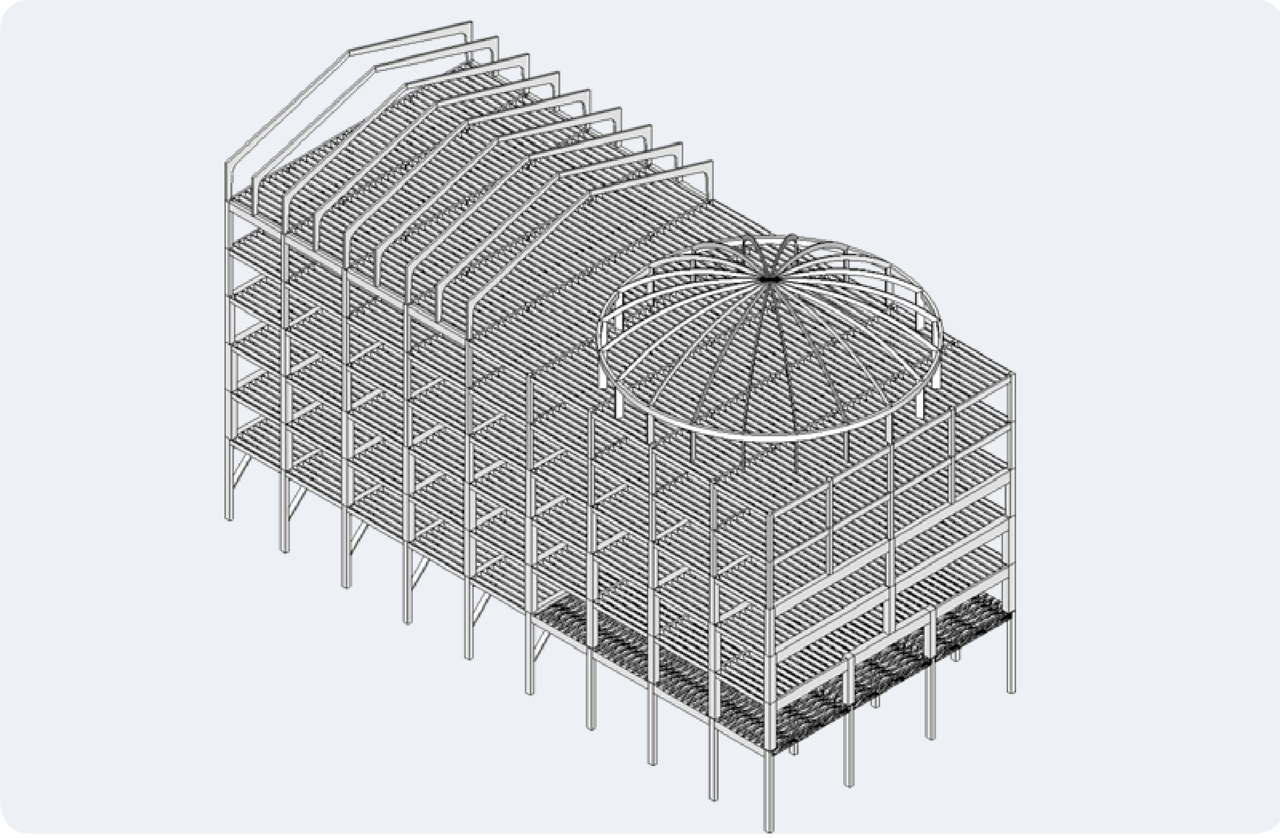


Figure 34: General conceptual design of Worked Example 1

11.1.2 Summary of Design Criteria

- Dimension limitations: Height: 25m, Width: 18m Depth: 40m
- Design the floors with as much open space as practical
- Top floor – 18m dome and 18m square clear span spaces with roof deck between
- Remaining floors supported on posts placed on a 4.5m x 4.5m typical grid, with variations in office and retail spaces
- Floor height requirements are:
  - Ground floor: 4.5m floor to ceiling
  - Level one – five: 2.5m floor to ceiling with 500mm floor thickness
  - Level six: Dome roof with Diameter (d) = 18m and rise (r) = d/5 = 3.6m
    - > Total height at this point: 4.5 + (5 x 3.0) + Dome (and walls) (3.6 +1.88) = 24.98m
  - Level six: Tudor Arch with a horizontal span of 18m and rise (r) = 5m
    - > Total building height at this point = 25m

11.1.3 Calculations

Determination of Wind Actions

For a multi-storey structure such as this, many of the controlling load cases are dependent on wind loading, so the structural design begins by determining the appropriate wind actions. This calculation is summarized below:

1. Use the *AS1170.0 Structural design actions – General principles*. Section 3 – Annual Probability of Exceedance Table 3.1 provides the importance levels and Table 3.3 the appropriate annual probability exceedance. For importance level 2 the annual probability is 1:500 for wind.

2. Calculate the wind speeds, pressures and actions per *AS1170.2:2011* Structural design actions - Part 2: Wind actions. Use Appendix F of *AS1170.2* if a flag is to be put on the top of the structure.The site variables assumed here are:

a.	Region A3	… Figure 3.1 (A)
b.	Exposure Category 2	… 4.2.1
c.	V_{500} = 45 m/s	… Table 3.1
d.	Direction Multiplier:	… Table 3.2
	i. M_d = 1.0 (NW)	
	ii. M_d = 0.85 (SW)	
e.	Terrain Height Multiplier[1]: $M_{z,cat}$ = 1.10	… Table 4.1 (A)
f.	Shielding Multiplier: M_s = 1.0	… Table 4.3
g.	Topographic Multiplier: M_t = 1.0	… 4.4

3. Determine the site wind speeds, $V_{sit\beta}$ using the variables assumed in 9b. and the equation:

$V_{sit\beta} = V_r\, M_d\, (M_{z,cat}\, M_s\, M_t)$ … *Equation 1*

$V_{sit\beta}$ = V500 $M_d\, (M_{z,cat}\, M_s\, M_t)$

$V_{sit\beta}$ = 45 * 1.0 * 1.1 * 1.0 * 1.0 = 49.50 m/s … M_d=1.0 (NW)

$V_{sit\beta}$ = 45 * 0.85 * 1.1 * 1.0 * 1.0 = 42.08 m/s … M_d=0.85 (SW)

4. Determine the design wind speed, $V_{des\theta}$ from the site wind speeds:

$V_{des\theta}$ =49.50 m/s. (Controlling wind speed)

5. Determine the design wind pressures, calculated per section 2.4.1 and distributed forces using the equation;

$p = (0.5\, \rho_{air})\, [V_{des\theta}]^2\, C_{fig}\, C_{dyn}$ … *Equation 2*

a. ρ_{air} is the density of air = 1.2 kg/m^3

b. C_{fig} is a combination of internal ($C_{p,i}$) and external ($C_{p,e}$) pressure coefficients. These apply to the structure in combinations that produce the highest loads on the elements.

For instance, use $C_{p,i}$ from Table 5.1(A) for all walls equally permeable.

Use	$C_{p,i}$ = -0.3 or 0, whichever is worse for each surface.	
Use	$C_{p,e}$ = 0.8 for windward wall	… Table 5.2(A)
	$C_{p,e}$ = -0.5 for leeward wall	… Table 5.2 (B)
	$C_{p,e}$ = -0.5 for side walls[2]	… Table 5.2(C)
	$C_{p,e}$ = External pressure coefficient for curved roofs	… Appendix C
	= Windward Quarter (U) = -2.93	… Table C3
	= Centre Halve (T) = -2.17	… Table C3
	= Leeward Quarter (D) = -1.86	… Table C3
Use	K_a = 1.0	… Table 5.4
	$K_{c,i}$ = 1.0	… Table 5.5
	$K_{c,e}$ = 0.8	… Table 5.5
	K_l = 1.0	… 5.4.4
	K_p = 1.0	… 5.4.5
	C_f = 1.0	… 5.5
	K_c = 1.0	… 5.4.3

$C_{fig,i} = C_{p,i}\, K_{c,I}$ (Internal pressures) = -0.3 … 5.2 (1)

$C_{fig,e} = C_{p,e}\, K_a\, K_{c,e}\, K_l\, K_p$ (External pressures) = 0.51 Windward Wall … 5.2 (2)

$C_{fig} = C_f\, K_a\, K_c$ (frictional drag forces) = 1.0 … 5.2 (3)

c. $$C_{dyn} = \frac{1 + 2I_h \sqrt{g_v^2\, B_s + \frac{H_s\, g_R^2\, SE_l}{\zeta}}}{1 + 2g_v\, I_h}$$ … 6.2 (1)

C_{dyn} = 1.09

[1]Linear interpolation required for intermediate values of height z and terrain category

[2] Horizontal distance from windward edge, depth, is assumed to be between 1*h* to 2*h*

[3] $C_{fig,e}$ *= -0.32 Leeward Wall, -0.32 Side Wall. For curved Roof, -1.88 Windward Quarter (U), -1.39 Centre Halve (T), -1.19 Leeward Quarter (D).*

See *AS1170.2* Section 6 - Dynamic Response Factor, for a complete list of the variables and their respective equations.

Using *Equation 2* and all of the variables above, the design wind pressures are at h = 23.2m:

p = -0.481	kPa	… using $C_{fig,i}$
p = 1.027	kPa Windward Wall	… using $C_{fig,e}$
p = -0.642	kPa Leeward Wall	
p = -0.642	kPa Side Wall	
p = -3.763	kPa Windward Quarter (U)	… Curved Roof
p = -2.788	kPa Centre Half (T)	
p = -2.391	kPa Leeward Quarter (D)	
p = 1.604	kPa	… using C_{fig}

6. Determine the wind actions per Clause 2.5 of the *AS1170.2* design actions using the following equation:

$F = \Sigma(p_z A_z)$ … *Equation 3*

A_z = reference area, in square metres, at height z, upon which the pressure at that height (p_z) acts.

Specific wind actions were calculated for individual elements and are detailed in Table 71 below.

Table 71: Details the Wind Actions used in calculating the listed elements design demands.

Member	Level	Location	Wind Direction	Wind Height (m)	kP
Glulam Dome Beam	6	Roof	Windward Quarter (U)	23.2	-3.76
Tudor Arch	6	Roof	Windward (W)	23.21	0.87
			Leeward (L)		-0.37
			Upwind Roof (U)		-0.62
			Downwind Roof (D)		-0.58
Bracing Wall	2	Wall	Windward (W)	9	1.27
Portal Frame	Ground	Wall	Windward (W)	2.25	1.23

Design of Individual Elements

The calculations below describe the design approach for the various elements within the structure. A curved glulam beam, Tudor arch beam, roof deck beam, various floor elements, a portal frame and a fire rated beam are designed in this section. Note, some demands have been conservatively designed from what is recommended in *AS 1720.1*. For example, the shear is calculated at a distance d from the support, whereas Clause 3.2.5 allows a distance of 1.5d. This provides a higher factor of safety within the element and exceedingly satisfies the guideline set in the code.

Design a Curved Glulam Beam

This beam is a part of the ribbed dome for the sixth storey restaurant. The dome has a 3.6m rise and an 18m clear span. Figures 35 to 37 show the various sections of the curved glulam beam.

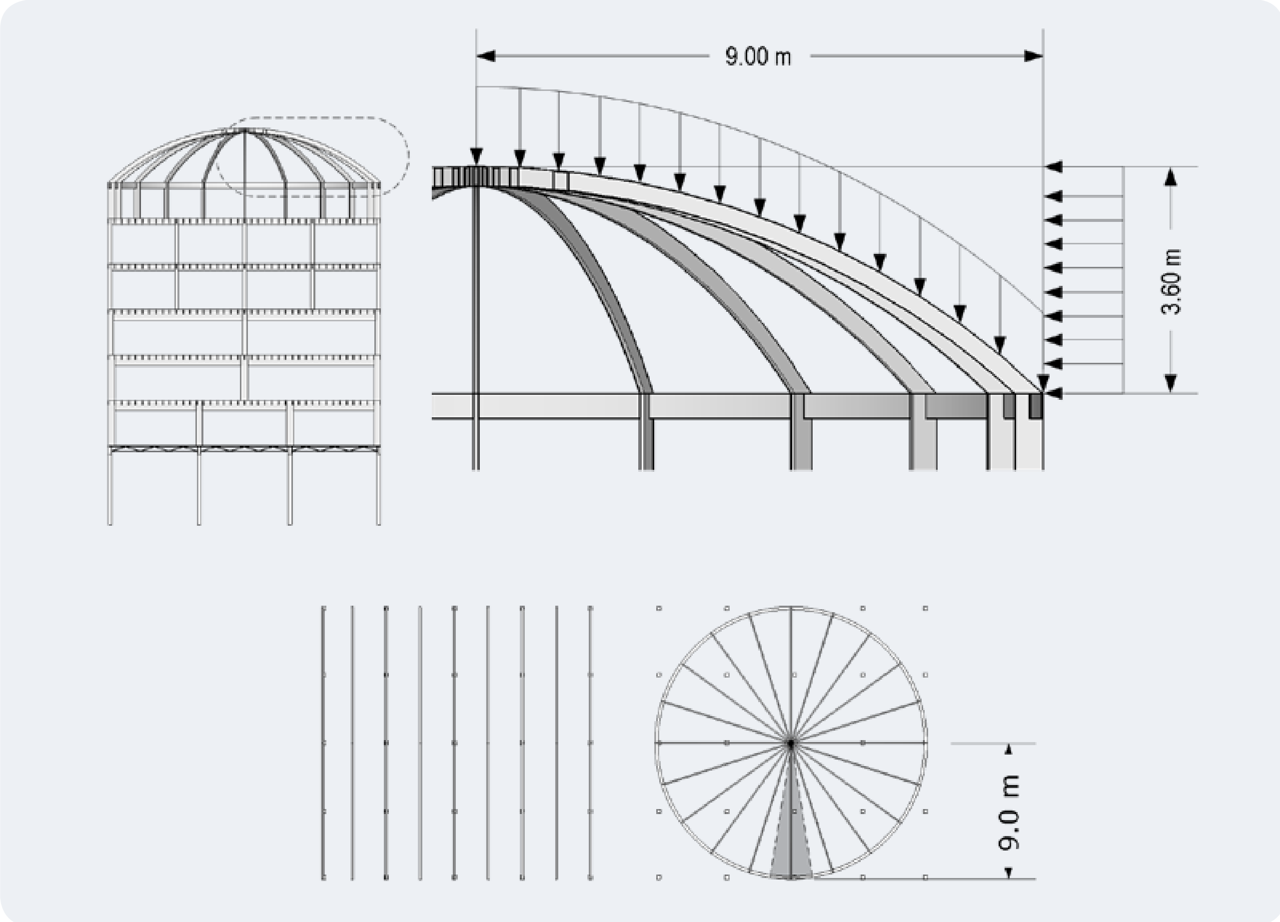


Figure 35: Curved glulam beam for ribbed dome

Using the properties of a circle, shown in the figure below, and the known variables provided, calculate the Radius (r), Angle (A), and Length (L).

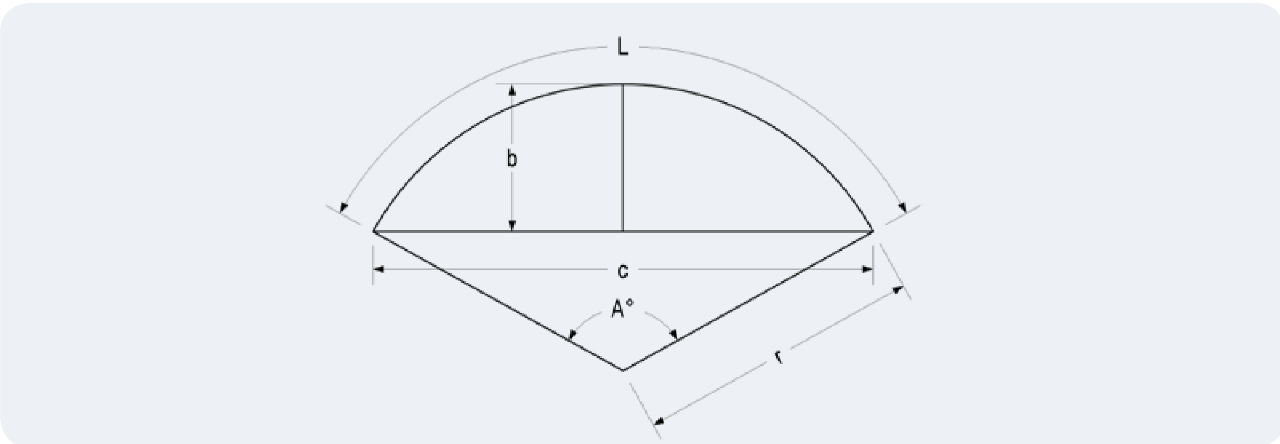


Figure 36: Properties of a circle

Knowing c = 18m and b = 3.6m,

$$r = \frac{4b^2 + c^2}{8b} = 13.05\text{m}$$

also using b and c we can determine the Angle A˚,

$$A^{o} = 4\tan^{-1}\left(\frac{2b}{c}\right) = 87.2^{\circ}$$

knowing both the Angle and Radius you can determine to Length of the arch (L),

$$L = \frac{87.2}{360}\, 2\,\pi\, 13.05 = 19.86\text{m}$$

Using the *AS1720.1* 2010 Design Standards Appendix E13.1 (b) shows a diagram of a Constant radius curved beam. Using this diagram and the Equations in E13.2.1 a design capacity in bending (M_d) and shear (V_d) can be calculated and compared to a known demand (M* and V*). The demand has been calculated using computer modeling software as well as input from the calculated variables seen above.

Using the *AS1170* Structural Design Actions Parts 0, 1 and 2 a combination for the ultimate limit states was used to calculate the strength and deflection demand of the Glulam Beam. Section 4.2.2(e) $E_d = [0.9G + W_u]$ provided the controlling case of M* = 8.201kN-m, V* = 12.147kN, and a deflection demand of 12.181mm. Knowing these demands, you can calculate an appropriate member with sufficient capacity suitable to support these demands.

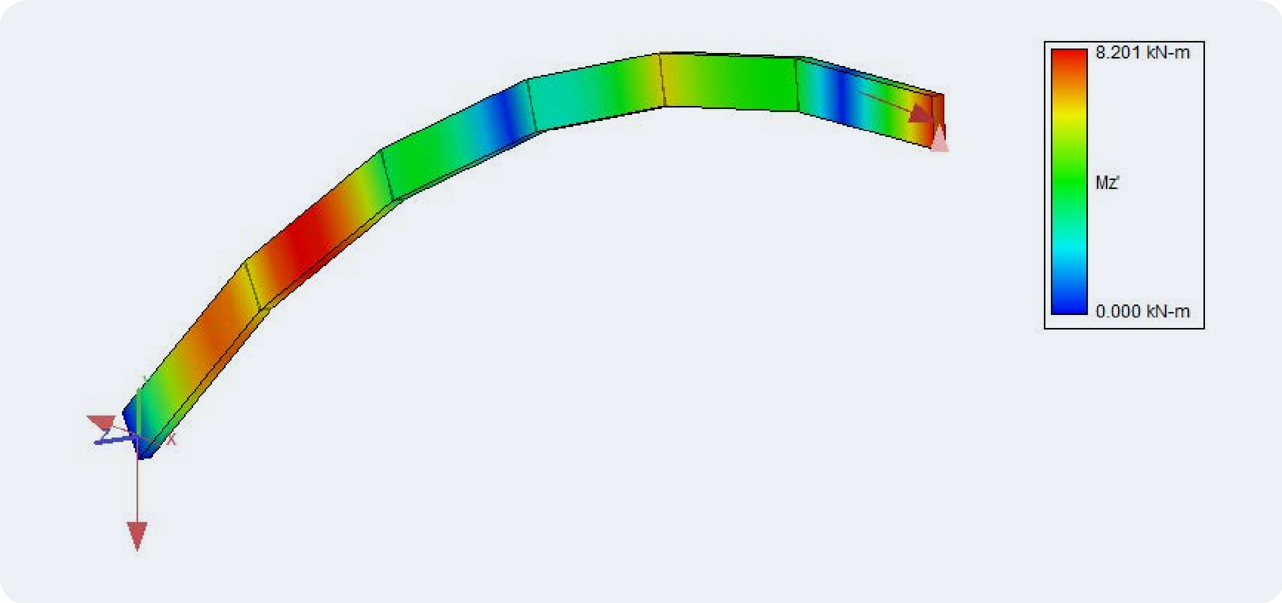


Figure 37: Cured glulam beam software output

Beam dimensions: 75 x 300 mm GL18 Glulam Beam. b = 75mm, d = 300mm.

Bending Strength:

$M_d \geq M^*$ … E13 (1)

where

Md = lesser of

$\phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_{sh}\, k_r\, f_b'\, Z$ … E13 (2)

$\phi\, k_1\, k_4\, k_6\, k_9\, k_v\, f_{tp}'\, (2\, A_{ap}\, R_{cl}\, /3)$ … E13 (3)

$\phi\, k_1\, k_4\, k_6\, k_9\, k_v\, k_{tp}\, f_{tp}'\, Z$ … E13 (4)

As an example the calculation for E13 (4) is detailed below which is the lessor of the 3 capacity equations. All variables are taken from the AS1720.1 Design Standards.

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
$k_v = 24/(A_{ap}\, R_{cl}\, \beta)^2$	= 0.45	… E13.2.5
k_{tp}	= 40	… E13.2.6
f_{tp}' = SD6	= 0.5	… H2.2
Z = bd2/6	= 0.00113 m^3	… 3.2.1.1
M_d = 8.53 kN-m >	**M* = 8.201 kN-m**	**OK**

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.015 m^2	… 3.2.5

V_d = 63.75 kN > V^* = 12.147 kN OK

Deflection Limit:

Using L/240 for the capacity limit sourced from a Timber Construction Manual, an allowable capacity limit of 41.38mm is calculated, where L = 19,860/2 = 9,930mm. The aforementioned deflection demand of **12.181mm** is well under this allowable limit.

Design a Tudor Arch Beam

The Tudor arch is 5m high at the apex and has an 18m span. It is treated as a 3-pin arch design with pinned connections at the apex and at the column bases. The arches are continuous curved glulam beams to provide moment resistance at the eave. The haunches have a radius of 0.65m, determined by the minimum radius available from the glulam fabricator and the eave height is 2.41m. The position and configuration of the arch analyzed is shown in Figure 38 below.

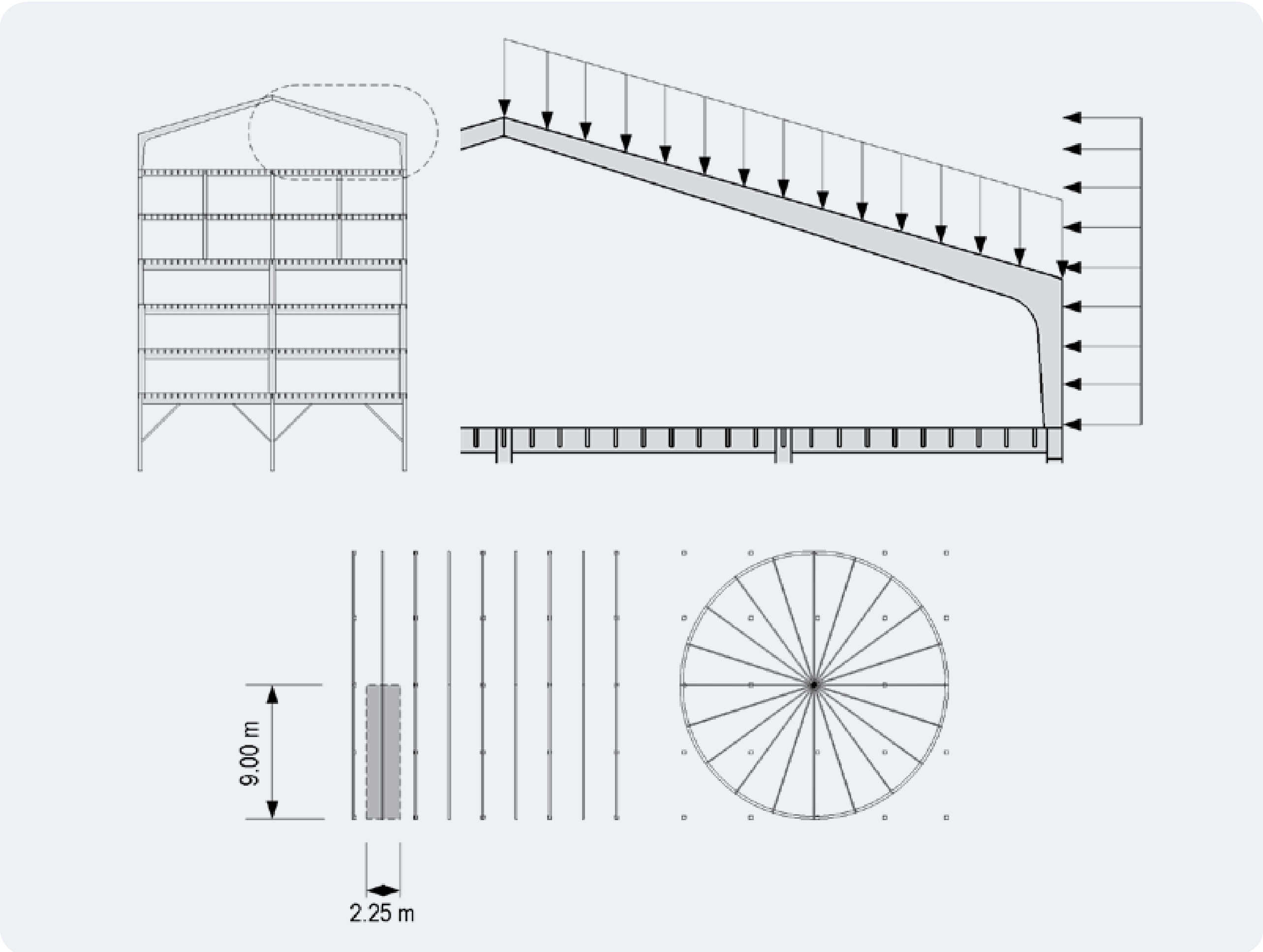


Figure 38: Illustration of the Tudor arch beam computed in this example.

Using the *AS1720.1* 2010 Design Standards Appendix E13.1 (e) shows a diagram of a Pitch Chambered beam. Using this diagram and the Equations in E13.2.1 a design capacity in bending (M_d) and shear (V_d) can be calculated and compared to a known demand (M* and V*). The demand has been calculated using computer modeling software as well as input from the calculated variables seen above.

Using the *AS1170* Structural Design Actions Parts 0, 1 and 2 a combination for the ultimate limit states was used to calculate the strength and deflection demand of the Glulam Beam. Section 4.2.2(d) Ed = [1.2G + Wu + ΨQ] provided the controlling case of M* = 12.76kN-m, V* = 5.94kN, and a deflection demand of 12.02mm. Knowing these demands, you can calculate an appropriate member with sufficient capacity suitable to support these demands.

Beam dimensions: 125 x 500 mm GL18 Glulam Beam. b = 125mm, d = 500mm.

Bending Strength:

$M_d \geq M^*$ … E13 (1)

where

M_d = lesser of

$\phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_{sh}\, k_r\, f_b'\, Z$ … E13 (2)

$\phi\, k_1\, k_4\, k_6\, k_9\, k_v\, f_{tp}'\, (2\, A_{ap}\, R_{cl}\, /3)$ … E13 (3)

$\phi\, k_1\, k_4\, k_6\, k_9\, k_v\, k_{tp}\, f_{tp}'\, Z$ … E13 (4)

As an example the calculation for E13 (4) is detailed below which is the lessor of the 3 capacity equations. All variables are taken from the *AS1720.1* Design Standards.

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
$k_v = 35/(A_{ap}\, R_{cl}\, \beta)^2$	= 0.93	… E13.2.5
k_{tp}	= 7	… E13.2.6
f_{tp}' = SD6	= 0.5	… H2.2
Z = bd2/6	= 5.2083 m^3	… 3.2.1.1

M_d = 14.36 kN-m > M* = 12.76 kN-m OK

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.0417 m^2	… 3.2.1.1

V_d = 177.08 kN > V* = 5.94 kN OK

Deflection Limit:

Using L/240 for the capacity limit sourced from a Timber Construction Manual, an allowable capacity limit of *47.51mm* is calculated, where L = 22.806mm. The aforementioned deflection demand of ***12.02mm*** is well under this allowable limit.

Design a Roof Deck Beam

There is a roof deck located on the sixth floor level. This area is designed for 5kPa live loads. The roof is made up of simply supported glulam beams spanning 4.5m with a tributary width of 4.5m. Floor joist spanning between these beams are placed at 450mm on centre. The location and configuration of the deck is shown in Figure 39 below.

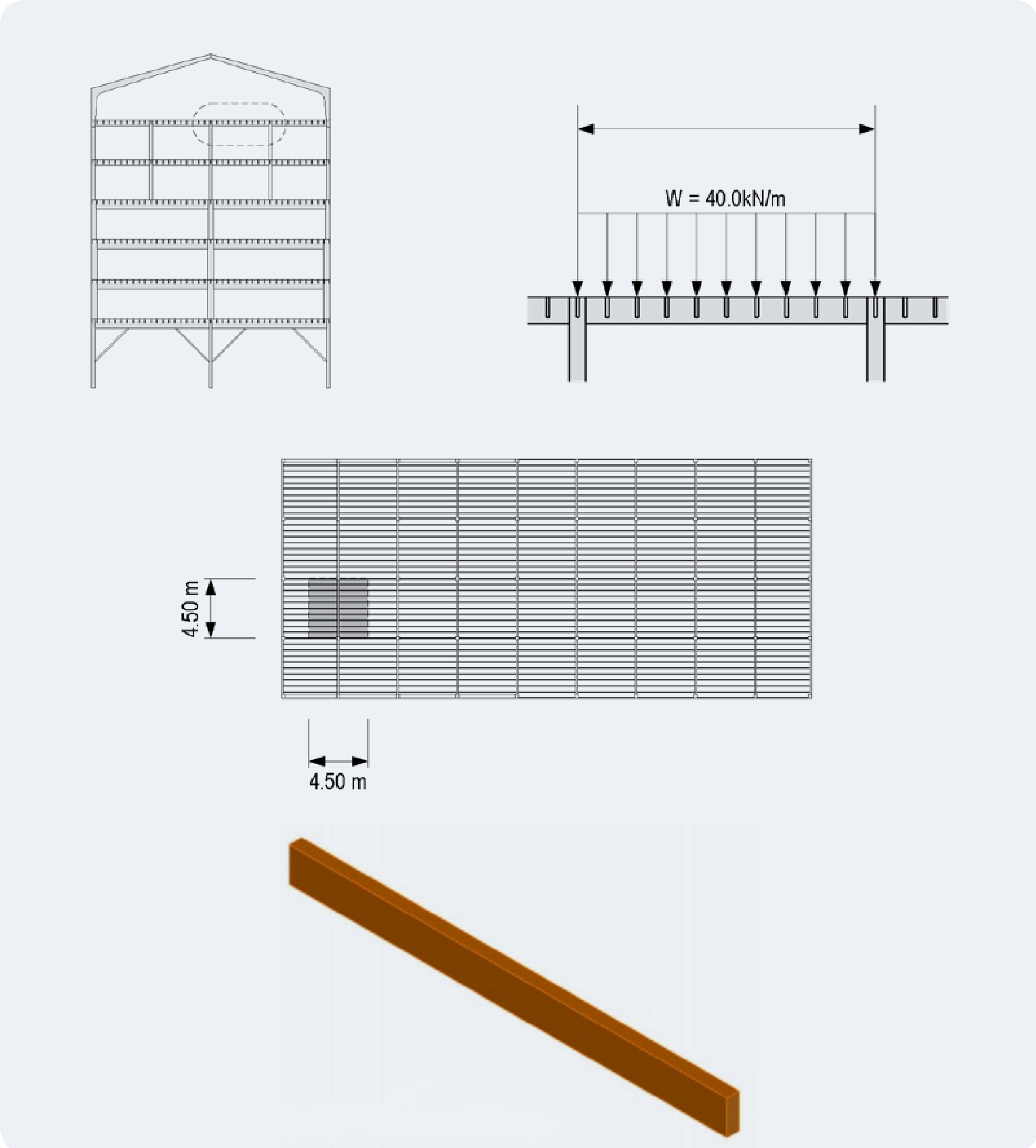


Figure 39: Structural location and configuration of the roof deck beam in this design example.

Beam dimensions: 125 x 400 mm GL18 Glulam Beam. b = 125mm, d = 400mm.

a. Uniformly Distributed Loads (UDL):

Dead load	= 1.1 kPa * trib. width	= 4.95 kN/m	
Self-Weight	= (density*b*d*g)/1000^3 Where density = 560 kg/m^3	= 0.27 kN/m	… g = 9.81m/s^2
Roof Deck	= 5 kPa * trib. width	= 22.5 kN/m	
Q		= 22.5 kN/m	
G		= 5.22 kN/m	

Unfactored Reactions (needed for fourth floor beam calculations below):

Q_r = (Q*L)/2	= 50.6 kN	
G_r = (G*L)/2	= 11.7 kN	

b. Load Combinations from *AS1170.0*

The controlling Strength Limit State	= 1.2G + 1.5Q	… 4.2.2 (b)
$SLS_{(UDL)}$	= 40.0 kN/m	… Factored Load (W)

c. Calculate the beam's Demand:

M*(UDL) = (W*L^2)/8	= 101.3 kN-m	
V*(UDL) = W*(L/2 - d)	= 74.0 kN	
Deflection Limit: (5*W*L^4)/384*E*I	= 0.017 m = 17.31 mm	
		… E = 18500*10^3 kPa … I = 6.67*10^{-4} m^4

d. Calculate the beam's Capacity:

Bending Strength:

$M_d \geq M^*$ … 3.2(1)

where

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f_b'\, Z$ … 3.2(2)

ϕ	= 0.85	… 2.3
k^1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
Lay	= 1500 mm	
r	= 0.25	
ρb	= 0.89	
S1 = 1.25*(d/b)*(Lay/d)$^{0.5}$	= 7.75	
ρbS1	= 6.89	
k_{12}	= 1	… 3.2.4
f_b' = GL18	= 45000 kPa	… 7.3.1
Z	= 0.00333 m^3	

Md = 102 kN-m > M* = 101.3 kN-m OK

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.0333 m^2	… 3.2.5

V_d = 113.22 kN > V* = 74.0 kN OK

Deflection Limit:

Using L/240, an allowable capacity limit of 18.75mm is calculated, where L = 4,500mm. The aforementioned deflection demand of ***17.31mm*** is under this allowable limit.

Design a Fifth Floor 4.5m Floor Beam

Similar to the roof deck, the fifth floor is made up of simply supported glulam beams spanning 4.5m with a tributary width of 4.5m. Floor joists are spaced at 450mm on centre. These beams are designed for 3kPa residential loads. Figure 40 indicates the location of the beam under analysis.

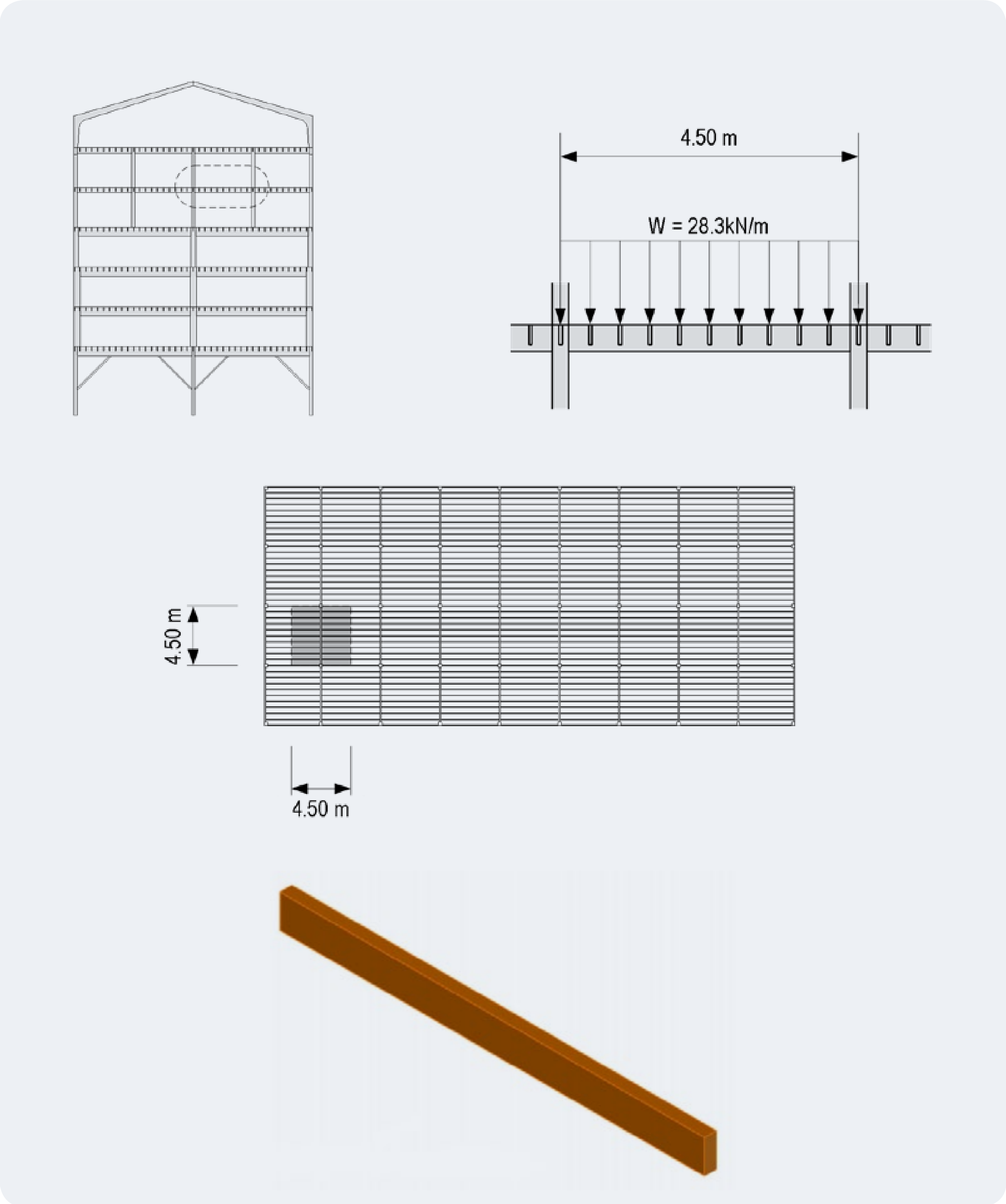


Figure 40: Structural location and configuration of the fifth level floor beam.

Beam dimensions: 100 x 400 mm GL18 Glulam Beam. b = 100mm, d = 400mm.

a. Uniformly Distributed Loads (UDL):

Dead load	= 0.85 kPa * trib. width	= 3.8	
Partition Load	= 0.6 kPa * trib. width	= 2.7 kN/m	
Self-Weight	= (density*b*d*g)/1000^3 Where density = 560 kg/m^3	= 0.22 kN/m	… g = 9.81m/s^2
Residential Load	= 3 kPa * trib. width	= 3.5 kN/m	
Q		= 13.5 kN/m	
G		= 6.7 kN/m	

Unfactored Reactions (needed for fourth floor beam calculations below):

Q_r = (Q*L)/2	= 30.4 kN	
G_r = (G*L)/2	= 15.1 kN	

b. Load Combinations from AS1170.0

The controlling Strength Limit State	= 1.2G + 1.5Q	… 4.2.2 (b)
$SLS_{(UDL)}$	= 28.3 kN/m	… Factored Load (W)

c. Calculate the beam's Demand:

$M^*_{(UDL)}$ = (W*L^2)/8	= 71.6 kN-m	
$V^*_{(UDL)}$ = W*(L/2 - d)	= 52.4 kN	
Deflection Limit: (5*W*L^4)/384*E*I	= 0.015 m	
	= 15.32 mm	
		… E = 18500*10^3 kPa
		… I = 5.33*10^{-4} m^4

d. Calculate the beam's Capacity:

Bending Strength:

$M_d \geq M^*$ … 3.2(1)

where

$M_d = \phi\ k_1\ k_4\ k_6\ k_9\ k_{12}\ f_b'\ Z$ … 3.2(2)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
Lay	= 1500 mm	
r	= 0.25	
ρb	= 0.89	
S1 = 1.25*(d/b)*$(Lay/d)_{0.5}$	= 9.68	
ρbS1	= 8.62	
k_{12}	= 1	… 3.2.4
f_b' = GL18	= 45000 kPa	… 7.3.1
Z	= 0.00267 m^3	

M_d = 81.6 kN-m > M* = 71.6 kN-m OK

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\ k_1\ k_4\ k_6\ f_s'\ A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.0267 m^2	… 3.2.5

V_d = 90.67 kN > V* = 52.4 kN OK

Deflection Limit:

Using L/240, an allowable capacity limit of ***18.75mm*** is calculated, where L = 4,500mm. The aforementioned deflection demand of ***15.32mm*** is under this allowable limit.

Design a Fourth Floor 9m Floor Beam

The support system below the fourth floor transition from a 4.5 x 4.5m grid to 4.5 x 9m. The fourth floor beams are simply supported glulam spanning 9m with a tributary width of 4.5m. Floor joists @ 450mm on centre. The beams receive a point load at mid-span from the columns above. A floor load reduction factor, $\Psi_a = 0.68$ applies over the fourth and fifth floor area when calculating imposed loads. Figure 41 illustrates the loading conditions and configuration of the fourth floor beam.

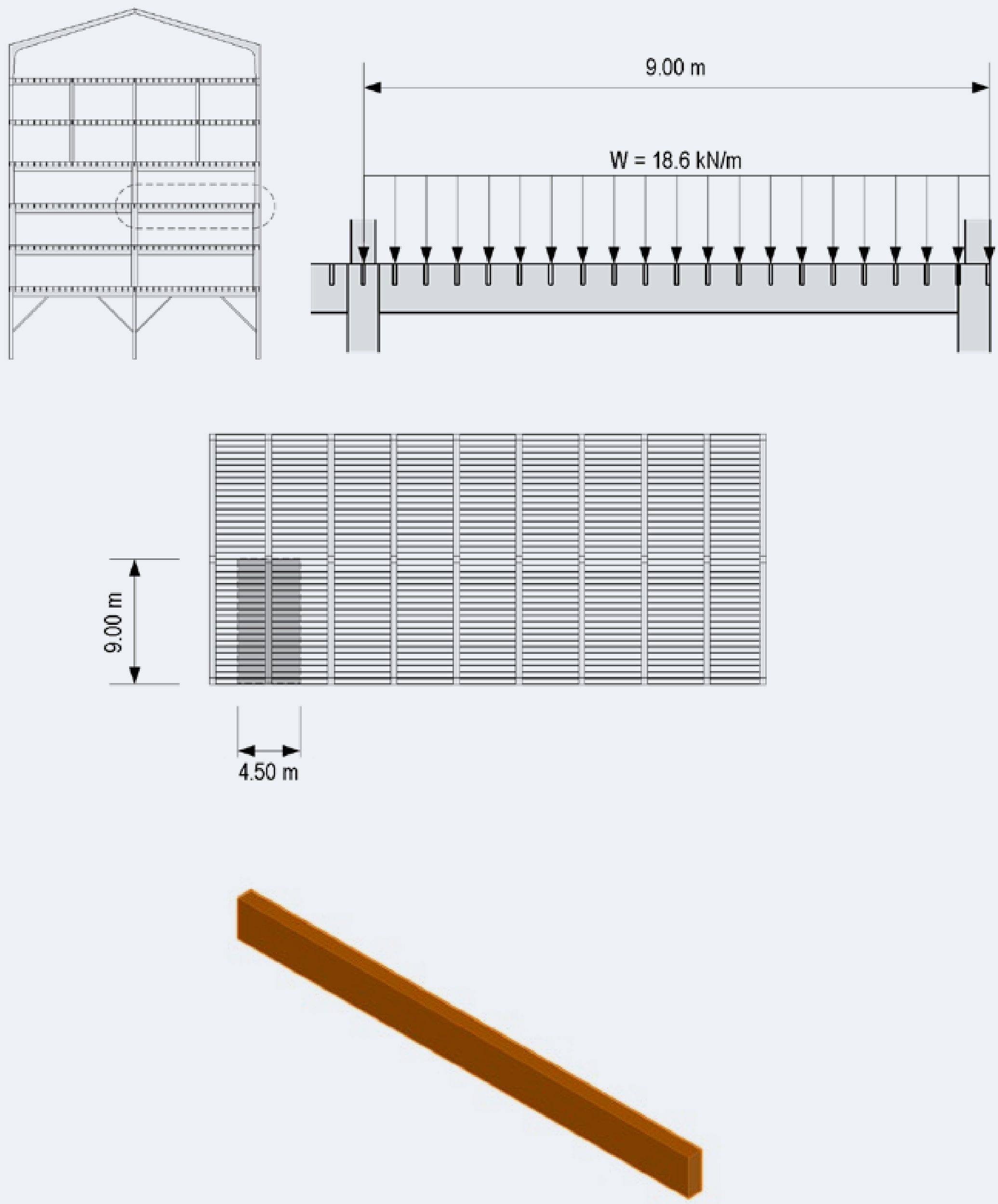


Figure 41: Structural location and configuration of the fourth level floor beam in this design example.

Beam dimensions: 350 x 700 mm GL18 Glulam Beam. b = 350mm, d = 700mm.

a.	Uniformly Distributed Loads (UDL):		
	Dead load = 0.85 kPa * trib. width	= 3.8	
	Partition Load = 0.6 kPa * trib. width	= 2.7 kN/m	
	Self-Weight = (density*b*d*g)/1000^3 Where density = 560 kg/m^3	= 1.35 kN/m	… g = 9.81m/s^2
	Residential Load = 3 * 0.68 kPa * trib. width	= 9.2 kN/m	
	Q	= 9.2 kN/m	
	G	= 7.85 kN/m	
b.	Above Floor Loads:		
	Qa = $Q_{r(5th\ Floor*\Psi a\ +\ Roof)}$*2	= 142.5 kN	
	Ga = $G_{r(5th\ Floor*\Psi a\ +\ Roof)}$*2	= 43.94 kN	
c.	Load Combinations from AS1170.0		
	The controlling Strength Limit State	= 1.2G + 1.5Q	… 1170.0-4.2.2 (b)
	$SLS_{(UDL)}$	= 23.22 kN/m	… Factored Load (W)
	$SLS_{(P)}$	= 280.0 kN	… Factored Load (P)
d.	Calculate the beam's Demand:		
	$M^*_{(UDL)}$ = $(W*L^2)/8$	= 235.1 kN-m	
	$M^*_{(P)}$ = (P*L)/4	= 630.0 kN-m	
	$M^*_{(SUM)}$	= **865.1 kN-m**	
	$V^*_{(UDL)}$ = W*(L/2 - d) = 88.2	kN	
	$V^*_{(P)}$ = (P/2)	= 140.0 kN	
	$V^*_{(sum)}$	= **228.2 kN**	
	Deflection Limit: $(5*W*L^4)/384*E*I$	= 0.010 m	
		= 10.72 mm	
	Deflection Limit: $(P*L^3)/48*E*I$	= 0.023 m	
		= 22.66 mm	
	Deflection Limit $_{(SUM)}$	= **33.38 mm**	
			… E =$18500*10^3$ kPa
			… I = $1.00*10^{-2}$ m^4
e.	Calculate the beam's Capacity:		
	Bending Strength:		
	$M_d \geq M^*$		… 3.2 (1)
where			
	$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f_b'\, Z$		… 3.2 (2)
	ϕ	= 0.85	… 2.3
	k_1	= 0.8	… 2.4.1.1
	k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
	k_6	= 1	… 2.4.3
	k_9	= 1	… 7.4.3
	Lay	= 1500 mm	
	r	= 0.25	
	ρb	= 0.89	
	S1 = $1.25*(d/b)*(Lay/d)^{0.5}$	= 3.66	
	ρbS1	= 3.26	
	k_{12}	= 1	… 3.2.4
	f_b' = GL18	= 45000 kPa	… 7.3.1
	Z	= 0.0285 m^3	
	Md = 874.65 kN-m **>**	**M* = 865.1 kN-m**	**OK**

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 Pa	… 7.3.1
A_s = (2/3) (b*d)	= 0.1633 m^2	… 3.2.5

V_d = 555.22 kN > V* = 228.2 kN OK

Deflection Limit:

Using L/240, an allowable capacity limit of *37.50 mm* is calculated, where L = 9,000mm. The aforementioned deflection demand of ***33.38 mm*** is under this allowable limit.

Design a Third Floor 9m 4hr Fire Resistant Floor Beam

The third floor beams are simply supported glulam spanning 9m with a tributary width of 4.5m. Floor joists are at 450mm on centre. A floor load reduction factor, Ψ_a = 0.77 applies over the third Floor Tributary area of 40.5m^2 when calculating imposed loads. A four-hour fire separation is required at the third floor to separate commercial and residential uses. Using the *AS1720.4-2006 Timber Structures – Fire Resistance for Structural Adequacy of Timber Members* Design Standard we can calculate whether or not the nominated beam will withstand the impact of fire for a time duration of 4 hours. Figure 42 below provides the structural location of the beam under analysis for a 4 hour fire resistance.

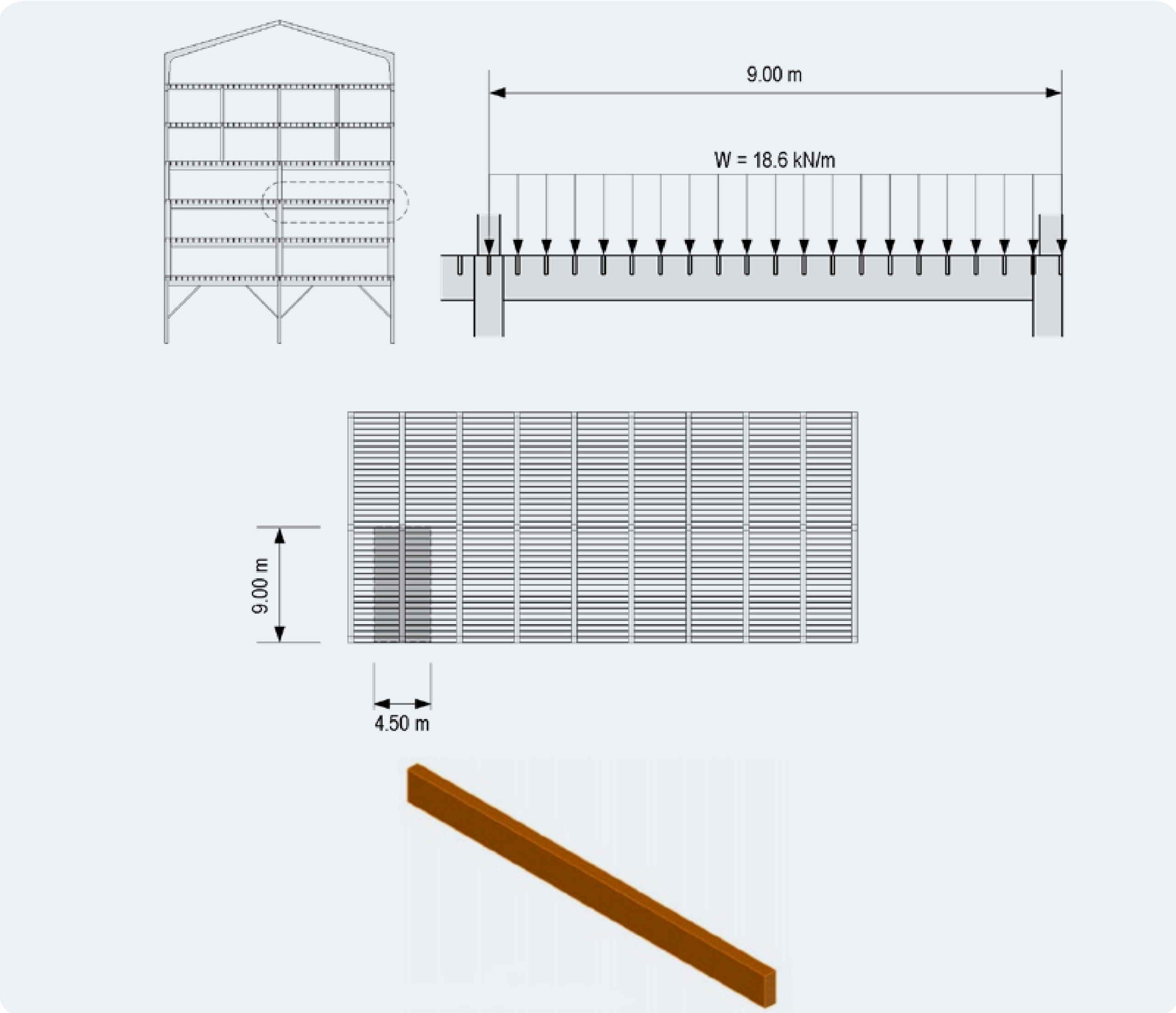


Figure 42: Structural location and configuration of the third level 4 hour fire resistant beam in this design example.

Beam dimensions: 450 x 700 mm GL18 Glulam Beam. b = 450mm, d = 700mm.

Assume 3 sides of the 4 are open to the elements.

a. Notional Charring Rate … 2.4

c	$= 0.4 + (280/\delta)^2$		…Equation 2.1
δ	= density of Douglas fir @ 12% Moisture content		
	$= 560\ kg/m^3$		
c	= 0.65 mm/min		

b. Effective Depth of Charring … 2.5

d_c	= c*t + 7.5		…Equation 2.2
t	= 240 minutes (4hrs)		
d_c	= 163.50mm		

c. Size of Effective Residual Section ` … 2.6

b	$= 450 - (d_c*2)$	= 123 mm	
d	$= 700 - d_c$	= 536.5 mm	

d. Design Load … 2.8

Strength Limit State		= 1G + thermal effect + 1Q	…1170.0-4.2.4
Dead load	= 0.85 kPa * trib. width	= 3.8 kN/m	
Partition Load	= 0.6 kPa * trib. width	= 2.7 kN/m	
Self-Weight	$= (density*b*d*g)/1000^3$ Where density $= 560\ kg/m^3$	= 1.73 kN/m	… $g = 9.81 m/s^2$
Residential Load	= 0.77 * 3 kPa * trib. width	= 10.4 kN/m	
Q		= **10.4** kN/m	
G		= **8.2** kN/m	
W		= 1G + 1Q (no thermal effect on remaining wood) = 18.6 kN/m	
$M^*_{(UDL)}$	$= (W*L^2)/8$	= **188.3** kN-m	
$V^*_{(UDL)}$	= W*(L/2- d)	= **70.7** kN	

e. Strength of effective Residual Section

Bending Strength:

$M_d \geq M^*$ … 3.2 (1)

where

$M_d = \phi\ k1\ k4\ k6\ k9\ k12\ fb'\ Z$ … 3.2 (2)

ϕ	= 0.85	… 2.3
k_1	= 0.94 (5hr duration)	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
Lay	= 1500 mm	
r=Live load/total load	= 0.56	
ρb	= 0.845	
S1 = 1.25*(d/b)*(Lay/d)0.5	= 9.44	
ρbS1	= 7.98	
k_{12}	= 1	… 3.2.4
f_b' = GL18	= 45000 kPa	… 7.3.1
Z	$= 0.00564\ m^3$	

M_d = 209.36 kN-m > M* = 188.3 kN-m OK

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$		… 3.2 (14)
ϕ	= 0.85	… 2.3
k_1	= 0.97	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.0422 m^2	… 3.2.5

V_d = 174.15 kN > V* = 70.7 kN OK

It is calculated that the beam will last 243 minutes against bending limits therefore stands up to 4 hours of fire resistance.

Design a Second Level Bracing Wall

This is designed according to *AS1684.2 Design Standards – Section 8.3.6 Wall Bracing.*

The second floor wall needs to resist the lateral loads from the third floor sheathing. Half the height of the second floor wall pushes on the third floor and the area above. This must be taken into account when calculating the area.
The second floor is at a height of 7.5m, therefore the vertical distance from half of the second floor to the top of the building = 25 – 7.5 – 3/2 = 16m.

a. Brace Wall Capacity

Wind Pressure =		1.27 kPa	Windward + Leeward Pressure @9m.
Area =		288 m^2	
Racking Force = p x A	=	365.8 kN	
Bracing Capacity =		6.4 kN/m	… T 8.18 (h) – Method A
Height Multiplier =		0.9	… Table 8.19
Wall Resistance Length	=	365.8 / (6.4*0.9)	
	=	**63.5m** of bracing wall required	

Maximum distance between braced walls at right angles to the building width shall not exceed 4.6m with a roof pitch of 17.1 degrees. This allows for residential internal floor plans to be maintained within the interior floor design. Assume 5 bracing walls are required consisting of 2 external walls of 15m lengths each, and 3 internal walls at 15m lengths each. Total wall length = 75m > 63.5m. Stud spacing is 600mm and plywood thickness on both sides of the wall is 7mm and F11 Grade. Fastener spacing is 150mm on centre using 30 x 2.8 ϕ galvanised flat head nails. M12 Rods are to be used at each end of the sheathed section top plate to bottom plate.

Internal Wall Resistance	= 3 * 15 * 6.4 * 0.9
	= 259.2 kN
External Wall Resistance	= 2 * 15 * 6.4 * 0.9
	= 172.8 kN
Sum	= **432.0 kN > 365.76 kN OK**

b. Top Plate Connection Capacity

Assume joists are at 450mm ctrs

Number of Joists per internal wall = 33
Number of Joists per external wall = 33
Using Table 8.22 (i) M12 Bolts in JD4 seasoned timber provided 5.1kN of Shear capacity per joist top plate connection

Total number of bolts and blocking pieces per internal wall = 66
Total number of bolts and blocking pieces per external wall = 66
Ensure the block piece is large enough to avoid splitting

Top Plate Connection Capacity:	Internal walls	= 3 * (66/2) * 5.1
		= 504.9 kN
	External walls	= 2 * (6/2) * 5.1
		= 336.6 kN
	Sum	= **841.5 kN > 365.76 kN OK**

c. Bottom Plate Connection Capacity

Assume double joists are at 450mm ctrs

Number of double joists per internal wall	= 33
Number of double joists per external wall	= 33

Using Table 8.24 (b) M12 Bolts in JD4 seasoned timber provided 20kN of Shear capacity

Total number of bolts per internal wall	= 33
Total number of bolts per external wall	= 33

Ensure the block piece is large enough to avoid splitting

Top Plate Connection Capacity:	Internal walls	= 3 * 33 * 20	
		= 2000.0 kN	
	External walls	= 2 * (33) * 20	
		= 1333.3 kN	
	Sum	= **3333.3 kN > 365.76 kN**	**OK**

As a cost saving measure 80% of the bottom joists could be single which would reduce the connection capacity but still remain at an acceptable level. i.e. They double joist and M12 bolt totals would be 6. The Capacity would = 600 kN > 365.76 kN.

d. Holdown force to resist uplift from overturning moments.

Force/m	= 365.78kn / 18m	= 20.32 kN/m

The distance between the brace walls is 4.5m and the height of each is 2.5m (with a 500mm ceiling depth). A tributary 4.5m length will experience 91.44kN of force. The 2.5m height of the brace wall generates a 228.6kN-m overturning moment. The deadweight of the brace wall, assumed to be 1.17kN/m over the 15m length generates an opposing deadweight moment resistance of 131.625kN-m. The summation of the two overturning moments calculates a net overturning moment of 95.975kN-m which across the 15m length produces a required holdown force of 6.47kN.

Design an Open Web Floor Joist:

This open web floor joist is to span 6m on Level 1 and support a dead and partition load of 0.85kPa and 0.6kPa respectively, and a commercial office load of 3kPa. The open we floor joist is illustrated in Figure 43 below.

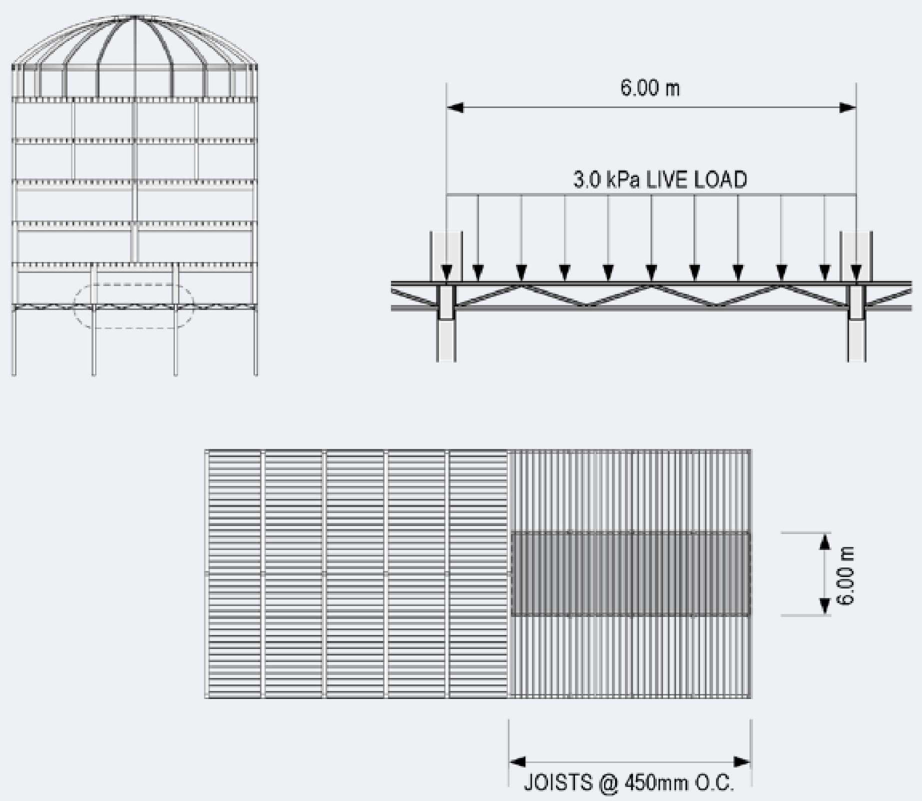


Figure 43: Structural location and configuration of the open web floor joist in this design example.

Using the span tables provided by Manufacturers aligning the design factors with capacity standards allows the correct truss member to be chosen. The Manufacturer used in this case study is Pryda and the span table is shown below in Figure 44.

Commercial Office Loads

Pryda Longreach

Dead Load = Nominal +0.50 kPa, Live Load = 3.0 kPa; 2.7 kN;
Truss Spacing = 450 c/c

Floor Truss ID	Chord Sizes	Max Span for 450 c/c
PL25/3	90x35 MGP10	2800
	90x45 MGP12	4900
	90x45 F17	5200
PL30/3	90x35 MGP10	3900
	90x45 MGP12	5600
	90x45 F17	5800
PL35/3	90x35 MGP10	4200
	90x45 MGP12	6300
	90x45 F17	6400
PL40/3	90x35 MGP10	4600
	90x45 MGP12	6800
	90x45 F17	6900
PL45/3	90x35 MGP10	4900
	90x45 MGP12	7300
	90x45 F17	7500

(i) Assumes 22mm particleboard flooring, using 565mm standard web lengths

Figure 44: Tabular illustration deriving maximum spans for various chord sizes.

This longreach truss design meets the specified demands. A Longreach truss with 90x45 F17 chord sizes, top and bottom, with standard 565mm web lengths spaced at 450 c/c is able to span the 6m length and carry the demand requirements.

Pryda Longreach Trusses

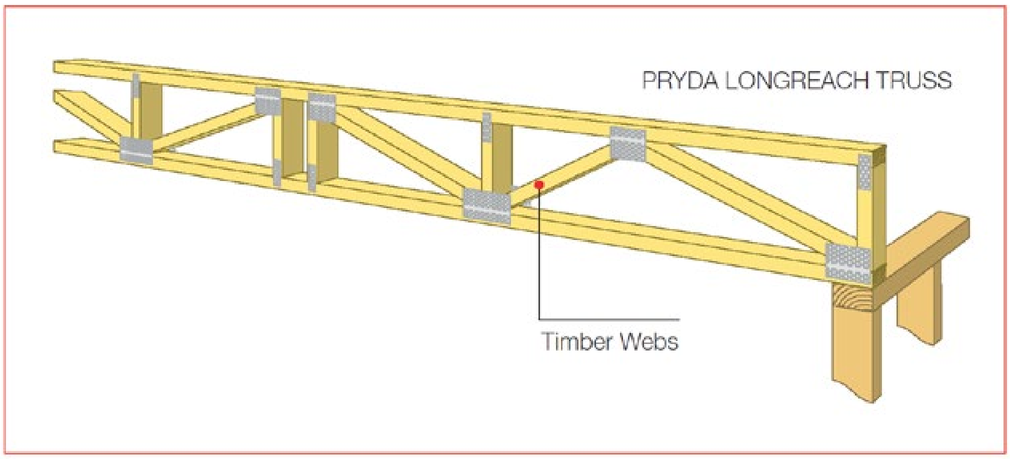


Figure 45: 3-Dimensional Sketch of a Pryda Longreach Truss

Design a Portal Frame

Situated on the ground floor of the seven story building. The portal frame is 4.5m in height and 9m in length, thus 2 frames take up the 18m width. A zero degree pitch on the beam requires corner support bracing at each of the two corner joints. The frames are spaced at 4.5m centres along the building. A 3.54m support brace will attach to the column and overhead beam at a 45 degree angle with a triangular plywood gusset fastened to each corner. An illustration of the portal frame is shown in Figure 46.

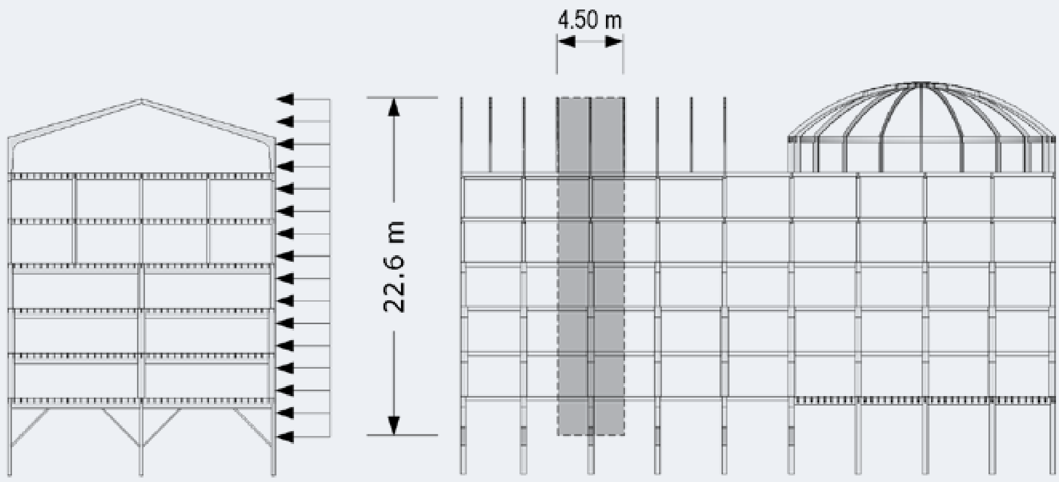


Figure 46: Illustration of the configuration of the portal frame in this design example.

Beam dimensions: 350 x 700 mm GL18 Glulam Beam. b = 350mm, d = 700mm. Column Dimensions: 350 x 250. Diagonal Brace Dimensions: 350 x 100. This analysis looks at the portal frame corner connections, uplift, and the combined axial and bending on the columns.

a. Uniformly Distributed Loads (UDL):

Dead load	= 1.1 kPa * trib.width	= 4.95 kN/m	
Self-Weight	= (density*b*d*g)/1000^3	= 2.0 kN/m	… g = 9.81m/s^2
Where density	= 560 kg/m^3		
Floor Load	= 3 kPa * trib.width	= 13.5 kN/m	
Q		= 13.5 kN/m	
G		= 6.7 kN/m	

b. Concentrated Load on end of Frame (exterior end has ½ the point load on the interior):

PQ	= 688.5 kN	
PG	= 284.9 kN	

c. Load Combinations from *AS1170.0*

Strength Limit State	= $1.2G + 1.5\psi_L Q$	… 4.2.2 (c)
	= $1.2G + Wu + \psi_c Q$	… 4.2.2 (d)
	= $0.9G + Wu$	… 4.2.2 (e)
ψ_L	= 0.4	… Table 4.1
ψ_c	= 0.4	… Table 4.1

d. Portal Frame Moment Demand:

Wind Pressure	=	1.234 kPa	Windward + Leeward Pressure @2.25 m
Area	=	102.4 m^2	(25 m – 2.25 m) x 4.5 m = 102.4 m^2
Racking Force	=	p x A / 2 frames	= 63.2 kN

Moment on Portal Frame Corners = M* = 63.2/2 * 3.606 = 114 kN-m

e. Portal Frame Corner Connection:

Use 2.8 ϕ galvanised flat head nails spaced at 50mm intervals to secure the plywood to the frame. Using the *AS1720.1* Design Standard Section 4.2, the design capacity of nailed joint connections can be analysed. This case study determines the capacity of in-plane moment that a Type 1 joint can resist.

$M_{d,j} \geq M^*$ … 4.2 (3)

where

$$M_{d,j} = \phi\, k_1\, k_{13}\, k_{14}\, k_{16}\, k_{17}\, r_{max}\, Q_k \sum_{i=1}^{n} \left(\frac{r_i}{r_{max}}\right)^{3/2}$$ … 4.2 (4)

ϕ	= 0.80		… Table 2.2
k_1	= 0.77		… Table 2.3
k_{13}	= 1	Side Grain	
k_{14}	= 2	Double Shear	
k_{16}	= 1.1	Plywood Gussets	
k_{17}	= 1.2	> 120 Nails	… Table 4.3 (b)
r_{max}*	= 1.85m		
Q_k	= 0.47kN	J4 2.8 ϕ Nail	… Table 4.1 (a)
n	= 170	Nail Count	
$\sum_{i=1}^{n} \left(\frac{r_i}{r_{max}}\right)^{3/2}$	= 83.2		
$M_{d,j}$	= 117.6 kN-m	> 114 kN-m OK Use plywood both sides	

*To calculate r_{max} the centroid of the nail group must be located. This can be done by determining the horizontal and vertical coordinates for each nail. The centroid is located at the average of the horizontal and average of the vertical coordinates. A spread sheet program can be used to calculate the coordinates and sum the capacities.

f. Axial Demand on Column:

N^*_c = 1.2 x (6.7 x 9 + 284.9) + 1.5 x 0.4 x (13.5 x 9 + 688.5) = 900.2 kN

… 1.2G + 1.5ψ_LQ

N^*_c = 1.2 x (6.7 x 9 + 284.9) + 0.4 x (13.5 x 9 + 688.5) = 738.2 kN

… 1.2G + Wu +ψ_cQ

g. Axial Column Capacity:

$N_{d,c} \geq N^*_c$ … 3.3 (1)

where

$N_{d,c} = \phi\, k_1\, k_4\, k_6\, k_{12}\, f_c'\, A_c$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
S_4 = 3.5d/b	= 4.9	
k_{12}	= 1.0	… 3.2.4
f_c' = GL18	= 45000 kPa	… 7.3.1
A_c = (b*d)	= 0.0875 m^2	… 3.3.1.1

$N_{d,c}$ = 2677 kN > N^*_c = 900.2 kN OK

h. Bending Demand on Column:

M* = 63.2/2 * (4.5 -2.5) = 63.2 kN-m

i. Column Bending Capacity:

$M_d \geq M^*$		… 3.2 (1)

where

$M_d = \phi\ k_1\ k_4\ k_6\ k_9\ k_{12}\ f_b'\ Z$		… 3.2 (2)
ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
Lay	= 2000 mm	
b	= 250 mm	
G_{13}	= 0.85	
L	= 3800 mm	
S_4 = The lesser of Lay/b & G_{13} x L/b	= 8	
r	= 1.0	
ρb	= 1.02	
$\rho b S_4$	= 8.16	
k_{12}	= 1	… 3.3.3
f_b' = GL18	= 45000 kPa	… 7.3.1
Z	= 0.0036 m^3	

M_d = 111.6 kN-m > M* = 63.2 kN-m OK

j. Combined Bending and Axial Action:

$[M^* / M_d]^2 + N^*_c / N_{d,c} \leq 1.0$		… 3.5.1
$[63.2/111.6]^2 + 900.2/2677$	= **0.66 ≤ 1.0**	**OK**

11.2 Worked Example 2: Ten Storey Mixed-Use Building

11.2.1 Conceptual Design

In this example, the initial concept is a ten storey, mixed-use structure that includes retail space on the ground floor, office spaces on levels one through three and residential housing units on levels four through ten. The general concept design of the structure is illustrated in Figure 47. The design incorporates a swimming pool on level ten (as shown in Figure 48). The structure is designed to provide an open floor plate that provides potential flexibility for tenant improvements or modifications.

The ten storey structure on top of a concrete first floor retail space with a swimming pool on the top floor shows the effectiveness of heavy dimension mass glue laminated timber (glulam) beams and cross laminated timber (CLT) in the construction of tall timber buildings.

The exterior footprint of the structure consists of a 24m wide x 47.4m long rectangle cross-section with a height of 37m. The loads are resisted by columns and brace frames that allow the exterior walls to consist entirely of glass. The column spacing and brace frames are integrated within the structure with the intent to provide an open floorplan where partitions can be continuously relocated to accommodate the user's needs. The columns are spaced 5.85m centre to centre and seven brace frame locations are situated in various directions. Two brace frames are located on either side of the long exterior walls, one is placed on each of the short exterior walls, and one is located in the short plan direction at the center of the building.

The floor-to-floor height of the building levels is dependent on the different uses for each floor. The height between the ground floor and level one is 4m, which encompasses the retail space. Levels one through ten are 3.3m floor-to-floor for the office and residential spaces. These clearances allow adequate ceiling heights considering their use. Glulam beams are used to support CLT floors and provide a shallow structural system that expands the vertical clearances. Figure 50 and Figure 51 show the structure's dimensions.

To ensure a safe structure that conforms to code requirements and serviceability, two stairways have been incorporated into the design. Additionally, elevators and utilities have the ability to pass through the floors at these locations. Although a fire-rating for this structure is not established in this design calculation, *AS 1720.4* and NCC 2016 – Vol 1 should be used to calculate the fire rating for the elements within the structure.

Varying materials were used in the design of this structure. Reinforced concrete is utilised in the construction of the ground level floor, level one and any level under the ground, as well as the foundations. The design of the reinforced concrete sections is outside the scope of this worked example. Heavy timber columns and braced frames are assumed as pinned connections at the top of the concrete of the level one floor. The exterior shell of the structure consists of glass. See Figure 49.

The structure is intended to maintain a nominal 100 year design life based on proper maintenance. The NCC and Australian Standards are used as a guideline on the basis of structural durability. The building elements are designed to provide adequate performance throughout the design life period. Material selection and detailing of the structural elements are designed to comply with the minimum design period. See Section 9.9 Table 68 for further description of design life constraints.

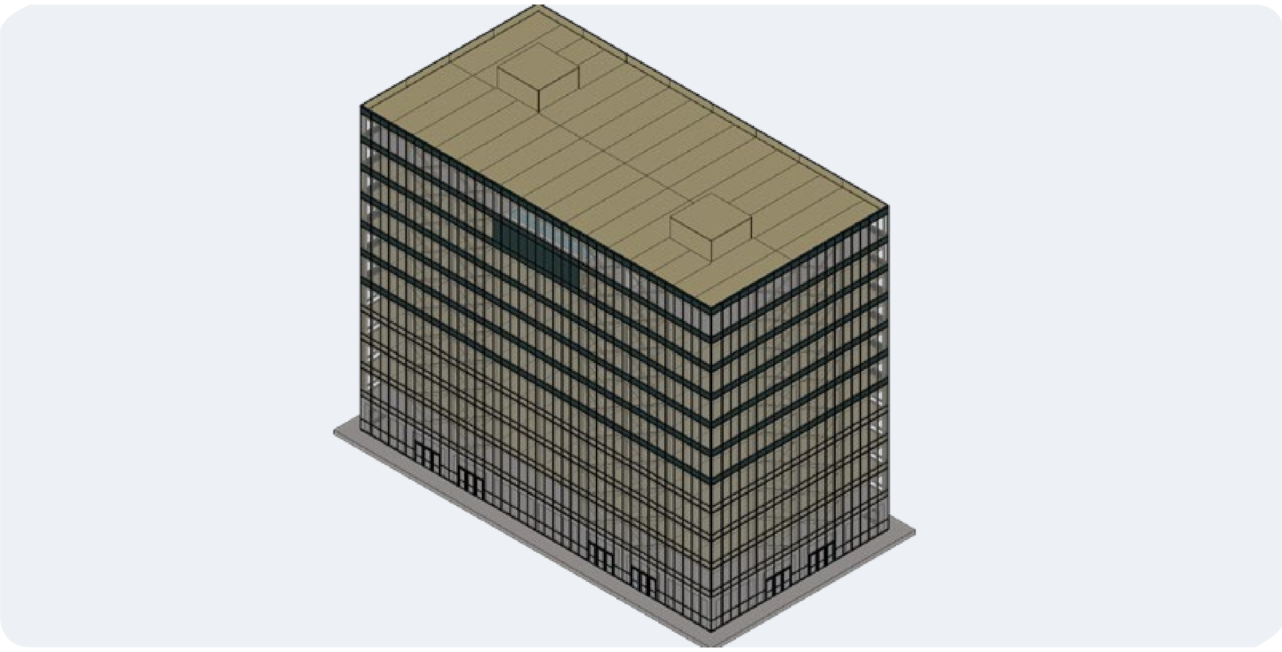


Figure 47: General conceptual design of Worked Example 2.

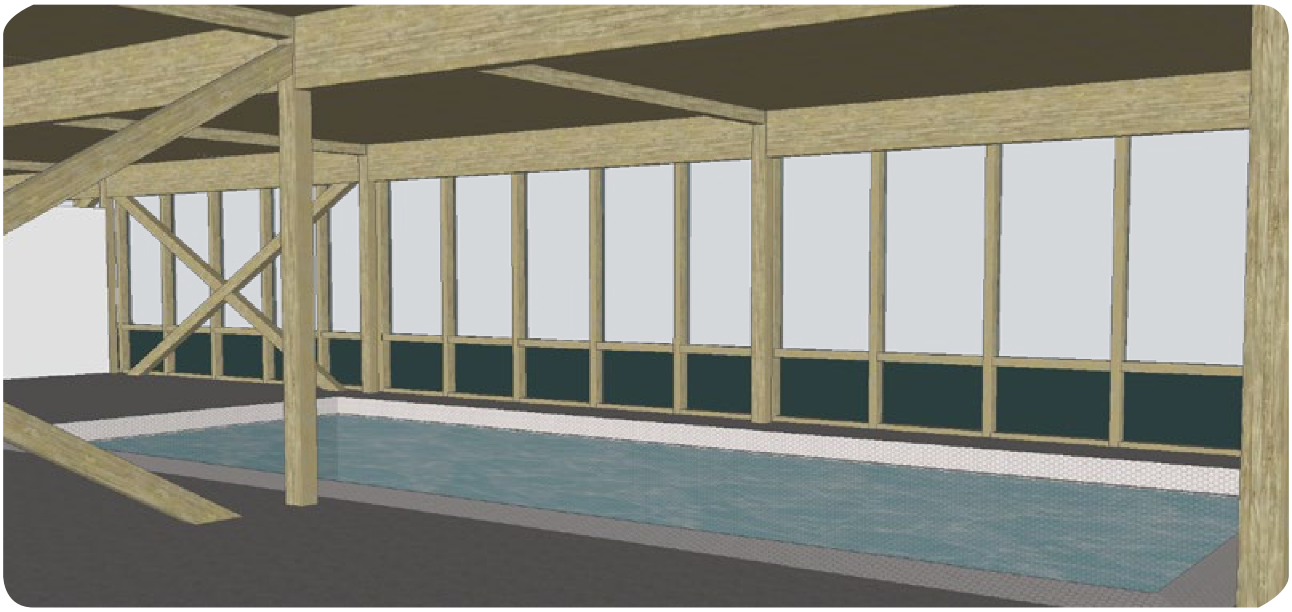


Figure 48: Conceptual design of inset pool on level ten

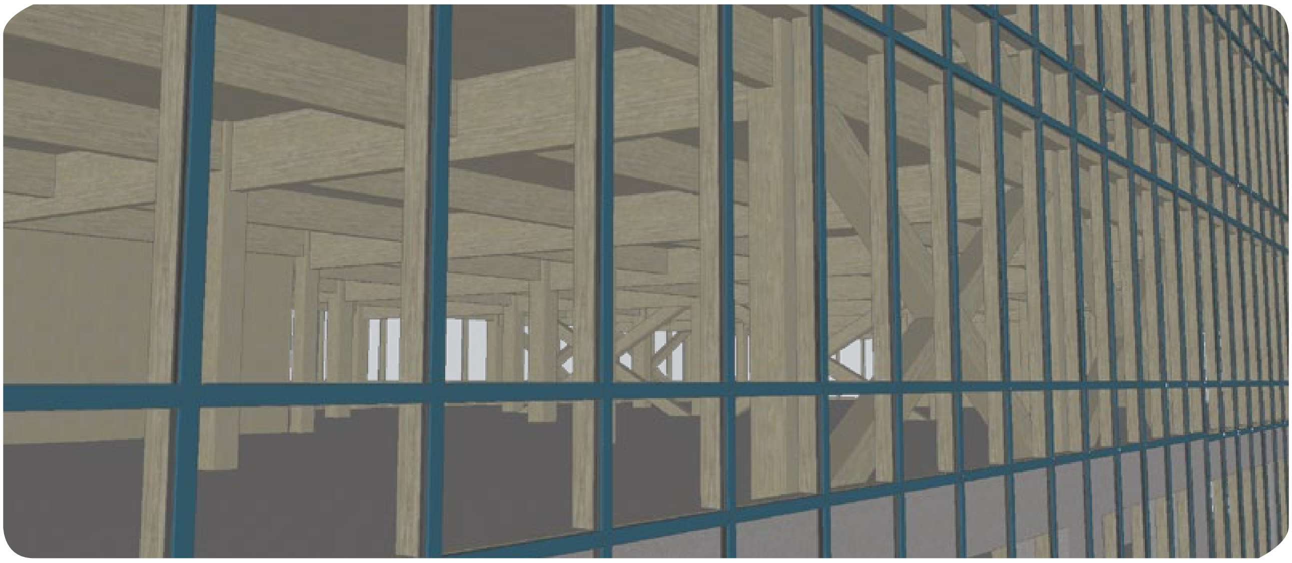


Figure 49: Conceptual exterior glass shell design.

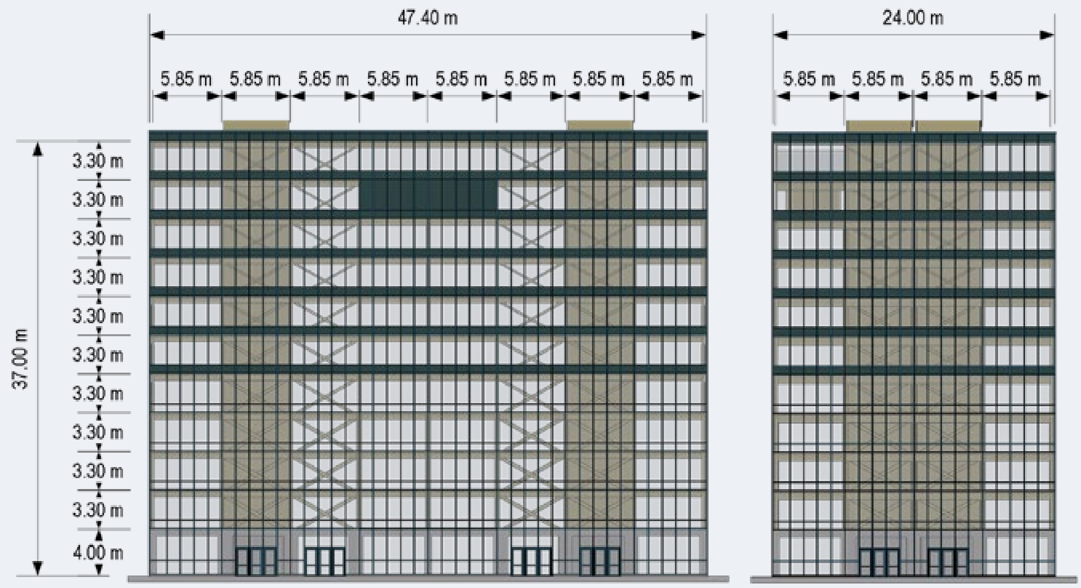


Figure 50: Conceptual design structure dimensions.

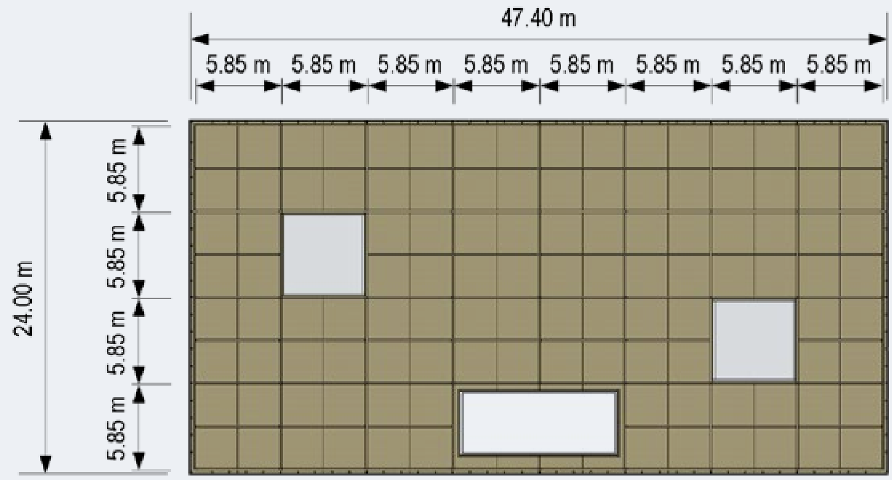


Figure 51: Conceptual design Level 10 framing dimensions.

Fire Rating

The fire-rating for each of the structural elements has not been calculated under this design example; however, designing Fire Resistance Levels is mandatory. A design one-hour fire-resistive calculation has been completed for a beam and column within the structure to illustrate the natural resistivity prior to implementing other fire resistance strategies. As outlined in NCC Volume 1 Section C1.1 provides the Fire Resistance Level times for each class of building. Fire resistance is incorporated into the design calculations in *AS 1720.4*, utilising the charring rate and effective depth of charring.

The computed example below incorporates a Fire Resistance Rating (FRR) following the National Building Code of Canada (NBCC) and PFC-6046 to illustrate the effects of fiber reinforced glulam. The Australian Code does not contemplate the use of reinforced elements, therefore the methodology from the PFC-6046 and NBCC was used. In order to directly compare the results of unreinforced versus reinforced, an identical calculation approach is required. Note, the fire resistance of high strength fiber reinforced elements greatly surpasses the unreinforced elements, this is shown in the design computation below.

In comparison to *AS 1720.4*, the FRL and FRR are not directly interchangeable factors. The FRL of a structure consists of a three part rating of structural adequacy, integrity and insulation; whereas the FRR provides a single rating. The FRR is similarly related to the Fire Resistance Period (FRP) of an element. Both the FRL and FRR calculation yield comparable results on large unreinforced dimension lumber; however, precaution should be taken when using small elements. The FRL approach cannot be conducted when using fiber reinforced elements, the appropriate code must be referenced when utilising these elements.

The target Fire Resistance Rating (FRR) is 1 hour or greater to allow for egress in the event of a fire. The FRR values for a beam and column that have been calculated have been calculated for both an unreinforced glulam and a reinforced glulam to show the advantage of utilising high strength fiber reinforced glulams in tall timber structures.

Mitigation strategies can be implemented to increase the fire resistance within the structure such as:

- Installing plasterboard/gypsum
- Implementation of a sprinkler system
- Utilising non-combustible materials (ie. Insulation)
- Design incorporating sacrificial timber
- Use of fire separation barriers (ie. Fire doors)
- Use of protected connectors (ie. Embedding, cladding)
- Treatment of the wood with fire retardant treatments and/or intumescent

Note, additional loading for increasing the fire resistance rating by the utilised plasterboard should be incorporated into design loads. The use of sacrificial timber is a methodology that accounts for fire occurrence by using larger elements than required for loading. The use of larger timber cross sections than required for load carrying capacity allows the extra wood to serve as a char barrier and slows down the charring rate in the net section required for load carrying capacity in the element, subsequently increasing the fire-resistance time.

As indicated in NCC Section C1.13 fire protected elements can be utilised where non-combustible materials are required in structures with Class 2, 3 or 5 provided the effective height of the structure is less than 25m. Given the suitability of the above improvement methods for fire resistance a local variance can be obtained for the 25m height restriction.

See Worked Example 1 for a more complete example of FRL design calculations.

Special Design Considerations

This structure is designed using high strength FiRP® Products (FiRP® refers to 'fibre reinforced plastics') to allow holes to be cut through structural members for services which increases height clearances within the structure as well as providing a particular aesthetic appeal. Using FiRP® glulam enables duct work and various other utilities to pass through the beams instead of underneath them, which saves money by reducing the overall building height without reducing the individual storey heights. Shear panels are used to accommodate concentrated forces at various locations. See the Figures 52 and 53 below.

Figure 52: Example of utilities running through high strength fibre reinforced glulam beams instead of underneath such as required in unreinforced glulam

Figure 53: Example illustration of shear panels utilised around services to provide for adequate net shear resistance in the glulam element

11.2.2 Summary of Design Criteria

- Dimension limitations: Height: 37m, Width 24m, Depth: 47.4m
- Structural Importance Level: 3
- Open concept design where possible
- Level ten contains an inset pool
- Floors are supported with columns on a 5.85m x 5.85m grid
- Floor height requirements are:
  - Ground Floor: 4m floor to top of level one
  - Level one - ten: 3.3m floor to floor with 105mm floor thickness between the beams.
- Building materials consist of:
  - Ground floor and level one: design material is concrete. This design is outside the scope of this sample.
  - Levels two - ten & Roof: design material is Cross Laminated Timber (CLT)
  - Horizontal beams throughout: design material is glue laminate beams (glulam)
  - Columns & Braces throughout: design material is glue laminate members (glulam)

11.2.3 Calculations

Determination of Wind Actions

For a multi-storey structure such as this, many of the controlling load cases are dependent on wind loading, so the structural design begins by determining the appropriate wind actions. This calculation is summarised below:

1. Use the *AS1170.0* Structural design actions – General principles. Section 3 – Annual Probability of Exceedance Table 3.1 provides the importance levels and Table 3.3 the appropriate annual probability exceedance. For importance level 3 the annual probability is 1:1000 for wind.

2. Calculate the wind speeds, pressures and actions per *AS1170.2:2011* Structural design actions - Part 2: Wind actions. Use Appendix F of *AS1170.2* if a flag is to be put on the top of the structure. The site variables assumed here are:

   a. Region A3 … Figure 3.1(A)

   b. Terrain Category 2 … 4.2.1

   This value was assumed based on a mid-level terrain with well-scattered obstructions

   c. V_{1000} = 46 m/s … Table 3.1

   Regional wind speed is the inverse of I/R therefore V_{1000} is derived

   d. Direction Multiplier: … Table 3.2

   i. M_d = 1.0 (NW)

   ii. M_d = 0.85 (SW)

   e. Terrain Height Multiplier : $M_{z,cat}$ = 1.15 … Table 4.1(A)

   This value is height dependent (where h=37m). Based on the multipliers for 30 and 40 metres, the terrain multiplier is interpolated between these values.

   f. Shielding Multiplier: M_s = 1.0 … Table 4.3

   Shielding multiplier assumed based on Clause 4.3.1

   g. Topographic Multiplier: M_t = 1.0 … 4.4

   Structure was assumed to be constructed on flat terrain

3. Determine the site wind speeds, $V_{sit\beta}$ using the variables assumed in 9b. and the equation:

   $V_{sit\beta} = V_r\, M_d\, (M_{z,cat}\, M_s\, M_t)$ … *Equation 1*

   $V_{sit\beta} = V_{500}\, M_d\, (M_{z,cat}\, M_s\, M_t)$

   $V_{sit\beta} = 46 * 1.0 * 1.15 * 1.0 * 1.0$ = 52.81 m/s … M_d=1.0 (NW)

   $V_{sit\beta} = 46 * 0.85 * 1.15 * 1.0 * 1.0$ = 44.97m/s … M_d=0.85 (SW)

4. Determine the design wind speed, $V_{des\vartheta}$ from the site wind speeds:

   $V_{des\vartheta}$ = 52.81 m/s (Controlling wind speed)

   As a conservative and simplified approach $V_{des,\vartheta} = V_{sit,\beta}$ …2.3

5. Determine the design wind pressures, calculated per section 2.4.1 and distributed forces using Equation 2. This calculation provides example for the wind pressure on the roof.

   $p = (0.5\ \rho_{air})\ [V_{des\vartheta}]^2\ C_{fig}\ C_{dyn}$ *… Equation 2.*

   a. ρ_{air} is the density of air = 1.2 kg/m^3

   b. Based on Clause 5.2(a) for enclosed buildings, internal pressures, external pressures and friction drag forces must be considered in both x and z directions. See below equations. C_{fig} is a combination of internal ($C_{p,i}$) and external ($C_{p,e}$) pressure coefficients. These apply to the structure in combinations that produce the highest loads on the elements.

$C_{fig,i} = C_{p,i}K_{c,I}$	Internal Pressures	…5.2 (1)
$C_{fig,e} = C_{p,e}K_aK_{c,e}K_lL_p$	External Pressures	…5.2 (2)
$C_{fig} = C_fK_aK_c$	Frictional Drag	…5.2 (3)

For instance, use $C_{p,i}$ from Table 5.1(A) for all walls equally permeable.

Use	$C_{p,i}$ = -0.3	
Use	$C_{p,e}$ = 0.8 for windward wall	… Table 5.2 (A)
	$C_{p,ez}$ = - 0.5 for leeward wall	… Table 5.2 (B)
	$C_{p,ex}$ = - 0.3 for leeward wall	… Table 5.2 (B)
	$C_{p,ez}$ = -0.65 for side walls[5]	… Table 5.2 (C)
	$C_{p,ex}$ = -0.5 for side walls[2]	… Table 5.2 (C)
Use	$K_{a\text{-}roof}$ =1.0	… Table 5.4
	$K_{a\text{-}}$ = 0.8 leeward and windward	… Table 5.4
	$K_{c,i}$ = 0.8	… Table 5.5
	$K_{c,e}$ = 0.8	… Table 5.5
	K_l = 1.0	… 5.4.4
	K_p = 1.0	… 5.4.5
	C_f = 0.0	… 5.5
	K_c = 1.0	… 5.4.3

$C_{fig,i}$	= -0.192	… 5.2 (1)
$C_{fig,e}$	= 0.51 Windward Wall[6]	… 5.2 (2)
C_{fig}	= 0.0	… 5.2 (3)

   c. $C_{dyn} = [1+2I_h[g_v^2B_s+(H_s+g_R^2SE_l)/\zeta]^{1/2}]/(1+2g_vL_h)$ … 6.2 (1)

   Where h = 37m

   I_h = 0.158 …Table 6.1

   g_v = 3.7

   ζ = 0.01

   H_s = 2 …6.2 (3)

   g_R = 2.327 …6.2 (4)

   Assuming an n_a=0.851

[4] Linear interpolation required for intermediate values of height z and terrain category

[5] Horizontal distance from windward edge, depth, is assumed to be between 1h to 2h

[5] $C_{fig,e}$ = -0.32 Leeward Wall, 0.51 Windward Wall, -0.52 Side Wall.

i. $$B_s = \frac{1}{1+\frac{\sqrt{0.26(h-s)^2+0.46b_{sh}^2}}{L_h}}$$ …6.2 (2)

At h = 37:

L_h = 117.89 …6.2 (3)

b_{sh} = 24

s = 37

B_s = 0.879

ii. $$S = \frac{1}{\left[1+\frac{3.5n_a h(1+g_v I_h)}{V_{des,\theta e}}\right]\left[1+\frac{4n_a b_{0h}(1+g_v I_h)}{V_{des,\theta}}\right]}$$ …6.2 (5)

S = 0.067

iii. $$E_t = \frac{\pi N}{\left(1+70.8N^2\right)^{5/6}}$$ …6.2 (6)

Where $N=n_a L_h [1+(g_v I_h)]/V_{des,\theta}$

N=3.01

E_t=0.0432

Therefore, C_{dyn} = 1.029

See *AS1170.2* Section 6 - Dynamic Response Factor, for a complete list of the variables and their respective equations.

Using *Equation 2* and all of the variables above, the design wind pressures are at h = 37m in the z-direction:

p =	-0.545	kPa	… using $C_{fig,i}$
p =	1.454	kPa Windward Wall	… using $C_{fig,e}$
p =	-0.909	kPa Leeward Wall	
p =	-1.477	kPa Side Wall	
p =	-2.120	kPa	… Max Roof Pressure

Determine the wind actions per Clause 2.5 of the *AS1170.2* design actions using the following equation:

$F = \Sigma(p_z A_z)$ … *Equation 3*

A_z = reference area, in square metres, at height z, upon which the pressure at that height (p_z) acts.

Specific wind actions were calculated for individual building stories (including the roof) in the z-direction and are detailed in Table 72. Figure 54 illustrates the wind loads applied to the structure on either face.

Table 72: Details of Wind Actions used in calculating the listed design demands.

Pressure z Vertical Projection (kN/m)								
	z	Vdesθ	pz, i	pz, e(z) W'ward	pz, e(z) L'ward	pz, e(z) Side	Pz, e + Pz, i	pz Drag
Roof	37	52.81	-0.545	1.454	-0.909	-1.477	-2.022	0
Level 10	33.7	52.20	-1.059	2.823	-1.765	-2.867	-3.926	0
Level 9	30.4	51.59	-1.028	2.741	-1.713	-2.784	-3.812	0
Level 8	27.1	50.99	-0.998	2.662	-1.664	-2.703	-3.702	0
Level 7	23.8	50.38	-0.969	2.585	-1.615	-2.625	-3.594	0
Level 6	20.5	49.77	-0.941	2.510	-1.569	-2.549	-3.490	0
Level 5	17.2	48.91	-0.904	2.411	-1.507	-2.449	-3.353	0
Level 4	13.9	46.28	-0.804	2.143	-1.339	-2.177	-2.980	0
Level 3	10.6	46.00	-0.791	2.110	-1.319	-2.143	-2.934	0
Level 2	7.3	41.86	-0.650	1.734	-1.084	-1.761	-2.411	0
Level 1	4	41.86	-0.717	1.913	-1.196	-1.943	-2.660	0

Roof Pressures with wind in the Z direction (kPa)				
	pz, e 0 to 0.5h max	pz, e 0 to 0.5h min	pz, e 0.5h to h max	pz, e 0.5h to h min
Roof	-2.120	-0.826	-1.294	-0.413

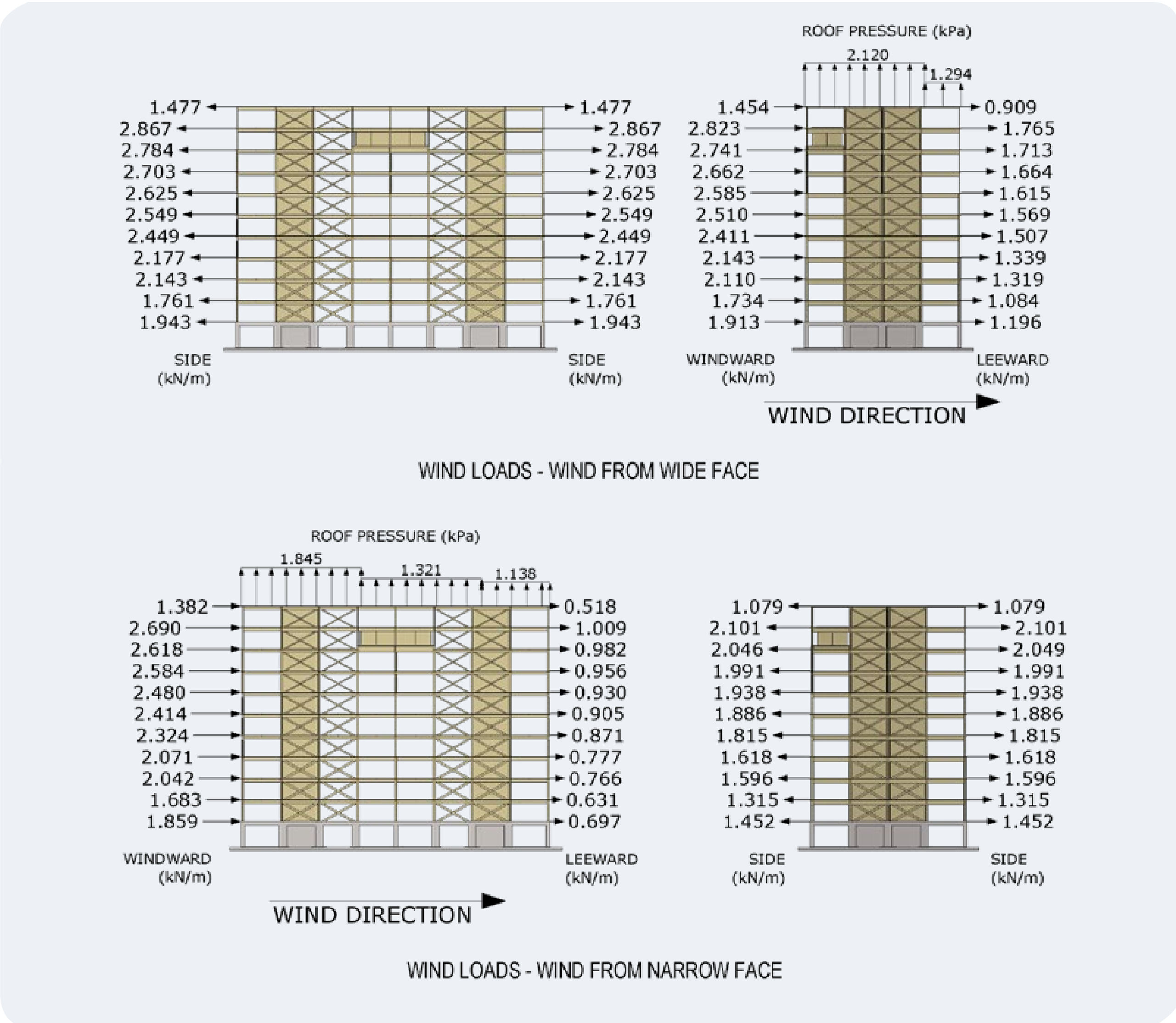


Figure 54: Illustration of the Wind Actions imposed on the structure.

Determination of Earthquake Actions

Earthquake loading has been considered for this example and applied to the structural elements. Detailing for the seismic stability system are calculated in accordance to *AS/NZS 1170.4*. This calculation is summarized below:

1. Use the *AS1170.4* Structural design actions – Earthquake Actions. Section 3 – Annual Probability of Exceedance Table 3.1 provides the importance levels and Table 3.3 the appropriate annual probability exceedance. For importance level 3 the annual probability is 1:1000 for earthquake.

6. To determine which modal analysis is required, the following design parameters must be defined:

   a. Probability Factor: $k_p = 1.3$ … Table 3.1

   b. Hazard factor: z = 0.09 … Table 3.2

   Assumed location: Goulburn

   c. Sub-soil class: C_e … 4.2.3

   Assumed mid-range class to be conservative

   d. Earthquake Design Category: II … Table 2.1

   i. $K_p z = 0.117$

   ii. H< 50m

7. Utilising the above values in conjunction with Figure 2.2, Clause 5.2, 5.4 and Static Analysis calculations are required.

8. Section 6.1 follows the Equivalent Static Analysis procedure. Utilising Clause 6.2.1 the earthquake base shear is calculated.

   $V = ([k_p Z C_h(T_1) S_p]/\mu) W_t$ … *Equation 4*

   a. $C_h T_1$= value of spectral shape factor for the fundamental natural period. Subsequently determine the structural performance factor and ductility factor.

   i. $C_h(T_1) = 1.25$ …Table 6.4

   Using the Equations for Spectra for $0.1 < T_1 \leq 1.5$ (after T_1 is determined in b)

   ii. $\mu = 2$ …Table 6.5 (A)

   For “other” wood timber structure

   iii. $S_p = 0.77$ …Table 6.5 (A)

   For “other” wood timber structure

   b. Calculate the natural period

   $T_1 = 1.25 k_t h_n^{0.75}$ …6.2 (7)

   Use k_t=0.05 …6.2.3

   h_n=37 …6.2.3

   T_1=0.9376

   c. Determine the seismic weight for each level in the structure as per Clause 6.2.2- Gravity Load. Below is an example calculation for Level 3.

   $W_i = \sum G_i + \sum \Psi_c Q_i$ …6.2 (6)

   i. $\Psi_c = 0.3$ …6.2.2

   For all other applications

   W_i = (465kN+825kN)+0.3(5kPa*(5.85^2x32bays))

   W_i = 2932.7kN

A tabulation of the structure self-weight at various levels is in Table 73.

Table 73: Seismic Wight (kN) per Building Level

Building Levels	Self wt. (kN)	Dead Load (kN)	Water (kN)	Live Load (kPa)	Seismic Weight (kN)
Level 1 - Concrete	11,148.5	825	-	5	13,616
Level 2 to 3	465	825	-	5	2,932.7
Level 4 to 8	465	825	-	3	2,275.6
Level 9 & 10	505	1398	1588	3	4,476.6
Roof	222	-	-	-	222

Where the total weight of the structure, W_t = 40,035kN

Using *Equation 4* and the variables above, the design base shear is calculated as follows:

V = 0.117*1.25*0.77/2*40,035kN

V = 2,254.2 kN

9. The vertical distribution of horizontal forces acting on the structure are calculated as per Clause 6.3 and the following equations.

$$F_i = k_{F,i}V \qquad \text{…6.3 (1)}$$

$$= \frac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_j h_j^k\right)}\left[k_p Z C_h(T_1)\frac{S_p}{\mu}\right] \qquad \text{…6.3 (2)}$$

i. k = 1.2188

Where k is linearly interpolated between 1.0 and 2.0 for $0.5 < T_1 < 2.5$ …6.3 (2)

ii. $\sum W_j h_j^k$ = is the sum of the product between the seismic weight and height of level i throughout the entire structure.

Utilising the above equations, the design force for each level can be computed. Table 74 below presents the distribution of seismic forces on the structure. For further information, and a complete list of variables, see *AS1170.4* Section 6.3.

Table 74: Vertical Distribution of Horizontal Forces

Level	F_i (kN)	$W_i h_i^k$	W_i*(kN)*	h_i*(m)*
Roof (11)	34.8	18,100.9	222.0	37
Level 10	626.9	325,721.9	4,476.6	33.7
Level 9	552.9	287,274.7	4,476.6	30.4
Level 8	244.3	126,946.8	2,275.6	27.1
Level 7	208.6	108,365.2	2,275.6	23.8
Level 6	173.9	90,340.5	2,275.6	20.5
Level 5	140.4	72,942.0	2,275.6	17.2
Level 4	108.3	56,262.7	2,275.6	13.9
Level 3	100.3	52,110.2	2,932.7	10.6
Level 2	63.7	33,074.7	2,932.7	7.3

10. Eccentricities are applied to the predetermined earthquake loadings summarized in Table 3.0 to calculate the torsional effects on the structure. As stated in Clause 6.6 eccentricities are applied at +/- 0.1b from centre. The forces utilised for this eccentricity calculation are summarized in Figure 55.

11. Determine the P-Delta Effects of the structure in accordance to Clause 6.7.2 and 6.7.3. Using the following equation

$$\theta = d_{st} \sum_{j=i}^{n} W_j / [h_{si} \mu \sum_{j=i}^{n} F_j]$$ *…Equation 5*

a. Based on the information in Clause 6.7.3.1 P-Delta effects require calculation. Clause 6.7.3.2 requires the static forces, moments and storey drift to be scaled by the following equation:

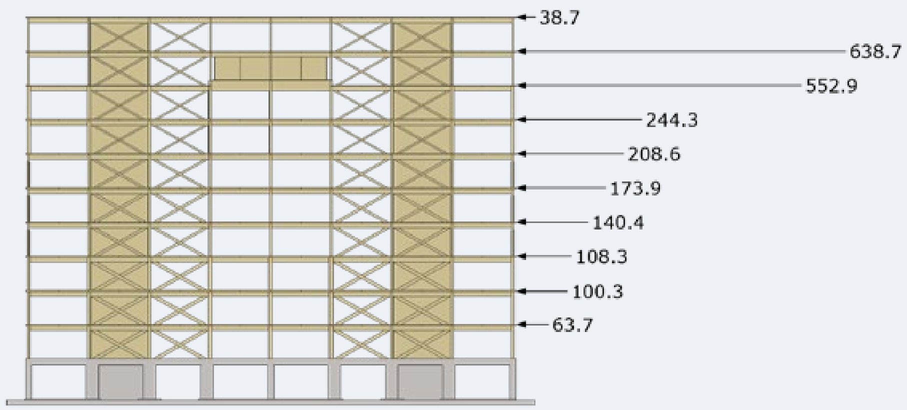
$$Scaling\ Factor = \frac{0.9}{(1-\theta)} \geq 1.0$$ *…Equation 6*

Table 75 provides a summary of the scaled P-Delta forces in the structure.

Table 75: P-Delta Effects

Level	dst(x)	dst(z)	u (x)	u (z)	(0.9/(1 – θ))
Roof (11)	5.92	165.94	0.007	0.189	1.109
Level 10	7.44	91.96	0.009	0.117	1.019
Level 9	8.22	32.72	0.011	0.044	Ok
Level 8	7.92	32.96	0.011	0.046	Ok
Level 7	6.62	16.45	0.010	0.024	Ok
Level 6	6.10	18.46	0.009	0.029	Ok
Level 5	5.09	18.33	0.008	0.030	Ok
Level 4	4.06	20.26	0.007	0.036	Ok
Level 3	2.83	9.38	0.005	0.018	Ok
Level 2	2.13	10.60	0.004	0.022	Ok

Therefore, F_{11} and F_{10} must be multiplied by the scaling factor to account for the P-Delta effects on the structure.



SEISMIC LOADS

Design of Individual Elements

The calculations below describe the design approach for the various elements within the structure. Various floor beams, cross laminated timber floor and roof panels, columns and braces are designed in this section. Note, some demands have been conservatively designed from what is prescribed in *AS 1720.1*. For example, the shear is calculated at a distance d from the support, whereas Clause 3.2.5 allows a distance of 1.5d. This provides a slightly higher factor of safety within the element and exceedingly satisfies the guideline set in the code.

Design a Third Floor 5.85m Floor Beam

The floor design is based on CLT floor panels supported by glulam beams. Using *AS1720.1*, a design capacity in bending (*M*) and shear (*V*) can be calculated and compared to a demand (*M** and *V**) determined using the variables listed below.

The controlling load combination for ultimate limit states used to calculate the strength and deflection demand was 1.2G + 1.5Q. The main floor beams are made up of simply supported glulam beams spanning 5.85m with a tributary width of 2.925m. A visual representation of the member selected is shown in Figure 56, displaying the location of the member in the structure. The third floor has been designed for 5kPa office storage space load, 0.5kPa partition load and a 0.1kPa superimposed dead load for miscellaneous floor finishes. Based on the calculated demands, an appropriate member with sufficient capacity was selected. The design procedure is as follows:

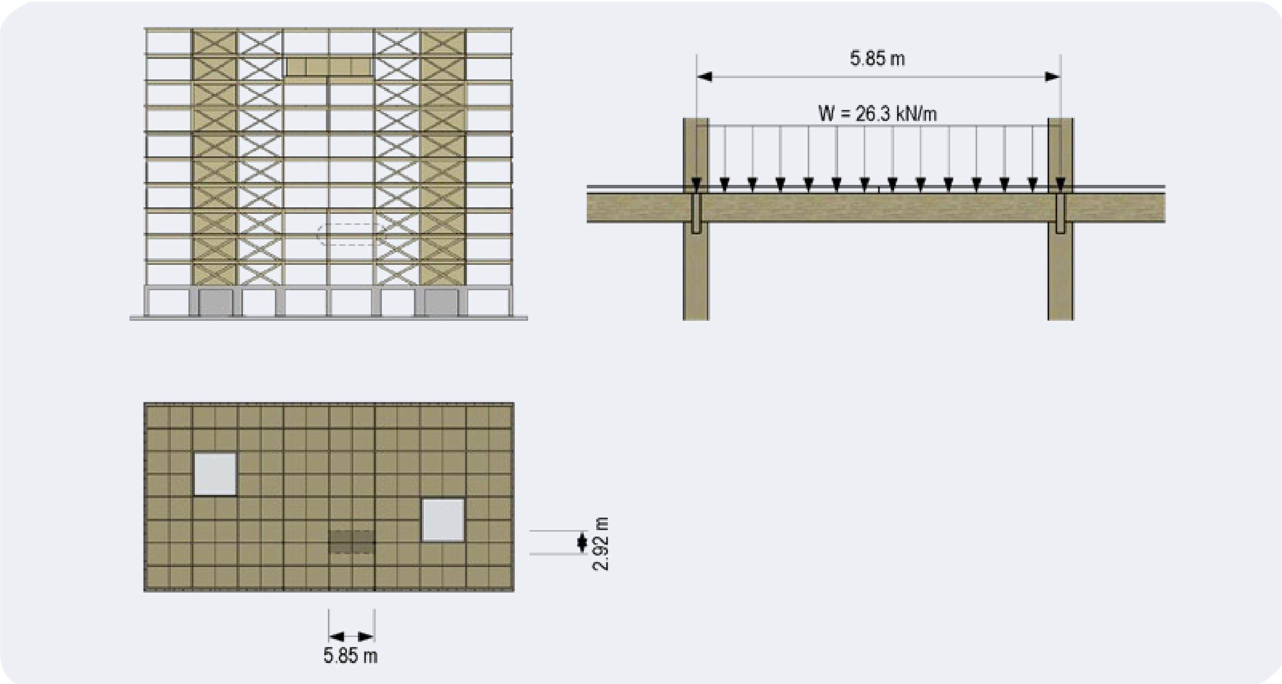


Figure 56: Structural location and configuration of the glulam floor beam in this example calculation.

Selected Glulam Beam dimensions: 130 x 457 mm GL18. b = 130mm, d = 457mm.

a.	Uniformly Distributed Loads (UDL):			
	Dead load	= .1 kPa * trib. width	= 0.29 kN/m	
	Self-Weight	= (density*b*d*g)/1000^3	= 0.33 kN/m	… g = 9.81m/s^2
		Where density = 560 kg/m^3		
	Deck Weight	= .53 kPa * trib. width	= 1.55 kN/m	
	Partition Load	= .5 kPa * trib. width	= 1.46 kN/m	
	Live Load	= 5 kPa * trib. width	= 14.6 kN/m	
	Q		= 14.6 kN/m	
	G		= 3.63 kN/m	
b.	Load Combinations from *AS1170.0*			
	The controlling Strength Limit State		= 1.2G + 1.5Q	… 4.2.2 (b)
	$SLS_{(UDL)}$		= 26.3 kN/m	Factored Load (W)

c. Calculate the beam's Demand:

M^*	$= (W^*L^2)/8$	= 112.5 kN-m	
V^*	$= W^*(L/2 - d)$	= 64.9 kN	
Live Load Deflection: $(5^*W^*L^4)/384^*E^*I$		= 0.011 m	
		= 11.64 mm	
			… $E = 18500^*10^3$ kPa
			… $I = 10.35^*10^{-4}$ m^4

d. Calculate the beam's Capacity:

Bending Strength:

$M_d \geq M^*$ … 3.2 (1)

where

$M_d = \phi\ k_1\ k_4\ k_6\ k_9\ k_{12}\ f_b'\ Z$ … 3.2 (2)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
k_{12}	= 1	… 3.2.4
Lay	= 500mm	
r	= 0.834	
ρb	= 0.827	
$S1 = 1.25^*(d/b)^*(Lay/d)^{0.5}$	= 4.597	
ρbS1	= 3.804	
f_b' = GL18	= 45000 kPa	… 7.3.1
Z	= 0.02797 m^3	

M_d = 129.1 kN-m **>** **M^* = 112.5 kN-m** **OK**

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\ k_1\ k_4\ k_6\ f_s'\ A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
$A_s = (2/3)\ (b^*d)$	= 0.0396 m^2	… 3.2.5

V_d = 134.7 kN **>** **V^* = 64.9 kN** **OK**

Deflection Limit:

Using L/240, an allowable capacity limit of 24.38mm is calculated, where L = 5,850mm. The aforementioned deflection demand of **11.64mm** is under this allowable limit.

Design a Third Floor Cross Laminated Timber Floor Panel

The floor panels throughout the building are Cross Laminated Timber (CLT) spanning 2.925m with a tributary width of 2.925m and supported by glulam beams. The third floor has been designed for 5kPa office storage space load, 0.5kPa partition load and a 0.1kPa superimposed dead load for miscellaneous floor finishes. Figure 57 displays the location of the member within the structure. Based on the calculated demands, an appropriate member with sufficient capacity was selected. The design procedure is as follows:

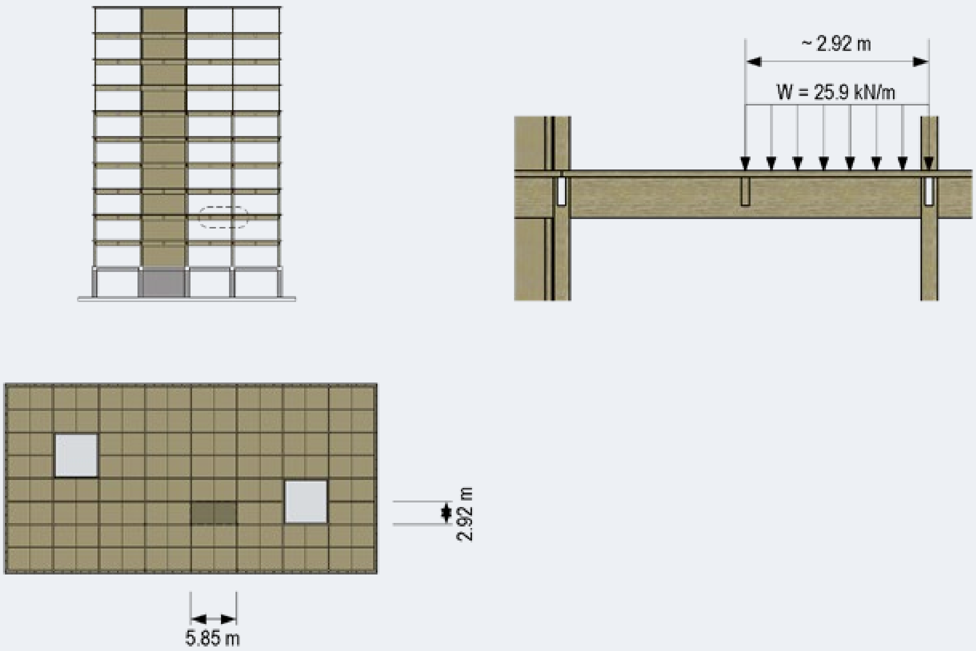


Figure 57: Structural location and configuration of the CLT floor panel.

Selected CLT Panel dimensions: 2925 x 105 mm. b = 2925mm, d = 105mm. Note the span to depth ration of 1:28 is acceptable in this case versus the typical limit of 1:25 due to the governing load criteria.

a. Uniformly Distributed Loads (UDL):

Dead load	= .1 kPa * trib. width	= 0.29 kN/m
Self-Weight	= .53 kPa * trib. width	= 1.55 kN/m
Partition Load	= .5 kPa * trib. width	= 1.46 kN/m
Live Load	= 5 kPa * trib. width	= 14.6 kN/m
Q		= 14.6 kN/m
G		= 3.3 kN/m

b. Load Combinations from *AS1170.0*

The controlling Strength Limit State	= 1.2G + 1.5Q	… 4.2.2 (b)
$SLS_{(UDL)}$	= 25.9 kN/m	… Factored Load (W)

c. Calculate the panel's Demand:

M* = $(W*L^2)/8$	= 27.7 kN-m	
V* = W*(L/2-d)	= 35.2 kN	
Live Load Deflection Limit: $(5*W*L^4)/384*E*I$	= 0.006 m	
	= 6.17 mm	
		… $E = 8000*10^3$ kPa
		… $I = 2.822*10^{-4}$ m^4

d. Calculate the panel's Capacity:

Bending Strength:

$M_d \geq M^*$			… 3.2 (1)

where

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f_b'\, Z$			… 3.2 (2)
ϕ		= 0.85	… 2.3
k_1		= 0.8	… 2.4.1.1
k_4 (EMC = 15)		= 1	… 2.4.2.1
k_6		= 1	… 2.4.3
k_9		= 1	… 7.4.3
k_{12}		= 1	… 3.2.4
Lay		= 2925mm	
r		= 0.45	
ρb		= 0.681	
S1 = 1.25*(d/b)*$(Lay/d)_{0.5}$		= 0.237	
ρbS1		= 0.161	
f_b'		= 12000 kPa	
Z		= 0.00368 m^3	
M_d = 43.8 kN-m	**>**	**M* = 27.7 kN-m**	**OK**

Shear Strength:

$V_d \geq V^*$			… 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$			… 3.2 (14)
ϕ		= 0.85	… 2.3
k_1		= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)		= 1	… 2.4.2.1
k_6		= 1	… 2.4.3
f_s'		= 3800 kPa	
A_s = (2/3) (b*d)		= 0.2048 m^2	… 3.2.5
V_d = 529.1 kN	**>**	**V* = 35.2 kN**	**OK**

Deflection Limit:

Using L/240, an allowable capacity limit of 12.19mm is calculated, where L = 2,925mm. The aforementioned deflection demand of **6.17mm** is under this allowable limit.

Design an 9th Floor 5.85m Floor Beam – Pool Support

Glulam floor beams support the swimming pool on level ten and are designed for 15kPa live load and 9kPa dead load. The floor is made up of simply supported glulam beams spanning 5.85m with a tributary width of 4.3875m. Floor joist spanning between these beams are placed at 1m on centre. Figure 58 displays the location of the member within the structure. Based on the calculated demands, an appropriate member with sufficient capacity was selected. The design procedure is as follows:

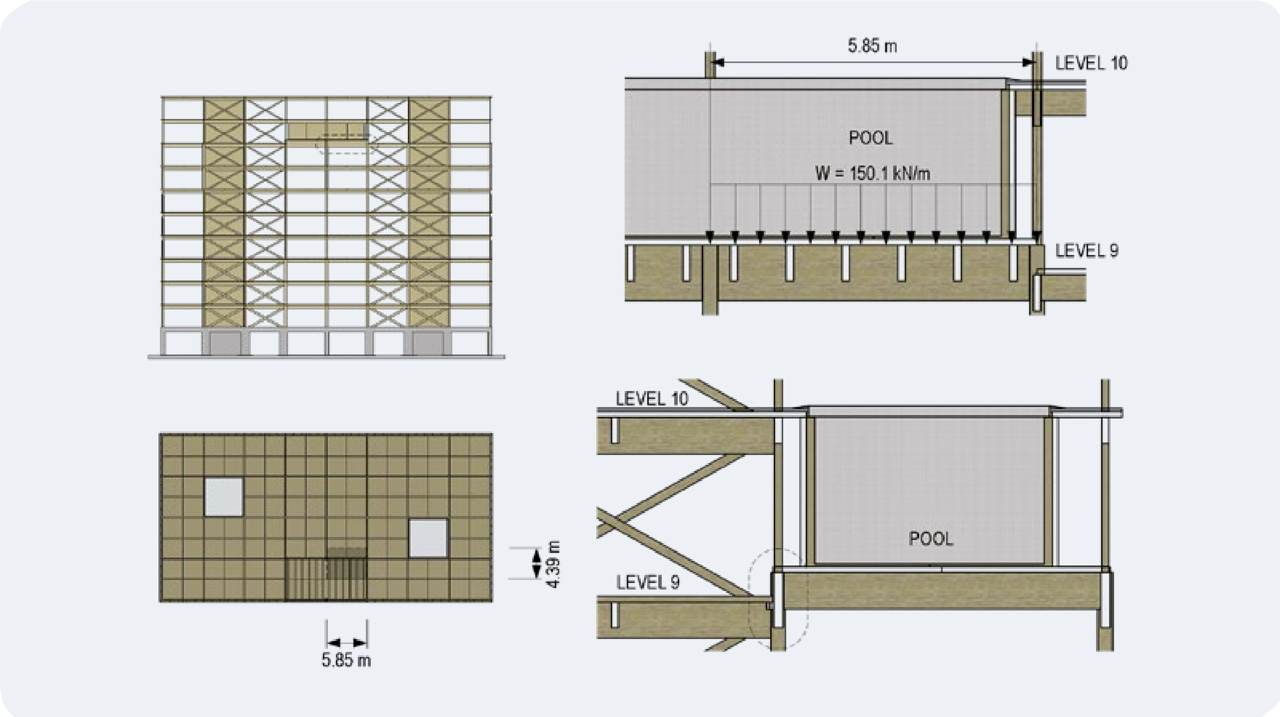


Figure 58: Structural location and configuration of the glulam floor beam supporting the pool.

Selected Glulam Beam dimensions: 171 x 991 mm GL18. b = 171mm, d = 991mm.

a. Uniformly Distributed Loads (UDL):

Dead load	= 9 kPa * trib. width	= 39.5 kN/m	
Self-Weight	= (density*b*d*g)/1000^3	= 0.93 kN/m	… g = 9.81m/s^2
	Where density = 560 kg/m^3		
Deck Weight	= .53 kPa * trib. width	= 2.33 kN/m	
Live Load	= 15 kPa * trib. Width	= 65.8 kN/m	
Q		= 65.8 kN/m	
G		= 42.8 kN/m	

b. Load Combinations from *AS1170.0*

The controlling Strength Limit State	= 1.2G + 1.5Q	… 4.2.2 (b)
$SLS_{(UDL)}$	= 150.1 kN/m	… Factored Load (W)

c. Calculate the beam's Demand:

M* = (W*L^2)/8	= 641.9 kN-m	
V* = W*(L/2 - d)	= 290.3 kN	
Live Load Deflection: (5*W*L^4)/384*E*I	= 0.0039m	
	= 3.92 mm	
		… E = 18500*10^3 kPa
		… I = 138.5*10^{-4} m^4

d. Calculate the panel's Capacity:

Bending Strength:

$M_d \geq M^*$			… 3.2 (1)

where

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f_b'\, Z$			… 3.2 (2)
ϕ		= 0.85	… 2.3
k_1		= 0.8	… 2.4.1.1
k_4 (EMC = 15)		= 1	… 2.4.2.1
k_6		= 1	… 2.4.3
k_9		= 1	… 7.4.3
k_{12}		= 1	… 3.2.4
Lay		= 500mm	
r		= 0.658	
ρb		= 0.839	
$S1 = 1.25*(d/b)*(Lay/d)^{0.5}$		= 5.145	
ρbS1		= 4.319	
f_b' = GL18		= 45000 kPa	
Z		= 0.02797 m^3	
M_d = 701.0 kN-m	**>**	**M* = 641.9 kN-m**	**OK**

Shear Strength:

$V_d \geq V^*$			… 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$			… 3.2 (14)
ϕ		= 0.85	… 2.3
k_1		= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)		= 1	… 2.4.2.1
k_6		= 1	… 2.4.3
f_s' = GL18		= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)		= 0.113 m^2	… 3.2.5
V_d = 384.0 kN	**>**	**V* = 290.3 kN**	**OK**

Deflection Limit:

Using L/240, an allowable capacity limit of 24.38mm is calculated, where L = 5,850mm. The aforementioned deflection demand of **3.92mm** is under this allowable limit.

Design a 5.85m Roof Beam

The roof design is based on CLT roof panels covered with a waterproof membrane and supported by glulam beams. Using *AS1720.1* a design capacity in bending (M_d) and shear (V_d) can be calculated and compared to a known demand (*M** and *V**) determined using the variables listed below.

The controlling load combination for ultimate limit states used to calculate the strength and deflection demand was 1.2G + 1.5Q. The roof beams are made up of simply supported glulam beams spanning 5.85m with a tributary width of 5.85m. The structural location of this element is depicted in Figure 59. These beams are designed for the loads listed below. Based on the calculated demands, an appropriate member with sufficient capacity was selected. The design procedure is as follows:

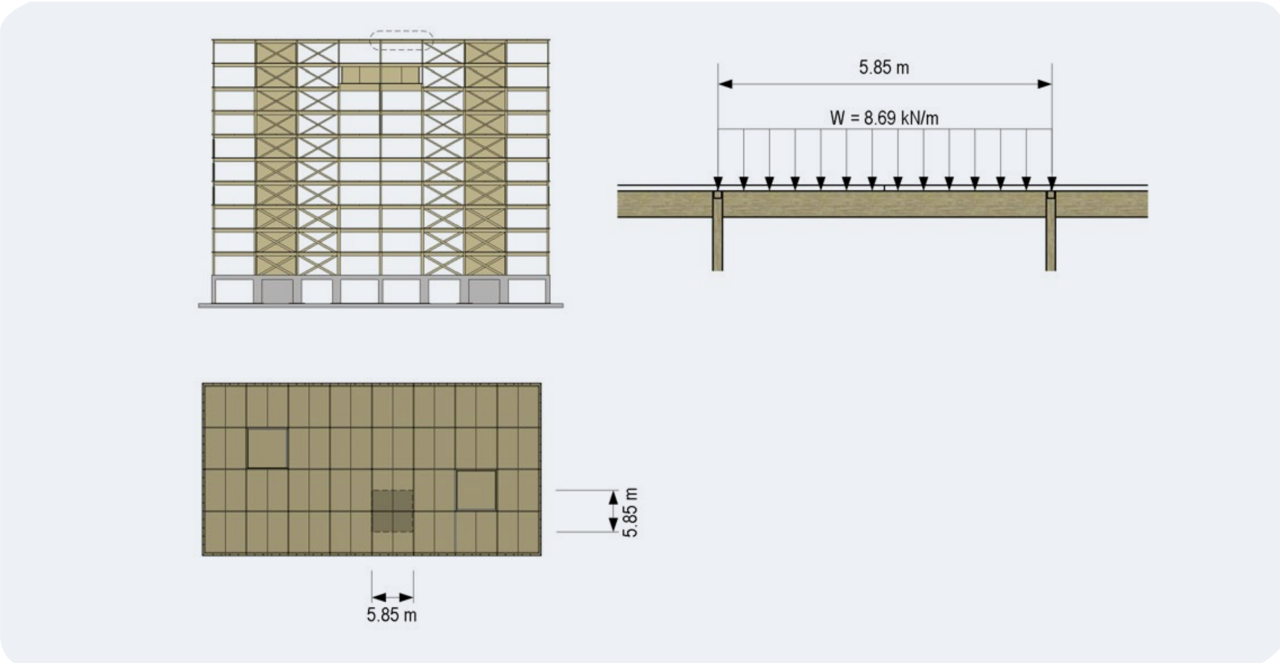


Figure 59: Structural location and configuration of the glulam beam supporting the roof.

Selected Glulam Beam dimensions: 130 x 385 mm GL18. b = 130mm, d = 385mm.

a.	Uniformly Distributed Loads (UDL):			
	Dead load	= .1 kPa * trib. width	= 0.59 kN/m	
	Self-Weight	= (density*b*d*g)/1000^3	= 0.27 kN/m	… g = 9.81m/s^2
		Where density = 560 kg/m^3		
	Roof Dead Load	= .78 kPa * trib. width	= 4.56 kN/m	
	Roof Live Load	= .25 kPa * trib. width	= 1.46 kN/m	
	Wind Reversal	= -2.12 kPa * trib. width	= -12.4 kN/m	
	Q		= 1.46 kN/m	
	G		= 5.42 kN/m	
b.	Load Combinations from *AS1170.0*			
c.	The controlling Strength Limit State		= 1.2G + 1.5Q	… 4.2.2 (b)
	$SLS_{(UDL)}$		= 8.69 kN/m	… Factored Load (W)
d.	Calculate the beam's Demand:			
	M*	= (W*L^2)/8	= 37.2 kN-m	
	V*	= W*(L/2 -d)	= 22.1 kN	
	Deflection Limit: (5*W*L^4)/384*E*I		= 0.0026m	
			= 2.58 mm	
				… E = 14000*10^3 kPa
				… I = 6.18*10^{-4} m^4

e. Calculate the panel's Capacity:

Bending Strength:

$M_d \geq M^*$		… 3.2 (1)

where

$M_d = \phi\ k_1\ k_4\ k_6\ k_9\ k_{12}\ f_b'\ Z$		… 3.2 (2)
ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
k_{12}	= 0.77	… 3.2.4
Lay	= 5850 mm	
r	= 0.252	
ρb	= 1.017	
S1 = 1.25*(d/b)*(Lay/d)$^{0.5}$	= 14.68	
ρbS1	= 14.43	
f_b' = GL18	= 45000 kPa	
Z	= 0.00321 m^3	

M_d = 73.4 kN-m **>** **M* = 37.2 kN-m** **OK**

Shear Strength:

$V_d \geq V^*$		… 3.2 (13)

where

$V_d = \phi\ k_1\ k_4\ k_6\ f_b'\ A_s$		… 3.2 (14)
ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 5000 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0334 m^2	… 3.2.5

V_d = 113.4 kN **>** **V* = 22.1 kN** **OK**

Deflection Limit:

Using L/240, an allowable capacity limit of 24.38mm is calculated, where L = 5,850mm. The aforementioned deflection demand of **2.58mm** is under this allowable limit.

Design a Cross Laminated Timber (CLT) Roof Panel

The controlling load combination for ultimate limit states used to calculate the strength and deflection demand was 1.2G + 1.5Q. The roof is made up of CLT panels, covered with a waterproof membrane, spanning 5.85m with a tributary width of 2.925m. These panels are designed for the loads listed below. Figure 60 below depicts the structural location of this element. Based on the calculated demands, an appropriate member with sufficient capacity was selected. The design procedure is as follows:

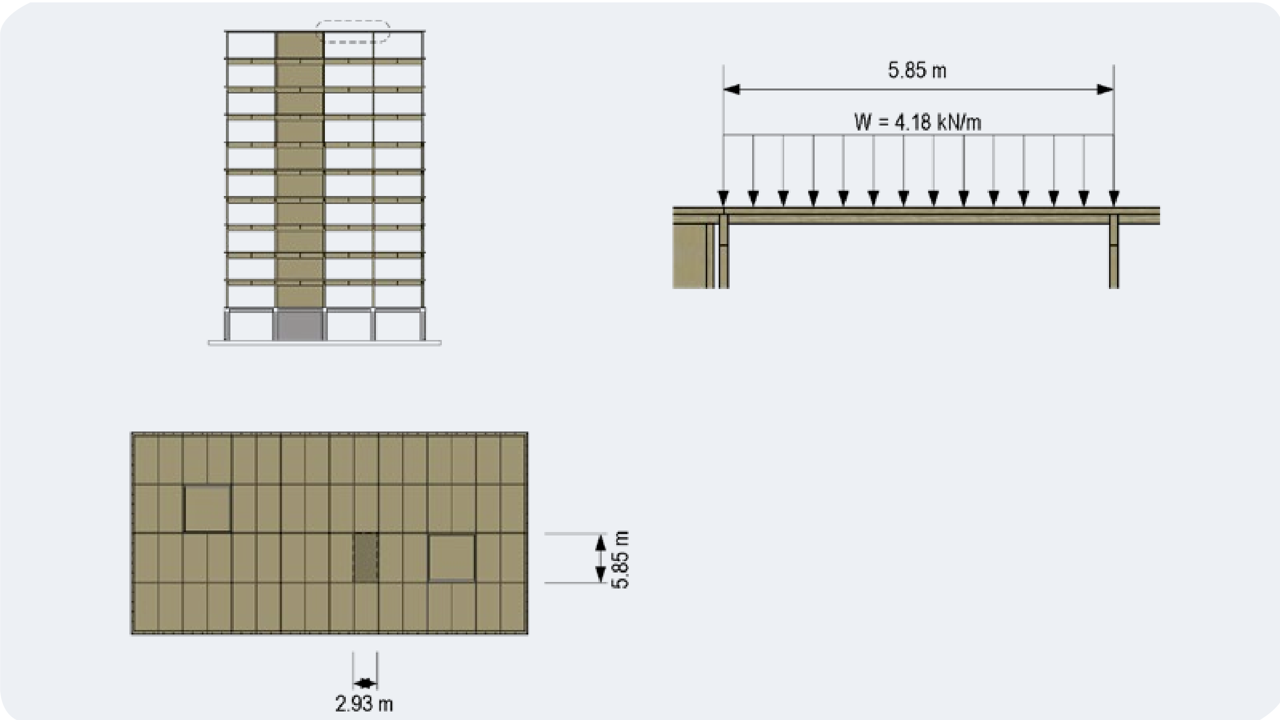


Figure 60: Structural location and configuration of the CLT roof panel.

Selected CLT Panel dimensions: 2925 x 105 mm. b = 2925mm, d = 105mm.

a.	Uniformly Distributed Loads (UDL):			
	Dead load	= .1 kPa * trib. width	= 0.29 kN/m	
	Self-Weight	= .53 kPa * trib. width	= 1.55 kN/m	
	Roofing	= .25 kPa* trib. width	= 0.73 kN/m	
	Roof Live Load	= .25 kPa * trib. width	= 0.73 kN/m	
	Wind Reversal	= -2.12 kPa * trib. width	= -6.2 kN/m	
	Q		= 0.73 kN/m	
	G		= 2.57 kN/m	
b.	Load Combinations from *AS1170.0*			
	The controlling Strength Limit State		= 1.2G + 1.5Q	… 4.2.2 (b)
	$SLS_{(UDL)}$		= 4.18 kN/m	… Factored Load (W)
c.	Calculate the beam's Demand:			
	M*	= $(W*L^2)/8$	= 17.9 kN-m	
	V*	= W*(L/2 -d)	= 11.8 kN	
	Deflection Limit:	$(5*W*L^4)/384*E*I$	= 0.005 m	
			= 4.94 mm	
				… $E = 8000*10^3$ kPa
				… $I = 2.822*10^{-4}$ m^4

d. Calculate the panel's Capacity:

Bending Strength:

$M_d \geq M^*$ … 3.2 (1)

where

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f_b'\, Z$ … 3.2 (2)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
k_9	= 1	… 7.4.3
k_{12}	= 0.77	… 3.2.4
Lay	= 5850mm	
r	= 0.262	
ρb	= 0.704	
$S1 = 1.25*(d/b)*(Lay/d)^{0.5}$	= 0.335	
ρbS1	= 0.236	
f_b'	= 12000 kPa	
Z	= 0.00538 m^3	

M_d = 43.8 kN-m > M^* = 17.9 kN-m OK

Shear Strength:

$V_d \geq V^*$ … 3.2 (13)

where

$V_d = \phi\, k_1\, k_4\, k_6\, f_s'\, A_s$ … 3.2 (14)

ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC ≤ 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_s' = GL18	= 3800 kPa	… 7.3.1
A_s = (2/3) (b*d)	= 0.205 m^2	… 3.2.5

V_d = 529 kN > V^* = 11.8 kN OK

Deflection Limit:

Using L/240, an allowable capacity limit of 24.38mm is calculated, where L = 5,850mm. The aforementioned deflection demand of **4.94mm** is under this allowable limit.

Design of a 7th Floor 3.3m Column

This member is designed in accordance to *AS 1170.0*-Section 3.3 – Column Design. This 3.3m column supports the glulam beams above, which transfer the occupancy load of 3kPa for the residential sector of the building to the column.

Using the *AS1720.1* design standards, the design capacity can be compared to a known demand within the structure. The demand has been determined using computer modelling software in conjunction with the variables listed below. The controlling case is determined as N* = 1214.3 kN. Utilising this demand, the associated member capacity is determined.

Selected Glulam Column dimensions: 260 x 260 mm GL17. b_1 = 260mm, b_2 = 260mm, tributary width= 2.925.

a. Uniformly Distributed Loads (UDL):

Dead load	= .62 kPa * trib.width	= 1.81 kN/m	
Q	= 3 kPa * trib.width	= 8.78 kN/m	
G	= (density $*b_1*b_2*g)/1000^3$	= 0.38 kN/m	… g = 9.81m/s^2
	Where density = 560 kg/m^3		

b. Load Combinations from *AS1170.0*

The controlling Strength Limit State = 1.2G + 1.5Q … 4.2.2 (b)

c. The demand of the column is based on the structural output. The maximum loading conditions on the structural element is **1214.3 kN (C), 0 kN (T), and 0 kN-m (M), 0kN (V).**
The governing failure mode is compression.

d. Compression Column Capacity- Parallel to the Grain:

$N_{d,c} \geq N^*_c$ … 3.3 (1)

where

$N_{d,c} = \phi\, k_1\, k_4\, k_6\, k_{12}\, f_c'\, A_c$		… 3.3 (2)
ϕ (Category 2)	= 0.85	… 2.3
$k_{1\text{-timber}}$ (5 month duration)	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
g_{13} (Flat ends)	= 0.7	…Table 3.2
L	= 3300mm	
$b = \min(b_1,b_2)$	= 260	
$S_4 = G_{13}*L/b$	= 8.88	…3.3(6)
r = (Live Load/Total Load)	= 0.00	
$\rho c = 11.39(E/f_c')^{-0.407} r^{-0.074}$	= 0.99	
k_{12}	= 1	… 3.2.4
f_c' = GL17	= 33000 kPa	… 7.3.1
$A_c = (b_1*b_2)$	= 0.0676 m^2	… 3.3.1.1

$N_{d,c}$ = 1517 kN > **N^*_c = 1214.3 kN** **OK**

e. Column Bearing Capacity- Parallel to the Grain:

$N_{d,l} \geq N_l^*$ … 3.2 (17)

where

$N_{d,l} = \phi\, k_1\, k_4\, k_6\, f_l' A_l$		… 3.2 (18)
ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_l' = GL17	= 40000 kPa	… 7.3.1
$A_l = (b_1*b_2)$	= 0.0676 m^2	… 3.2.6.2

$N_{d,l}$ = 1839 kN > **N_l^* =1214.3 kN** **OK**

f. Column Bearing Capacity- Parallel to the Grain:

$N_{dt} \geq N_t^*$		… 3.4 (1)

where

$N_{d,t} = \phi\ k_1\ k_4\ k_6\ f_t' A_t$		… 3.4 (2)
ϕ	= 0.85	… 2.3
k_1	= 0.8	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_t' = GL17	= 20000 kPa	… 7.3.1
$A_t = 0.6*(b_1*b_2)$	= 0.0406m^2	… 3.4.1

$N_{d,t}$ = 552 kN **>** **N_l^* = 0 kN** **OK**

Design of a 7th to 8th Floor 6.7m Brace

Similar to the calculations for the seventh floor column, the demands for the brace were determined using computer modeling software in conjunction with the variables listed below. The bracing resists the lateral loads imposed on the structure.

Selected Brace dimensions: 260 x 195 mm GL17. b1 = 195mm, b2 = 260mm.

a. Uniformly Distributed Loads (UDL):

Dead load	= .62 kPa	= 0.12 kN/m	
Q	= 3 kPa	= 0.59 kN/m	
G	= (density $*b_1*b_2*g)/1000^3$ Where density = 560 kg/m^3	= 0.59 kN/m	… g = 9.81m/s^2

b. Load Combinations from *AS1170.0*

The controlling Strength Limit State = 1.2G + Wu+0.4Q … 4.2.2 (b)

c. The demand of the column is based on the structural output. The maximum loading conditions on the structural element is **236.4 kN (C), 211.0 kN (T), and 0.8 kN-m (M), 0.67kN (V).** The governing failure mode is compression.

d. Brace Compression Capacity:

$N_{d,c} \geq N^*_c$		… 3.3 (1)

where

$N_{d,c} = \phi\ k_1\ k_4\ k_6\ k_{12}\ f_c'\ A_c$		… 3.2 (14)
ϕ	= 0.85	… 2.3
$k_{1\text{-timber}}$	= 1	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
g_{13} (Restrained at both ends)	= 0.7	…Table 3.2
L	= 6673 mm	
$b=\min(b_1,b_2)$	= 195mm	
$S_4 = G_{13}*L/b$	= 23.95	
r=(Live Load/Total Load)	= 0.82	
ρc	= 0.91	
k_{12}	= 0.42	… 3.2.4
f_c' = GL17	= 33000 kPa	… 7.3.1
$A_c = (b_1*b_2)$	= 0.0507 m^2	… 3.3.1.1

$N_{d,c}$ = 597 kN **>** **N^*_c = 236.4 kN** **OK**

e. Brace Bearing Capacity- Parallel to the Grain:

$N_{d,l} \geq N_l^*$ … 3.2 (17)

where

$N_{d,l} = \phi\ k_1\ k_4\ k_6\ f_l'A_l$ … 3.2 (18)

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_l' = GL17	= 40000 kPa	… 7.3.1
$A_l = (b_1*b_2)$	= 0.0507 m^2	… 3.2.6.2

$N_{d,l}$ = 1724 kN > N_l^* = 236.4 kN OK

f. Brace Tension Capacity- Parallel to the Grain:

$N_{dt} \geq N_t^*$ … 3.4 (1)

where

$N_{d,t} = \phi\ k_1\ k_4\ k_6\ f_t'A_t$ … 3.4 (2)

ϕ	= 0.85	… 2.3
k_1	= 1	… 2.4.1.1
k_4 (EMC = 15)	= 1	… 2.4.2.1
k_6	= 1	… 2.4.3
f_t' = GL17	= 20000 kPa	… 7.3.1
$A_t = 0.6*(b_1*b_2)$	= 0.0304 m^2	… 3.4.1

$N_{d,t}$ = 517 kN > N_l^* = 211.0 kN OK

Connection Design

Based on the demand forces presented above, the connection capacities can be determined. The connections are designed based on the demand required from each individual element and in accordance to Clause 4.4.3. – Design Capacity for Bolted Connections. Figure 61 to Figure 63 illustrate a typical connection within the structure based on the controlling loading conditions applied (from Level 2). The fixed end moment connection (FEM) is designed to accommodate timber plugs such that the connection is hidden from the sides. However for clarity the plugs are not shown. See figures below for an example of hidden FEM's. Further the use of high strength fiber allows a much more effective an economical design. The use of high strength fibre enhanced connectors is beyond the scope of this case study.

The column and brace bolt capacity check is utilising the demand on level seven and the beam bolt capacity is derived from the demand specified for the Main Floor Beam.

a. Column Bolt Connection Capacity for a Type 1 Joint: Section 260 x 260 mm GL17

$N_{d,i} \geq N^*$ … 4.4 (2)

where

$N_{d,j} = \phi\ k_1\ k_{16}\ k_{17}\ nQ_{sk}$ … 4.4 (3)

ϕ	= 0.65	… 2.3
$k_{1\text{-joint}}$	= 0.69	… 2.4.1.1
k_{16}	= 1.2	… 4.4.3.2
k_{17}	= 1.0	…4.4.3.2
n	= 4	… 4.4.3.2
$Q_{sk}=Q_{skl}$	= 40.9	… 4.4.5
From Table 4.9(C)- Seasoned Timber		
Bolt dia	= 24mm	
Group	= JD4	
b_{eff}	= 110	
# of planes	= 2	

$N_{d,j}$ = 146.7 kN > N^* = 0 kN OK

b. Brace Bolt Connection Capacity: Section 260 x 195 mm GL17

$N_{d,i} \geq N^*$		… 4.4 (2)

where

$N_{d,j} = \phi\, k_1\, k_{16}\, k_{17}\, nQ_{sk}$		… 344 (3)
ϕ	= 0.65	… 2.3
k_1	= 1.14	… 2.4.1.1
k_{16}	= 1	… 4.4.3.2
k_{17}	= 1	… 4.4.3.2
n	= 8	… 4.4.3.2
Q_{sk}	= 81.8	… 4.4.5
Bolt dia	= 24mm	
Group	= JD4	
b_{eff}	= 110	
# of planes	= 2	

$N_{d,j}$ = 242.5 kN **>** **N* = 236.4 kN** **OK**

c. Main Floor Beam Bolt Connection Capacity: Section 130 x 457 mm GL18

$N_{d,i} \geq N^*$		… 4.4 (2)

where

$N_{d,j} = \phi\, k_1\, k_{16}\, k_{17}\, nQ_{sk}$		… 344 (3)
ϕ	= 0.65	… 2.3
k_1	= 1.69	… 2.4.1.1
k_{16}	= 1	… 4.4.3.2
k_{17}	= 1	… 4.4.3.2
n	= 4	… 4.4.3.2
Q_{sk}	= 40.9	… 4.4.5
Bolt dia	= 24mm	
Group	= JD4	
b_{eff}	= 110	
# of planes	= 2	

$N_{d,j}$ = 146.7 kN **>** **N* = 74.9 kN** **OK**

The bolt connections are design to inhibit shear forces from controlling the bolt capacity. Instead, loads are transferred using bearing connection mechanisms. Thus, the bolt capacity in the column and beam is dependent on the tension within the member; whereas the bolt capacity of the brace is dependent on the maximum compression or tension forces within the element. Note, these connections are designed solely based on the design of the timber elements, thus the steel elements still require design. Based on this, various steel elements necessitate design consideration including: bearing plates, kerf plates, and various welds throughout the structure. The bearing plates should be calculated for flat plate bending and should have a weld of sufficient strength connecting the bearing plate to the kerf plate. Kerf plates should be designed by analysing the capacity for gross tension, net tension and compression (for braces). In addition, it is essential that various other welds throughout the structure are included in design considerations.

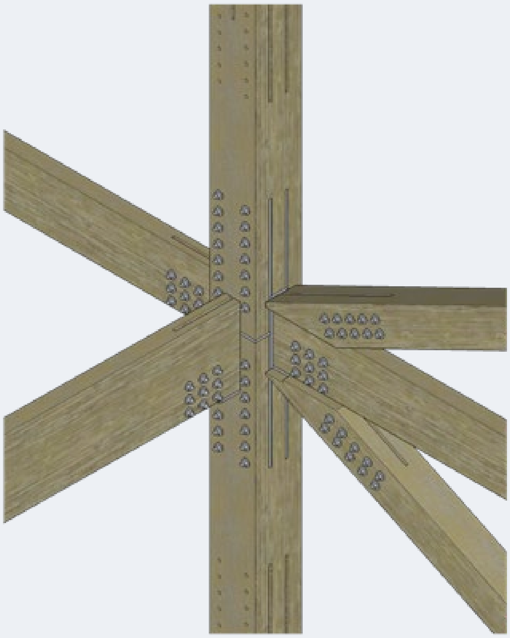


Figure 61: Example connection design fixating the columns, braces and beams together, taken from level two.

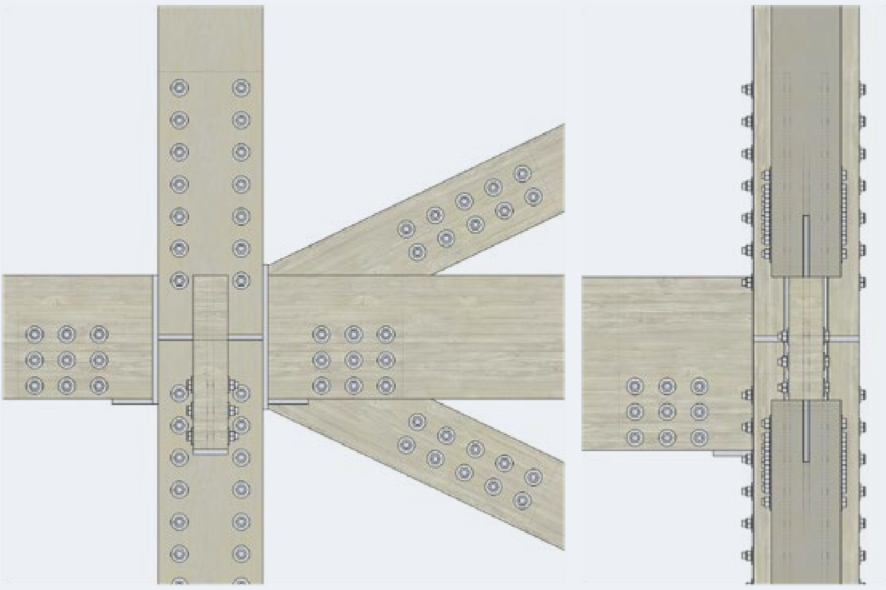


Figure 62: Connection design illustrating bolt pattern at level two.

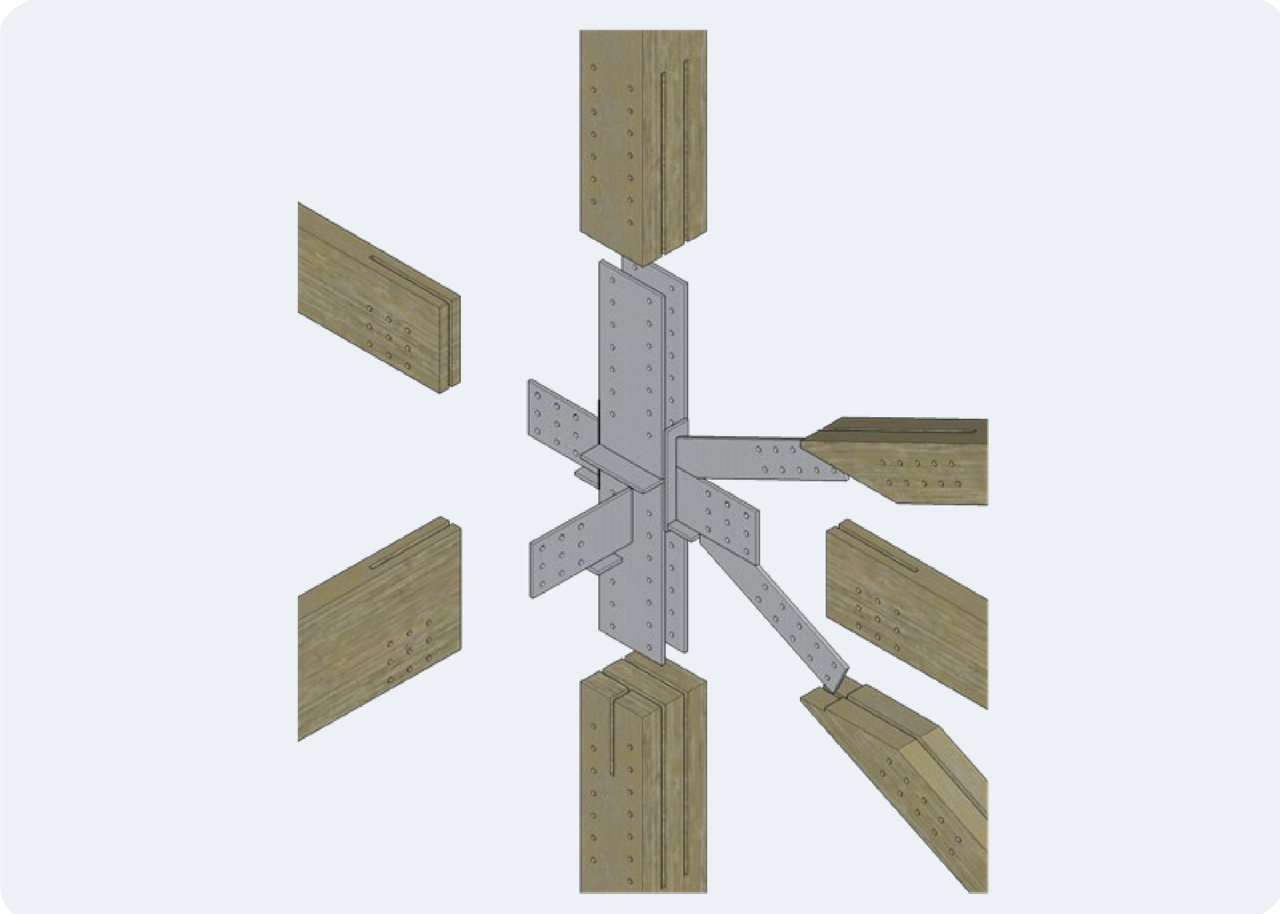


Figure 63: Steel connection design.

Fire Resistance Level (FRL)

To exhibit the fire resistance (1-hour) of a typical element within the structure, the following FRR calculation is summarized below for a third floor beam and a seventh floor column. The first FRR calculation has been completed without high strength fibre reinforcement utilising the National Building Code of Canada (NBCC). The fire resistance rating is determined by the use of the factored demand and capacity forces, and the calculations are summarized below. If FiRP® Fibre Reinforced Polymer Reinforced Glue-laminated beams products are desired, PFC-6046 Section 06180 should be referenced to complete a separate FRR_r calculation. The use of FiRP® glulam beams and columns provides for a FRR well in excess of 1 hour.

For a beam exposed on three sides:

$FRR=0.1fb[4-(b/d)]$ *…Equation 7*

Where b=130 mm; d= 457 mm

a. Determine the load factor f, see Appendix D of the NBCC:

Demand:	= 112.3 kN-m
Capacity:	= 129.1 kN-m
% Loading=demand/capacity	= 87.00%
K_e	= 1
f	= 1.02

Therefore FRR=49.36 minutes. If FiRP® Reinforcement is utilised the capacity is improved dramatically and the demand/capacity approaches 50% and the f value becomes 1.3 and the FRR_r becomes 62.9 minutes which exceeds the required 1 hour fire rating. See below.

$FRR_r=0.1fb[4-(b/d)]$ *…Equation 7a*

Where b=130 mm ; d= 457 mm

a. Determine the load factor f, see PFC 6046:

Demand:	= 112.3 kN-m
Capacity:	= 258.2 kN-m
% Loading=demand/capacity	= 44.00%
K_e	= 1
f	= 1.3

The FRR for columns is derived from the following equation:

For a column exposed on four sides:

$FRR = 0.1 f b[3-(b/d)]$ ...*Equation 8*

Where b=260 mm.; d= 260 mm

a. Determine the load factor f, see Appendix D of NBCC:

Compressive demand:	= 1,214.3 kN
Capacity:	= 1,517 kN
% Loading	= demand/capacity = 80.05%
K_e	= 1
f	= 1.05

Therefore FRR = 54.59 minutes. If FiRP® Reinforcement is utilised the capacity is improved dramatically and the demand/capacity approaches 50% and the f value becomes 1.3 and the FRR_r becomes 67.4 minutes which exceeds the required 1 hour fire rating, see below.

For a column exposed on four sides:

$FRR_r = 0.1 f b[3-(b/d)]$ ...*Equation 8a*

Where b=260 mm.; d= 260 mm

a. Determine the load factor f, see PFC 6046:

Compressive demand:	= 1,214.3 kN
Capacity:	= 2,648 kN
% Loading	= demand/capacity = 46%
K_e	= 1
f	= 1.30

These elements fail to pass the one hour fire resistance in the sizes shown unless they are reinforced with FiRP® Products. Based on this, fire resistance of these elements may be the governing design parameter unless FiRP® Products are utilised or other methods of improving the FRR discussed in earlier sections. The required FRL for each structure component is displayed in Specification C1.1 of the NCC for each building element and class, based on this the element sizes should be altered or treated depending on the minimum requirements. The use of FiRP® Glulams would provide for FRR values well in excess of one hour while also providing for holes and service access through the glulam elements. In addition the use of high strength fibre allows much more effective FEM's.

11.3 Worked Examples Conclusion

This design guide provides practicing structural engineers and other building professionals with an outline of the parameters encompassing timber structure design.

The two worked examples provide an applicable, step-by-step approach to designing timber components in multi-storey timber buildings. Diverse structural requirements are included. The examples demonstrate the process and procedures required from forming the initial design concept to finalising the 'demand versus available capacity' design verifications. The sources of design requirements are included in each calculation.

Worked Example 1 includes the design of a five storey multi-use structure with a dome, Tudor arch auditorium, and roof deck, and follows an open floor plate design approach. The solution provides insight on calculations for wind load actions, fire resistance, individual member design, Tudor arch design, lateral loading and an open web floor joist.

The detailed calculations for Worked Example 1 focus on wind load actions, fire resistance (4-hour), and individual member design. Fire resistance of the element is dependent on the charring (rate and effective depth) of the element and subsequently determining the bending and shear capacity based on the remaining residual capacity. The individual elements designed include the floor beams, curved glulam dome, Tudor arch, and portal frame. The curved glulam, floor beams, and arch are dependent on the bending, shear and deflection of the element. The portal frame contains the analysis of the axial compression and bending capacity of the columns as well as the effects of combined axial and bending of the element.

Worked Example 2 outlines the design of a ten storey multi-use structure on top of a concrete first floor with a pool on the tenth floor and, similar to Worked Example 1, it follows an open floor plate design approach. In addition to wind load actions and individual member design, this example provides calculations for earthquake actions and pool support-member design, bracing and connections.

Detailed calculations for Worked Example 2 focus on wind action, earthquake action and individual element design. The individual elements are designed according to system types: the floor, roof, column and bracing systems. Each system compares the element demand force with the capacity of the section. The earthquake action loading utilises site-specific information to derive the elasticity of the structure and determine the horizontal shear force imposed on the structure as well as the vertical distribution of horizontal forces on the structure. Bending, shear and deflection are controlling parameters outlined for roof and floor systems. Columns are designed to withstand compression, and bearing and tension parallel to the grain. The connection demand is derived from the forces within the connecting elements.

Each worked example provides a step-by-step process and illustrations of the design concept as well as loading scenarios. Importantly, all major structural components in both example buildings are made of timber products, glulam and CLT. This emphasises the capacity of timber to enable diversity in structural design and demonstrates how larger buildings can be designed with structural integrity while maintaining a high aesthetic appeal. Figures 64 and 65 illustrate built examples similar to the design concepts presented.

Figure 64: Design concept image taken from Arch Daily – International House

Figure 65: Design concept image taken from Jobsite Australia – Inspiring Projects from the 2017 Timber Design Awards

12 Glossary

Term	Explanation
Across the grain	At right angles to the general direction of the fibres or wood elements.
Action	A force or load applied to a structure or an imposed deformation such as moisture change effects or settlement.
Adhesive	A substance used to bond two surfaces together.
Along the grain	Parallel to the general direction of the fibres or wood elements.
Anisotropic	Exhibiting different properties when measured along its different axes.
Assembly	A part of a structure consisting of several members such as a roof truss or a floor diaphragm.
Back sawn	Timbers sawn so that the growth rings are inclined at less than 45 degrees to the wide face.
Bio-deterioration	The breaking down of timber by natural or biological agents such as fungi and insects.
Board	1. A piece of sawn or dressed timber of greater width than thickness. Usually 19 mm to 38 mm thick and 75 mm or more wide. 2. A manufactured wood products supplied as rigid or semi-rigid sheets such as fibreboard and particle board.
Box beam	A built-up beam with solid timber flanges and plywood or wood-base panel product webs.
Buckling	Sideways deflection of a structural member under compression.
Building element	A principal part of a building, such as a roof, wall or floor.
Characteristic value	The characteristic value of an action or material property is its appropriate representative test value, before combination or safety factors are applied to it.
Component	A member made up of various parts often manufactured as a product, or part of a force such as the vertical component.
Compression	A state or condition of being pushed or shortened by a force.
Compression failure	In wood, failure where individual fibres are pushed end to end into each other until they buckle and are pushed out of the side of the piece.
Conduction	Heat transfer through a solid material from a higher temperature area to a lower temperature one.
Connection	All the components used to attach one building component to another: timber to timber or timber to another material. It will include fasteners and usually connectors.
Connector	Proprietary and custom-made timber, plastic or metal items used with fasteners to join timber to timber or timber to another material. It includes gussets, nailplates and beam hangers.
Creep	Increase in deformation following prolonged loading.
Cross laminated timber	Cross laminated timber (CLT) is engineered wood panels made by joining layers of timber together with the grain direction of alternating layers at right angles
Decay	Softening, weakening, or total decomposition of wood substance by fungi.
Deformation	Deflection or displacement of a member, component or assembly, or the slip in a connection.
Density	With wood, density is the mass of wood substance and moisture enclosed within a piece divided by its volume. As the piece's mass varies with its moisture content (MC), density is often expressed at a specified MC, usually 12%.
Design value	For an action or group of actions or material property, the appropriate characteristic value or values modified as necessary by the relevant combination and safety factors.
Dowel	A cylindrical timber or steel rod driven directly into timber or a pre-drilled hole to make a joint. Dowels are generally without a nut or thread.

Term	Explanation
Durability	1. The natural resistance of timber to bio-deterioration. 2. In building, the efficacy of assemblies and details to preserve or protect the building's fabric from decay or deterioration.
Durability class	A rating for the natural resistance of a species' heartwood to bio-deterioration expressed on a 1-4 scale for exposure in-ground and out of ground contact to AS 5604. Durability Class 1 timber is rated as highly durable while Durability Class 4 timber is rated as non-durable.
Element	A single part of a connection, component, or structure.
End grain	The grain shown on a cross cut surface of wood.
Engineered wood product	A general term for a manufactured product made from sections of solid timber, veneer or wood strands, particles or fibres arranged and usually bonded together with an adhesive under heat and pressure to form a structurally reliable material that avoids or minimises the natural variability found in logs or sawn timber. Glulam, plywood, LVL and oriented strand board are engineered wood products.
Equilibrium moisture content	The MC at which timber neither gains nor loses moisture from the surrounding atmosphere. It will change with changes in humidity and temperature.
Expansion	In timber, the expansion of wood fibres caused by the uptake of moisture in service. Unit expansion is the rate of expansion expected with a 1% MC increase.
Fasteners	Timber, plastic or metal items used to directly join timber to timber or timber to another material such as nails, dowels, screws, bolts and similar fixings.
Fibre saturation point (FSP)	The stage in wood drying where the cell walls are saturated with bound water but the cell cavities are free of water. It is usually considered to be about 26% moisture content.
Fibreboard	Manufactured products made from a mix of wood fibres and usually an adhesive binder arranged into a mat and pressed into a sheet. Variations in the fibre length, matt thickness and pressing pressure produce a range of products such as low density, medium density and high density fibreboard. High-density fibre (HDF) boards such as hardboard are typically made without adhesives and are the only product type of the group regularly used structurally.
Fin plate	Metal or plywood plates set into shaped recesses in a timber section, or sandwiched between two timber elements to make a connection. They are usually fixed with steel dowels, screws, nails or occasionally bolts
Fire resistance level	The required resistance of an element to fire, expressed in minutes, for each of three categories: structural adequacy / integrity / insulation.
Fixing	Any item used to secure parts of a frame or element together.
Flashing	A strip of impervious material fitted to provide a barrier to moisture movement into the building envelope or interior.
Foundation	The soil, subsoil or rock upon which a structure is supported.
Frame	1. The main timbers of a structure fitted and joined together. 2. A three dimensional self-contained structural system of interconnecting members that functions with or without horizontal diaphragms or floor bracing systems.
Glue laminated timber	Sections of sawn timber glued together to form larger, more structurally reliable timber elements. The sections are often joined along their length into laminates, then glue together on their wide face or on their edges.
Grade	The designation of the quality or capacity of a log, piece of timber or other wood product determined in accordance with standard rules.
Grain	1. The general direction of the fibres or wood elements relative to the main axis of the piece. 2. The direction, size, arrangement, appearance or quality of the fibres in wood or timber
Gusset plate	A plate, usually of steel or plywood, used to join or reinforce principal members in the same plane.
Hardboard	High density fibreboard
Hardness	A wood property that enables it to resist indentation. It is often determined by the Janka hardness test.

Term	Explanation
Hardwood	A general term for broad leafed trees classified botanically as Angiosperm or the wood and timber produced by these trees.
Hazard class	A rating for a timber component's exposure to biological and other hazards in an application. Hazard classes are rated on a 1-6 scale: Hazard Class H1 represents the lowest level of hazard and H6 represents the highest level.
Heart	The zone of low-quality wood from the original sapling or growing tip of a hardwood tree found in the centre of the log. It generally has low strength and durability and high shrinkage rates.
Heartwood	The zone of wood making up the centre part of the tree, beneath the sapwood but excluding the heart. Heartwood may contain phenolic compounds, gums, resins, and other materials that usually make it darker and more decay resistant than sapwood.
Hygroscopic	A material that gives off and absorbs moisture to remain in equilibrium with the surrounding atmosphere.
Joint group	A grouping of timber species based on their likely performance in a joint or connection. Species are classified into one of six joint groups for unseasoned timber, J1 to J6, and one of six joint groups for seasoned timber, JD1 to JD6, based on the average species density.
Laminated strand lumber	An engineered wood product made from wood strands arranged roughly longitudinally, and bonded together with an adhesive under heat and pressure into a large billet. This is then resawn into market sizes.
Laminated timber	A built up product made of layers or laminations of wood, arranged with the grain of each layer parallel to each other and joined together with glue, nails or other fasteners.
Laminated veneer lumber (LVL)	An engineered wood product made from peeled veneers bonded together with an adhesive under heat and pressure into panels with the grain of most veneers running parallel to each other along the board. The panel is then resawn into market sizes.
Lightweight timber construction	Rugged timber frame construction assembled from lightweight sawn, fabricated and panel products with fasteners, steel strap and other connectors. The thickness of sawn products is generally 45 mm or less.
Limit state	The states beyond which the structure no longer satisfies the design performance requirements.
Longitudinal	The direction along the grain and parallel to the axis of the wood's cells. In practice, it often means along the piece of wood.
Lyctid susceptibility	The susceptibility of the sapwood of some hardwood to attack by the larva of lyctid beetles.
Massive timber construction	In the NCC, a solid wood element not less than 75 mm thick formed from chemically bonded laminated timber, including cross laminated timber, laminated veneer lumber and glued laminated timber. In practice, fasteners can also be used to form solid timber panels.
Member	A beam or column within a structure or assembly.
Mid-rise	Generally, a building with an effective height of less than 25 metres.
Moisture content	The amount of moisture contained in wood, expressed as a percentage of the oven dry mass.
Nail laminated timber	Sections of sawn timber nailed together to form larger, more structurally reliable timber elements. The sections may joined along their length into laminates or butt jointed before being nailed on their wide face.
Nail-on plate	Generally sheet metal connector plates with pre-punched holes ready to receive fasteners such as nails. Plates can be flat, include a 900 twist, be simply folded or welded to a steel section to form a connecting surface to other material.
Nailplate connector	Sheet metal connectors stamped so that nails are formed and protrude on one face. These make a reliable connection when pressed into the timber on each side of a joint.
Oriented strand board	An engineered wood product made from flakes or large chips of wood bonded together with an adhesive under heat and pressure. The fibre direction within each layer is generally in the same direction, but in some cases the direction alternates between layers.
Prefabrication	The design and off-site manufacture of a project specific component, assembly or system that is utilised, in part or as a whole, to build a structure.
Particle board	An engineered wood product made from timber particles combined with adhesive bonded together under heat and pressure into sheets.
Permanent action	Dead loads, such as the self-weight of the structure or fittings, ancillaries and fixed equipment.

Term	Explanation
Pitch	The slope of a roof.
Pith	The zone of low-quality wood from the original sapling or growing tip of a softwood tree found in the centre of the log. It generally has low strength and durability and high shrinkage rates.
Platform frame construction	A building method where the floor form platforms supported on top of the wall frames. The next level of walls is then built off this platform. Wall frames are braced to form vertical load-bearing diaphragms with the roofs and floors acting as horizontal diaphragms.
Plywood	An engineered wood product made from peeled veneers bonded together with an adhesive under heat and pressure into panels with the veneer's grain direction alternating between layers. The grain on the outside veneer runs along the sheet with the grain of alternate inner layers running across the sheet.
Portal	A planar frame where the lateral and bending forces are transferred between the portal rafters and columns by moment-resisting connections.
Post and beam structure	A structural frame with repetitively arranged columns and beams.
Preservative	Any substance that is effective in preventing fungi, borer and insect attack in wood for a reasonable period of time.
Purlin	One of a series of horizontal framing timbers supporting the roofing material. Purlins usually span at right angles to the slope of the roof.
Racking	The effect caused by horizontal actions in the plane of a wall or other horizontal element. An element's rack resistance is its resistance to going out of square.
Radial	The direction coincident with a radius from the centre of the log to the circumference. In practice, it often means at right angles with a growth ring.
Rafter	One of a series of roof support timbers that provide principal support for the roofing material. Rafters usually span parallel to the slope of the roof.
Sapwood	The zone of wood near the outside of the stem, beneath the bark that contain living cells and reserve materials such as starches. The sapwood is generally lighter in color than the heartwood and is lower durability.
Serviceability limit states	Limit states beyond which specified service criteria are no longer met.
Sheet metal connector	A shaped connector made of sheet metal and perforated so that nails can be driven through. Also known as a nail-on plate.
Shrinkage	In timber, the contraction of wood fibres caused by the loss of moisture during production or in service. Shrinkage can be expressed as a percentage of the dimension of the wood when green. Unit shrinkage is the rate of shrinkage expected with a 1% MC decrease.
Slip	The relative movement between two loaded members within a mechanically-fastened connection that joins them.
Softwood	A general term for trees that, in most cases, have needle or scale-like leaves, classified botanically as gymnosperms. It includes all conifers. It also includes the wood produced by these trees.
Span	The distance between structural support, measured horizontally.
Strength	The ability of a member to sustain stress without failure.
Strength group	A classification of timber species based on their mechanical properties into groups of similar strength in an element. Species are classified into one of seven strength groups for unseasoned timber (S1 the strongest to S7 the weakest) and one of eight for seasoned timber (SD 1 the strongest to SD 8 the weakest).
Stressed skin panel	Prefabricated, engineered modules where joist or studs of solid timber, LVL, I-beams or floor trusses are fixed or bonded to structural panels of ply, OSB, HDF or particleboard on the top and bottom faces. The assembly acts as a composite system with the panels acting as tension or compression chords and the joists acting as web-members.
Stud frame wall	A wall frame assembled from closely spaced vertical studs fixed between horizontal timber plates. Sheathing over the studs or noggings fitted between them reduce the risk of the studs buckling. Sheet bracing or steel straps and ties connecting the stud to the plates allow the frames to resist uplift and racking loads.

Term	Explanation
Tangential	The direction coincident or parallel with a tangent at the circumference of the log. In practice, it often means coincident with a growth ring.
Tension	A state or condition of being pulled or stretched by a force.
Tension failure	In wood, failure where individual fibres are pulled apart or rupture as a result of tensile stresses.
Termites	Soft-bodied social insects that eat wood and other cellulous. Some species cause commercial damage in buildings.
Timber	A general term for natural or sawn wood in a form suitable for building or structural purposes.
Truss	A frame of members in the same plane joined only at their end and interconnected to form triangles. If loads are applied only at the joints, primary stresses in the elements are axial compression or tension.
Ultimate limit state	Limit states associated with collapse or other forms of structural failure that may endanger the safety of people.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