41

Timber Garden Retaining Walls

Up to 1.0 m High

Technical Design Guide issued by Forest and Wood Products Australia

design and build
woodsolutions.com.au

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Aveo Group, Strongbuild - Bella Vista luxury retirement community (10-storey luxury retirement village build with cross-laminated timber - CLT).

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-47-8

Prepared by:
Timber Queensland Ltd

Technical Reviewer:
Colin MacKenzie

First Published: November 2017

Contents

1 Introduction

Timber sleepers or round posts and logs can be used to create effective and relatively inexpensive garden edges, steps and walls to terrace sloping sites. Galvanized steel posts can also be used in conjunction with sleepers to achieve satisfactory designs. Timber fences can also be integrated with garden walls to add privacy and character to front yards.

This Guide contains recommendations for the construction of timber garden walls up to 1 metre high.

1.1 Performance

Retaining walls designed and constructed in accordance with this Guide could be expected to achieve a service life of 15 years or greater. Where a longer service life is required, hardwood timber should be limited to In-ground Durability Class 1 and softwood should be treated to H5 level.

For these more durable options, service life in excess of 25 years can be expected.

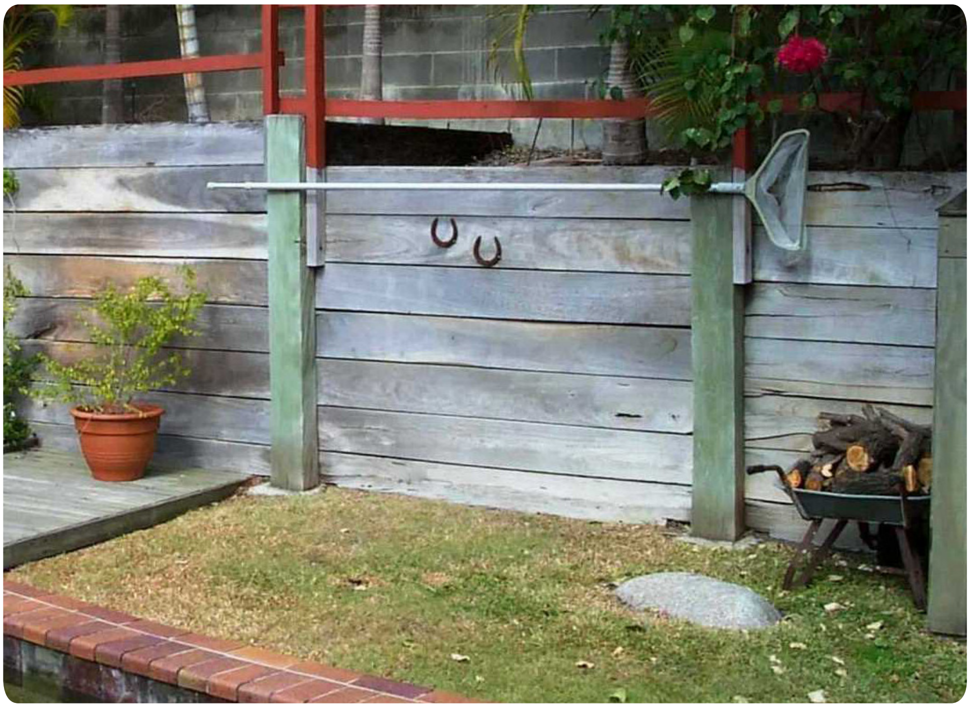


Figure 1.1: 25 year old hardwood retaining wall constructed using treated Class 1 durability posts and Class 1 and Class 2 Wales

1.2 Building Approval and Certification

Local authorities generally do not require building approval or engineer certification for retaining walls up to 1 metre high. Walls (irrespective of height) closer than 1.5 metres to a building, swimming pool, deck etc., and retaining walls greater than 1 metre high will generally require building approval and certification by an engineer.

1.3 Ground Conditions

The sizes and other recommendations in this Guide assume that posts are embedded into firm natural ground (e.g. stiff clay, shale or dense sand), with a maximum ground slope of 1 in 6. Special design may be required for steeper slopes and poor subgrade soil conditions such as uncompacted fill.

Figure 1.2: It is critical that the footing embedment of posts is adequate to prevent rotation of the posts. The treated pine rounds in this wall are still sound. The wall is failing due to inadequate embedment of posts.

2 Materials

2.1 New Timber

There is no Australian Standard or other grading requirement for landscaping timber. The quality of sawn 'sleepers' and round logs can vary greatly between different suppliers.

Timber sold for landscaping is likely to contain natural characteristics such as knots, gum veins, heart, insect damage, want, wane etc., and there could be some bow, cupping, twisting etc. Where a higher than normal appearance or performance is required, timber should be selected with limitations on the size and number of these characteristics.

Structurally graded timber (e.g. F14 hardwood and F7 treated pine) will have prescribed limits for natural characteristics and will generally be higher quality than 'landscaping' timbers.

Softwood logs and sleepers shall be preservative treated to hazard level H4 or higher.

Figure 2.1: Combination of durable hardwood posts and H4 CCA treated rounds

Hardwood should be In-ground Durability Class 1 or 2 and resistant to termites. Hardwood containing sapwood shall be preservative treated to H4 or better.

Table 2.1 provides a list of some readily available termite resistant hardwood and cypress species and their in-ground durability class.

Table 2.1: Hardwood and Cypress Durability

Species	In-Ground
Blackbutt	2
Blackbutt, Western Australian	2
Bloodwood, red and brown	1
Box, grey and grey coast	1
Cypress, white	2
Gum, red forest	1
Gum, river red	2
Gum, spotted	2
Ironbark, red and grey	1
Ironwood, Cooktown	1
Jarrah	2
Mahogany, red	2
Stringybark, Darwin	1
Wandoo	1

**Based on BCA's Provision C2.2 Table C2.2*

Figure 2.2: High quality hardwood sleepers

Figure 2.3: Low quality hardwood sleepers

2.2 Recycled Timber Railway Sleepers

Recycled hardwood railway sleepers provide another viable option for construction of timber retaining walls. When new, these sleepers are usually supplied to rail authorities under stringent specifications including species, durability and preservative treatment. Railway sleepers are much 'heavier' (larger in cross-section) than normal landscaping sleepers so may not suit use with 'off the shelf' steel post systems.

As they will have typically been in use for many decades, some existing degradation due to decay, termites and weathering can be expected. If this has severely degraded the sleepers then they will have a lesser life expectancy than equivalent material when new.

Any recycled sleepers that exhibit significant surface or possible internal degradation caused by decay or termites should be discarded. Sleepers with live termite activity should also be discarded.

Drilling the sleepers with a 4 mm diameter drill where internal degradation is suspected will reveal if any areas of concern. Drills will resist solid timber but easily penetrate decayed or termite damaged timber.

Figure 2.3: Recycled hardwood railway sleepers complete with rail bearing plates.

2.3 Steel Posts

Steel posts should be hot-dip galvanized in accordance with AS/NZ 4680:2006 - Hot-dip galvanized (zinc) coatings on fabricated ferrous articles. A minimum zinc coating thickness of 50 microns is recommended. The specification and design for steel posts should be in accordance with the manufacturer's requirements. Figure 2.4 shows typical design options using steel posts.

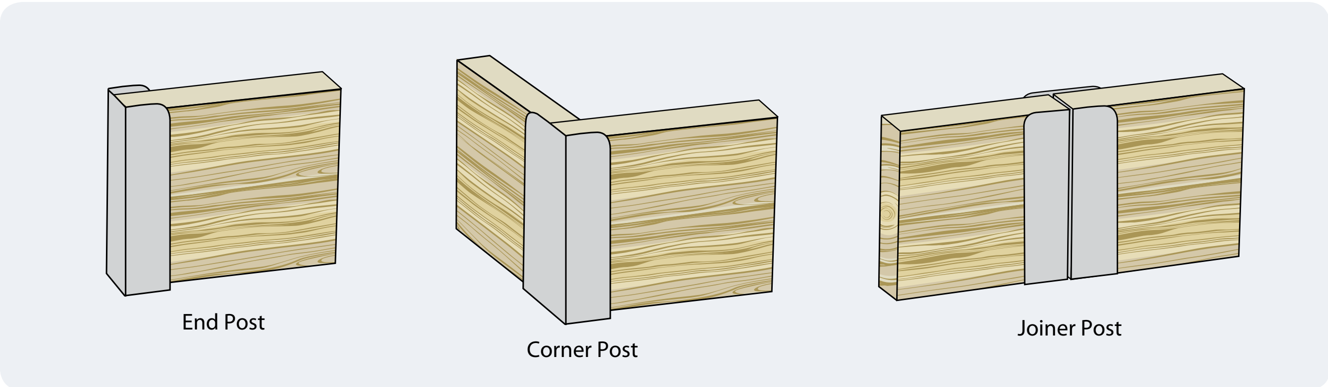


Figure 2.4: Timber Sleepers with Steel Posts

Figure 2.5: Poor quality galvanized steel 'H' section post showing corrosion approximately 8 years after installation

2.4 Preservative Treatment

AS 1604.1 – Specification for preservative treatment, Part 1: Sawn and round timber, specifies the minimum penetration and retention requirements for various preservative treatment types.

Common preservatives used to treat timber for retaining wall applications are CCA, ACQ, copper quaternary and copper azole.

For Hazard Class H4, AS 1604.1 requires the following penetration to be achieved in the timber:-

In-ground Durability Class 1 and 2 timber – The preservative must penetrate all of the sapwood. Penetration of the heartwood is not required.

In-Ground Durability Class 3 and Class 4 timber (Softwood) – For sawn timber, the preservative shall penetrate not less than 10 mm from any surface, or, unpenetrated heartwood shall comprise less than 20% of the cross section and shall not exceed 50% of the width of the face and not extend more than halfway through the piece.

For round timber, penetration shall be not less than 10 mm from the surface.

Figure 2.6 illustrates the minimum penetration requirements in sawn and round in-ground durability Class 3 and 4 timber.

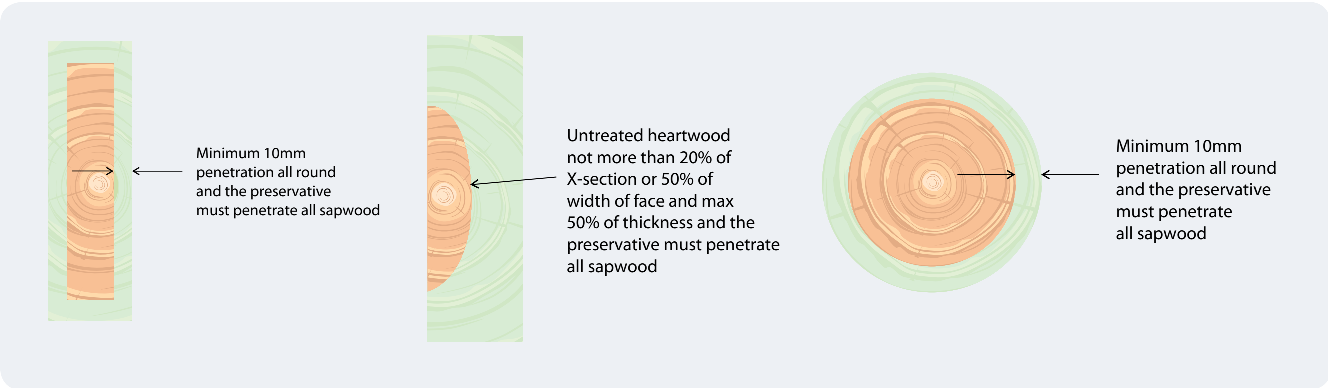


Figure 2.6: Preservative treatment Requirements for In-Ground Durability Class 3 and 4 Timber such as Softwood

Figure 2.7: Premature failure of incorrectly treated pine sleeper wall in school playground

Figure 2.6: Non-compliant CCA treated pine sleepers that failed after 5 years. Sleepers contained excessive amounts of untreated heartwood.

2.5 Corrosion

Timber that is treated with copper based preservatives such as described above, that is in contact with galvanized steel, can cause premature corrosion of the steel due to incompatibility of the copper in the preservative and the zinc coating on the steel. To minimise the potential for this to occur, the contact surfaces (timber and or steel) should be separated with a plastic or bituminous damp proof course or coated with an appropriate ‘paint’ such as bituminous paint.

3 Construction

3.1 General

Timber retaining walls up to one metre high can be constructed as shown in Figure 3.1.

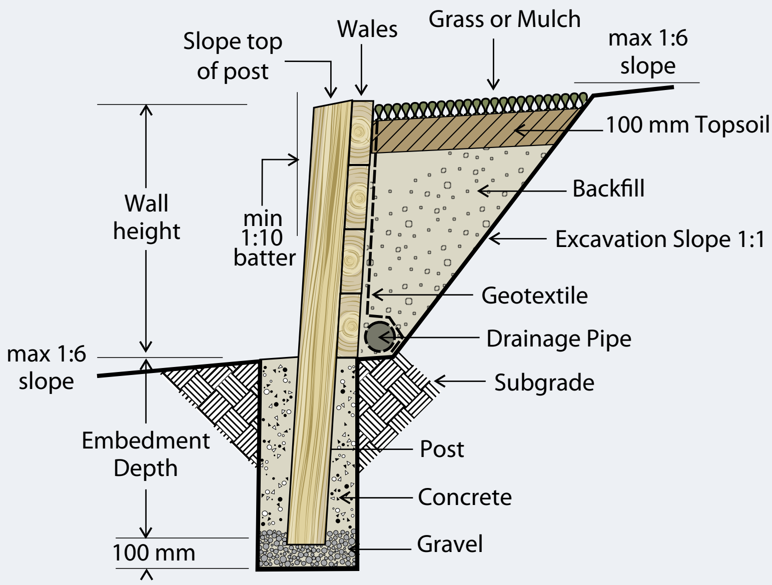


Figure 3.1: Retaining Wall Construction

3.2 Member Sizes and Embedment

Minimum member sizes and post embedment depths should be as outlined in Tables 3.1 and 3.2 for preservative treated softwood and hardwood, respectively.

Alternatives for round and sleeper post and wale arrangements are shown in Figures 3.2 (a) to 3.2 (i).

Table 3.1 – Preservative Treated Softwood (SWD)

Table 3.1: Preservative Treated Softwood (SWD)

Member	Refer Figure	Post Spacing	1200				1500				2400			
		Wall Height	400	600	800	1000	400	600	800	1000	400	600	800	1000
Sleeper Posts	3.2 (a)	Size (bxd)	150x50	150x75	200x75	N/A	150x150	150x75	200x75	N/A	200x100	200x100	N/A	N/A
		Hole Dia. (Ø)	300	300	450	N/A	300	300	450	N/A	300	450400	N/A	N/A
		Depth	400	600	600	N/A	400	600	650	N/A	400	600	N/A	N/A
Slab Posts	3.2 (b)	Thickness (t)	90	90	90	125	90	90	90	125	100	100	125	2/125
		Hole Diam.	300	300	300	300	300	300	300	300	300	300	300	450
		Depth	400	600	600	750	400	600	650	750	400	600	900	950
Single Round Posts	3.2 (c)	Post Dia. (Ø)	100	125	150	180	100	125	175	200	100	150	200	225
		Hole Dia. (Ø)	300	300	300	300	300	300	300	450	300	300	450	450
		Depth	400	600	800	1000	400	600	800	1000	600	800	1000	1400
Double Round Posts	3.2 (d)	Post Dia. (Ø)	100	100	125	150	100	100	150	175	100	125	175	200
		Hole Dia.	300	300	300	300	300	300	300	450	300	300	450	450
		Depth	400	600	800	1000	400	600	800	1200	600	800	1000	1400
Sleeper Wales	3.2 (e)	Size (bxd)	150x50	150x50	200x50	200x50	150x50	150x50	200x50	200x75	200x100	200x100	200x100	200x100
Slab Wales	3.2 (f)	Thickness (t)	90	90	90	90	90	90	90	90	90	100	100	125
Round Wales	3.2 (g)	Dia. (Ø)	75	75	75	75	75	75	75	75	100	100	100	100
Split Wales (1/2 round)	3.2 (h)	Dia. (Ø)	75	75	100	125	75	100	125	N/A	N/A	N/A	N/A	N/A
Winged Split Wales	3.2 (i)	Thickness (t)	100	100	100	100	100	100	100	100	N/A	N/A	N/A	N/A

Table 3.2: Hardwood (HWD) (refer Figures 3.2 (a) and 3.2 (e))

Member	Wall Height	400	600	800	1000
Posts	**Post Spacing 1200**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	800	1000
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50
Posts	**Post Spacing 1500**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	800	1200
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50
Posts	**Post Spacing 2400**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	1000	1400
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50

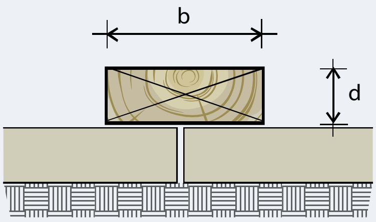


Figure 3.2 (a): Sleeper posts (HWD & SWD)

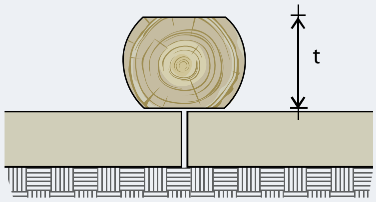


Figure 3.2 (a): Sleeper posts (HWD & SWD)

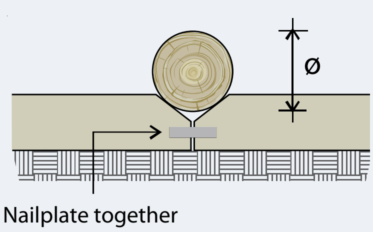


Figure 3.2 (c): Single round post (SWD)

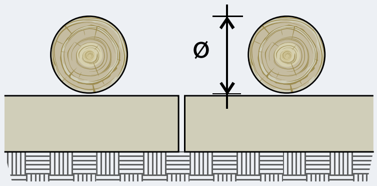


Figure 3.2 (d): Double round posts (SWD)

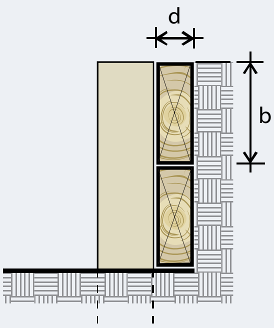


Figure 3.2 (e): Sleeper wales (HWD & SWD)

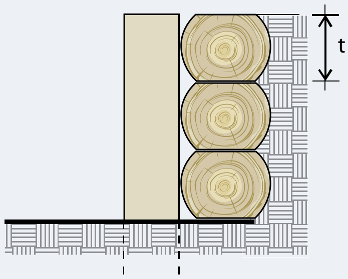


Figure 3.2 (f): Slab wales (SWD)

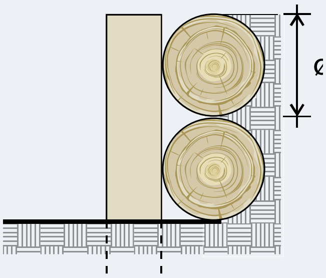


Figure 3.2 (g): Round wales (SWD)

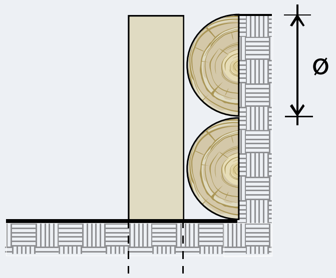


Figure 3.2 (h): Split wales (SWD)

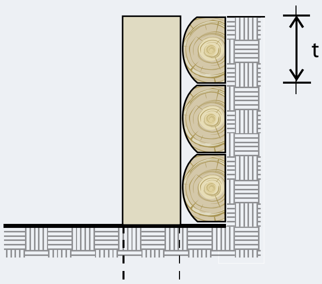


Figure 3.2 (i): Winged split wales (SWD)

Walls exceeding 400 mm high should have a 1:10 batter as shown in Fig 3.1.

The batter allows for the wall to have some lateral movement and still appear stable.

Post holes should have a 100 mm layer of coarse gravel installed prior to placing posts and backfilling with concrete.

'No-fines' concrete is recommended for hardwood posts.

Walls up to 400 mm high may have concealed posts, with wales fixed to the outside of posts with hot dipped galvanised batten screws, coach screws or bolts (refer Fig 3.3a). Walls above 400 mm shall have wales behind posts (refer Fig 3.3b). Typical methods of positioning posts at ends and corners are shown on Figure 2.

Wales may require temporary fixing to posts with wire ties, skew nails or batten screws.

The top of posts should be cut with a slight slope to shed water and should be sealed with a preservative emulsion.

Figure 3.3: Sloping cut to top of ironbark retaining wall post in this 23 year old wall

3.3 Drainage

Drainage should be provided behind walls exceeding 400 mm high to prevent additional loads due to the build-up of ground water. A slotted polyethylene drainage pipe should be provided at the base of the wall, discharging to a suitable outlet. To prevent backfill material from flowing through gaps in the wall and to assist in drainage, geotextile should be placed to the inside face of wales and between the drainage pipe and the backfill. See Figure 3.1.

Backfill should be a free flowing material such as ashes, sand or gravel– not clay. Backfill should be installed no sooner than three days after concreting posts.

Figure 3.4: Well designed and constructed treated pine retaining wall. Wall is battered and tops of posts have angle cut to shed water

3.4 Retaining Wall Posts and Garden Edges

Typical corner and end restraints are shown in Figure 3.5 (a) to 3.5 (c). Alternatively, posts can be positioned as shown on Figure 3.xx.

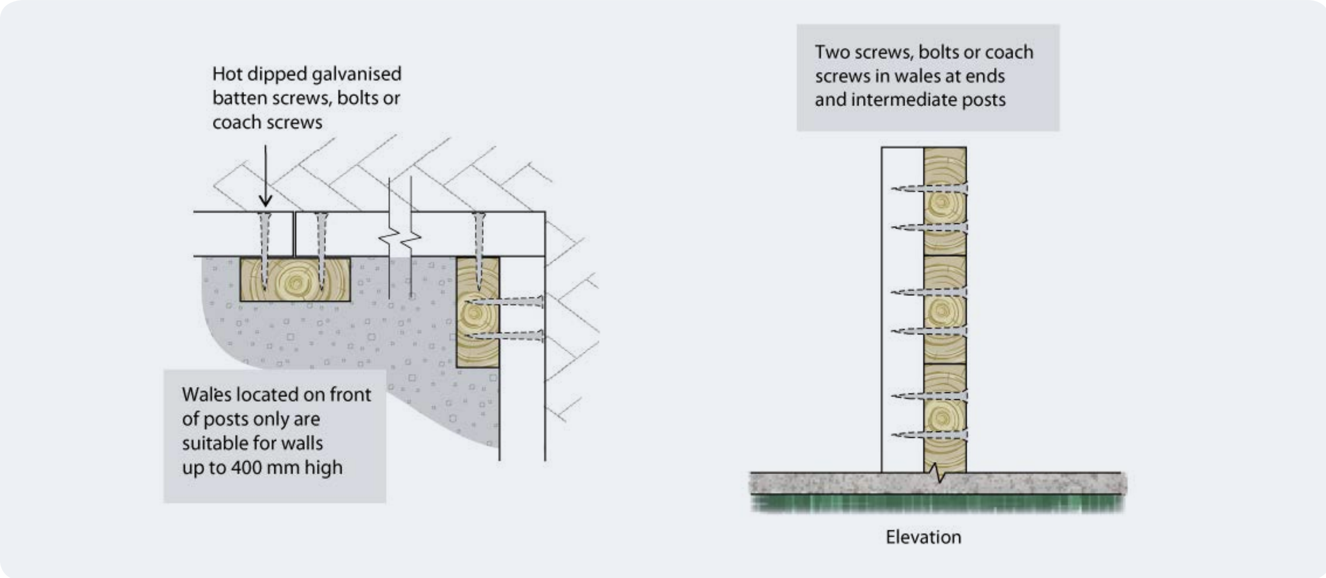


Figure 3.5 (a): Sleeper Post Arrangement (Inside Wall)

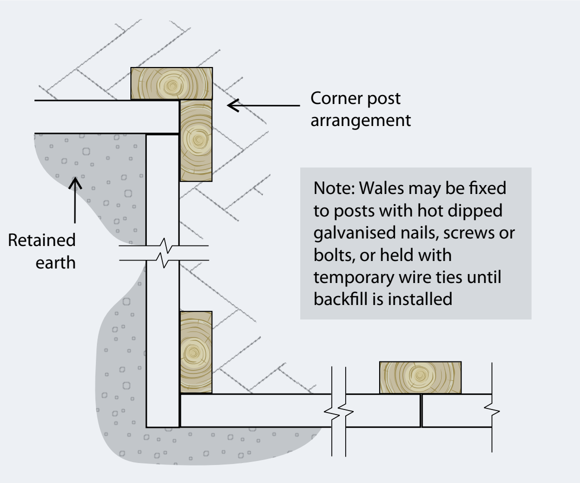


Figure 3.5 (b): Sleeper Post Arrangement (Outside Wall)

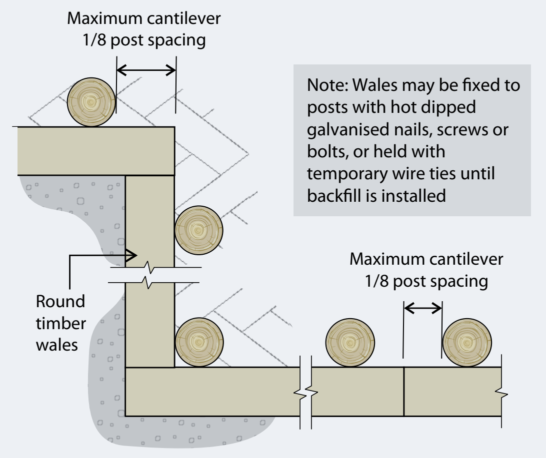


Figure 3.5 (c): Round Post Arrangement (Inside Wall)

Single sleepers and rounds can be used to form garden edges.

Garden edges can be self-supporting or may require restraining at corners and ends such as shown in Figure 3.5.

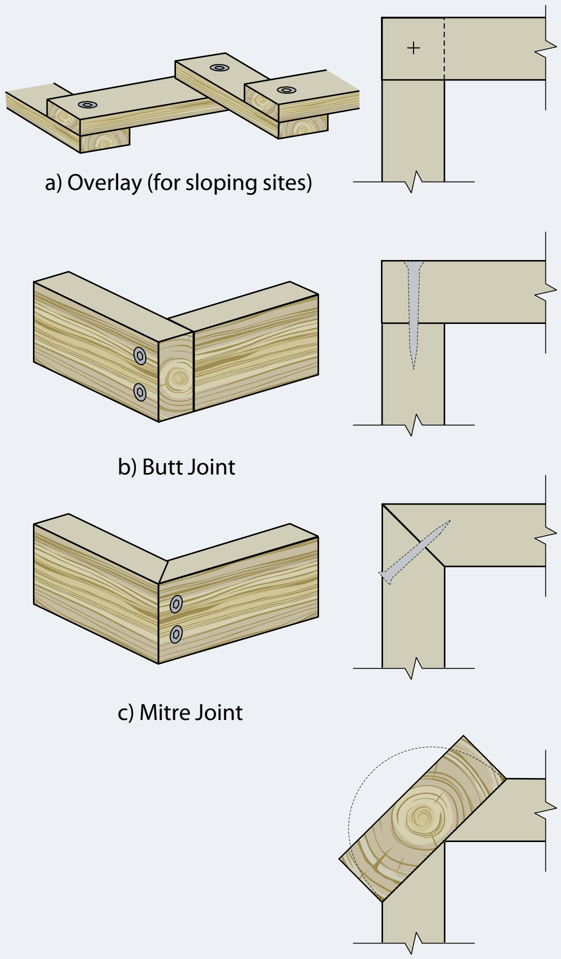


Figure 3.6: Garden Edge Arrangements

3.5 Steps

Single sleepers can be used to form steps or to terrace gently sloping ground.

Ends can be restrained by one of the methods shown in Figures 3.5 or 3.6. Alternatively, end restraint can be achieved by fixing to retaining walls as shown on Figure 3.8.

Tread options include bricks, pavers, gravel, compacted earth, or timber sleepers 'on flat'.

Note: Pine sleepers are not recommended to be used on the flat unless the piece can be securely fixed evenly across the face with at least two fixings at ends and intermediate supports.

Figure 3.7: Inadequately fixed (screws of insufficient length) resulting in distortion (twisting) of sleepers used on flat.

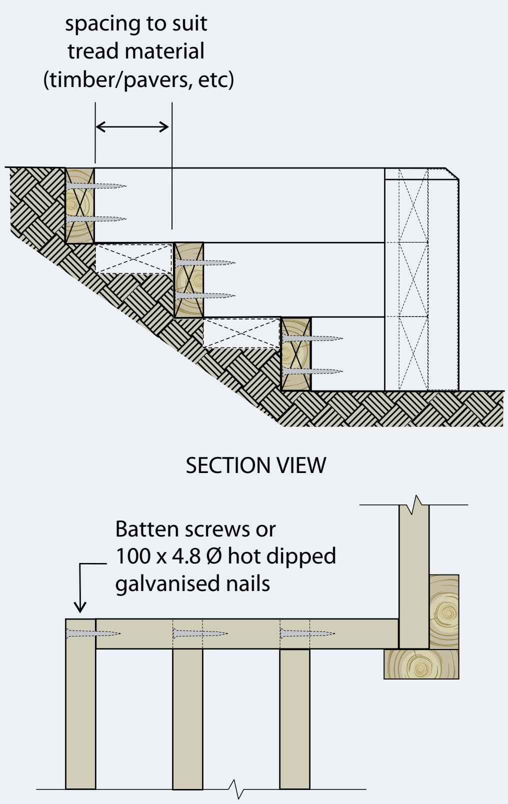


Figure 3.8: Steps

4 Maintenance

Treated timber is treated to resist termite and fungal attack only. Weathering and sun exposure without appropriate maintenance will impact on the appearance and life of treated sleepers and posts.

The appearance and longevity of timber retaining walls can be significantly enhanced by regular maintenance. The following maintenance procedures should be considered.

- All fresh cut end grain and other surfaces: Apply a liberal coating of copper naphthenate oil (CN Oil)
- Tops of posts: Apply CN oil or CN emulsion every 2 – 3 years
- Face of walls: Apply penetrating pigmented (preferably light colours) oil based stains or CN oil every 2 to 3 years. This will assist in minimising weathering of the timber and maintain visual appeal.
- Base of posts: Keep base of posts clear of any dirt, leaves and other debris. Apply a generous coating of CN emulsion around the base of the posts annually.

Figure 4.1: Treated pine retaining wall (and fence) being regularly maintained with application of timber preservative oil.

Figure 4.2: Hardwood retaining wall and fence posts being maintained with CN oil.

5 Safe Working and Disposal of Off-cuts

Working with timber produces dust particles. Protection of the eyes, nose and mouth when sanding, sawing and planing is highly recommended. Refer to tool manufacturers for safe working recommendations for particular items of equipment.

As with all treated timber, do not burn offcuts or sawdust. Preservative treated offcuts and sawdust should be disposed of by approved local authority methods.

6 References

Australian Standards

AS 1604.1 – 2012. Specification for preservative treatment, Part 1: Sawn and round timber

Additional Sources

Timber retaining walls for residential applications. Timber Queensland, March 2014

41

This guide was published in Nov 2017

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

Timber Garden Retaining Walls

Up to 1.0 m High

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions™ design and build woodsolutions.com.au

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Aveo Group, Strongbuild - Bella Vista luxury retirement community (10-storey luxury retirement village build with cross-laminated timber - CLT).

WoodSolutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-47-8

Prepared by:
Timber Queensland Ltd

Technical Reviewer:
Colin MacKenzie

First Published: November 2017

Contents

1 Introduction

Timber sleepers or round posts and logs can be used to create effective and relatively inexpensive garden edges, steps and walls to terrace sloping sites. Galvanized steel posts can also be used in conjunction with sleepers to achieve satisfactory designs. Timber fences can also be integrated with garden walls to add privacy and character to front yards.

This Guide contains recommendations for the construction of timber garden walls up to 1 metre high.

1.1 Performance

Retaining walls designed and constructed in accordance with this Guide could be expected to achieve a service life of 15 years or greater. Where a longer service life is required, hardwood timber should be limited to In-ground Durability Class 1 and softwood should be treated to H5 level.

For these more durable options, service life in excess of 25 years can be expected.

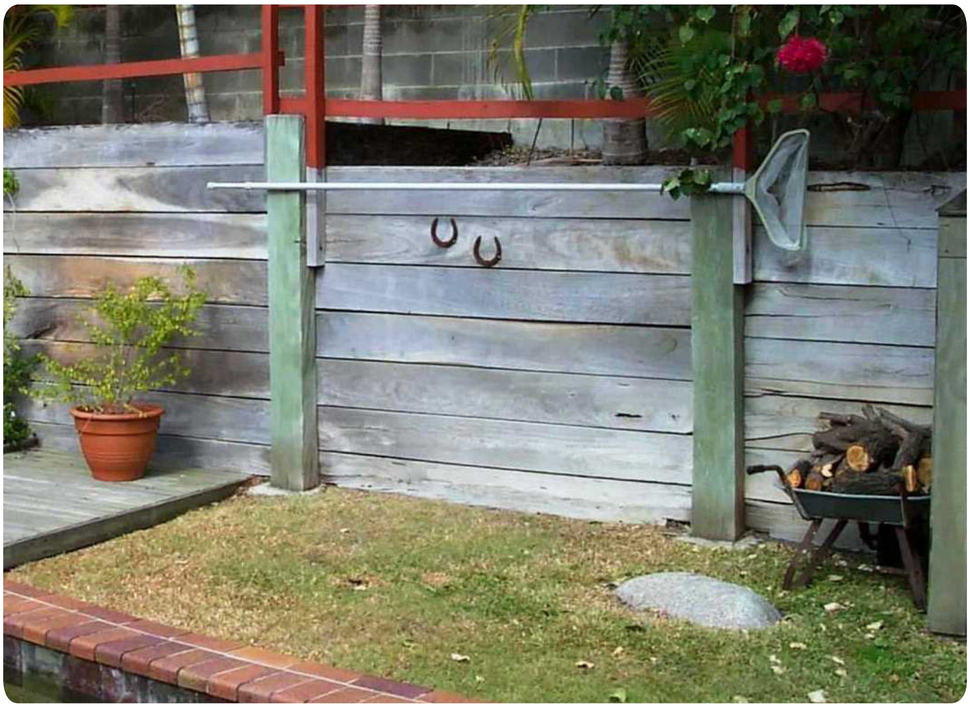


Figure 1.1: 25 year old hardwood retaining wall constructed using treated Class 1 durability posts and Class 1 and Class 2 Wales

1.2 Building Approval and Certification

Local authorities generally do not require building approval or engineer certification for retaining walls up to 1 metre high. Walls (irrespective of height) closer than 1.5 metres to a building, swimming pool, deck etc., and retaining walls greater than 1 metre high will generally require building approval and certification by an engineer.

1.3 Ground Conditions

The sizes and other recommendations in this Guide assume that posts are embedded into firm natural ground (e.g. stiff clay, shale or dense sand), with a maximum ground slope of 1 in 6. Special design may be required for steeper slopes and poor subgrade soil conditions such as uncompacted fill.

Figure 1.2: It is critical that the footing embedment of posts is adequate to prevent rotation of the posts. The treated pine rounds in this wall are still sound. The wall is failing due to inadequate embedment of posts.

2 Materials

2.1 New Timber

There is no Australian Standard or other grading requirement for landscaping timber. The quality of sawn 'sleepers' and round logs can vary greatly between different suppliers.

Timber sold for landscaping is likely to contain natural characteristics such as knots, gum veins, heart, insect damage, want, wane etc., and there could be some bow, cupping, twisting etc. Where a higher than normal appearance or performance is required, timber should be selected with limitations on the size and number of these characteristics.

Structurally graded timber (e.g. F14 hardwood and F7 treated pine) will have prescribed limits for natural characteristics and will generally be higher quality than 'landscaping' timbers.

Softwood logs and sleepers shall be preservative treated to hazard level H4 or higher.

Figure 2.1: Combination of durable hardwood posts and H4 CCA treated rounds

Hardwood should be In-ground Durability Class 1 or 2 and resistant to termites. Hardwood containing sapwood shall be preservative treated to H4 or better.

Table 2.1 provides a list of some readily available termite resistant hardwood and cypress species and their in-ground durability class.

Table 2.1: Hardwood and Cypress Durability

Species	In-Ground
Blackbutt	2
Blackbutt, Western Australian	2
Bloodwood, red and brown	1
Box, grey and grey coast	1
Cypress, white	2
Gum, red forest	1
Gum, river red	2
Gum, spotted	2
Ironbark, red and grey	1
Ironwood, Cooktown	1
Jarrah	2
Mahogany, red	2
Stringybark, Darwin	1
Wandoo	1

**Based on BCA's Provision C2.2 Table C2.2*

Figure 2.2: High quality hardwood sleepers

Figure 2.3: Low quality hardwood sleepers

2.2 Recycled Timber Railway Sleepers

Recycled hardwood railway sleepers provide another viable option for construction of timber retaining walls. When new, these sleepers are usually supplied to rail authorities under stringent specifications including species, durability and preservative treatment. Railway sleepers are much 'heavier' (larger in cross-section) than normal landscaping sleepers so may not suit use with 'off the shelf' steel post systems.

As they will have typically been in use for many decades, some existing degradation due to decay, termites and weathering can be expected. If this has severely degraded the sleepers then they will have a lesser life expectancy than equivalent material when new.

Any recycled sleepers that exhibit significant surface or possible internal degradation caused by decay or termites should be discarded. Sleepers with live termite activity should also be discarded.

Drilling the sleepers with a 4 mm diameter drill where internal degradation is suspected will reveal if any areas of concern. Drills will resist solid timber but easily penetrate decayed or termite damaged timber.

Figure 2.3: Recycled hardwood railway sleepers complete with rail bearing plates.

2.3 Steel Posts

Steel posts should be hot-dip galvanized in accordance with AS/NZ 4680:2006 - Hot-dip galvanized (zinc) coatings on fabricated ferrous articles. A minimum zinc coating thickness of 50 microns is recommended. The specification and design for steel posts should be in accordance with the manufacturer's requirements. Figure 2.4 shows typical design options using steel posts.

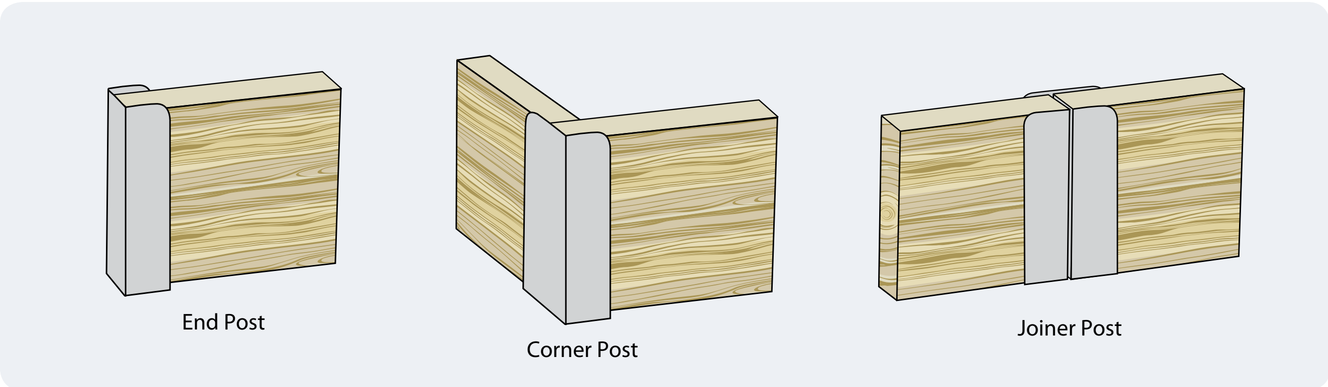


Figure 2.4: Timber Sleepers with Steel Posts

Figure 2.5: Poor quality galvanized steel 'H' section post showing corrosion approximately 8 years after installation

2.4 Preservative Treatment

AS 1604.1 – Specification for preservative treatment, Part 1: Sawn and round timber, specifies the minimum penetration and retention requirements for various preservative treatment types.

Common preservatives used to treat timber for retaining wall applications are CCA, ACQ, copper quaternary and copper azole.

For Hazard Class H4, AS 1604.1 requires the following penetration to be achieved in the timber:-

In-ground Durability Class 1 and 2 timber – The preservative must penetrate all of the sapwood. Penetration of the heartwood is not required.

In-Ground Durability Class 3 and Class 4 timber (Softwood) – For sawn timber, the preservative shall penetrate not less than 10 mm from any surface, or, unpenetrated heartwood shall comprise less than 20% of the cross section and shall not exceed 50% of the width of the face and not extend more than halfway through the piece.

For round timber, penetration shall be not less than 10 mm from the surface.

Figure 2.6 illustrates the minimum penetration requirements in sawn and round in-ground durability Class 3 and 4 timber.

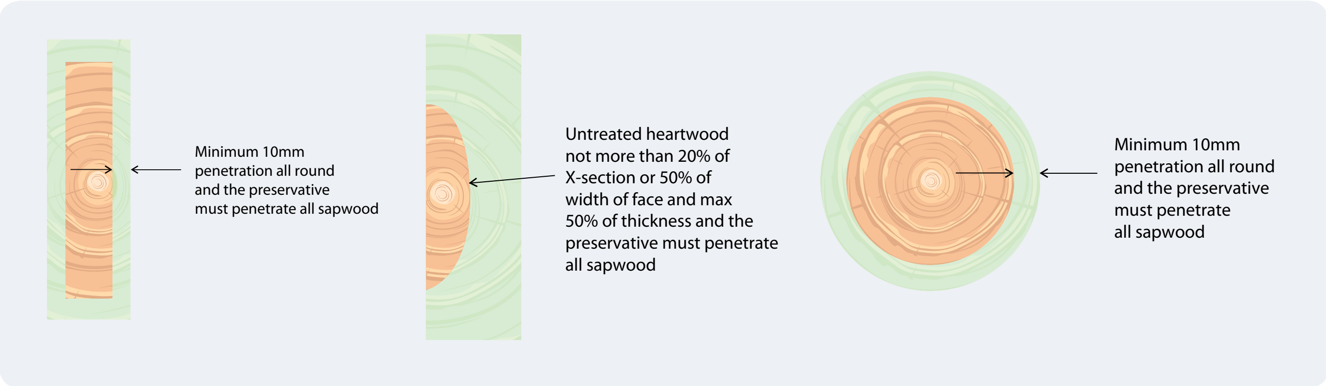


Figure 2.6: Preservative treatment Requirements for In-Ground Durability Class 3 and 4 Timber such as Softwood

Figure 2.7: Premature failure of incorrectly treated pine sleeper wall in school playground


Figure 2.6: Non-compliant CCA treated pine sleepers that failed after 5 years. Sleepers contained excessive amounts of untreated heartwood.

2.5 Corrosion

Timber that is treated with copper based preservatives such as described above, that is in contact with galvanized steel, can cause premature corrosion of the steel due to incompatibility of the copper in the preservative and the zinc coating on the steel. To minimise the potential for this to occur, the contact surfaces (timber and or steel) should be separated with a plastic or bituminous damp proof course or coated with an appropriate 'paint' such as bituminous paint.

3 Construction

3.1 General

Timber retaining walls up to one metre high can be constructed as shown in Figure 3.1.

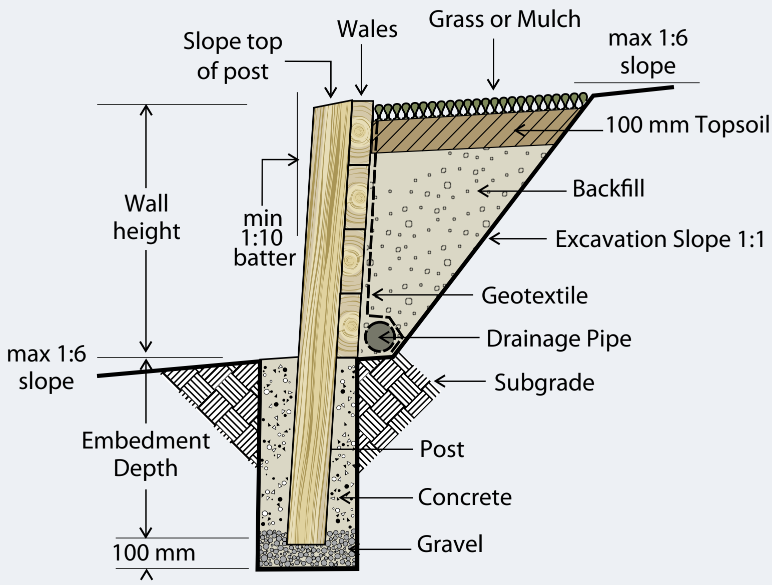


Figure 3.1: Retaining Wall Construction

3.2 Member Sizes and Embedment

Minimum member sizes and post embedment depths should be as outlined in Tables 3.1 and 3.2 for preservative treated softwood and hardwood, respectively.

Alternatives for round and sleeper post and wale arrangements are shown in Figures 3.2 (a) to 3.2 (i).

Table 3.1 – Preservative Treated Softwood (SWD)

Table 3.1: Preservative Treated Softwood (SWD)

Member	Refer Figure	Post Spacing	1200				1500				2400			
		Wall Height	**400**	**600**	**800**	**1000**	**400**	**600**	**800**	**1000**	**400**	**600**	**800**	**1000**
Sleeper Posts	3.2 (a)	**Size (bxd)**	150x50	150x75	200x75	N/A	150x150	150x75	200x75	N/A	200x100	200x100	N/A	N/A
		Hole Dia. (Ø)	300	300	450	N/A	300	300	450	N/A	300	450400	N/A	N/A
		Depth	400	600	600	N/A	400	600	650	N/A	400	600	N/A	N/A
Slab Posts	3.2 (b)	**Thickness (t)**	90	90	90	125	90	90	90	125	100	100	125	2/125
		Hole Diam.	300	300	300	300	300	300	300	300	300	300	300	450
		Depth	400	600	600	750	400	600	650	750	400	600	900	950
Single Round Posts	3.2 (c)	**Post Dia. (Ø)**	100	125	150	180	100	125	175	200	100	150	200	225
		Hole Dia. (Ø)	300	300	300	300	300	300	300	450	300	300	450	450
		Depth	400	600	800	1000	400	600	800	1000	600	800	1000	1400
Double Round Posts	3.2 (d)	**Post Dia. (Ø)**	100	100	125	150	100	100	150	175	100	125	175	200
		Hole Dia.	300	300	300	300	300	300	300	450	300	300	450	450
		Depth	400	600	800	1000	400	600	800	1200	600	800	1000	1400
Sleeper Wales	3.2 (e)	**Size (bxd)**	150x50	150x50	200x50	200x50	150x50	150x50	200x50	200x75	200x100	200x100	200x100	200x100
Slab Wales	3.2 (f)	**Thickness (t)**	90	90	90	90	90	90	90	90	90	100	100	125
Round Wales	3.2 (g)	**Dia. (Ø)**	75	75	75	75	75	75	75	75	100	100	100	100
Split Wales (1/2 round)	3.2 (h)	**Dia. (Ø)**	75	75	100	125	75	100	125	N/A	N/A	N/A	N/A	N/A
Winged Split Wales	3.2 (i)	**Thickness (t)**	100	100	100	100	100	100	100	100	N/A	N/A	N/A	N/A

Table 3.2: Hardwood (HWD) (refer Figures 3.2 (a) and 3.2 (e))

Member	Wall Height	400	600	800	1000
Posts	**Post Spacing 1200**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	800	1000
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50
Posts	**Post Spacing 1500**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	800	1200
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50
Posts	**Post Spacing 2400**				
	Size (bxd)	200 x 50	200 x 75	200 x 75	200 x 100
	Hole Dia.	300	300	300	300
	Depth	400	600	1000	1400
Wales	Size (bxd)	200 x 50	200 x 50	200 x 50	200 x 50

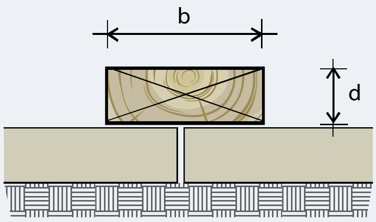


Figure 3.2 (a): Sleeper posts (HWD & SWD)

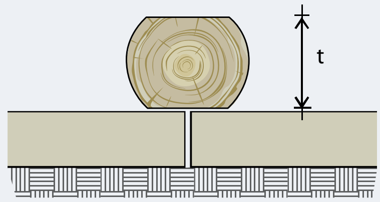


Figure 3.2 (a): Sleeper posts (HWD & SWD)

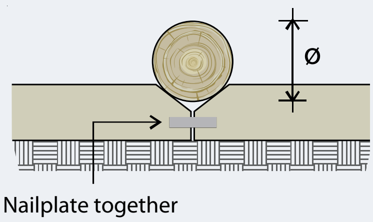


Figure 3.2 (c): Single round post (SWD)

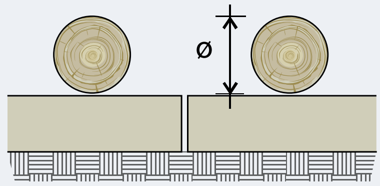


Figure 3.2 (d): Double round posts (SWD)

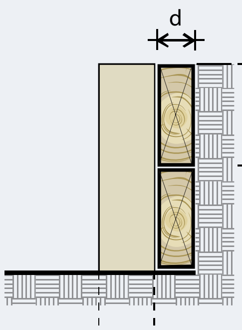


Figure 3.2 (e): Sleeper wales (HWD & SWD)

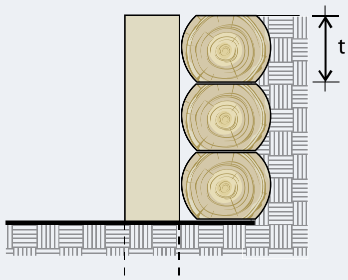


Figure 3.2 (f): Slab wales (SWD)

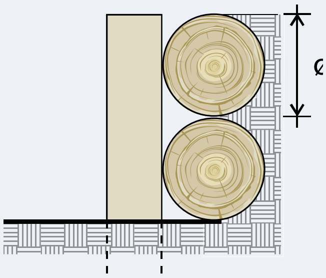


Figure 3.2 (g): Round wales (SWD)

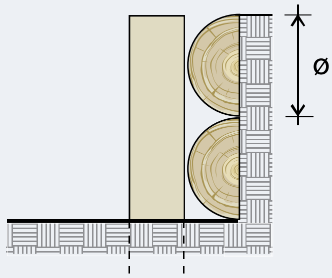


Figure 3.2 (h): Split wales (SWD)

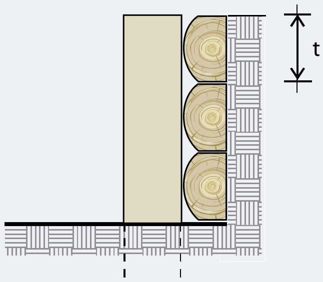


Figure 3.2 (i): Winged split wales (SWD)

Walls exceeding 400 mm high should have a 1:10 batter as shown in Fig 3.1.

The batter allows for the wall to have some lateral movement and still appear stable.

Post holes should have a 100 mm layer of coarse gravel installed prior to placing posts and backfilling with concrete.

'No-fines' concrete is recommended for hardwood posts.

Walls up to 400 mm high may have concealed posts, with wales fixed to the outside of posts with hot dipped galvanised batten screws, coach screws or bolts (refer Fig 3.3a). Walls above 400 mm shall have wales behind posts (refer Fig 3.3b). Typical methods of positioning posts at ends and corners are shown on Figure 2.

Wales may require temporary fixing to posts with wire ties, skew nails or batten screws.

The top of posts should be cut with a slight slope to shed water and should be sealed with a preservative emulsion.

Figure 3.3: Sloping cut to top of ironbark retaining wall post in this 23 year old wall

3.3 Drainage

Drainage should be provided behind walls exceeding 400 mm high to prevent additional loads due to the build-up of ground water. A slotted polyethylene drainage pipe should be provided at the base of the wall, discharging to a suitable outlet. To prevent backfill material from flowing through gaps in the wall and to assist in drainage, geotextile should be placed to the inside face of wales and between the drainage pipe and the backfill. See Figure 3.1.

Backfill should be a free flowing material such as ashes, sand or gravel– not clay. Backfill should be installed no sooner than three days after concreting posts.

Figure 3.4: Well designed and constructed treated pine retaining wall. Wall is battered and tops of posts have angle cut to shed water

3.4 Retaining Wall Posts and Garden Edges

Typical corner and end restraints are shown in Figure 3.5 (a) to 3.5 (c). Alternatively, posts can be positioned as shown on Figure 3.xx.

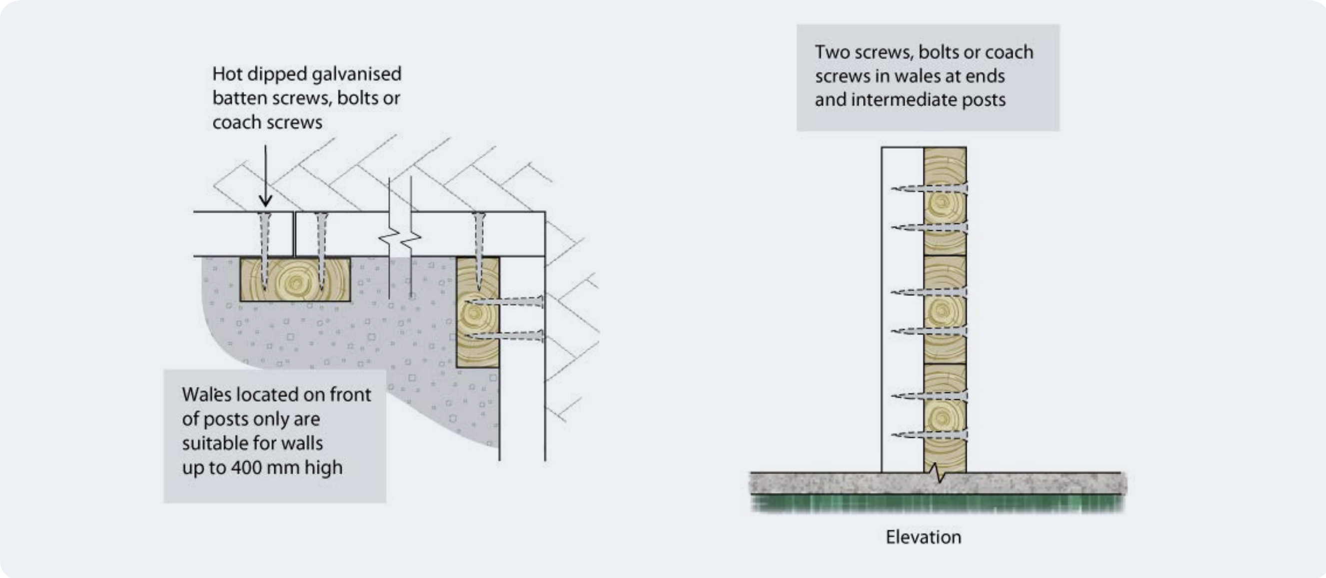


Figure 3.5 (a): Sleeper Post Arrangement (Inside Wall)

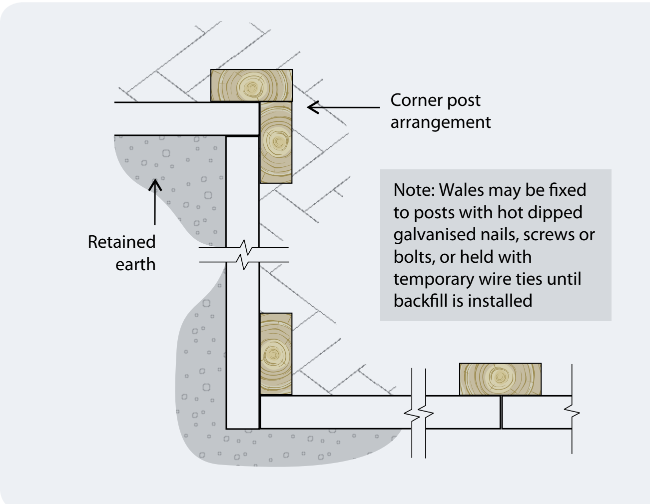


Figure 3.5 (b): Sleeper Post Arrangement (Outside Wall)

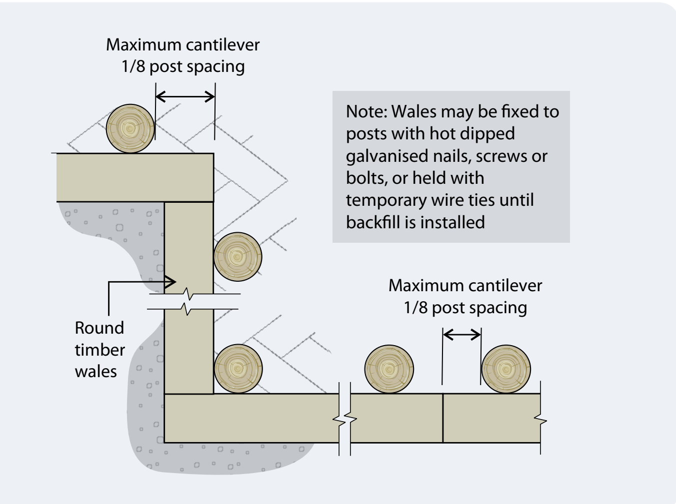


Figure 3.5 (c): Round Post Arrangement (Inside Wall)

Single sleepers and rounds can be used to form garden edges.

Garden edges can be self-supporting or may require restraining at corners and ends such as shown in Figure 3.5.

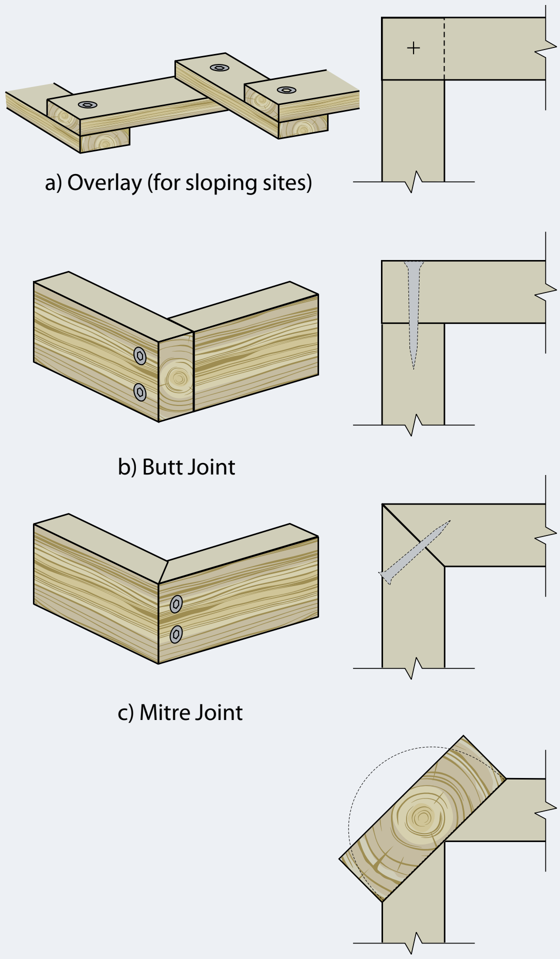


Figure 3.6: Garden Edge Arrangements

3.5 Steps

Single sleepers can be used to form steps or to terrace gently sloping ground.

Ends can be restrained by one of the methods shown in Figures 3.5 or 3.6. Alternatively, end restraint can be achieved by fixing to retaining walls as shown on Figure 3.8.

Tread options include bricks, pavers, gravel, compacted earth, or timber sleepers 'on flat'.

Note: Pine sleepers are not recommended to be used on the flat unless the piece can be securely fixed evenly across the face with at least two fixings at ends and intermediate supports.

Figure 3.7: Inadequately fixed (screws of insufficient length) resulting in distortion (twisting) of sleepers used on flat.

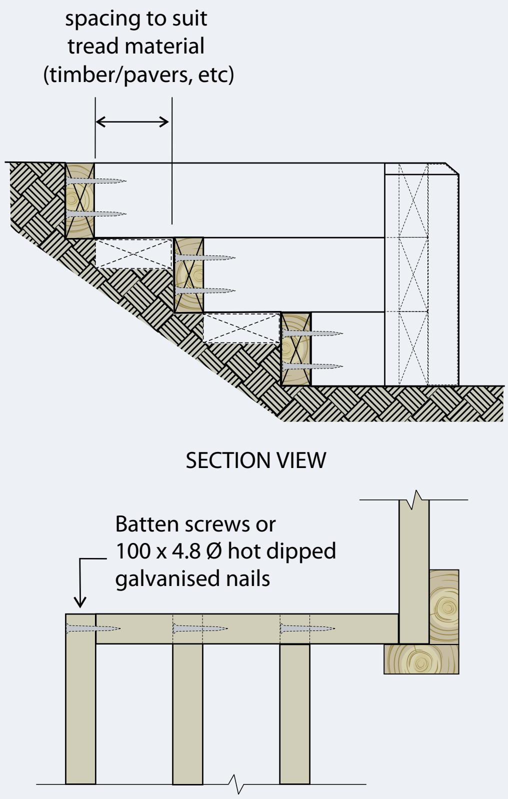


Figure 3.8: Steps

4 Maintenance

Treated timber is treated to resist termite and fungal attack only. Weathering and sun exposure without appropriate maintenance will impact on the appearance and life of treated sleepers and posts.

The appearance and longevity of timber retaining walls can be significantly enhanced by regular maintenance. The following maintenance procedures should be considered.

- All fresh cut end grain and other surfaces: Apply a liberal coating of copper naphthenate oil (CN Oil)
- Tops of posts: Apply CN oil or CN emulsion every 2 – 3 years
- Face of walls: Apply penetrating pigmented (preferably light colours) oil based stains or CN oil every 2 to 3 years. This will assist in minimising weathering of the timber and maintain visual appeal.
- Base of posts: Keep base of posts clear of any dirt, leaves and other debris. Apply a generous coating of CN emulsion around the base of the posts annually.

Figure 4.1: Treated pine retaining wall (and fence) being regularly maintained with application of timber preservative oil.

Figure 4.2: Hardwood retaining wall and fence posts being maintained with CN oil.

5 Safe Working and Disposal of Off-cuts

Working with timber produces dust particles. Protection of the eyes, nose and mouth when sanding, sawing and planing is highly recommended. Refer to tool manufacturers for safe working recommendations for particular items of equipment.

As with all treated timber, do not burn offcuts or sawdust. Preservative treated offcuts and sawdust should be disposed of by approved local authority methods.

6 References

Australian Standards

AS 1604.1 – 2012. Specification for preservative treatment, Part 1: Sawn and round timber

Additional Sources

Timber retaining walls for residential applications. Timber Queensland, March 2014

42

This guide was revised in Oct 2020

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

Residential Care Buildings (Class 9c) Timber-based Deemed-to-Satisfy Solutions

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions™ design and build woodsolutions.com.au

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Aveo Group, Strongbuild - Bella Vista luxury retirement community (10-storey luxury retirement village build with cross-laminated timber - CLT).

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-45-4

Prepared by:
Timber Development Association
6/281 Pacific Highway
North Sydney, 2060

First Published: December 2017,
Revised: May 2018, October 2020

Contents

Contents

1 Introduction

Australia's population is ageing and demand for assisted-living accommodation is increasing. The Aged Care Financing Authority found that *"an additional 74,000 new beds will be required over the coming decade to meet the needs of an ageing Australia"*.

Residential Care Class 9c buildings are generally low- to mid-rise in height, which is ideally suited to timber construction. The regulatory requirements for this building classification differ from Class 2 (apartments) or 3 (hotels, etc) buildings covered under WoodSolutions Guide No 2[1] and 37R[2].

This Guide provides information and options to comply with the Building Code of Australia (BCA)[3] using timber structural systems for Class 9c Residential Care buildings. It discusses the various BCA's Deemed-to-Satisfy (DTS) solutions available, focusing on the fire and acoustic requirements, as well as recommending timber-based design and construction solutions to meet these requirements.

2 Guide Structure

The BCA provides more than one Deemed-to-Satisfy fire solution for meeting the Performance Requirements of a Class 9c Residential Care building. Each solution affects the viability of the range of timber-based solutions.

This Guide is divided into two parts:

1. Discussion of Deemed-to-Satisfy fire and acoustic solutions – Sections 3 to 5.

2. Timber design and construction solutions that meet the variety of regulatory compliance options – Section 6.

3 Regulatory Compliance

This section discusses the BCA's Deemed-to-Satisfy Provisions[3] for fire and acoustics, provides interpretation for some of these requirements and explains when they are needed. Requirements for Class 9c buildings other than fire and acoustic are not discussed within this Guide and readers should consult the BCA for other elements that make up a building design.

Words used in this guide that are in italics and in bold have a definition prescribed in the BCA. Further explanation of these terms are found in the Appendix A of this guide or the BCA.

3.1 Fire-resisting DTS Requirements Building Code of Australia (BCA)

The Building Code of Australia has three distinct fire Deemed-to-Satisfy solutions for a Class 9c residential care buildings: specifications for various internal walls and shafts; for external walls close to the property's boundaries or another building; and for floors and roofs.

Some Deemed-to-Satisfy solutions are optional; the designer needs to choose one method and stay with it. Each method will have advantages and disadvantages for the use of timber, or the type of timber system used. The three optional Deemed-to-Satisfy solutions are:

- **BCA Specification C1.1 Method:** This method follows the traditional fire-resisting methodology used for most building types, using the BCA's Specification C1.1 provisions. This method is generally applicable to low-rise Type C buildings.
- **Fire-Protected Timber Method:** This method follows the traditional Specification C1.1 provision as detailed in the first method, but utilises the concession within the specification that allows the replacement of non-combustible building elements with fire-protected timber elements. Structural timber elements that are encapsulated in a non-combustible fire-protective covering will meet this requirement. There are no limits on the Type of Construction or Rise in Storey for this method.
- **BCA Provision C2.5 Class 9c Building**: This method uses the specific provisions within the BCA for Class 9c buildings. Within this provision, there are fire-resistance requirements that are optional to the traditional Specification C1.1 method. There are no limits on the Type of Construction or Rise in Storey for this method.

These specifications vary, depending on the BCA's Type of Construction and Rise in Storey as well as additional or alternative requirements described in BCA's Provision. Some BCA Provisions allow for more than one option. Where this occurs, these options will be discussed in the relevant section of this Guide.

Section 4 discusses these requirements in relation to the building's Type of Construction. Discussion of building solutions for these requirements are found in Section 6.

The BCA has three building classifications that provide for residential care:

- ***Class 3 residential care buildings intended for accommodation of the aged, children or people with disabilities who are not related to each other and the term of the stay is not specified.***
- ***Class 9a residential care buildings are public health-care buildings that contain a residential care part. They are intended for aged occupants who require a high degree of care, i.e. nursing homes.***
- ***Class 9c residential care buildings are buildings intended for resident care only. They are intended for persons with varying care needs, i.e. low to high care. It is also intended to allow the level of care to change as the residents' needs change over time.***

Consideration of the appropriate Class is required as construction of Class 3 or 9a buildings may restrict the profile of the residents who can be accommodated as well as affect what structural timber is the appropriate solution.

3.2 Type of Construction

The fire resistance of a building is determined by the Type of Construction (BCA Provision C1.1) and the Type of Construction is dependent on the building classification, Class 9c in this case, and the Rise in Storey. The BCA has three Types of Construction, A, B and C, where Type C has the lowest fire resistance requirement and Type A the highest. The Rise in Storey for a building is a BCA-defined term meaning the greatest number of storeys at any part of the external wall of the building (BCA Provision C1.2). Table 3.1 provides the Type of Construction for the Rise in Storey for a Class 9c building.

Table 3.1: BCA's Types of Construction Limits

Rise in Storey	Type of Construction
Single Storey	C
Two storey concession*	C
Two storey	B
Three storey and above	A

**Based on BCA's Provision C1.5*

3.2.1 Two Storey Type C buildings

The BCA Provision C1.5 allows a building with a Rise of Storey of two to be considered as a Type C construction, if it has sprinklers (BCA Specification E1.5 only) throughout, and the maximum compartment size is no larger than 3,000 m^2 or 18,000 m^3 (see Table 3.2). Therefore, as sprinklers are always required for Class 9c building, as discussed in Section 3.3, irrespective of the Type of Construction, this concession will have considerable savings for timber construction when compared to Type B requirements.

For two-storey Class 9c buildings, the concession that allows Type C construction may have savings over solutions that meet Type B construction.

3.3 Sprinklers

The BCA Provision E1.5 (Table E1.5) requires a sprinkler system to be installed throughout the building, for any Rise in Storeys or Type of Construction.

NSW and Victoria have additional requirements for sprinklers such as Victoria requiring sprinklers to be installed in covered balconies (Victoria's Specification E1.5 Clause 2 (b)) and NSW requiring their own fire sprinkler standard[4] to be used. If building within these regions, refer to the State variations in conjunction with this Guide.

The inclusion of sprinklers gives several concessions to the fire resistance requirements, such as Fire Hazard Properties, discussed in Section 4.5.

4 Fire Resistance Deemed-to-Satisfy Details

Fire resistance requirements for a Class 9c building vary due to Rise in Storey, various specific provisions and concessions within the BCA, state variations, location or purpose within the structure and if the building element is loadbearing or non-loadbearing or used as shafts. The following discusses each requirement.

4.1 Fire Compartments (BCA Provision Spec C1.1 Table 5)

Table 4.1 describes the maximum area or volume of a building for each Type of Construction allowed under the BCA. Compartments that are greater than the area or volume described in Table 4.1 must be divided by a fire wall so that each area or volume doesn't exceed the maximum allowed for the Type of Construction. A fire wall is a wall that divides a storey or building into fire compartments and, depending on the Type of Construction, it will have a Fire Resistance Level (see Table 4.1).

Table 4.1: Type of construction compartment size limit and the fire wall FRL requirements.

Type of Construction	Maximum Fire Compartment Size		Fire Wall Fire Resistance Level
	Area m^2	Volume m^3	
C	3,000	18,000	90/90/90
B	5,500	33,000	120/120/120
A	8,000	48,000	120/120/120

4.2 BCA Provision Clause C2.5 (b) Specific Requirements

This is a BCA Provision specifically for Class 9c buildings and must be included in all design solutions. The clause has explicit requirements for the placement of Smoke-proof Walls as well as minimum fire resistance requirements for floors and an optional internal wall solution. These requirements vary from the general fire requirements within the BCA's Specification C1.1 and because they are specific requirements they have precedence over Specification C1.1.

Where the BCA gives specific requirements such as Provision C2.5, these requirements have precedence over general requirements such as Specification C1.1.

4.2.1 Smoke-proof Walls

Smoke-proof walls (BCA Provision C2.5) must be installed so that they divide the building into areas of not more than 500 m^2 and separate Ancillary Use Areas.

Ancillary Use Areas are areas that contain equipment or materials that have a high potential fire hazard, including:

- a kitchen and related food preparation areas with a combined floor area of more than 30 m^2
- a laundry, where items of equipment are of the type that are potential fire sources (e.g. gas-fired dryers)
- a storage room with a floor area of more than 10 m^2 that is used predominantly for the storage of administrative records.

Section 6.3.1 gives an example and information on timber systems and construction practices.

4.2.2 Floors

BCA's Provision C2.5 (b) (ii) requires all floors that are explicitly separating fire compartments within the building or fire compartment to have a minimum Fire Resistance Level of 60/60/60. As this requirement is 'specific' as compared to the 'general' fire resistance requirements of the BCA's Specification C1.1 Section 3, 4 and 5, the Provision C2.5 (b) (ii) must be followed.

This BCA Provision mainly affects Type B and C buildings, requiring that these floors have a Fire Resistance Level of 60/60/60. Under BCA Specification C1.1 Section 4 and 5, these floors may require no fire resistance or Fire Resistant Level of less than 60 minutes or another method such as fire protective covering or ceiling with an Incipient Spread of Fire rating.

This provision will also affect the concessions for mezzanine floors over storage areas not considered a storey, and floors above a space for the accommodation of motor vehicles or used for storage or any other ancillary purpose. These floors will also need to have a minimum Fire Resistance Level of 60/60/60.

For Type A construction, the Fire Resistance Level is 120/120/120. The BCA Specification C1.1 Clause 3.2 (a) and (b) excludes floors laid directly on the ground or above a space that is not a storey, garage, storage or other ancillary purposes. Where these floors support fire resistance construction, the BCA Specification C1.1 Clause 2.2 Fire protection for support of another part may transfer fire resistance onto these floors.

4.2.3 Internal Walls

The BCA Provision C2.5 (b) (iii) has an alternative fire resistance solution for internal walls other than those bounding lift and stair shafts. As it is an alternative solution to BCA Specification C1.1, it is discussed in more detail in Section 4.3.1.

4.3 Detailed Fire Resistance Requirements Including Options

The following specifies the fire-resisting requirements for a design to BCA Specification C1.1. It contains the fire resistance requirements needed for a Class 9c Residential Care building. For some elements, the BCA has more than one solution and, where this occurs, the options are discussed. Where there is only one solution for a building element, it implies there is only one method available.

4.3.1 Internal Walls

The fire resistance of an internal wall of a Class 9c building has more than one solution within the BCA. The following discusses the various solutions available.

BCA Specification C1.1 method

The fire resistance requirement for internal walls varies due to the Type of Construction, whether the walls are loadbearing or non-loadbearing, and the purpose of the room they bound. Table 4.2 describes the various fire resistance requirements for each wall type, construction type, and whether loadbearing or not. Internal walls that bound public corridors or lobbies, between or bounding Sole Occupancy Units (SOUs) and for shafts, except for fire-resisting lift and stair shafts (see Section 4.3.3).

Table 4.2: External Wall Fire resisting ratings

Element	**Load Bearing**[1]				**Non-Load Bearing**			
Type of Construction	**Type A**	**Type B**	**Type C Double storey**[2]	**Type C Single storey**	**Type A**	**Type B**	**Type C Single storey**	**Type C Double storey**
Bounding public corridors or lobbies	120/-/-	120/-/-	60/-/-	-/-/-	-/-/-	-/-/-	-/-/-	-/-/-
Between or bounding ***SOU***s	120/-/-	120/-/-	60/-/-	-/-/-	-/-/-	-/-/-	-/-/-	-/-/-
Shafts except fire resisting lift and stairs shafts	120/90/90	60/-/- Note 1	60/-/-	-/-/-	-/-/-	-/-/-	-/-/-	-/-/-
Other load-bearing walls	120/-/-	120/-/-	60/-/-	-/-/-	-/-/-	-/-/-	-/-/-	-/-/-

Notes:

1. Loadbearing shafts vary from Specification C1.1 Table 3.3 requirements as they may support the floor that has a fire-resistance requirement of 60/60/60.

2. If the BCA Provision concession C1.5 is used to obtained two-storey Type C construction, then the walls used to support the floor will be required to be fire resistant (60/–/–) due to the floors required to be 60/60/60 and the BCA's Specification C1.1 2.2 Support of Another Part transferring this requirement to the floor's support structure.

For Type A and B construction, the loadbearing walls must also be constructed from concrete, masonry or fire-protected timber. A way to circumvent the use of loadbearing walls within a structure is to utilise a post and beam framed or post and plate solution. Timber columns, beams and floors do not have a non-combustible requirement within the BCA. The non-combustibility requirements for floor, beams and columns may only come about when they support a loadbearing or fire resistance wall, as the BCA's Specification C1.1 Clause 2.2 Fire Protection for a Support of Another Part transfers the non-combustible requirement to the support structure.

The use of a post and beam framed or post and plate solution removes all internal loadbearing walls, and for Class 9C construction, there is no fire resistance requirement for most internal non-loadbearing walls (BCA Specification 3.1 (e) and 4.1 (f)). The exception is a fire-resisting lift and stair and service shafts where these shafts can be of fire-protected timber (discussed below), concrete, masonry or non-combustible construction to remain as a Deemed-to-Satisfy solution.

This fire-protected timber solution is only variable when the building has an effective height less than or equal to 25 m. Refer to Appendix A for more information on Fire-Protected Timber.

The use of a timber post and beam or post and plate structural system removes the need for loadbearing walls and therefore the requirements to be non-combustible or concrete and masonry. Timber columns, beams and floors have no requirement to be non-combustible unless they support an element that is required to be non-combustible.

Furthermore, there are no Fire Resistance Level requirements for all non-loadbearing walls for all Types of Construction, potentially reducing construction costs.

Internal walls Fire-Protected Timber method

For this solution, follow BCA Specification C1.1 as discussed above. For Type A and B construction, the loadbearing walls may be constructed from fire-protected timber, structural timber elements encapsulated in non-combustible insulated linings. There are limits on the extent Fire-Protected Timber can be used, such as the Rise in Storey, association construction. These limits are discussed in Appendix A.

Internal walls BCA Provision C2.5 (b) method

The BCA Provision C2.5 (b) (iii) has an alternative solution to its Specification C1.1 for the internal walls, other than those bounding lift and stair shafts. Provision C2.5 (b) (iii) requires all internal walls, other than those bounding lift and stair shafts, to have a minimum Fire Resistance Level of 60/-/-. The walls must be supported by the floor specified in BCA Provision C2.5 (b) (ii) (see Section 4.2.2). This solution states that all internal walls, other than those bounding lift and stair shafts, are to be fire-resistant, even walls that under BCA Specification C1.1 would not have any fire resistance, such as non-loadbearing walls bounding SOUs or partition walls within an SOU.

Bounding walls to lift and stair shafts must follow the BCA's Specification C1.1 requirements, discussed in Section 4.3.3.

The sub-compartmentation caused by all internal walls required to have fire resistance and the addition of smoke proof wall allows for the staged evacuation of residents from the building. The lower fire resistance allowed by BCA's Provision C2.5(b)(iii) also recognises the effectiveness of the required sprinkler systems in Class 9c buildings.

NSW variation

NSW has a variation to the BCA Provision C2.5 that changes the requirement for internal walls and these changes are summarised below.

For Type A, B and C buildings, all non-loadbearing internal walls between Sole Occupancy Units and bounding a public corridor in a residential part of the building must be lined on each side with 13 mm standard grade plasterboard or material with at least an equivalent level of fire protection. Where cavity insulation is used, the insulation is to be non-combustible. The exceptions where this does not apply are fire walls and smoke-proof walls.

These walls must extend to the underside of the floor above or a ceiling lined with 13 mm standard grade plasterboard or a non-combustible roof covering. Also, any penetrations above the door head height, construction joint, space between the top of the wall and the floor, ceiling or roof must be sealed to prevent smoke transfer.

Loadbearing internal walls for Type A and B must comply with the requirements of Specification C1.1 (see Section 4.3.1). For Type C construction, loadbearing walls between Sole Occupancy Units and bounding a public corridor in a residential part of the building must be lined and sealed as if they were non-loadbearing, as discussed above. Where there is no fire resistance required, any penetrations above the door head height, construction joint, space between the top of the wall and the floor, ceiling or roof must be sealed to prevent smoke transfer.

4.3.2 Walls Immediately Below the Roof

The BCA has a concession for internal wall immediate below the roof. It is applicable for internal walls using the BCA Specification C1.1 method and fire-protected timber method. It is not appropriate for internal walls using the BCA Provision C2.5 (b) method, as all the internal walls are required to be fire-resistant to 60/–/-.

For Type A construction (BCA Specification C1.1 3.7) that have an effective height of less than 25 m and the roof has no fire resistant requirements, the internal walls immediately below the roof, other than fire wall and shafts, may be reduced to:

- with Rise in Storeys exceeding 3: FRL 60/60/60
- with Rise in Storeys not exceeding 3: no FRL.

For Type B construction, BCA Specification C1.1 4.1 (g) provides a concession for internal walls, other than fire walls and shaft walls, removing the need for these walls to be fire-resistant.

4.3.3 Fire-resisting Lift and Stair Shafts

Stair Shafts

All stairways that are essential to exit the building must be fire-isolated (BCA Provision D1.3 (b)).

For a Type A and B buildings, the shaft walls are required to be:

- loadbearing: 120/120/120
- non-loadbearing: –/120/120.

In addition, loadbearing shaft walls must be constructed from concrete, masonry or fire-protected timber.

For Type C buildings, only if a stair is required, i.e. when the two storeys concession is used and if the stair is a required exit, the shaft wall is to be fire rated to 60/60/60 (BCA Specification C1.1. Table 5).

The stairway and landings contained in the fire-resistant shaft must be constructed from non-combustible materials or meet the BCA concession for timber stairways (BCA Provision D2.25). It must also be designed so that if there is a local failure, it will not cause structural damage or impair the fire-resistance of the shaft (BCA Provision D2.2).

The BCA's timber stairways concession (BCA Provision D2.25) requires the stairs to be constructed from the following:

- have the timber finished thickness of not less than 44 mm
- have an average density of not less than 800 kg/m^3 at a moisture content of 12%
- the building is protected throughout by a sprinkler system (other than an FPAA101D system) complying with Specification E1.5 that extends to within the fire-isolated enclosure
- fire protection being provided to the underside of stair flights and landings using one layer of 13 mm fire-protective grade plasterboard.

Lift Shafts (BCA Provision C2.10 b)

A lift shaft in the residential area of a Class 9c building must have a shaft with a Fire Resistance Level of:

- Type A and B construction: 120/120/120
- Type C construction: 60/60/60.

4.3.4 External Walls

The fire resistance of external walls depends on the distance from a fire-source feature (another building or boundary), the building's Type of Construction, and whether they are loadbearing or not. Table 4.3 lists numerous Fire Resistance Levels for various distances from a fire-source feature and if the wall is loadbearing or non-loadbearing.

BCA Provision C2.5 does not have any special or concessional requirements for external walls and relies on the requirements contained within BCA Specification C1.1.

Type C construction has a concession that allows the external walls to be fire rated from the outside direction only (BCA Specification C1.1 Clause 5.1 b).

Table 4.3: External Wall Fire resisting ratings

Distance to fire source feature	Load Bearing			Non-Load Bearing		
	Type A	Type B	Type C	Type A	Type B	Type C
<1.5 m	120/120/120	120/120/120	90/90/90	-/120/120	-/120/120	-/90/90
1.5 m to <3 m	120/90/90	120/90/60	60/60/60	-/90/90	-/90/60	-/60/60
3 m to <9 m	120/60/30	120/30/30	-/-/-	-/-/-	-/-/-	-/-/-
9 m to <18 m	120/60/30	120/30/ -	-/-/-	-/-/-	-/-/-	-/-/-
18 m plus	120/60/30	-/-/-	-/-/-	-/-/-	-/-/-	-/-/-

For Type A and B buildings, BCA Provision C1.9 (a) requires all components that make up the external wall, including the cladding, to be constructed from non-combustible materials. The BCA Provision C1.13 concession allows any component required to be non-combustible to be substituted by fire-protected timber, refer to Appendix A.

Another way to remove the non-combustible requirement for a Type B buildings is to convert the building classification to Type C, by using the concession given in the BCA Provision C1.5. This concession has no non-combustibility requirements for external walls.

For Type C buildings, the BCA has no non-combustibility requirements for external walls, and where a wall is required to be fire-resisting, only have the Fire Resistance Level from the outside.

Vertical separation of openings in external walls (BCA Provision C2.6)

For Type A buildings there is an additional requirement in external walls. Any window or other opening in that wall that is above another opening in the storey below will require protection from the spread of fire via the facade (BCA Provision C2.6 (a)). However, as the building is required to be fully sprinkled, this requirement is removed by BCA Provision C2.6 (b) (iii).

There are no requirements for separation of opening in a Type B and C buildings.

4.3.5 Columns

The BCA has provisions for both internal and external columns, in and around a building. The requirements vary depending on the Type of Construction and location within the building.

Internal columns, except top most floor

Internal columns, except for the topmost floor, used within a Type A and B buildings are required to have fire resistance in accordance with Table 4.4. For Type C buildings, there is no fire-resistance, except where two-storey Type C buildings are used. In this case, they may support a fire-resistance floor and will inherit fire-resistance from the floor, due to BCA's Specification C1.1 Clause 2.2 Fire Protection for a Support of Another Part (see Table 4.4).

Table 4.4: Fire Resistant Levels for internal columns.

	Type A	Type B	Type C Double-storey[1]	Type C Single storey
Internal Column	120/-/-	120/-/-	60/-/-	-/-/-

Note:

1. If the BCA concession C1.5 is used to obtained two-storey Type C construction, then any columns used to support the floor will be required to be fire resistant (60/ – / –) due to the floors required to be 60/60/60 and the BCA's Specification C1.1 2.2 Support of Another Part transferring this requirement to the floor's support structure.

Columns on the top storey (storey under roof) Type A

For Type A construction with an effective height of not more than 25 m and with a roof that is not fire-resistant (see Section 4.4.5 on methods to remove fire resistance requirements), internal columns may have a reduction in Fire Resistance Level to:

- with Rise in Storeys not exceeding 3: –/–/-
- with Rise in Storeys exceeding 3: 60/60/60.

For Type B buildings, the storey immediately below the roof does not need to have a Fire Resistance Level, refer BCA Specification 4.1 (g).

For Type C buildings, there are no Fire Resistance Level required

Internal columns may have a higher Fire Resistance Level than the floor or roof that they may support. This is because there is no concession or exemption for their Fire Resistance Level, unlike for walls and floors. One way to bypass this is to replace columns with a loadbearing walls.

External columns

The fire resistance of columns on the outside of the building varies with the Type of Construction and the distance from the fire source feature. See Table 3.6 for Fire Resistance Levels as per Type of Construction and the distance from fire source feature. For Type A, B and C non-loadbearing external columns, there is no fire-resistance requirements.

Table 3.6: Fire Resistant Levels for external columns.

Distance to fire source feature	Loadbearing		
	Type A	Type B	Type C
<1.5 m	120/–/-	120/–/-	90/–/-
1.5 m to <3 m	120/-/-	120/-/-	60/–/-
3 m to <18 m	120/–/-	120/–/-	-/-/-
18 m plus	-/-/-	-/-/-	-/-/-

4.3.6 Internal Beams and Trusses

For Type A buildings, beams or trusses must have a Fire Resistance Level of 120/–/-, except for when they are included in a roof that is not required to be fire-rated (seeSection 4.4.5).

For Type B and C buildings, there are no direct fire resistance requirements for beams and trusses, except where they are incorporated within a fire-resisting building element.

For all Type of Construction, a beam or truss may be required to have a fire-resistance by virtue of BCA's Specification C1.1 Clause 2.2 Support of Another Part. This requirement transfers the fire-resistance requirements of the elements it supports onto the beam or truss.

4.3.7 Roofs

Fire-resistance requirements vary with the Type of Construction, but in all cases a requirement or a concession can be employed to remove the need to fire rate the roof at all.

For Type A buildings, roofs do not need to be fire-resistant as a complying sprinkler is used throughout the building (BCA Specification C1.1. Clause 3.5). For Type B and C buildings, there are no fire-resistant requirements for roofs.

Although roofs are not required to be fire-resistant, Support of Another Part (BCA Specification C1.1 Clause 2.2) may infer that the roof or elements within the roof may need fire resistance as they could provide lateral support to the external walls. The BCA contains a concession (BCA Specification C1.1. Clause 2.2 (b) (iii)) that excludes roofs from this requirement.

4.3.8 Balconies and Verandahs

The BCA contains a concession (BCA Specification C1.1 Clause 2.5 (f)) for balconies and verandahs that remove any need to provide fire-resistant requirements. For Type A buildings, balconies and verandahs must not form part of the only path of travel to a required exit. They must also be no more than a rise of two storeys above the lowest storey, and any supporting columns are to be non-combustible or built from fire-protected timber (refer Appendix A).

For Type A buildings three-storey or more, the balcony or verandah is required to be considered as a floor and construction should be as for a floor (see Section 4.2.2). For Type B and C buildings, balconies and verandahs must not form part of the only path of travel to a required exit, and this condition removes any fire-resisting requirements.

4.4 Fire Hazard Properties

The BCA controls the fire performance of materials used as coverings for floors, walls and ceilings. The following discusses the requirements.

4.4.1 Floor Coverings BCA Specification C1.8 Clause 3

For Residential Use Areas within a Class 9c building, the Critical Radiant Flux must not exceed 2.2 kW/m^2. For all other areas, except fire-isolated exits and fire control rooms, the Critical Radiant Flux must not exceed 1.2 kW/m^2. The Critical Radiant Flux for fire-isolated exits and fire control rooms are limited to 4.5 kW/m^2.

As Class 9c buildings are always sprinkled, the maximum smoke development rate of 750 per cent-minutes is not required. The BCA also allows the floor to return up the wall for a distance of 150 mm. Floor coverings turned up the wall greater than 150 mm must comply with BCA Group Number (see section 4.5.2).

Critical Radiant Flux performance of tongue and grooved timber floorboards can be found on the WoodSolutions website under the timber species section. The Critical Radiant Flux for a number of timber species is recorded with other species information. If the species contains no Critical Radiant Flux numbers, then there is no publicly available information for that species. Refer to timber product suppliers for further details.

Engineered timber flooring products such as Oriented Strand Board, particleboard, plywood and Cross Laminated Timber, etc, are proprietary, and the information must be sourced from the supplier of the product.

4.4.2 Wall and Ceiling Linings

The surface fire spread performance for wall and ceiling linings is controlled by the product's Group Number. For Class 9c buildings the allowable Group Number depends on the location of the material within the building. The Residential Use Areas are considered by the BCA as Specific Areas (BCA Notes to Table 3 Specification C1.10 Clause 4). The Group Number for wall and ceiling products in different areas within a sprinkled Class 9c building must not be less than the following (BCA Specification C1.10 Clause 4):

- Residential use areas (Specific Area) – 3
- Public corridor – 2
- Fire isolated exits and fire control rooms – 1
- Other areas (not mentioned above) – 3.

Class 9c buildings are always sprinkled so the requirement for the smoke growth rate index must not be more than 100 or the average specific extinction area is less than 250 m^2/kg is not applicable (BCA Specification C1.10 4 (a)).

The performance of solid timber linings and decorative veneers on particleboard or Medium Density Fibreboard (MDF) can also be found from the Wood Solutions website under the timber species. For systems or species not found on the website, consult the product supplier.

5 Acoustics Deemed-to-Satisfy Detailed

The BCA provides requirements for residential buildings so that unnecessary sound is not transmitted between separate living spaces so as to cause illness or loss of amenity as a result of the noise.

Class 9c buildings have a lower sound attenuation requirement than Class 2 and 3 buildings, and the reason suggested by the BCA Guide[5] is that Class 2 and 3 buildings are 'noisier' than a unit in Class 9c buildings. The BCA's Guide suggests Class 2 and 3 buildings have televisions, stereos and other activities that may not be found in a Class 9c building; therefore the level of airborne sound attenuation required in a Class 9c building is less. Unlike Class 2 and 3 buildings, the spectrum adaption terms on airborne noise (Ctr) and impact sound insulation on floors are not applied or modified the sound performance of building elements in a Class 9c building.

5.1 Sound Insulation Rating of Floors (BCA Provision F5.4 (b))

The BCA requires the floor separating in a Class 9c Sole Occupancy Units to have a Weighted Sound Reduction Index (Rw) not less than 45, and there is no direct impact noise requirement for Class 9c buildings.

5.2 Sound Insulation Rating of Walls (BCA Provision Clause F5.5 (c) and (d))

A wall in a Class 9c building that separates different Sole Occupancy Units, or a Sole Occupancy Unit from a kitchen, bathroom (not contained within the unit), sanitary compartment, laundry, plant room or utility room, must have a Weighted Sound Reduction Index (Rw) not less than 45. Further, a timber-framed wall that separates Sole Occupancy Units from a kitchen or laundry must have an impact sound insulation rating that is satisfied if the wall has two or more separate leaves, not rigidly mechanically connected. The exception of rigidly mechanically connection is at the periphery edges (BCA Provision Clause F5.3 (b) (ii)), i.e. top, bottom and side of the wall.

5.3 Sound insulation rating of internal services (BCA Provision Clause F5.6)

Ducts, soil, waste, water supply pipes or stormwater, including a duct or pipe that is located in a wall or floor cavity, must be separated from any Sole Occupancy Unit by construction with an Rw + Ctr (airborne) not less than the following:

- adjacent room is a habitable room (other than a kitchen) – 40
- adjacent room is a kitchen or non-habitable room – 25.

5.4 Door to SOU

For Class 9c buildings, the BCA has no acoustic requirements for doors in acoustic-rated walls.

6 Design and Construction Solutions

6.1 Design Approach

There are always trade-offs in building design as key considerations are pulling design solutions in different directions. For Class 9c buildings, the significant design considerations are around occupancy, layout, acoustic, fire and structural design. For a successful design, each design issue needs to be balanced off equally with each other.

As discussed in previous sections, the BCA's Deemed-to-Satisfy solutions may give more than one method for compliance for a particular building element, an example being the three solutions for internal fire-rated walls. There are four distinct structural timber building systems, discussed in Section 6.2. For each timber building system, a BCA's Deemed-to-Satisfy solution may work better than another. The Guide, where appropriate, discusses where the various structural timber building systems may work the best with a specific BCA's Deemed-to-Satisfy solution.

6.2 Timber Building Systems

There are four distinct timber construction systems possible for use in Class 9c Residential Care timber buildings. These are lightweight timber-frames (Figure 6.1), post and beam (Figure 6.2), solid timber panels (Cross Laminated Timber (CLT) or Laminated Veneer Lumber (LVL)) (Figure 6.3) and post and plate (Figure 6.4). Refer to WoodSolutions Guide 46[6] for more details on each timber construction system. Each timber construction system may have advantages and disadvantages when it comes to compliance for a particular building element; the Guide will discuss the merits of each in the following section.

Figure 6.1: Lightweight Timber Frame - The Green (Image: Frasers Property Australia)

Figure 6.2: Timber Post and Beam, International House, Lend Lease (Image: TDA)

Figure 6.3: CLT Forte Living, Lend Lease (Image: PlanetArk)

Figure 6.4: Post and Plate – Brock Commons. (Image: Brudder)

6.2.1 Internal Wall

As discussed in Section 4.3.1, the BCA provides three options for internal wall compliance: BCA Specification C1.1 method, Fire-Protected timber method and BCA Provision C2.5 (b) (iii) method (Section 4.3.1). The fundamental difference between the compliance method is the treatment of the internal walls. The BCA Provision C2.5 (b) (iii) method requires all the internal walls (loadbearing and non-loadbearing), irrespective of the Type of Construction, to be fire-resistant, but at a common rating, i.e. not less than 60/–/–. The BCA Specification C1.1 and Fire-Protected timber method do not require non-loadbearing walls to be fire rated. For the later BCA solutions, this provides an advantage as it removes any need to fire-rated internal walls, except for those bounding lift and stair shafts.

The BCA Provision C2.5 (b) (iii) method may favour the use of timber-framed or panel construction timber systems, as these timber construction systems have many loadbearing walls, requiring fire rating. Therefore, the consequence of BCA Provision C2.5 (b) (iii) method requiring all walls to be fire-rated has fewer cost impacts, as a number of these walls already need to be fire-rated.

For the BCA Specification C1.1 method, post and beam or post and plate construction timber systems would be favoured, as these timber systems have no loadbearing walls except for those bounding lift and stair shafts. Post and beam or post and plate construction timber system allow the use of non-loadbearing non-fire-rated internal walls, which are considerably cheaper than fire-rated walls.

There is no straight forward answer for which timber structural system is superior, investigation into all is required.

6.2.2 External Walls

The BCA has only one solution for external walls being compliance to BCA Specification C1.1. This Deemed-to-Satisfy solution requires the external walls to be non-combustible for Type A and B construction, whereas for Type C construction there are no non-combustible limits. Also, in some situations, the external wall is required to be fire-rated. The fire rating depends on whether the wall is loadbearing, the Type of Construction and its distance from a fire-source feature.

Solutions for lightweight timber frames or solid timber panels building systems favour fire-protected timber solution (Appendix A). For post and beam and post and plate systems, they easily allow a non-loadbearing curtain wall system to be used. Curtain walls have no fire resistance, but there is the situation due to the distance away from a fire-source feature, or the curtain wall is protected by external wall sprinklers, there is no need to fire-rated the external wall. In these situations, the curtain wall is only required to be non-combustible.

The same non-loadbearing curtain wall solution could be employed for lightweight timber-framed or solid timber panel systems, but in many instances, these buildings systems rely on external walls to be loadbearing.

Consideration of which BCA's Deemed-to-Satisfy solution is used for the internal and external walls may dictate which structural timber system works best.

6.3 Building Solutions Details

6.3.1 Smoke-proof Wall Layout (BCA Specification C2.5)

The BCA Provision C2.5 (b) (i) requires smoke-proof walls to be placed so that the area within the smoke-proof walls is not greater than 500 m^2. They are also required around Ancillary Use Areas such as kitchens, laundry and storage rooms. Therefore, when considering where to place smoke proof walls, the Ancillary Use Areas should be considered first, as they are required in these locations irrespective of area limits. Once the Ancillary Use Areas smoke-proof walls are determined, the remainder of the area can be assessed to see if the area is still required to be divided further.

Other locations that smoke-proof walls can coincide with are walls required to be fire rated. As fire-rated walls generally have higher construction requirements than smoke-proof walls, they can be used to divide the area further.

Smoke-proof wall example

The example in Figure 6.5 is from the WoodSolutions Cost Plan on Aged Care, WoodSolutions Guide No 287. Refer to that Guide for more information on the building's design.

Step 1 – Assign the Ancillary Use Area wall first – the kitchen location is marked in blue within Figure 6.5. The storeroom is not included as it is less than 10 m^2, which is below the area required to be considered by the BCA.

Step 2 – As the total floor area is greater than 500 m^2, the building needs to be divided into areas no bigger than 500 m^2. The most logical spot is the door between the dining room and the residential area, marked in orange in Figure 6.5.

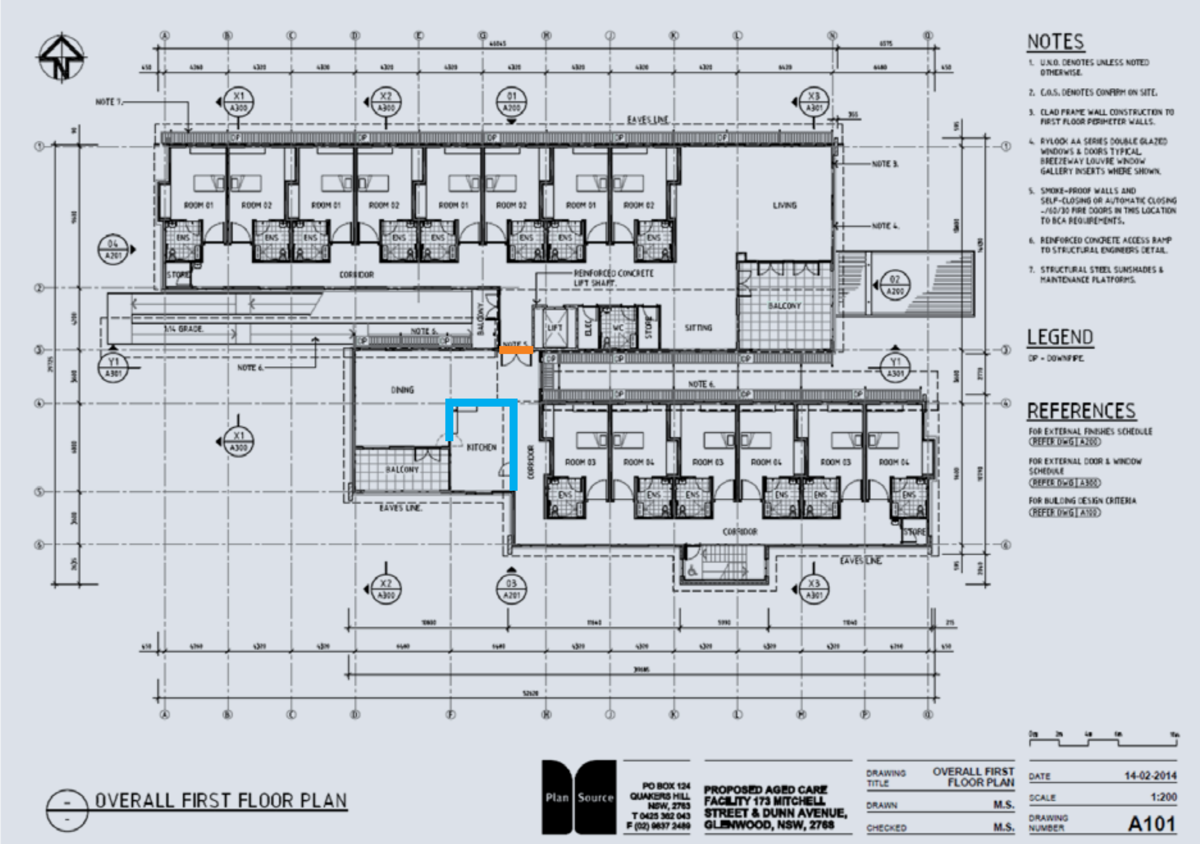


Figure 6.5: Floor plan of Residential Care facility showing Smoke-proof wall locations

The construction of a smoke-proof wall can be achieved by extending standard grade 13 mm plasterboard to the underside of the floor above or non-combustible roof covering (see Figure 6.6).

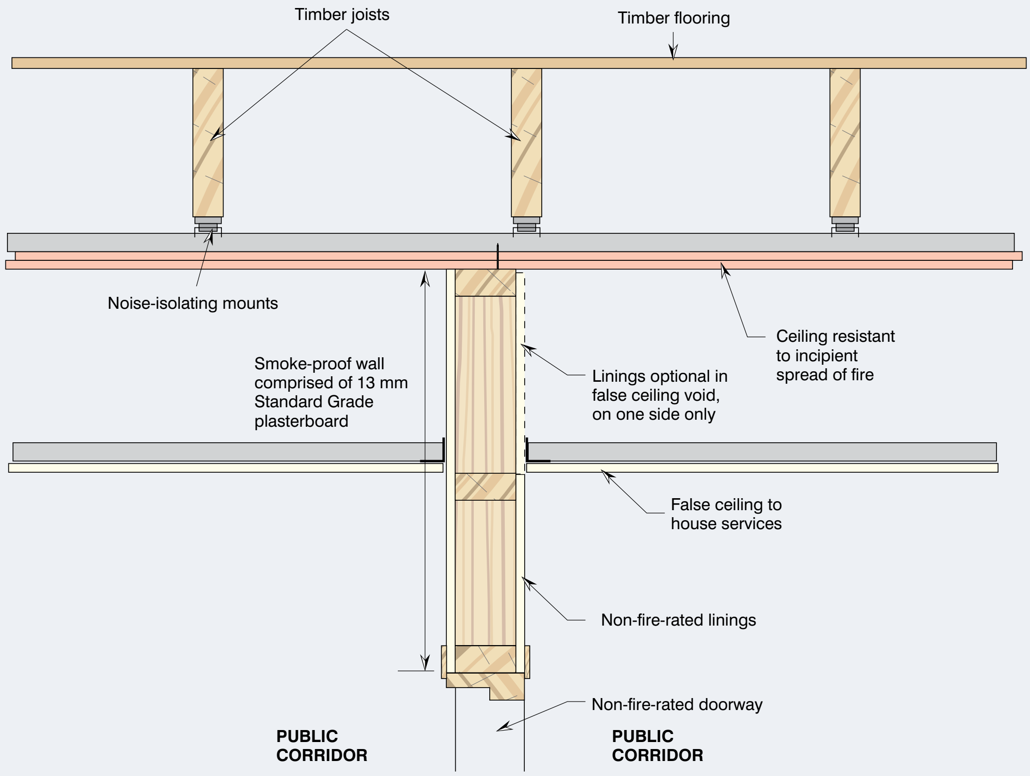


Figure 6.6: Smoke-proof Wall Construction

NSW variations

Smoke-proof walls in NSW differ from the general BCA requirements in that they are to be lined on both sides of the wall with 13 mm standard grade plasterboard and, if provided with cavity insulation, contain only non-combustible insulation. The construction joint in the wall must also be smoke sealed with intumescent putty or the like.

6.3.2 Floors

The BCA requires floors for Type B and C is to have a minimum Fire Resistance Level of 60/60/60 and for Type A, 120/120/120. Furthermore, the floor is to have sound transmission and insulation of a Weighted Sound Reduction Index (Rw) of not less than 45, when between Residential Care areas. This acoustic requirement does not include any impact noise obligation and generally speaking is much less demanding than the acoustic requirements for Class 2 or 3 buildings.

Timber-framed floors

Fundamentally, timber-framed floors (see Figure 6.7) are made up of components that aid fire resistance and acoustic performance:

1. Floor covering or topping – acoustic improvement
2. Floor joist and structural flooring – acoustic improvement and fire rating
3. Insulation – acoustic improvement
4. Ceiling – acoustic improvement and fire rating.

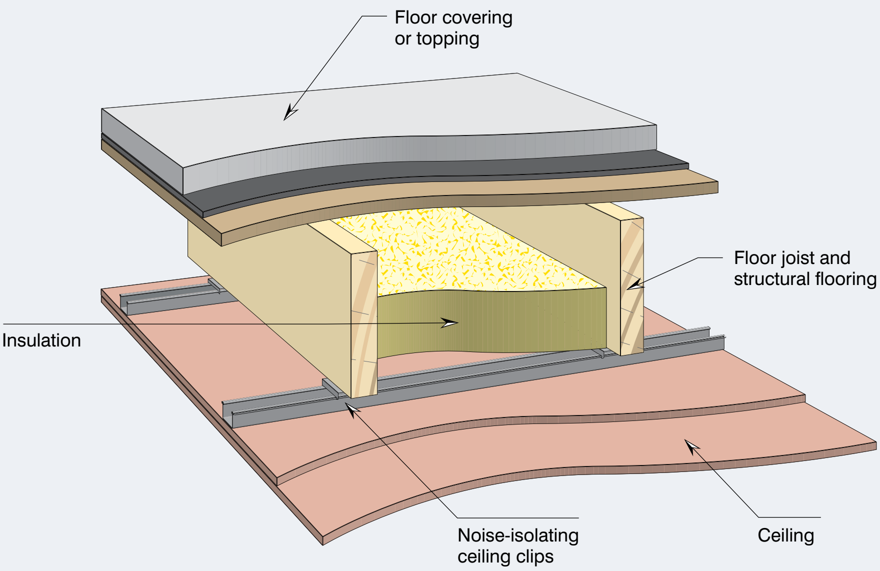


Figure 6.7: Illustration of components in a fire and acoustic rated timber-framed floor.

Changing the makeup of each component will affect the fire and acoustic performances. According to CSR's Red Book[8], to achieve a 60/60/60 fire-resistance timber-framed floor systems requires two layers of 13 mm fire-protective plasterboard, while 120/120/120 timber-framed floor systems require three layers of 16 mm fire-protective plasterboard.

The acoustic requirement of Rw 45 can be achieved with the direct fixed two layers of 13 mm fire-protective plasterboard to the underside of the floor joist and with 90 mm glass wool insulation between the joists. Where three layers of 16 mm fire-protective plasterboard are used, no insulation is required.

Acoustic performance above BCA minimum requirements can easily be achieved by the addition of furring channels, which improve the acoustic performance by 4 dB, and resiliently mounted furring channels, which increase the acoustic performance by a further 9 dB (based on CSR's The Red Book).

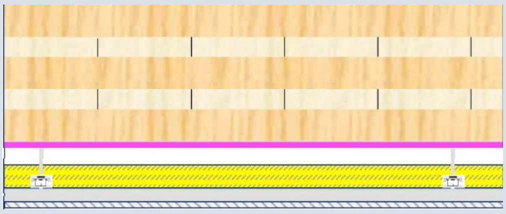
There are many acoustic solutions for timber-framed floors, and the acoustic performance of framed floors is predominately dependent on the linings used for the ceiling. For further fire and acoustic information, as well as compliance evidence, refer to the relevant lining supplier.

Solid Panel Floor (CLT or LVL)

The fire resistance of CLT or LVL floors varies from manufacturer to manufacturer so a direct reference to the supplier of these products for performance information is required.

For the acoustic performance of CLT, refer to WoodSolution Guide No 44[9]. Within this Guide, all the floor systems exceed the BCA minimum requirement for Class 9c residential care buildings. Refer to Table 6.1 for a compliant CLT system.
For LVL based floor systems, refer to the lining or LVL manufacture for information.

Table 6.1: CLT Acoustic Rated Floor System.

Configuration	
	140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 185mm suspended ceiling with Resilient mounts 75mm glasswool insulation Lining

System	**Test**	**Floor Covering**	**Lining**	**Thickness (mm)**	**R_w**	**$R_w + C_{tr}$**	**STC**	**$L_{n,w}$**	**IIC**
F04-02	T1617-08	Bare CLT	1x13mm standard plasterboard	354	57 - 59	**51 - 53**	58 - 59	**59 - 61**	49 - 51
F06-01	T1617-09	Bare CLT	2x13mm standard plasterboard	367	59 - 61	**53 - 55**	59 - 61	**57 - 58**	52 - 53

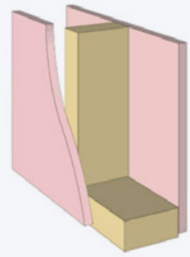
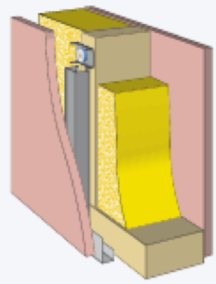
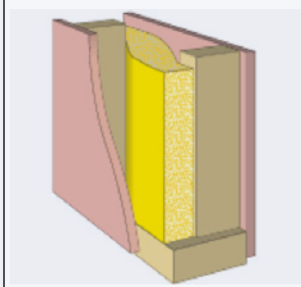
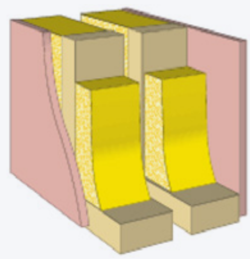
6.3.3 Internal Walls

As discussed above, the BCA gives more than one way for internal walls to comply with fire-resistant requirements. Irrespective of the pathway for achieving fire resistance, the acoustic provision for internal walls that separate different units, units from a kitchen, bathroom (not contained within the unit), sanitary compartment, laundry, plant room or utility room are all the same, requiring to have Weighted Sound Reduction Index (Rw) not less than 45. Furthermore, where the Sole Occupancy Unit's wall separates a kitchen or laundry, it must also have impact noise resistance. which can be achieved through separate leaf construction. The exception is that connections are allowed at the edge of the wall, i.e. top, bottom or end of wall length. The following discusses the timber solution for various fire compliance methods.

BCA Provision C2.5 (b) (iii) Method

This BCA provision requires all walls except the fire-resistant lift and stair shaft to have a Fire Resistance Level of 60/- / – irrespective of the application. Timber-framed walls require at least 16 mm fire-protective plasterboard applied to both sides of the studs to achieve this Fire Resistance Level. Table 6.2 has minimum complying published timber-framed systems; refer to plasterboard suppliers for evidence of fire and acoustic performance.

Table 6.2: Minimum Fire and Acoustic Compliant Timber-Framed Systems

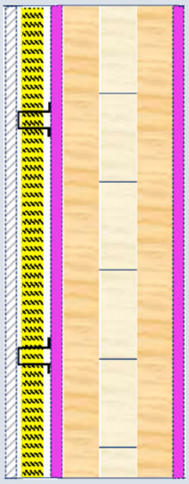
Description	FRL	Acoustic Requirements	Minimum Compliant System	Graphic
Partitions not required to be acoustic rated	60/ - / -	No requirement	**Single stud** • 16 mm fire grade plasterboard to both sides of stud • Stud size 70 x 45 mm • Acoutic rating: NA	
Wall that separates different SOU, SOU and bathroom (not contained within the unit), sanitary compartment and plant room or utilities	60/ - / -	R_w not less than 45	**Single Stud with resilient mounts** • 16 mm fire grade plasterboard to both sides of stud • Stud size 70 x 45 mm • 75 mm glasswool insulation • Acoutic rating: R_w 49	
Wall between SOU and kitchen or laundry	60/ - / -	R_w not less than 45 and two or more separate leaves without rigid mechanical connection except at the periphery.	**Staggered stud** • 16 mm fire grade plasterboard to both sides of stud • Stud size 70 x 45 mm • 75 mm glasswool insulation • Acoutic rating: R_w 49	
			Double stud • 16 mm fire grade plasterboard to both sides of studs • Stud size 70 x 45 mm • 75 mm glasswool insulation • Acoutic rating: R_w 57	

Note: Acoustic and Fire Rating information from CSR Red Book Fire & Acoustic Design Guide

For CLT systems fire resistance requirements can be found from CLT suppliers. For acoustic information, again refer to WoodSolution Guide No 44[9]. Table 6.3 provides CLT walls that meet the acoustic performance and Table 6.4 provides CLT wall solutions that require the separation of leaves.

Table 6.3: Minimum Acoustic Compliant CLT Systems.

Configuration

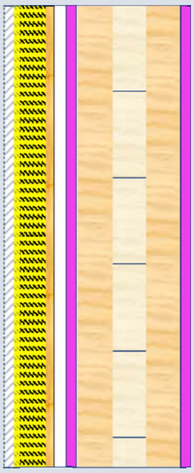


Lining
28mm furring channel
Clip or mount (min. 45mm cavity)
50mm glasswool insulation
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Connection	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W05-02	T1617-94	1x13mm standard plasterboard	Adjustable clip	180	**52**	**42**	53 - 54
W06-01	T1617-37	1x13mm standard plasterboard	Resilient mount	180	**51 - 52**	**42 - 43**	53
W06-02	T1617-38	2x13mm standard plasterboard	Resilient mount	193	**56**	**47**	57 - 58

Figure 6.4: Minimum acoustic compliant CLT systems with leaves separation.

Configuration



Lining
Stud frame
75mm glasswool insulation
Gap
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Stud Frame	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W02-02	T1617-88	1x13mm standard plasterboard	70mm timber stud 20mm gap	225	**56**	**48**	56 - 57
W04-02	T1617-43	1x13mm standard plasterboard	64mm steel stud 20mm gap	219	**58**	**48 - 49**	58 - 59

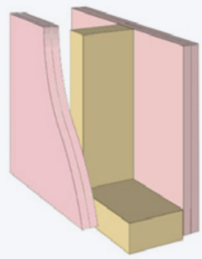
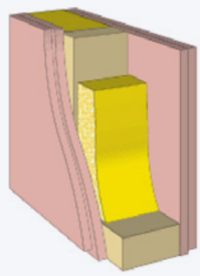
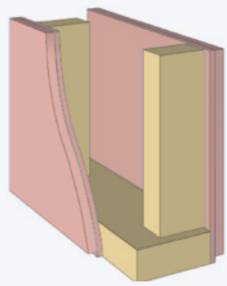
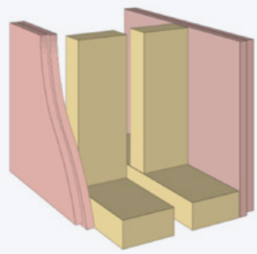
BCA Specification C1.1 and Fire-Protected Timber Method

For Type A and B construction, loadbearing walls that bound public corridors or lobbies between Sole Occupancy Unit are generally required to have a Fire Resistance Level of 120/–/-. For Type C construction that is two-storey in height, a Fire Resistance Level of 60/–/– is generally required on the lower storey. For non-loadbearing walls, there is no Fire Resistance Level required.

Table 6.5 has minimum complying timber framed systems a Fire Resistance Level of 120/–/-.

For a Fire Resistance Level of 60/–/– refer to Table 6.2.

Table 6.5: Minimum fire 120/ – / – compliant timber-framed systems.

Description	FRL	Acoustic Requirements	Minimum Compliant System	Graphic
Partitions not required to be acoustic rated	120/ - / -	No requirement	**Single stud** • 2 x 16 mm fire grade plaster-board to both sides of stud • Stud size 70 x 45 mm • Acoutic rating: NA	
Wall that separates different SOU, SOU and bathroom (not contained within the unit), sanitary compartment and plant room or utilities	120/ - / -	R_w not less than 45	**Single Stud** • 2 x 16 mm fire grade plaster-board to both sides of stud • Stud size 70 mm • 75 x 45 mm glasswool insulation • Acoutic rating: R_w 46	
Wall between ***SOU*** and kitchen or laundry	120/ - / -	R_w not less than 45 and be two or more separate leaves without rigid mechanical connection except at the periphery	**Staggered stud** • 2 x 16 mm fire grade plaster-board to both sides of studs • Stud size 70 x 45 mm • Acoutic rating: R_w 48	
			Double stud • 2 x 16 mm fire grade plaster-board to both sides of studs • Stud size 70 x 45 mm • Acoutic rating: R_w 51	

Note: Acoustic and Fire Rating information from CSR Red Book Fire & Acoustic Design Guide

6.4 Fire Resistant and Acoustic Detailing

6.4.1 Lightweight Structural Timber-Framed Systems – Excluding Fire-Protected Timber Systems

Fire-resistant and acoustic rating detailing of lightweight timber-framed fire-rated systems, other than Fire-Protected Systems, are fundamentally the same irrespective of the building classifications. WoodSolutions Guide No 2[10] contained information on the joining of floors to walls, walls to walls and associated cavities, penetrations, etc.

6.4.2 Solid Timber Panel Systems – CLT and LVL- excluding Fire-Protected Timber Systems

As solid panel systems solutions (CLT or LVL) are proprietary products, refer to supplying companies for information.

6.4.3 Fire-Protected Timber Lightweight Structural Timber-Framed and Solid Timber Panel Systems

Refer to WoodSolutions Guides No 37R[2] and 37C[11].

Appendix A : Fire-Protected Timber

Fire-protected timber is timber elements protected by non-combustible linings, and the timber can be either lightweight timber-framed or mass timber. The non-combustible linings differ if they are for lightweight timber-framing or mass timber, and is summarised in Table A.1.

Table A.1 Fire-Protected Timber Non-combustible linings requirements.

	Application	DTS Solution	AS1530.4 Test Method Criteria
Lightweight Timber-Framing	Loadbearing and required fire-resistant walls	2 x 13 mm fire-protective plasterboard	To achieve a Resistance to the Incipient Spread of Fire of not less than 45 minutes
Mass Timber	Inside a fire-isolated stairway or lift shaft	1 x 13 mm fire-protective plasterboard	The time allowed for fire protective covering and timber interface to exceeds 300°C – 20 minutes
	External walls within 1 m of an allotment boundary or 2 m of a building on the same allotment	2 x 13 mm fire-protective plasterboard	The time allowed for fire protective covering and timber interface to exceeds 300°C – 45 minutes
	All other applications	1 x 16 mm fire-protective plasterboard	The time allowed for fire protective covering and timber interface to exceeds 300°C – 30 minutes

Fire-protected timber has several other conditions for compliance:

- the building has an effective height of not more than 25 m; and
- the building has a sprinkler system complying with Specification E1.5 (other than an FPAA101D or FPAA101H system) throughout; and
- any insulation installed in the cavity of the timber building element required to have a Fire Resistance Level and be non-combustible; and
- cavity barriers are provided in accordance with BCA Specification C1.13.

Appendix B : Definition

Aged care building: a Class 9c building for residential accommodation of aged persons who due to varying degrees of incapacity associated with the ageing process are provided with personal care services and 24-hour staff assistance to evacuate the building during an emergency.

Ancillary Use Areas: are areas that contain equipment or materials that have a high potential fire hazard such as:

- kitchen and related food preparation areas greater than 30 m^2
- laundry – where items of equipment are potential fire sources (e.g. gas-fired dryers)
- storage rooms greater than 10 m^2 used predominantly for the storage of administrative records.

Average Specific Extinction Area: the average specific extinction area for smoke as determined by AS/NZS 3837.

Class 9c: a residential care building.

Critical Radiant Flux: is a measure determined by AS ISO 9239.1 and means the critical heat flux where a piloted flame is extinguished.

Curtain Wall: a non-loadbearing external wall that is not a panel wall.

Deemed-to-Satisfy Solution: a method of satisfying the Deemed-to-Satisfy Provisions.

External wall: an outer wall of a building which is not a common wall.

Fire compartment: for Deemed-to-Satisfy Provisions – any part of a building separated from the remainder by walls and/or floors with each having an FRL not less than that required for a fire wall for that type of construction and where all openings in the separating construction are protected in accordance with the Deemed-to-Satisfy Provisions of the relevant Part.

Fire Protective Covering:

- 13 mm fire-protective grade plasterboard; or
- 12 mm cellulose cement flat sheeting complying with AS/NZS 2908.2 or ISO 8336; or
- 12 mm fibrous plaster reinforced with 13 mm x 13 mm x 0.7 mm galvanised steel wire mesh located not more than 6 mm from the exposed face; or
- other materials not less fire-protective than 13 mm fire-protective grade plasterboard, fixed in accordance with the normal trade practice for a fire-protective covering.

Fire Source Feature:

- the far boundary of a road, river, lake or the like adjoining the allotment; or
- a side or rear boundary of the allotment; or
- an external wall of another building on the allotment which is not a Class 10 building.

Fire wall: a wall with an appropriate resistance to the spread of fire that divides a storey or building into fire compartments.

Group Number: the number of one of four groups of materials used in the regulation of fire hazard properties and applied to materials used as a finish, surface, lining, or attachment to a wall or ceiling.

Habitable room: a room used for normal domestic activities, including a bedroom, living room, lounge room, music room, television room, kitchen, dining room, sewing room, study, playroom, family room, home theatre and sunroom but excludes a bathroom, laundry, water closet, pantry, walk-in wardrobe, corridor, hallway, lobby, photographic darkroom, clothes-drying room and other spaces of a specialised nature occupied infrequently or for extended periods.

Internal wall: excludes a common wall or a party wall.

Non-combustible:

(a) applied to a material — not deemed combustible as determined by AS 1530.1 — Combustibility Tests for Materials; and

(b) applied to construction or part of a building — constructed wholly of materials that are not deemed combustible.

Materials deemed not combustible under BCA Provision C1.12 include Plasterboard, perforated gypsum lath with a normal paper finish, fibrous-plaster sheet, fibre-reinforced cement sheeting and pre-finished metal sheeting having a combustible surface finish not exceeding 1 mm.

Panel Wall: a non-loadbearing external wall, in frame or similar construction that is wholly supported at each storey.

Performance Solution: a method of complying with the Performance Requirements other than by a Deemed-to-Satisfy Solution.

Public corridor: an enclosed corridor, hallway or the like which serves as a means of egress from 2 or more Sole Occupancy Units to a required exit from the storey concerned.

Sole Occupancy Unit: a room or other part of a building for occupation by one or joint owners, lessee, tenant or other occupier to the exclusion of any other owner, lessee, tenant or other occupier and includes a room or suite of associated rooms in a Class 9c building, which includes sleeping facilities and any area for the exclusive use of a resident.

Smoke Growth Rate Index: the index number for smoke used in the regulation of fire hazard properties and applied to materials used as a finish, surface, lining or attachment to a wall or ceiling.

Residential care building: a Class 3, 9a or 9c building that is a place of residence where 10% or more of persons who reside there need physical assistance in conducting their daily activities and to evacuate the building during an emergency (including any aged care building or residential aged care building) but does not include a hospital.

Resident use area: part of a Class 9c building normally used by residents and includes sole Occupancy Units, lounges, dining areas, activity rooms and the like but excludes offices, storage areas, commercial kitchens, commercial laundries and other spaces not for the use of residents.

Rise in Storeys: the greatest number of storeys calculated in accordance with BCA C1.2.

Type of Construction: A minimum Type of fire-resisting construction of a building.

Shaft: the walls and other parts of a building bounding vertical chute, duct or similar passage but not a chimney or flue.

Appendix C : References

1. WoodSolutions, Guide No 2 – Timber-framed Construction for Multi-residential Buildings Class 2 and 3 – Design and construction guide for BCA compliant sound and fire-rated construction.
2. WoodSolutions, Guide No 37R – Mid-rise Timber Buildings – Multi-residential Class 2 and 3.
3. National Construction Code, 2019 Building Code of Australia – Volume One.
4. NSW Planning and Infrastructure Fire Sprinkler Standard, 2013.
5. National Construction Code, 2019 Guide to the BCA Volume One.
6. WoodSolutions, Guide No 46, Wood Construction Systems.
7. WoodSolutions, Guide No 28, Rethinking Aged Care Construction – Consider Timber.
8. CSR Gyprock, The Red Book, Fire and Acoustic Design Guide.
9. WoodSolution Guide No 44, CLT Acoustic Performance.
10. WoodSolutions, Guide No 2, Timber-framed Construction for Multi-residential Buildings Class 2 & 3 – Design and construction guide for BCA compliant sound and fire-rated construction.
11. WoodSolutions, Guide No 37C, Mid-rise Timber Buildings Commercial and Education.

Interior Design Guide 43

Wood Solutions
design and build
woodsolutions.com.au

This guide was published in Jan 2018
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Reimagining Wood-based Office Fitout Systems

Design Criteria and Design Concepts

Design Guide issued by Forest and Wood Products Australia

WoodSolutions 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 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.

design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-49-2

Authored by:

Professor Perry Forsythe and Kevin Bradley

Acknowledgements:
Thanks to Tran Dang for assistance with workshop and technical design implementation

School of the Built Environment Faculty
of Design Architecture and Building
University of Technology Sydney

First Published: January 2018

Contents

1 Introduction

Timber has a unique place in the construction industry. At one end of the scale, it is a fundamental structural component (it has strength, can be organised into structural framing and allows diverse shapes); at the other end, fitout owners often specify timber as an expression of quality that projects the company's values to their clients and staff.

While timber is common in residential construction, its application has declined in commercial office fitouts. For example, its use in office partitioning has been almost completely replaced by steel-framed plasterboard. At best, it is used as a preferred and aesthetically appealing outer skin. Where it is used, many of the workstations and other products are manufactured overseas.

However, opportunities to introduce timber are emerging as trends toward open plan offices move away from constructed solutions and toward furniture-based design. The increasing emphasis on sustainability assessment, new ways of working, office workers who value richer engagement with their workspaces, and a broader awareness of the recurring embodied energy/carbon in each office re-fit, all point to new possibilities in office fitout.

This guide seeks to expand the view of timber as an 'aesthetic skin' to include new forms of engagement. It deals specifically with the development of design concepts for new wood-based office fitout systems revolving around environmental sustainability outcomes. The emphasis is on reducing the recurring physical waste and embodied carbon/energy brought about by the 'fitout-stripout-repeat' cycle that commonly arises from lease churn over the life of office buildings. This is primarily achieved by designing with a view to increased reuse and recycling, and by taking advantage of the inherent benefits of timber as a sustainable construction material. Designs particularly focus on partitions, workstations, furniture and suspended ceiling tiles.

2 Research-based Design

A parallel research report to this guide, 'Increasing Wood-based Office Fitout for Sustainable Life Cycle Benefits' (download at www.fwpa.com.au), provides evidence-based research findings that inform the design concepts developed in this document. For instance, that report:

- quantifies and qualifies office lease and fitout churn cycles
- quantifies and qualifies physical waste from stripout processes including problems for reusing/recycling waste
- compares embodied energy/carbon for wood versus traditional fitout, using a Life Cycle Assessment approach.

While these findings have markedly influenced design, attention is also given to other features central to successful fitout design, including architectural, supply chain, tenant, building owner, cost and fitout/stripout contractor needs.

2.1 Research into the 'Fitout-Stripout-Repeat' Sustainability Loop

The design approach in this guide is based on research-based evidence. A continuous cycle commonly evident from the research is defined in Figure 1. It involves a three-step process that, from a sustainability perspective, begins when an outgoing tenant reaches the end of their lease, triggering the ubiquitous 'Make Good' clause. Stage I, covers removal of their fitout; Stage 2 involves reinstatement of what was originally in place; Stage 3 covers the new fitout installed by the incoming tentant. Ultimately, this results in a repetitive cycle as tenants come and go from the building. Design decisions lack visibility at the 'Make Good' and 'Stripout' stages of the process, which creates disconnections that prevent closure of the sustainability loop and cause poor reuse and recycling outcomes.

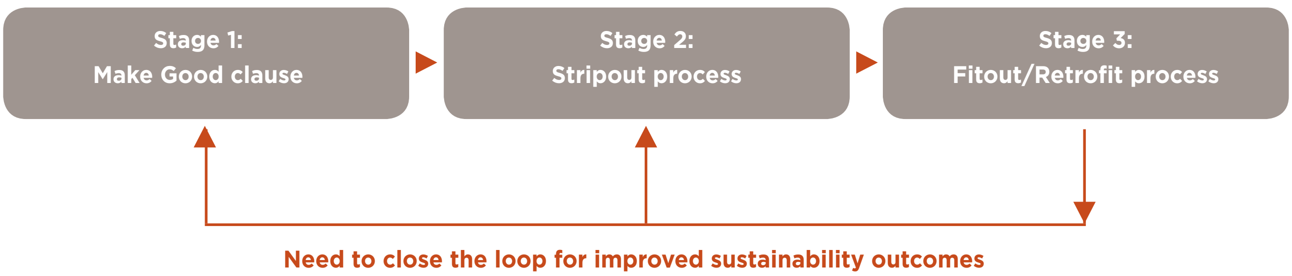


Figure 1: The 'fitout-stripout-repeat' cycle and the sustainability loop problem.

It is worth looking into the factors that influence each stage in the Figure 1 model more closely. For instance, the 'Make Good' clause potentially creates triple waste because it will likely induce:

- stripout by the outgoing tenant
- temporary reinstatement of the original fitout
- a new fitout by the incoming tenant.

The 'Stripout' stage (Stage 1) typically involves the building owner's capital works team and outgoing tenant. The actual process is characterised by the following constraints:

- Speed is paramount to reduce lost rent or usage of the building.
- Work is often done after hours at extra cost and extra process to meet inter-tenancy rules and to have uninterrupted access to the goods lift and loading bay.
- The goods lift and truck cartage cycles dominate other processes.

- Waste is separated into streams if easy and worthwhile in terms of reuse/recycling costs; if not at least cost neutral, it will go to landfill.
- Volumetric items (e.g. workstations) are generally not economically viable to reuse/recycle.
- Creating sufficient economies of scale to make reuse/recycling worthwhile is usually linked to the critical mass of a homogenous waste stream.
- Reuse/recycling is more viable if depots are close by, as this reduces cartage costs and improves the ability for truck shuttle times to coordinate with site processes.

The 'Fitout' stage (Stage 3) typically involves the building owner's leasing team and the incoming tenant. Issues include:

- Designing for aesthetics and corporate needs dominate.
- Sustainability may have an impact on design, depending on the tenant's targeted environmental expectations, the base building's sustainability rating and the building owner's environmental expectations.

The decisions made up front at the fitout design stage may not take into account what will happen at the stripout stage, in terms of reuse and recycling. This distances levels of accountability for the chosen fitout materials as the cycle progresses.

The current specification of wood primarily revolves around a conversation between client and designer concerning aesthetics. As an example, relatively little detail is provided about the underlying construction of partition walls because the main issue is the appearance of the outer skin. ***The first sustainability loop disconnection is made because*** the client is blind to, and often has no explicit opinion on, many construction materials, and has no vision of the likely end of life implications arising from their choices.

The designer then turns their attention to what it takes for the partition wall to be documented for construction. They may typically take a conservative approach and tap into conventional fitout construction methods in order to seek a competitive price (e.g. a steel-framed structure with plasterboard lining and glass partitions). Such assemblies can be difficult to deconstruct and so little consideration is made for the end-of-life treatment of the partition wall. ***The second disconnection in the sustainability loop is made***.

At the end of the partition wall's life, the stripout contractor will assess its removal based primarily on past knowledge of what they expect to exist beneath the outer plasterboard layer. The waste will likely be a split of separated materials that can be recycled and mixed materials that can't be easily separated or have limited reuse and recycling value. Unfortunately, the mixed materials usually go to landfill. ***The third disconnection in the sustainability loop is made.***

At each stage, some accountability for fitout materials at the end of their use is relinquished. Improving visibility and accountability along the stages in the loop would set the design direction for the sustainable use of wood in fitouts.

2.2 Quantifying and Qualifying Fitout Churn and Waste

- Fitout has a short life span that should be designed for reuse and recycling. Research indicates that fitout churn falls within a range of 5.0–8.9 years; small tenants tend to be in the lower range and large tenants in the upper range.
- Where no significant efforts have been made for reuse and recycling onsite, the research indicates that about 64% of the material goes to landfill. Where efforts have been made to improve waste management, the tables can be turned so that 61% of waste can be diverted from landfill. For details, refer to report *Increasing Wood-based Office Fitout for Sustainable Life Cycle Benefits*, downloadable at www.FWPA.com.au
- Saving materials from landfill involves a fundamental problem of mixed material. The difficulty in separating components leads to demolishers sending such materials to landfill. For details, refer to the report *Increasing Wood-based Office Fitout for Sustainable Life Cycle Benefits*, downloadable at www.FWPA.com.au.
- Recycling and reuse only occurs where there is sufficient critical mass, homogeneous materials are easy to separate, and/or there is easy access to reuse/recycling depots and/or resale markets. Wood often has insufficient critical mass and is hard to separate by virtue of being part of composite materials and assemblies – it often becomes part of mixed waste going to landfill.

3 Other Key Impacts on Design

3.1 Client and Worker Perceptions

Wilmot et al (2014) indicate that new materials are often preferred under the broad understanding that, “New materials are so cheap that the additional handling and transport required, makes reused or reprocessed materials unable to compete”. ‘New’ means you can search, compare and order from a catalogue with assured knowledge of material choice, quality, size, availability and warranty. There is much less room to manoeuvre when considering reused or recycled materials.

New fitout enables a company to have conversations with staff around the quality of their future work environment, which can be a significant component in the change management process. Companies need to have a well-articulated position on resource recovery and attract like-minded staff to value fitout reuse and recycling. To support this, there is a need to focus on retaining value when designing for reuse and recycling.

3.2 Office Futures

“The emphasis on making and prototyping, along with the rise of a project-centred workforce, will change the nature of workspace and how it’s provided. Look for the emergence of just-in-time settings designed to support the activities of fast-moving, constantly changing teams.” Gensler Workplace Forecast (2016)

Gensler says we have to be aware of “the emergence of just in time settings”. Doing business in office settings in the future is less likely to look very much like the offices of the past, or even the present. The physical space of buildings may not change, but the amount of space, type of space, location and base building technology is likely to keep changing. This change is, in part, driven by the increasing capacity of mobile connectivity and the natural expectation that work/productivity is not related to inhabiting 12 m^2 of personal commercial real estate for 7 hours a day. Technology increasingly blurs the physical footprint of the workplace, and this landscape has no defined boundaries. As Gensler indicates, it is likely that the design of the new workplace fitout will respond to this fluidity.

The new office is likely to mean less focus on permanent construction and more potential for a furniture-driven approach (including the use of furniture design principles to create flexible partitioning systems). This is an important trend that potentially benefits wood-based fitout because of the suitability of wood panel products to scalable, cost effective and automated furniture production processes. For instance, BIM-oriented 3D design and computerised wood cutting/processing equipment can accomplish much in terms of providing cost-effective, customisable and automated small batch production processes that are less reliant on overseas manufacture.

3.3 Supply Chain and Delivery

The proposed design concepts need to be compatible with the idiosyncrasies and complexities of the supply chain involved. Workstations that dominate project budgets often come from overseas with long lead times that effectively reduce the fitout contractor’s level of project control. Re-empowering fitout contractors would be useful in having greater control and efficiency; improved cost effectiveness; better customisation potential; and greater ability to assemble in Australia.

Greater integration of the supply chain is needed, including actively adopting concepts such as ‘design for manufacture and assembly’. In fact, given the obvious need for reuse and recycling in fitout, this term can be extended to ‘design for manufacture, assembly and ***disassembly***’ (DFMAD). Such an approach aims to offer a way to harness digital technology (across clients, designers, manufacturers, fitout and stripout contractors) by making use of technologies such as computer numeric cutting files (CNC), 3D printing files and parametric designs that can change to suit scalability needs. The intention is also to create greater visibility to reduce the disconnects in the sustainability loop.

There is potential to both compress and simplify the supply chain under a wood-based approach, including shorter lead times, reduced purchase of separate parts, and an improved ability to have a smooth transition from design to fabrication to site construction.

Such an approach could optimise use of basic carpentry skills to cover a large part of the fitout work, creating simplified workflows, and work and project management requirements.

4 Design Criteria

Fitout design concepts need to come up with systems that can offer multi-generational value, without feeling 'second hand'. Timber can be an underpinning material in fitouts that have refined potential for reuse and recycling markets. In terms of strategic design, criteria include:

4.1 Materiality and configurability

- Incorporate more timber in fitouts to create sufficient critical mass for reuse and recycling practices to develop.
- Design for reuse and recycling at the beginning of the fitout process.
- Use a relatively homogenous palette of materials.
- Use a minimal kit of parts.
- Develop multi-purpose scalable components.
- Design for re-configurability to extend reuse options.
- Develop assemblies that allow incorporation of services (e.g. cabling).

4.2 Deconstructability

- Ensure fast knockdown from volumetric to flat and stackable.
- Avoid special dismantling tools.
- Ensure compact stacking to optimise goods lift usage and efficient truck cartage.
- Ensure wood panels and workstation tops are easily separated from the support frames beneath.
- Create quiet deconstruction methods to allow more work during normal office hours.

4.3 Critical handling

- Ensure fast stripout that maintains flow from breakdown-to-lift, lift-to-truck, and truck-to-depot.

4.4 Changes in fitout style and scope

- Take advantage of open plan offices that allow greater utilisation of a furniture-based design approach.
- Broaden timber-based tender packages to compress and simplify the supply chain.

4.5 Digitally driven DFMAD

- Create scalability by using parametric design, i.e. where a 3D model of an assembly (such as a shelving unit) can be changed in size and the rest of the componentry and proportions change accordingly.
- Use 3D printing for components that join timber pieces together and can be automatically re-sized using parametric modelling.
- Use computer numeric cutting (CNC) machines to automatically cut pieces for an assembly, from a digital file.

5 Timber Design Systems – Key Product Areas

The focus in the design process needs to be around key fitout items that constitute the major proportion of fitout works/costs:[1]

- partitions – lightweight, reconfigurable and deconstructible
- workstations – scalable, reconfigurable and customisable in Australia
- furniture – including scalable shelf and storage units, pedestals and miscellaneous tables
- ceiling tiles for suspended ceilings – including utilisation of established product lines.

Figure 2: Key product areas.

The partition and workstation concepts use a (partly) common kit of parts that can be quickly disassembled and reassembled. They do not need specialist manufacturer teams to do this, thus ensuring simplicity, flexibility, economies of scale, construction efficiency and importantly, critical mass to encourage reuse and recycling.

The furniture concepts take advantage of open source designs that have been modified by the authors to allow scalable, parametric design capable of a file-to-factory-to-site approach. These designs utilise 3D modelling that can be sent to a CNC cutting machine and once this is done, sent as flat pack to site for easy and fast assembly. Such units can be made to order at short notice and do not need specialist manufacturers, designers or lengthy procurement times (or lengthy delivery distances). They are relatively homogenous and don't depend on specialist parts or proprietary manufacturing systems.

For completeness, existing products have been co-opted into the design field for ceiling tiles (these products are only suggestions that can be coupled with the above). These products already exist in established markets and include:

- ceiling tiles capable of both front-of-house and back-of-house applications
- existing plywood acoustic ceiling tile products that can be routed with noise capture patterns and sound absorbent backing according to project specific needs; high-quality powder coated finishes are possible.

[1] Less potential appears to exist in other parts of fitout such as floor coverings where carpet is the main option – it is cheap, fast to lay and easy to remove (especially tiles with pressure sensitive glue). Some carpet tile manufacturers also have advanced recycling schemes. The same applies to seating as there is an established need for these to be supplied separately and to meet specific comfort and ergonomic criteria.

These conceptual ideas have been developed to a level of functional feasibility that could be further developed by industry as new products. The models and physical prototyping that have been undertaken indicate proof of concept and function rather than trying to exhibit a finished working product. The general level of design development provides for industry wide uptake while still allowing individual proprietary input that may enhance competitive advantage.

5.1 Wood Partition Walls – A Modular Hollow Core System

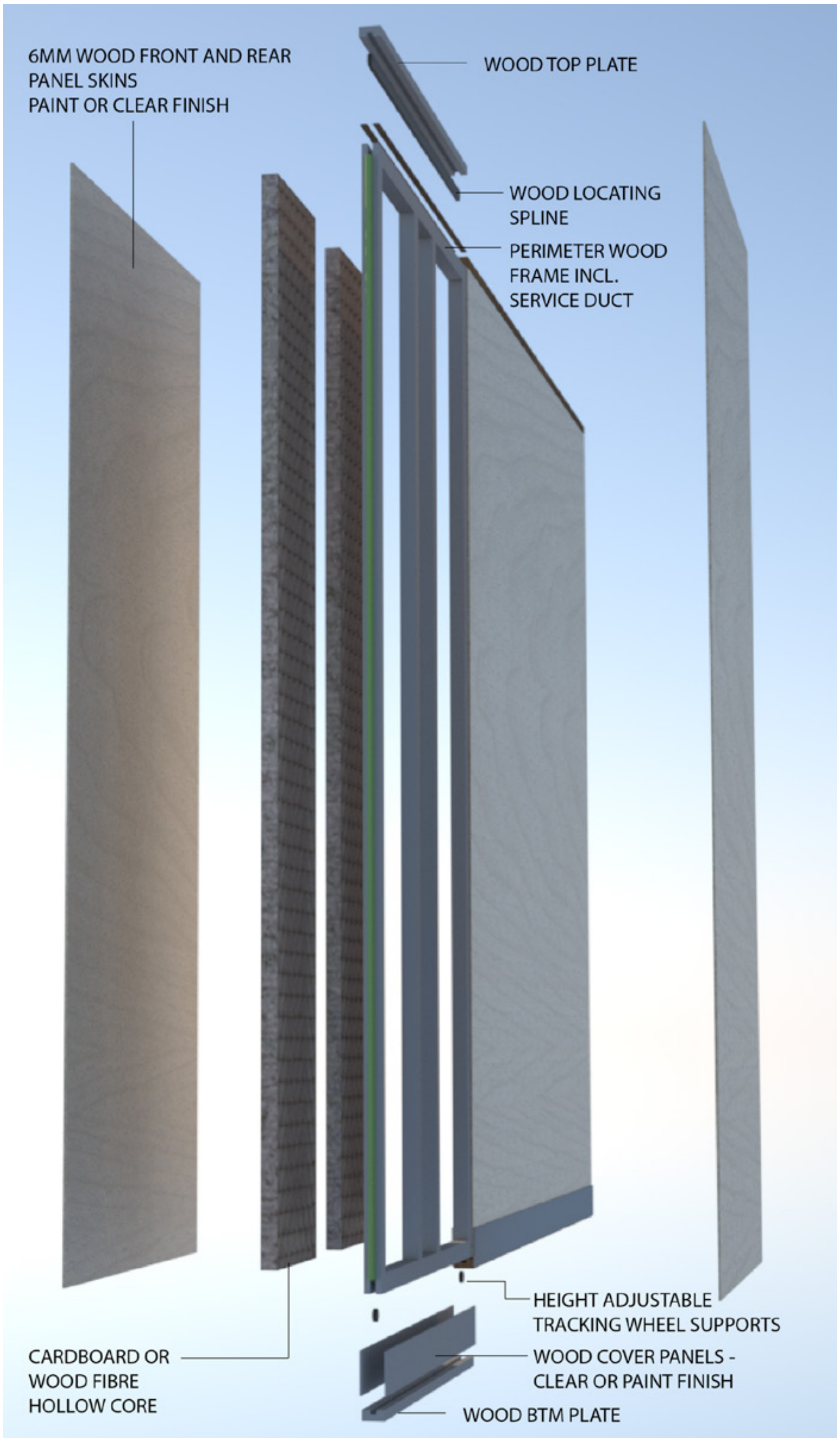


Figure 3: Timber partition panel concept, UTS 2016.

5.1.1 The concept

The concept for a new wood-based partitioning system is a response to the past reliance on multiple materials and trades to construct steel stud and plasterboard partition systems. The modular panel concept responds directly to the design criteria presented in Section 4 (as presented in Figure 4). For instance, it:

- responds to evolving work situations over the life of a lease
- is simple to install/deconstruct and is able to be reused without depending on multiple trades
- utilises a homogenous timber-oriented material palette to improve reuse and recycling consisting of timber skins, timber frame, minimal (starch-based) glue and expanded cardboard filling
- aims to leverage existing manufacturing businesses – primarily the door manufacturing industry discussed further below
- compresses and simplifies the supply chain both in material supply, work packaging and in transition from design to manufacture to assembly and finally, disassembly.

The approach does not require permanent structures or walls to be demolished at each churn in fitout. It does not rely on a lengthy construction process and its materials and construction technology are flexible.

While the outer skins for such panels can be pitched at competing with common plasterboard finishes using the likes of hardboard, this system also lends itself to higher-grade finishes, such as plywoods with high end decorative veneers. The hollow core, modular, timber panel is designed so that it can be used interchangeably as wall partitions, workstation partitions or work station benchtops (see section 5.2).

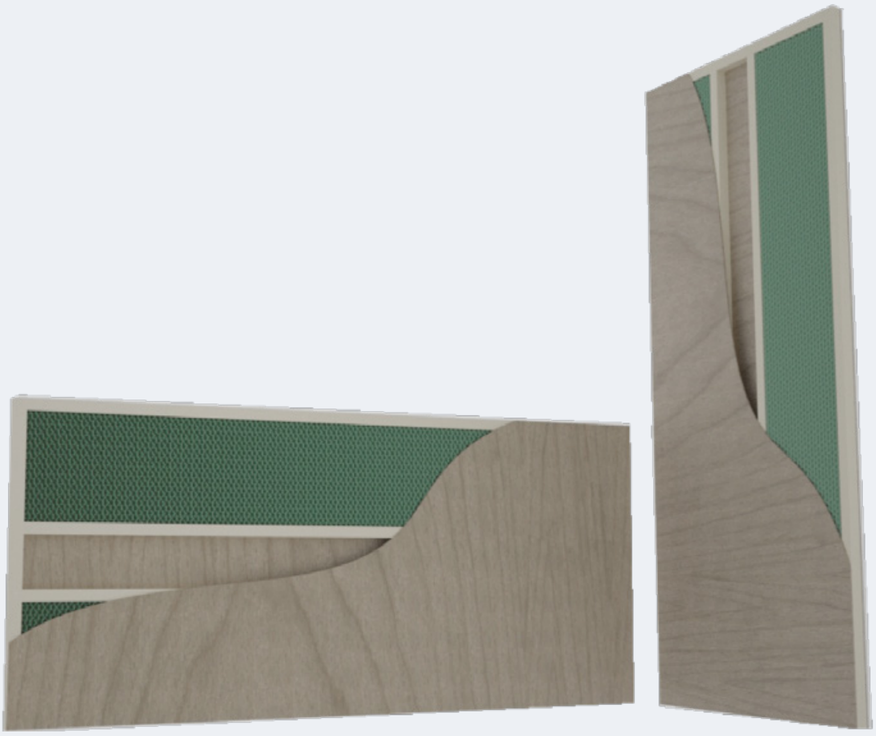


Figure 4: Final concept iteration, UTS 2016.

5.1.2 Having an impact at the right place in the supply chain

This approach adapts existing door manufacturing techniques that aim to provide a cost-effective and well-developed base.[2] For instance, door manufacture goes beyond basic raw materials by providing a more value-added and prefabricated product centred around wood. Even so, its location in the fitout delivery process means that it still allows design flexibility and is well known in day-to-day specification and purchasing by designers and fitout contractors alike.

[2] Hume Doors and Timber (www.humedoors.com.au/), one of the largest door manufacturers in Australia, assisted with prototyping, cost information and feedback about manufacturability criteria of UTS's original design concepts.

Door manufacture typically includes: economies of scale; cost effective and efficient manufacturing processes; well-developed compliance and quality assurance systems; certification under common sustainability assessment schemes including FSC and PEFC; ability to upgrade to meet fire and sound performance as part of normal manufacturing processes; Australia-wide distribution networks; recognition in existing supply chains in the fitout industry; supply of ancillary fitout materials.

Door manufacture also offers broad scope in terms of client-driven customisability, including customisable unit sizes as well as panel veneers, skins, cores and finishes. The existing carpentry workforce has deep tacit knowledge, thus offering a readily available workforce for installation needs

Figure 5: Manufacture and warehousing, Hume Doors, Lansvale NSW.

Figure 6: Distribution, Hume Doors, Lansvale, 2016.

5.1.3 Design Details

The primary unit size is targeted at a 2,700 x 1,200 x 45 mm panel so that panels can be dimensionally coordinated with typical ceiling grid modules. Height can be scaled to a maximum 3,000 mm, which should deal with most ceiling heights. Width can be scaled to a variety of typical door widths of 820 mm, 870 mm, 920 mm, and to less standard widths of 410, 420, 460, 520, 620, 720 and 770 mm.

The thickness profile is important as it includes thicker-than-normal door skins (2 x 6 mm thick wood panel skins) and deeper-than-normal timber edge rails at 33 mm wide x 50 mm deep. There is also 33 mm wide x 50 mm deep framing pieces defining the central services duct. This provides a strong and impact-resistant unit that will not sag under the intended non-structural office use loads. Even so, this thickness is considerably less than equivalent steel stud and plasterboard partition walls, thus yielding extra usable floor space.

Joining of panels is facilitated by a routed groove along all edges of the panels. This creates flexibility in terms of joining options:

- The grooves of opposing panels receive a spline (inserted into each panel). This can be widened to show off the natural features of a decorative timber spline or reduced to either hide the joint or create a shadow line using less expensive timber.
- At the bottom of panels, the groove is used to accept adjustable rollers commonly used by door manufacturers in sliding door assemblies. These allow the panels to be adjusted for plumb and aim to assist work flow by having one worker loading panels onto the track while the other pulls them into place and fixes them (at the bottom, behind the skirting piece, see Figure 9). On top, simple locating guides (short sections of spline material) fit into the top groove.
- At stop ends where edges are exposed or where a starter piece is required to build out from an existing wall, a milled timber 'T' section can be inserted into the groove. Again, this could be made from decorative or less-expensive timber according to aesthetic needs.

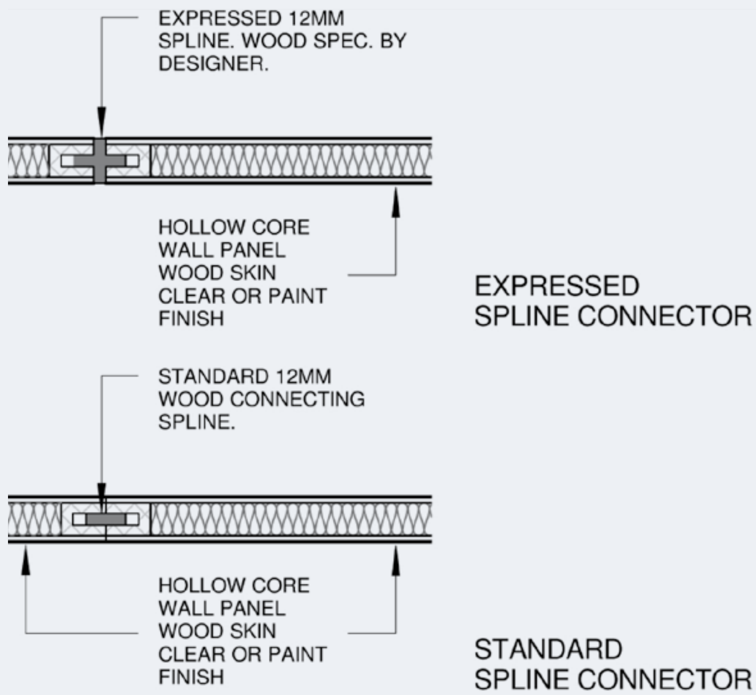


Figure 7: Spline used to join panels.

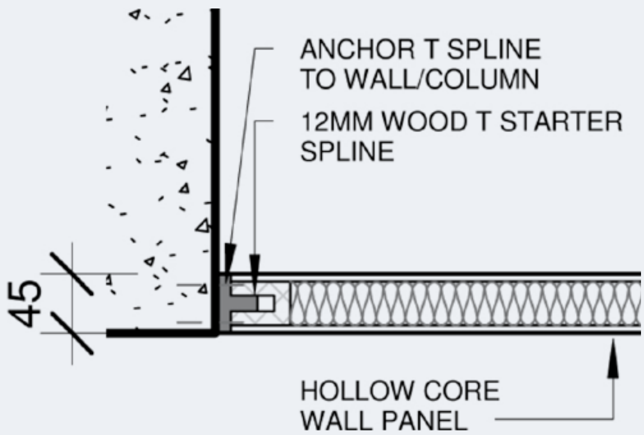


Figure 8: 'T' section for starter pieces and stop ends.

5.1.4 Channels, fixing and support

The hollow core partition needs a similar approach to the top and bottom channels used for locating partition walls in place. Two concepts can be used:

- engineered timber top and bottom channels (concept detail below), or
- extruded wood channels (such products mix waste wood fibre and resin) and can make highly detailed shapes similar to aluminium extrusions.[3]

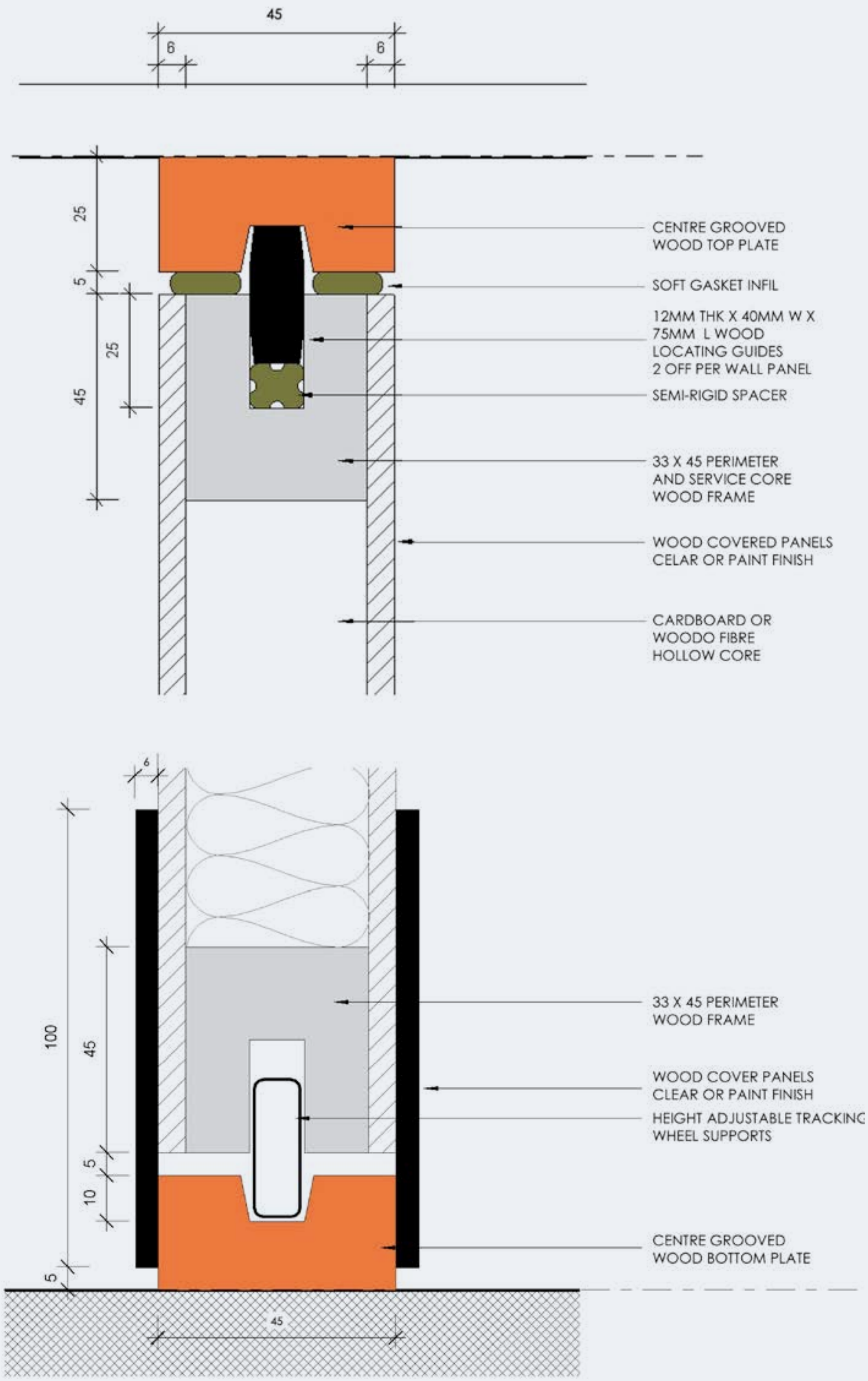


Figure 9: Concept top and bottom fixing channels utilising generic timber components (dimensions in millimeters).

[3] *Based on UTS design requirements, Innowood (www.innowood.com.au/), an extruded wood manufacturer, assisted in the concept development process and related the application of their products.*

An advantage of the wood extrusion is that it can be ordered off the shelf or customised at much lower batch sizes than achievable with aluminium. It can be handled and worked like timber – sawn, sanded, screwed and painted the same as timber. It has a grain, colour and texture gradations that flow through the full thickness of the material appearing quite similar to real timber.

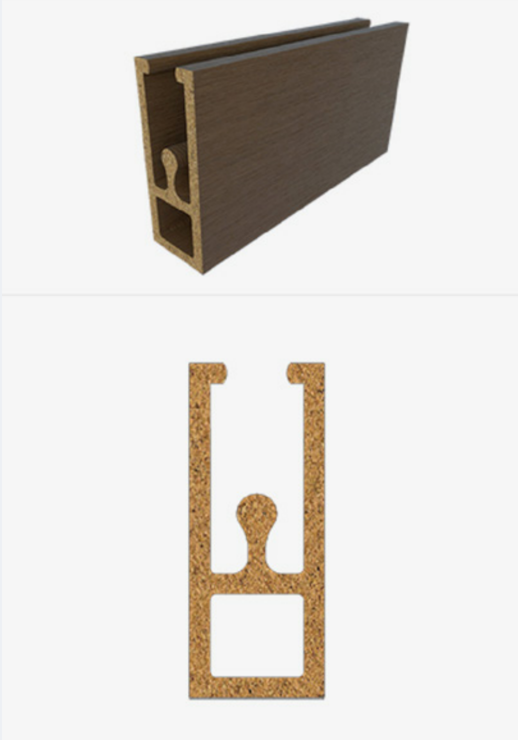


Figure 10: Example of commonly available extruded wood profile. Source: Innowood.

5.1.5 Services

The main provision for services in office fitouts concerns data and electrical cabling (including lighting and general purpose power outlets). The modular hollow core system includes provision for these services via a central void within each partition.

Minor water supply plumbing can potentially be accommodated using a similar approach. For instance, the 100 mm x 33 mm chase cavity should accommodate common water supply pipes.

Where more substantive services are required, there is the option to add a second 'facing' panel, which can include a dedicated services cavity between two panels. The relatively thin nature of the partitions (i.e. 45 mm) means that this approach carries a relatively minor wall width penalty that would only apply to a limited number of walls surrounding the likes of bathroom and kitchen areas.[4]

5.1.6 The wood-based hollow core design in practice

A wall in an office fitout plays a number of roles. The primary role is separation or the delineation of one space from another. There are other roles that are less obvious, but still considered in the design of the timber panel partition system:

- an enclosure for the reticulation of services to workstations or utilities
- a surface that can project an image through graphic application or material choice (veneer)
- a surface for mounting or hanging items.

The panel system uses the modular as a response to a uniform kit of parts, so the UTS designers designed a panel that can be installed on its side to become a 1200 mm high workstation partition. The panel is functional in two ways and potentially interchangeable over time if desired. As discussed later under workstations, the same basic panel concept can be used as a workstation bench top, which adds to the critical mass required to encourage reuse/recycling.

[4] The majority of bathroom facilities are part of the base building and may not occur in tenancy-based fitout construction. Mechanical, fire and air conditioning services require separate and dedicated attention and are not dealt with here.

It is envisaged that by designing for reuse, a market will develop for the salvaged partition panels and the related components that form the overall system. The nature of these related components means they can also be supplied separately and according to project specific needs.

After multiple reuses, it is likely that the panels would be down cycled for other uses such as mulch, animal bedding, feedstock for new wood panel production and resource recovery. Specifiers need to recognise that the scope of recyclability (including door skins in particular) may be material and coating-specific and should be chosen accordingly. For instance, materials such as hardboard do not commonly include any glues where utilising natural lignin to bond the wood fibres together. Inquiries should be made with individual materials manufacturers for further details on such issues.

Figure 11 shows a general idea of the partitions as wall and workstation panels. The image has the surfaces skin removed in part, to indicate how data and power can reticulate through the panel system from the roof void down to the individual workstation setting. The image also gives an indication of the workstation design discussed in the next section.

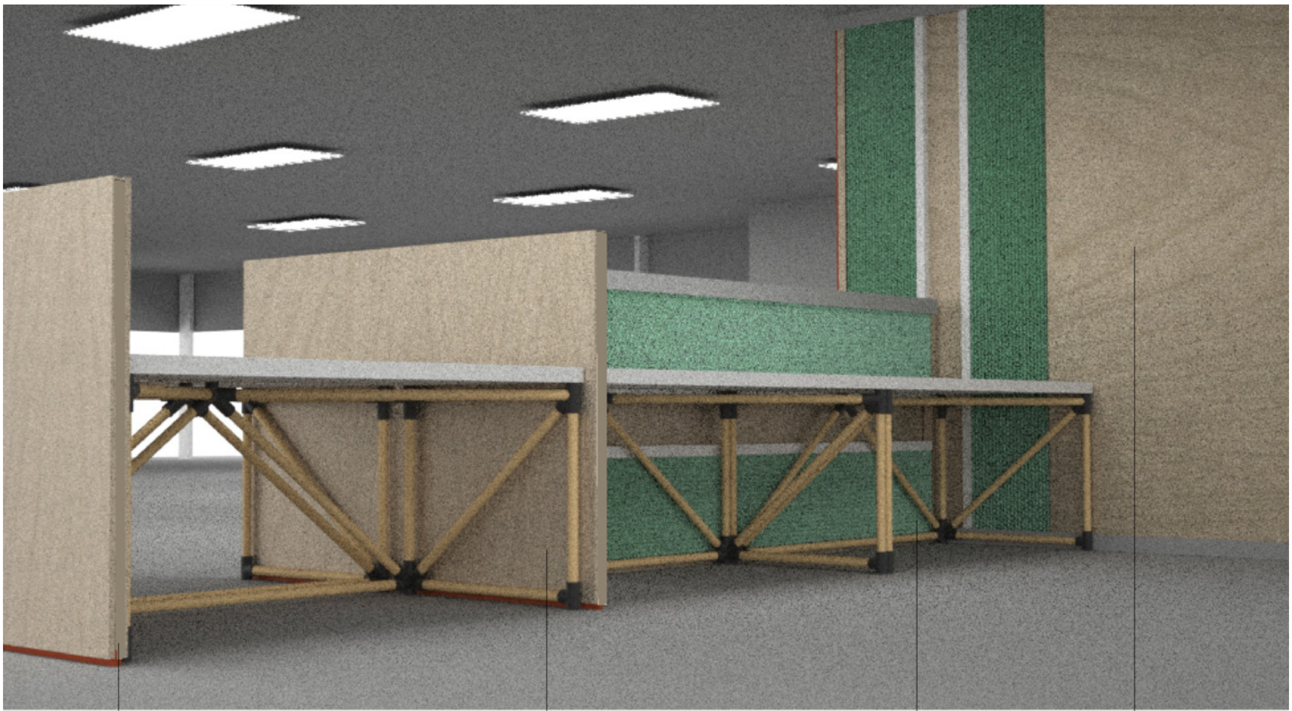


Figure 11: Concept arrangement of the UTS designed panel and workstation systems. Areas in green have the facing panel skin removed to show inner core material and service duct arrangement.

Figure 12: Partition prototype panels. Spline connector and assembly, UTS June 2016.

Figure 13: Prototype panel as a workstation partition, UTS June 2016.

5.2 Workstations Using a Reconfigurable Kit of Parts

The design paradox for workstations is that they are highly refined products, but cumbersome from a procurement and waste management perspective. For instance, they:

- have long lead times when initially purchased (often from overseas manufacturers)
- attract a large percentage of a fitout project's capital expense
- are constructed from a mix of many materials, with parts that are not interchangeable between manufacturers or between manufacturer's models
- are heavy and difficult to relocate
- are mostly worthless when they are no longer required, regardless of their condition.

Tenants nearly always own workstations and depreciate the cost over the life of the lease. The design context is driven by the combination of a near non-existent second-hand furniture market, futile attempts for disposal by the lessee, limited ability to transfer to an incoming lessee and, if all these avenues are exhausted, removed with a likely landfill destination.

5.2.1 The concept

The design questions for the future of workstations are: can they be nimble in terms of being designed to support the activities of fast-moving, constantly changing situations and can they avoid quick cycle obsolescence? These questions seek to look beyond what has gone before in workstation design and put changing business needs central to the capacity for responding to design change.

The design driver for the workstation is to afford the owner quick disassembly and reassembly without the need for specialist tools or trades people by utilising an interchangeable kit of parts and a homogenous material palette.

Steel, MDF, laminates, glues, rubber, and fabric form a common material palette of a workstation. Each material serves a role but can generally be seen as a hierarchy of material from steel as support, MDF/laminate as working surface, rubber as protective material, and fabric as image (colour). While there is a slim chance that the workstation would be relocated/reused at the end of its life, it is more likely that the combination of all these materials collectively render the system obsolete, regardless of it's quality, resulting in the cost of the repurposing of a workstation being higher than the cost of its landfill disposal.

Can workstations be made less complex in their use of material and still achieve functional and aesthetic performance?

The design driver for this workstation is to reduce it to structural and functional necessity with corresponding material application.

Finally, the design considers the full extent of the life of the workstation from the time it is considered on the plan, through the changes in business activity during its use, through to the time the stripout team is assessing how they will shift a fitout from a tenancy, and beyond to alternative wood-based reuses.

5.2.2 3D printable joiners and dowels

Figure 14: 3D printable joiners and dowels, UTS June 2016.

The idea behind the dowels and 3D printable joiners arose from thinking about how workstations are currently made of many materials and coatings, and conversely, how far a design could be stripped back to the point of only function and material.

The goal was to employ a very limited material palette in a pursuit of high reuse/recycle potential while combining infinite size combinations. Only basic dimensions of width, working surface height, and depth were reference points for conceptual modelling.

The joiners connect the dowels at junctions and support the work surface. Removal of a few screws at selected joiners sees the whole assembly pull apartment into its kit of individual parts, in minutes. Workstation partitions (see Section 5.1.6) can be added as required for privacy and other needs. Dowels can be made from decorative hardwood or less expensive softwood, which are both readily available. The printing material for the prototype joints is plastic based, but wood-based 3D printing filament are becoming more readily available.

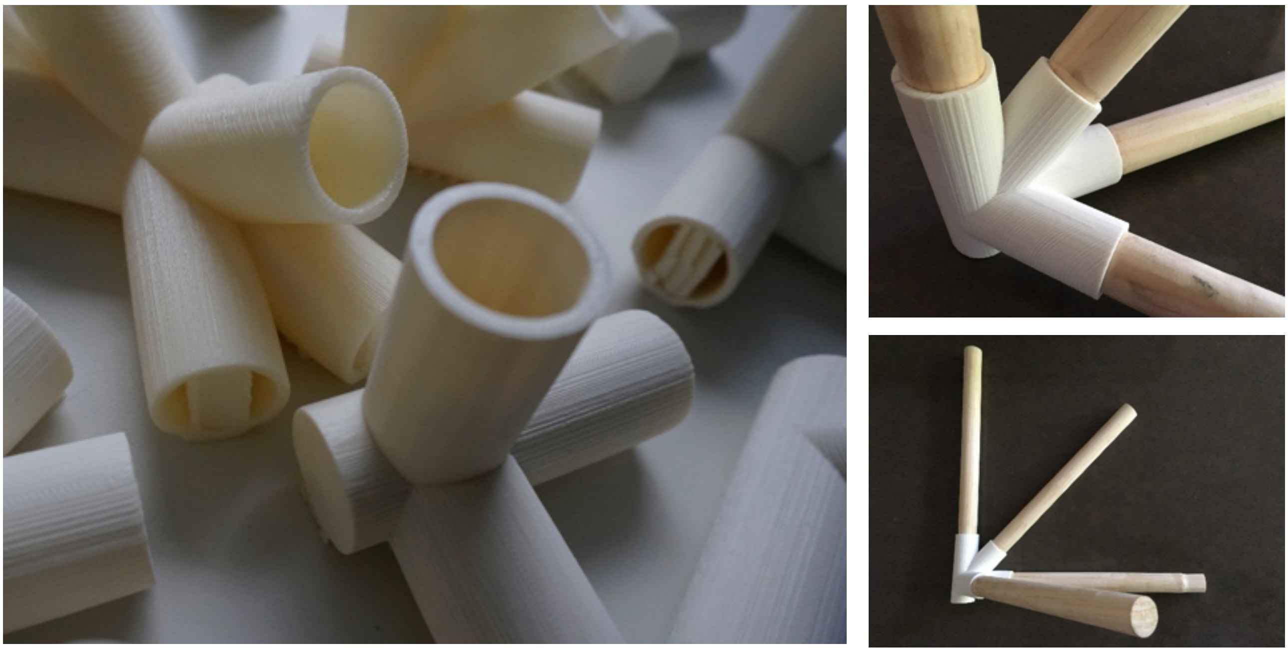


Figure 15: Printed joints, UTS 2016.

Research into 3D printed materials for frame joints should focus on structural capacity using wood filament, as this would allow for waste wood to be recycled directly back into the same product line. Further, 3D printing only tends to be practical and cost effective for small production runs but for larger projects, 3D printing should be ideally used as a precursor to larger-scale production options.

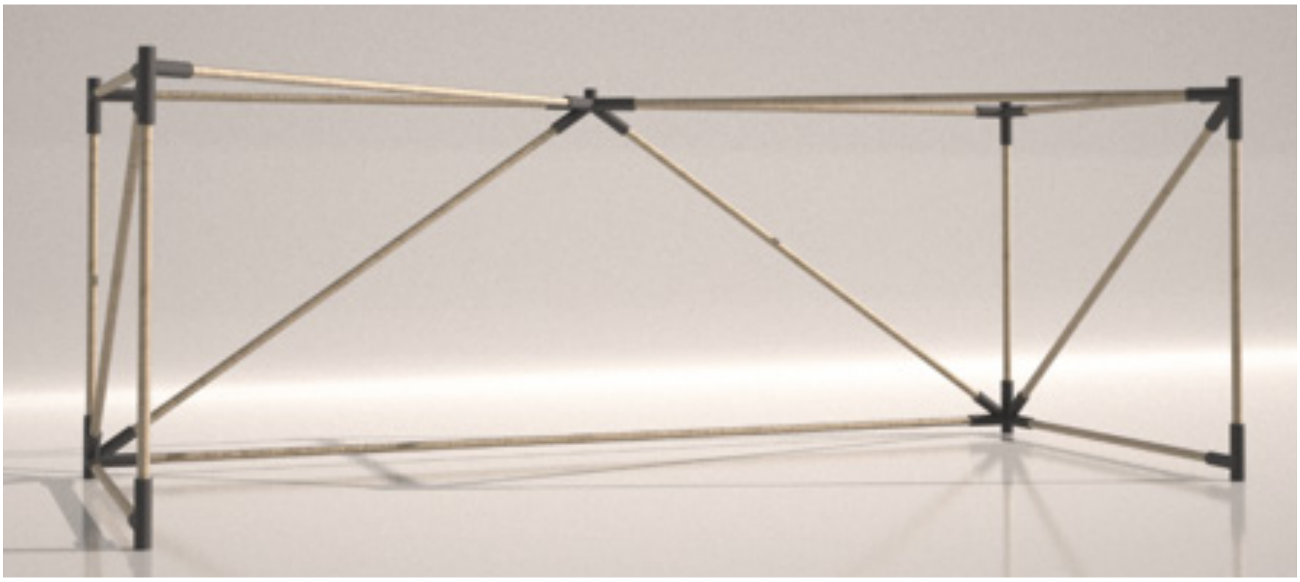


Figure 16: Concept workstation frame model (Rhino), UTS 2016.

Figure 17: Concept workstation model (Rhino), UTS 2016.

5.2.3 Details on parametric design

The design of the concept workstation was fine as a response to function, material and reuse. A designer could conceivably design a workstation by going through the same manual modelling process as UTS and print off the joiners, purchase the timber dowel members and the hollow core work surface. But this was not seen as offering the level of design flexibility required for broad-based usage. Parametric design was employed to bridge this gap by allowing continuous scalability of the work stations (i.e. the dowels and joiners). Here, the parametric plug-in (Grasshopper) was used in conjunction with common 3D CAD software (Rhino), to allow the designer to adjust the length, height, and depth of the workstation with corresponding parametric modelling working in the background. The parametric model changes the geometry of the joiners, the length of dowel members and the size of the work surface. Printing of the joints can be done directly from the software and a cutting schedule generated for the timber dowel.

To put the parametric design function into perspective, a designer could specify a number of workstation types with the joiners being automatically calculated and timber-cutting schedules generated. There is no reliance on long lead times, and possibly, workstations could be manufactured as needed when business activity changes.

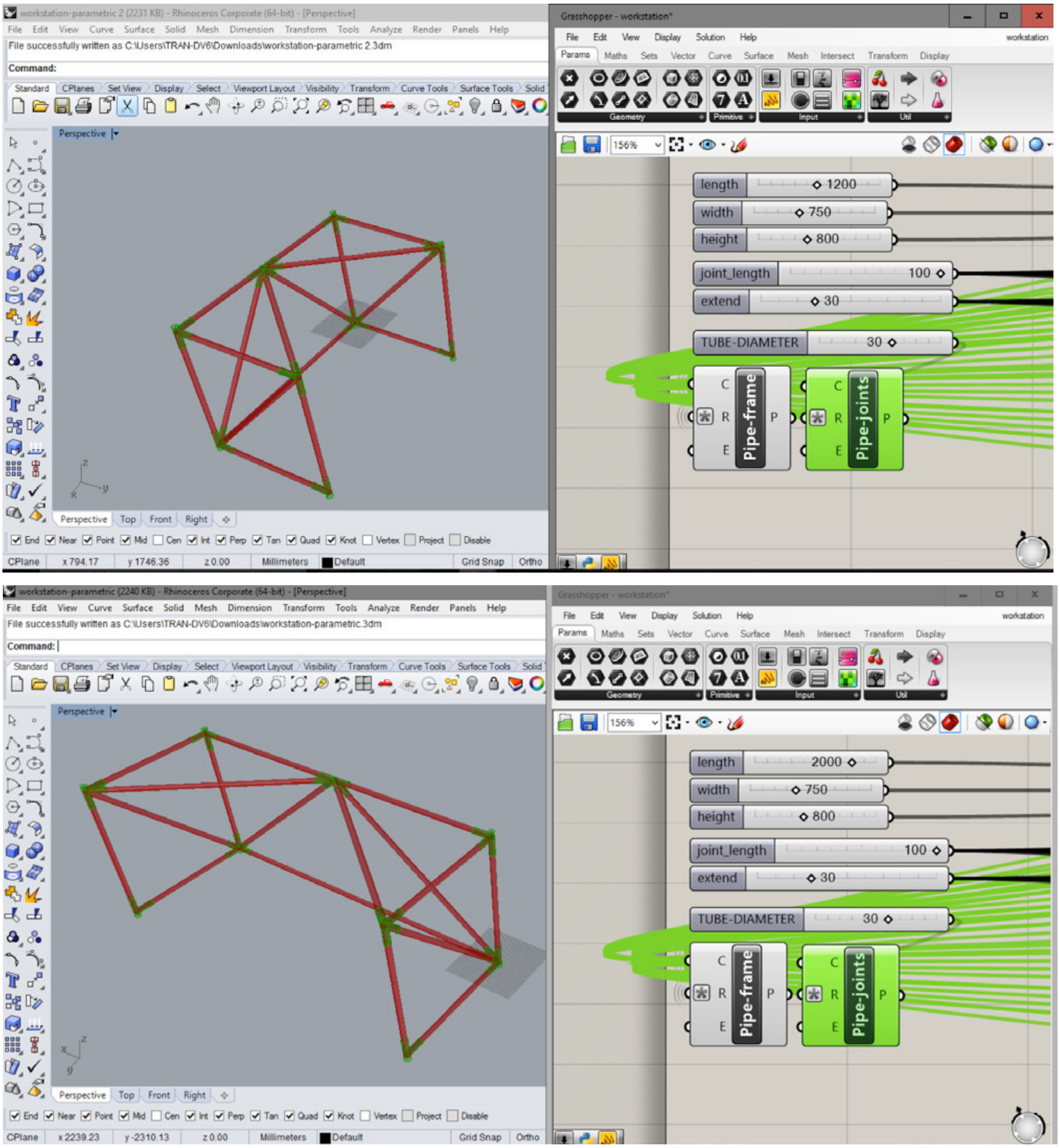


Figure 18: Parametric modelling Rhino and Grasshopper files, UTS 2016.

Ideally, the workstation design should be a web-based interface where workstations could be specified and printing files generated without the need to know Rhino or the parametric software Grasshopper.

Figure 19: UTS workstation prototype, June 2016

Figure 20: UTS workstation prototype, June 2016.

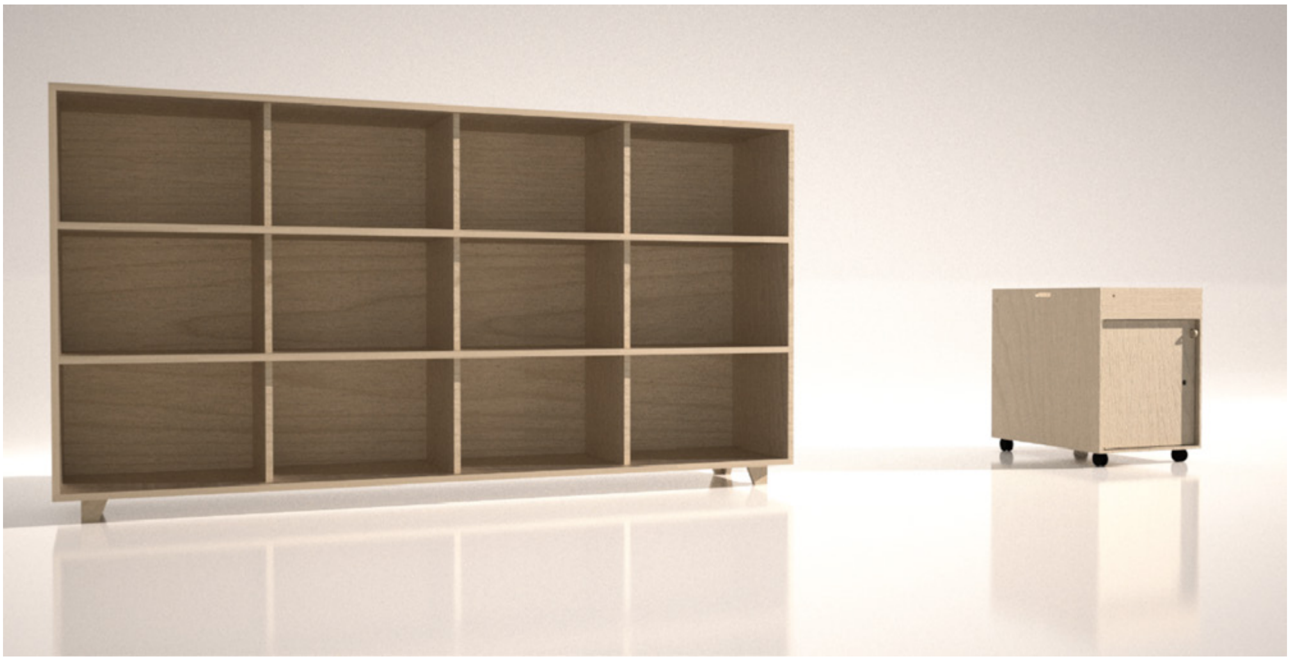


Figure 21: UTS drawing of parametric open source furniture 2016.

5.3.1 The concept

Commercial loose office furniture differs widely, depending on front-of-house or back-of-house applications.

Front-of-house is commonly viewed as a direct expression of the type and image of a business. It is an opportunity for a designer and client to showcase the nature and character of the business. Selection of this type and quality of furniture is nearly always applied to the reception area. It is quite individualistic and represents only a small percentage of the furniture and storage requirements in a fitout.

The UTS designers looked more towards general loose furniture and storage relating to back-of-house applications. This represents a larger opportunity for increased use of wood products.

General office furniture and storage lends itself to high volume repetition as it applies across the tenancy and is closely related in function and size of workstations. This repetitive nature of such furniture was found to have strong synergies with the previously discussed CNC technology. For instance, digital files tell the CNC machine how to cut the various pieces from a sheet of plywood, which may take only 10 minutes to do in practice, and then a further 10 minutes to fabricate it. Slot and tab type designs mean that there is very little need for other parts or components – even door hinges can be machined into the wood-base design that keeps things very simple for both assembly and disassembly. Here, designs for loose furniture and storage can be found as open source files on the internet. One company in particular, operates as an online studio for individual furniture designers working with wood panel products and CNC routing. *Open Desk* (www.opendesk.cc) has open source files that have been incorporated into this project. Initially, these files were used as proof of concept insofar as ensuring that CNC cutting delivered an appropriate end product. It was soon realised that by converting the designs to parametric files (see section 5.2.3), the furniture could become scalable to meet a much wider multitude of needs that could be utilised by individual fitout designers.

Figure 22: CNC made furniture by URS using *Open Desk* designs (Source: table www.opendesk.cc/zero/half-sheet-table and chair, UTS 2016, using Finn Lockers www.opendesk.cc/fin/fin-lockers#get-it-made and Zero Pedestal www.opendesk.cc/zero/pedestal)

Figure 23: Close up photograph of Zero Pedestal (Source: www.opendesk.cc/zero/pedestal).

5.3.2 Manufacturing open source furniture in practice

In terms of process, the *Open Desk* CAD files are downloaded, opened and require some work in checking for minor errors (in the jointing patterns) and potential conversion to a file type compatible with the CNC machine. The files are set up to cut pieces from a specified sheet size. Materials sourced for testing the files were readily available in plywood or other wood panel sheets.[5] Fabrication using the routed pieces is quite fast as the slot and tab arrangements and the lightweight materials provide simplicity and an intuitive approach to assembly.

[5] Materials used in specific parts of the CNC prototyping were supplied by Gunnersens (www.gunnersens.com.au/), one of the largest timber distribution companies in Australia.

Figure 24: CNC milling tools, UTS 2106.

Figure 25: Open Desk Pedestal manufacture, UTS CNC milling machine, UTS 2016.

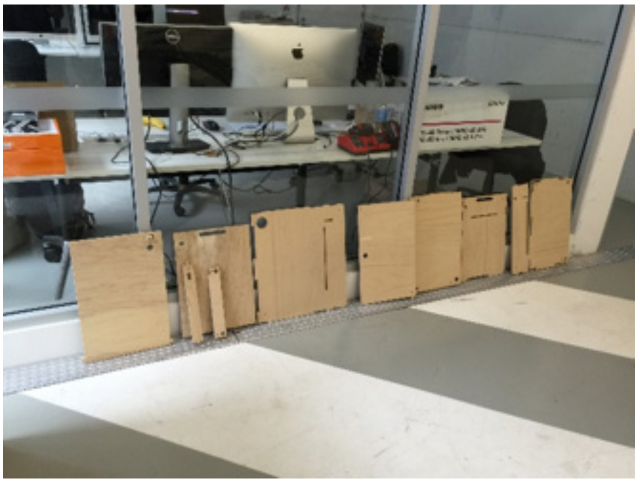


Figure 26: In-progress milling of the Open Desk Pedestal, UTS 2016

Figure 27: Completed Half Table, Roxanne Chair, Fin Storage, UTS 2016.

5.3.3 Open source products with parametric design applied

While the open source files offer access to manufacturing furniture, they do not offer flexibility in changing size to meet specific project requirements. While one option is to go back to the designer to make changes, UTS considered this as too rigid in approach for the design intention required for office fitout. Consequently, the position has been taken whereby open source furniture designs should have the capacity for user intervention. The native *Open Desk* files were rewritten in the same common software program and parametric plug-in mentioned previously (see section 5.2.3), using Rhino and Grasshopper. This approach was used to develop a select group of the open source files for storage furniture prototyping, as this was thought to have the widest applicability in office fitout situations.

The three storage units that had parametric design applied included the units shown in Figures 20 to 23. Each of these units can be adjusted in height, length, width, number of shelves, and number of dividers to suit a particular design or configuration, or respond to a number of varying workstation sizes.

5.4 Ceiling Tiles and Existing Products

Many office ceiling tiles are predominately mineral fibre products that are imported from China or the USA. They are a cheap surface that offers good acoustic performance. They are quite fragile and prone to breakage and water damage. Plywood products have potential as an alternative and should be considered where resilience, aesthetics and potential reuse capacity come into play or where ceiling tiles provided as part of the base building, remain in place from one tenant to the next.[6]

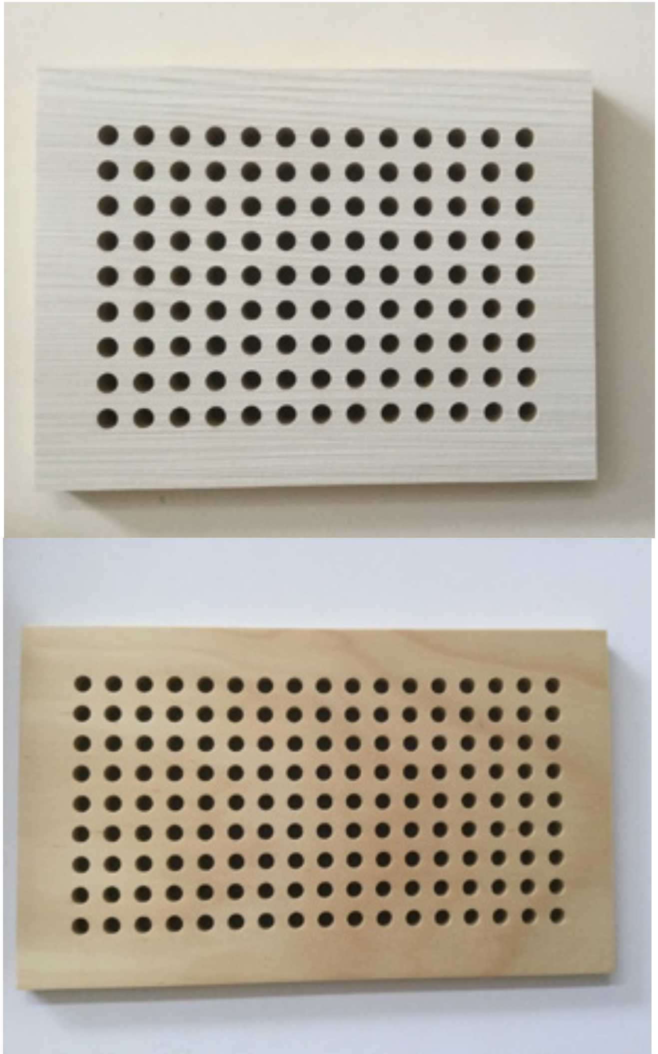


Figure 28: Ceiling tile example. Source: www.keystoneacoustics.com.au/key-plyKey Lena Sample, 2016.

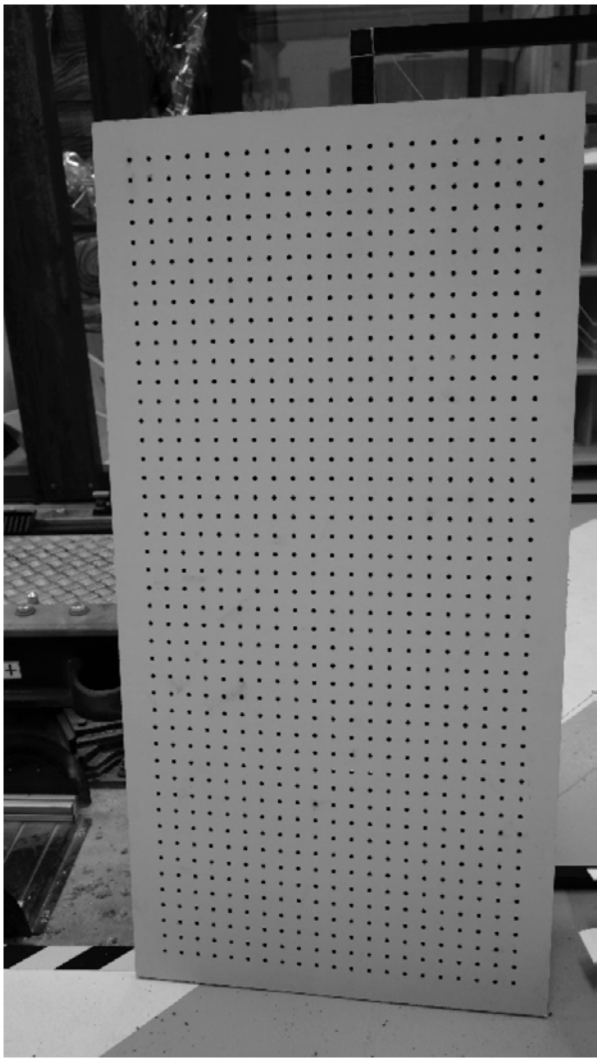


Figure 29: UTS CNC milled hardboard 1200x600 ceiling tile, UTS 2016.

[6] UTS manufactured its own standard 1195x595 mm ceiling tiles from 9 mm hardboard. While this material exhibits good recyclability potential – due to using natural lignin as a binder instead of glues – machining properties need to be improved as machining of slots and holes tends to result in furry edges.

6 A Life Cycle Assessment Comparing Wood-Based and Traditional Fitouts

Building on the previous description of wood fitout concepts, the concepts were compared with traditional office fitout to determine if any environmental benefits existed. The most appropriate and accepted method used to holistically assess the environmental impacts associated with a product – including construction products – is Life Cycle Assessment (Cole 1998; Junnila, Horvath & Guggemos 2006; Horne, Grant & Verghese 2009).

Ostensibly, the study used a typical 1,550m^2 floor plate to model both options (Figure 30). From this, materials, production and end-of-life processes were quantified to determine amounts of life cycle energy and greenhouses gas emissions (GHG) for each option. While the wood approach utilised the aforementioned designs, the traditional approach utilised steel stud and plasterboard partitions; aluminium and glass partitions;[7] a mix of suspended plasterboard ceilings tiles and decorative aluminium tiles in selected areas; workstations with MDF tops with metal support chassis. Of note, floor coverings were predominantly the same for both options and therefore represented a neutral variable in the comparison. The study is detailed fully in the parallel research report to this guide, 'Increasing Wood-based Office Fitout for Sustainable Life Cycle Benefits' (download at www.FWPA.com.au) – only key findings from this report are provided here.

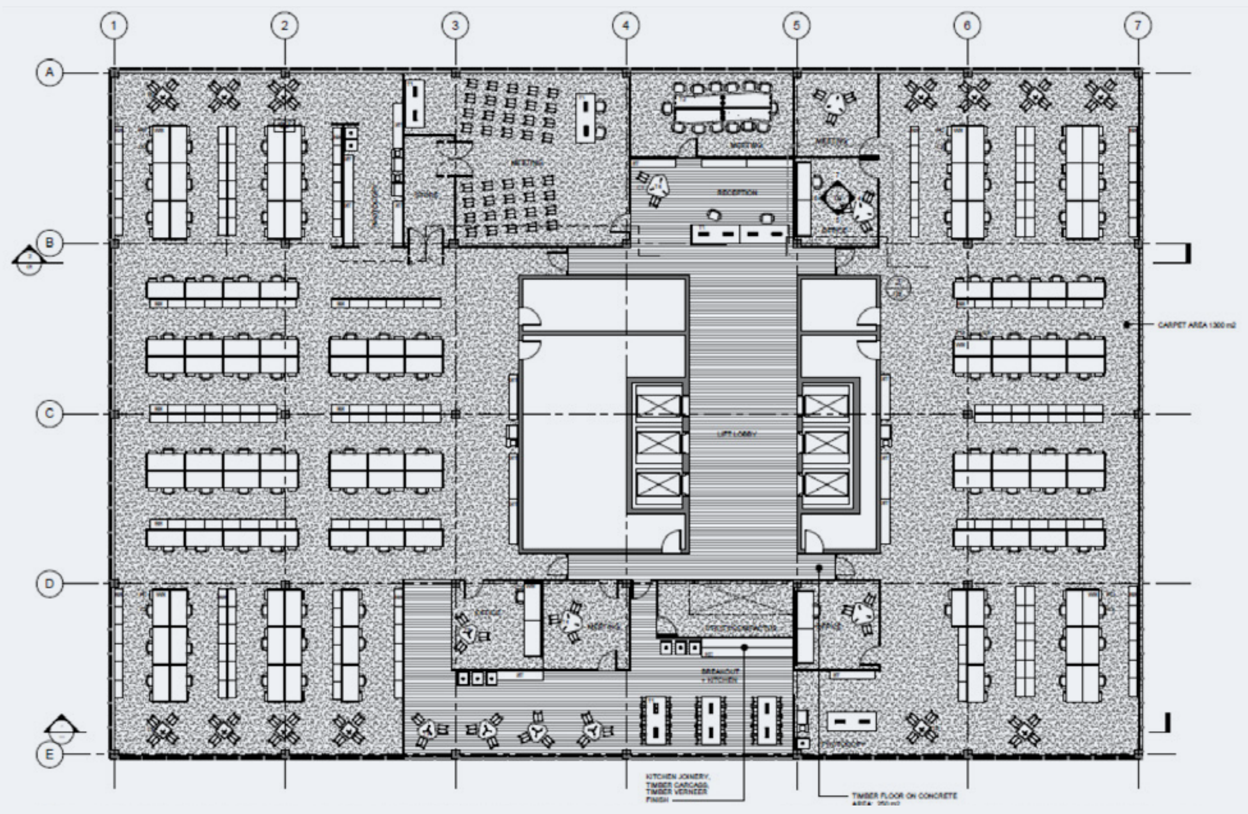
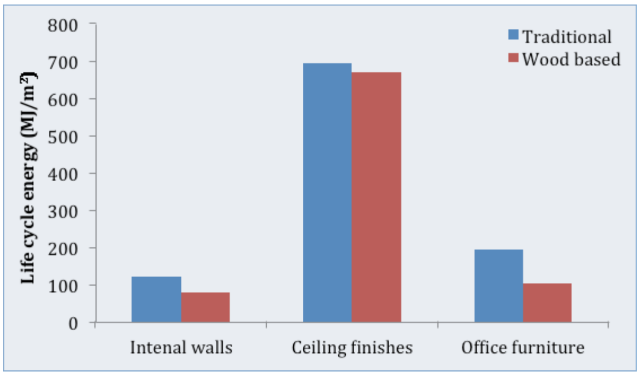


Figure 30: Hypothetical office space used for wood versus traditional LCA comparison.

[7] *In the wood version, aluminium extrusions were replaced with timber profiles.*

Drawing from the report findings, the traditional design consumes about 1,565,300 MJ or 1,009.8 MJ/m^2, while the wood-based design consumes 1,321,900 MJ or 852.8 MJ/m^2. The traditional approach therefore consumes about 16% more energy than the wood-based design.

Reading from Figure 31, the main areas of dominance concern the wood-based office furniture and partition walls. Here, office furniture consumes 195 MJ/m^2 and 103 MJ/m^2 respectively for traditional and wood-based design. Traditional office furniture consumes about 47% more energy than wood-based office furniture. Wood-based design of internal walls also provides a significant reduction in energy consumption – traditional represents 122 MJ/m^2 and the wood-based approach 80 MJ/m^2, a reduction of about 35%.



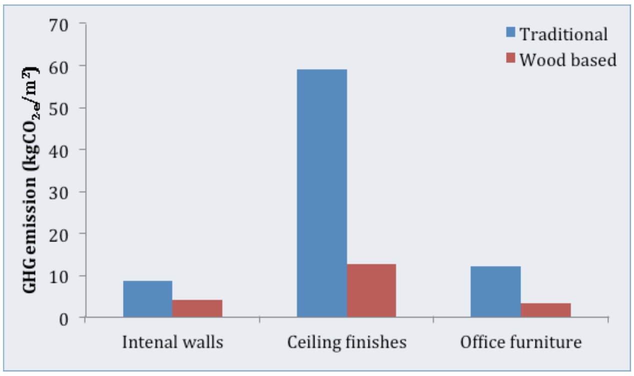


Figure 32: Comparing wood-based and traditional fitout designs.

With regards to GHG emissions the traditional design emits about 124,100 kgCO_2-e or 80 kgCO_2-e/m^2 compared to 31,400 kgCO_2-e or 20 kgCO_2Ðe/m^2 for the wood-based design. The traditional design emits about 75% per NLA more GHG than the wood-based design. The materials used in the traditional design, such as aluminium, steel and glass, are highly polluting materials that emit a high content of GHG during manufacturing processes. The traditional design of internal walls using metal stud and plasterboard emits about 54% more GHG than the wood-based partition panels. In addition traditional workstations and pedestals also emit about 72% more GHG than wood-based office furniture.

Given the above, key areas for life cycle improvement – where the wood-based approach improves on the traditional approach – include workstations (including wood instead of metal chassis) and partition walls (including hollow core panel walls).

7 Conclusions

The design concepts in this guide offer a canvas of production possibilities where it must be realised that the value of the material cannot be separated from the product. The designs particularly focus on partitions, workstations, furniture and suspended ceiling tiles. Key conclusions include:

- Workstations appear to consume the most significant resources and material mix. The strategic conclusions are to reduce the material palette to one material (wood-based products), a small kit of replaceable parts that could be used over, be of a material that does not rely on long lead times and could be made using a just-in-time approach.
- Collectively, there is not enough critical wood mass (that is sufficiently homogenous in terms of material type) to stimulate reuse and recycling markets. Increasing critical mass is important in order to make reuse and recycling economically viable during stripout processes.
- The design and construction industry employ established products and systems that are very familiar across the industry to designers, construction and stripout contractors. This report has cast construction and furniture systems in a different light by creating and discovering other construction methodologies that could benefit the industry.

Key benefits associated with the design concepts in this guide include:

- A significant increase in the use of wood products.
- A reduced material palette across all fitout items, focusing on a more homogenous set of wood-based materials.
- A significant simplification of construction methodologies across all fitout items that do not rely on mixed materials and metal fixing.
- A reduced number of trades involved in fitout.
- Workstations that can be made to order without the reliance on a lengthy ordering process.
- Workstations with components that can be easily disassembled and reassembled.
- Designs that are open source, including workstations that have parts that are not specific to any manufacturer's system.
- Designs that suit a furniture-driven approach to fitout that is well suited to open plan office designs.
- Reduced life cycle energy and much reduced GHG when compared to traditional fitout (especially for workstations and partition walls).

Even though this guide takes fitout design into a new mindset, the concepts are not reliant on each other and therefore allow individual uptake. The approach is also progressive in terms of taking advantage of the way wood lends itself to new digitally driven forms of production such as Design for Manufacture Assembly and Disassembly. Future work should consider developing this approach further and merging it with other arising technologies such as the Radio Frequency Identification tags (RFID tags) – already commonly used in retail and transport logistics. These tiny tags allow small amounts of digital information to be stored in the likes of furniture assemblies and can wirelessly read/write information, thus making it possible to track and store reuse, recycling and sustainability information for a given product both during, and up until end of life. This provides a systematic way to verify the sustainability of wood products.

.

8 References

Cole RJ, 1998, *Energy and greenhouse gas emissions associated with the construction of alternative structural systems*, Building and Environment, vol. 34, pp335-348.

Horne R, Grant T and Verghese K, 2009, *Life cycle assessment: principles, practice and prospects*, CSIRO Publishing, Collingwood, Vic.

Junnila S, Horvath A and Guggemos AA, 2006, *Life-Cycle Assessment of Office Buildings in Europe and the United States*, Journal of Infrastructure Systems no. March 2006, p8.

Gensler, 2016, *Workplace Forecast – A new ethos gives rise to new types of space*. www.gensler.com/2016-design-forecast-workplace.

Nolan G, 2011, *Timber in multi-residential, commercial and industrial building: Recognising opportunities and constraints.*

Wilmot K, McGee C and Milne G., 2014, *Market Research: Tenancy Fitout Material Procurement Attitudes and Practices*. Report prepared by the Institute for Sustainable Futures, University of Technology, Sydney, for Better Buildings Partnership.

44

WoodSolutions™
design and build
woodsolutions.com.au

This guide was published in March 2018
Please check woodsolutions.com.au/publications to ensure you have the latest edition

CLT Acoustic Performance

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Brock Commons Tallwood House, Acton Ostry Architects Inc

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-44-7

Prepared by:
PKA Acoustic Consulting
admin@pka.com.au

Acknowledgements
Author: Joel Parry-Jones

Reviewed by:
Peter Knowland, Simon Connolly

First Published: March 2018

Contents

1 Introduction

Cross Laminated Timber (CLT) is a modified timber building product in widespread use internationally. While its use in Australia is currently not widespread, the potential of the product for using in dividing walls, floors and ceilings is increasingly evident, particularly for multi storey residential buildings. Such buildings have particular acoustic requirements, mandated in Australia by the National Construction Code, Building Code of Australia (BCA).

Extensive acoustic research programs have been conducted across Europe and in North America on CLT's acoustic properties. The particulars, designed to meet local building codes, building practises and local materials, mean that it is not always possible to compare the results to the NCC's acoustic requirements.

PKA Acoustic Consulting (PKA) was commissioned by FWPA and the NSW Timber Development Association (TDA), to conduct and coordinate a research program into the acoustic performance of CLT products in various system configurations with the aim of compliance with the NCC.

CLT is available from a number of manufactures and suppliers. The study co-ordinated the supply of CLT panels from various suppliers for the acoustic assessment and testing. Information on the specific suppliers and products is set out in the table below.

Table 1.1: List of CLT and acoustic system suppliers.

Company	PKA Reference	Material Supply
Strongbuild Commercial Pty. Ltd.	Strongbuild	Binderholz CLT
Meyer Timber NSW Pty. Ltd.	Meyer Timber	Meyer Timber CLT
Stora Enso Australia Pty. Ltd.	Stora Enso	Stora Enso CLT
Xlam Australia Pty. Ltd.	Xlam	Xlam CLT
Dynamic Composite Technology Pty. Ltd.	DCTech	Proctor Q-Silence Floor Battens Proctor Q-Silence Floor Underlay DCT SolidEX Floor Screed DCT URSAcoustic

PKA cautions that this research program is limited to the sound insulation performance of CLT in an acoustic laboratory. The achievement of an acoustic result in an acoustic laboratory that complies with a particular code does not mean that compliance is automatically achieved on site.

Constructing buildings exclusively using CLT as load-bearing elements will require additional acoustic detailing of potential sound flanking paths to ensure compliance with the NCC's verification criteria when measured in- situ. Further acoustic testing may also be required in order to fully examine these issues.

Particular information on sound flanking in CLT constructions is available from the National Research Council of Canada: "Report to Research Consortium for Wood and Wood-Hybrid Mid-rise Buildings, Acoustics – Sound Insulation in Mid-Rise Wood Buildings" (2014) (Schoenwald, S.)

2 Executive Summary

Cross Laminated Timber (CLT) is a prefabricated solid engineered wood product made of several layers of timber boards stacked crosswise (at 90 degrees) and glued under pressure together to form a solid rectangular panel.

The Australian wood products industry, in conjunction with the Forest and Wood Products Australia Limited (FWPA), envisage considerable potential for the CLT as a construction material in amongst other areas, residential developments.

Such buildings normally have particular acoustic requirements, normally mandated in Australia by the National Construction Code, Building Code of Australia (NCC) or by a relevant Local Government Authority.

Previous acoustic research programs of CLT have been conducted across Europe and in North America. The applicability of this data to the local market is however limited due to the design of the testing to address codes that are not relevant to Australia. Further, the test elements often include construction materials that are not available or not in widespread use in Australia.

PKA Acoustic Consulting (PKA) was commissioned to coordinate a research program into the acoustic performance of CLT in various system configurations with the aim of compliance with the NCC.

The acoustic testing was carried out at New Zealand's Auckland University Acoustic Laboratory.

CLT panels are available from a number of different suppliers. This study coordinated the supply of test panels from various manufactures. The same CLT panel configuration from multiple suppliers was tested in order to consider the variability of CLT products.

2.1 Sound Insulation Criteria

The use of the NCC is mandated for attached and multistorey residential buildings in Australia.

Section F5 "Sound Transmission and Insulation" includes acoustic requirements for internal dividing elements as follows:

- F5.4 Dividing Floors
- F5.5 Dividing Walls
- F5.6 Services Isolation

Any CLT system configuration must, as a minimum, primarily address these requirements within the NCC.

Some Local Government Authorities consider that parts of Section F5 do not offer adequate amenity to occupants and have adopted more stringent acoustic requirements. Where possible CLT acoustic building systems should be designed or developed in recognition of the local government requirements. This report includes the Association of Australian Acoustical Consultants (AAAC) Star Rating system that is widely used as a preferred impact sound insulation rating of floors.

2.2 Complying Systems

The test program successfully identified various CLT systems that would meet the acoustic requirements of the NCC and the AAAC.

2.2.1 Dividing Walls

The acoustic research program identified that CLT panel in conjunction with plasterboard wall systems can comply with the Part F5.5 of the NCC pertaining to dividing walls separating sole-occupancy units (SOUs). The various wall configurations and applications are summarised as follows:

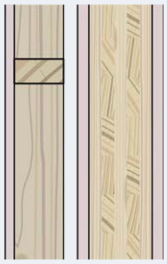
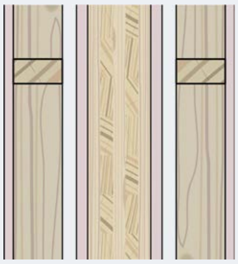
Criteria	Graphic	Brief Description	Thickness
$R_w + C_{tr} \geq 50$ Separating SOUs No cavity services Discontinuous construction		Separate stud one side Cavity insulation CLT panel with linings	219-238mm
$R_w + C_{tr} \geq 50$ Cavity services Discontinuous construction		Separate stud both sides Cavity insulation CLT panel with linings	328-354mm
$R_w + C_{tr} \geq 50$ Cavity services Discontinuous construction		Furring channel one side Separate stud one side Cavity insulation CLT panel with linings	296mm
$R_w + C_{tr} \geq 50$ Separating SOUs No cavity services Discontinuous construction		Double CLT panel with linings Cavity insulation	232mm
$R_w \geq 50$ Common wall* $R_w \geq 45$ Aged Care $R_w + C_{tr} \geq 40$ Shaft wall No cavity services Not discontinuous		Furring channel one side Cavity insulation CLT panel with linings	180-193mm

Figure 1.1: Complying dividing walls

Criteria	Graphic	Brief Description	Thickness
R_w ≥ 50 Common wall* R_w ≥ 45 Aged Care No cavity services Discontinuous construction		Separate stud one side Cavity insulation CLT panel with linings	219-225mm
R_w ≥ 50 Common wall* R_w ≥ 45 Aged Care No cavity services Discontinuous construction		Double CLT without linings Cavity insulation	200mm
$R_w + C_{tr}$ ≥ 25 Service Shaft wall Not discontinuous		CLT panel with or without linings	90-122mm

Figure 1.1: Complying dividing walls (continued)

** Common walls are walls separating SOUs from plant room, lift shaft, stairway, public corridor, lobbies or different building classifications.*

2.2.2 Dividing Floors

The acoustic research program identified that CLT panel in conjunction with floor toppings and plasterboard ceilings systems can comply with the Part F5.4 of the NCC pertaining to dividing floors separating sole-occupancy units (SOUs). The various floor configurations and applications are summarised as follows:

Criteria	Graphic	Brief Description	Thickness
$R_w + C_{tr} \geq 50$		Bare CLT panel with lining Cavity insulation Suspended or furring ceiling	249-367mm
$R_w + C_{tr} \geq 50$ $L_{nT,w} \leq 55$ (AAAC 3 Star)		Floor with acoustic underlay CLT panel with lining Cavity insulation Furring or suspended ceiling	286-404mm
$R_w + C_{tr} \geq 50$ $L_{nT,w} \leq 50$ (AAAC 4 Star)		Floor with acoustic underlay CLT panel with lining Cavity insulation Suspended ceiling	400-427mm
$R_w + C_{tr} \geq 50$ $L_{nT,w} \leq 50$ (AAAC 4 Star)		Resilient batten floor Cavity insulation CLT panel with lining Cavity insulation Suspended ceiling	417-443mm
$R_w + C_{tr} \geq 50$ $L_{nT,w} \leq 50$ (AAAC 5 Star)		Floor with resilient battens Cavity insulation CLT panel with lining Cavity insulation Suspended ceiling	453mm

Figure 1.2: Complying dividing floors.

2.3 Limitations

The acoustic research program identified that CLT panel in conjunction with floor toppings and plasterboard ceilings systems can comply with the Part F5.4 of the NCC pertaining to dividing floors separating sole-occupancy units (SOUs). The various floor configurations and applications are summarised in Section 8 of this guide.

3 Cross-laminated Timber (CLT)

3.1 CLT Description

Cross-laminated timber (CLT) typically refers to a prefabricated solid engineered wood product made of several layers of timber boards stacked crosswise (at 90 degrees) and glued under pressure to form a solid rectangular panel.

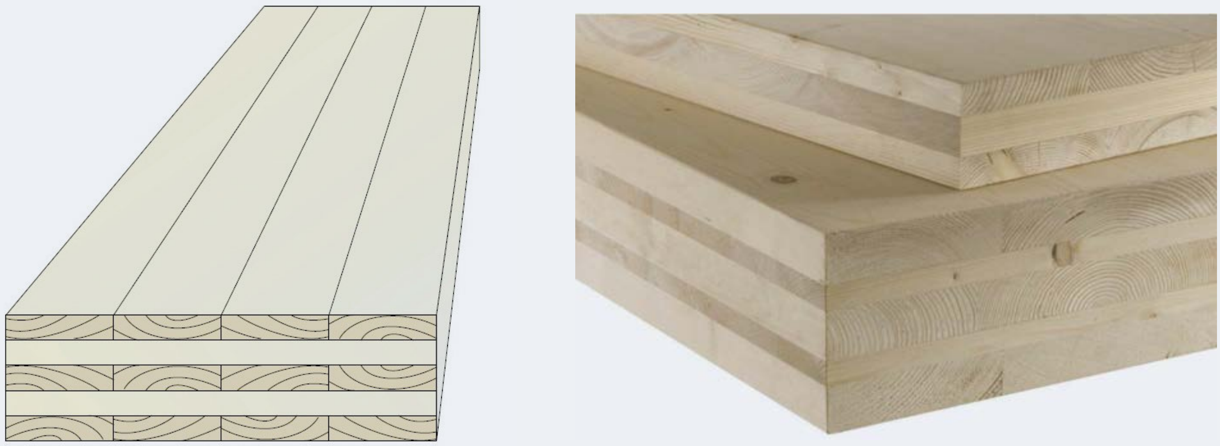


Figure 3.1: Cross-laminated timber (CLT) *Sources: Left - CLT Handbook: Cross-laminated Timber (U.S. Edition 2013) FP Innovations, Right - Massive Timber Construction Systems – Cross-laminated Timber (CLT) (2012) Forest and Wood Products Australia Limited*

A summary of the physical properties of CLT panels tested in this acoustic research program is set out in the table below.

Table 3.1: Physical properties of CLT panels.

Type	Product	Thickness mm	Layers	Density kg/m3	Mass kg/m2
Wall	Xlam	90	3 ply	472	42.5
Floor	Binderholz	140	5 ply	456	63.9
	Meyer			458	64.1
	Stora Enso			444	62.2
	Xlam			464	65.0
	Xlam	200	5 ply	464	92.8

3.2 CLT Connections

CLT panels are typically connected together on site due to production and transport limitations. There are several possible panel-to-panel connection options however in the preliminary discussions with the CLT suppliers it was determined that the most typical is the “cover-board” method which was used almost exclusively throughout the acoustic research program.

Cover-board connections involve the CLT panel edges profiled to take a strip of timber fastened with screws. During the acoustic testing it was apparent that sound leakage was occurring between the CLT panel connections. It was deemed necessary to seal the gap between the panels with sound- rated sealant prior to the installation of the cover-board to ensure the acoustic integrity of the CLT system was maintained.

For comparison purposes acoustic testing was performed of a CLT floor that was connected using a "half-lapped" method. This involves milling half of the connecting edge on both panels to allow an overlap. The panel edges are then typically fixed together with long self-tapping screws.

The two methods adopted in this acoustic research program are shown below.

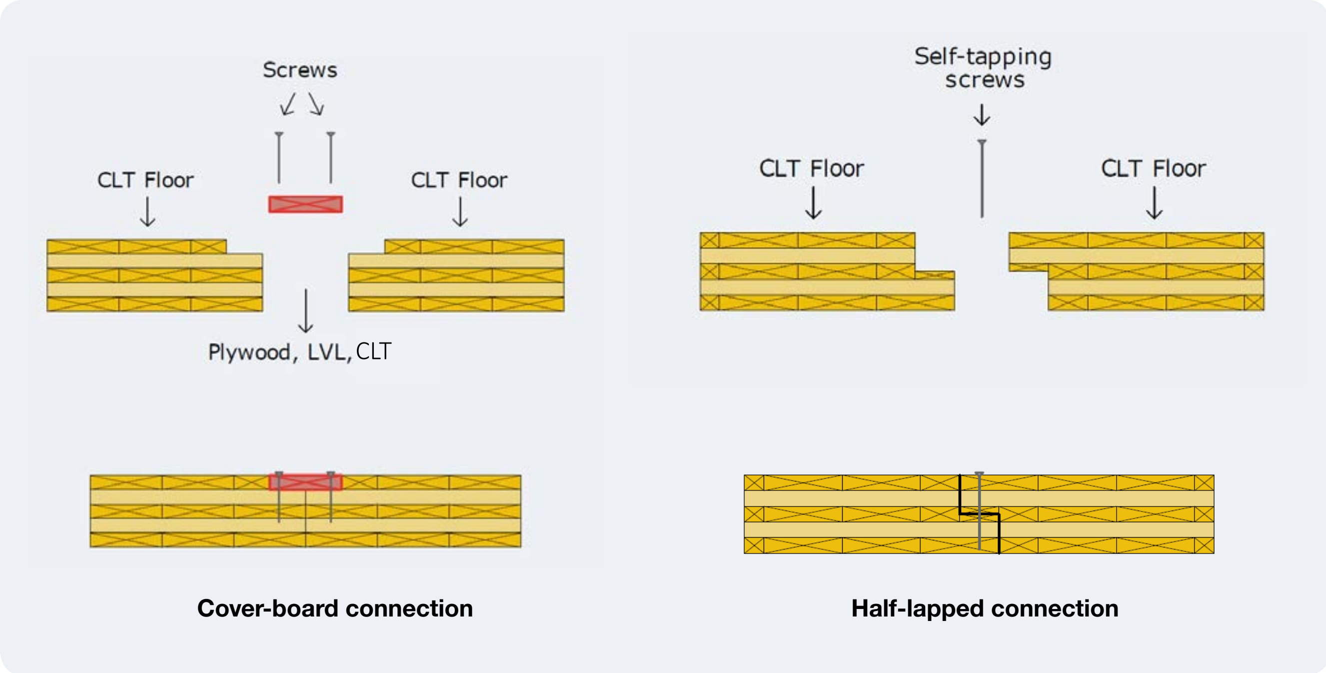


Figure 3.2: CLT connection types. *Source: CLT Handbook: Cross-laminated Timber (U.S. Edition 2013) FP Innovations*

3.3 CLT Build-up

NCC 2016 introduced a new deemed to satisfy solution for timber based building systems, called fire protected timber. One of the elements to fire protected timber is direct fixed fire rated plasterboard.

The acoustic research program adopted the direct fix fire-rated plasterboard approach as the default for CLT system configurations as follows:

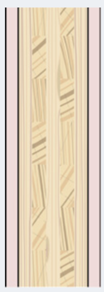
Building Element	Graphic	CLT Build-up
Wall		16mm fire-rated plasterboard screw fixed 90mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard screw fixed
Floor		140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard screw fixed

Figure 3.3: Fire protected timber

There were a few wall systems where one or both fire-rated plasterboard linings were removed to provide additional data on which future acoustic assessments could be based if fire rating is desired at the stud lining position. This data is detailed in Appendix A.

This report cautions that any air gaps or cavities that might arise from the direct fix lining being applied to the CLT has the potential to deteriorate the acoustic performance of the system. This can happen with the dabbed glue fixing method. If glue is to be used it must be applied evenly which typically requires a notched applicator.

To ensure that no gaps were present between the direct fix lining and CLT, the fire-rated plasterboard was directly fixed with screws at a maximum of 300mm centres throughout the entire acoustic testing program.

4 Assessment Methodology

PKA and TDA collaborated during the preliminary stages of the acoustic research program. The aim was to develop a comprehensive acoustic laboratory testing schedule of CLT wall and floor/ceiling systems that would allow the following:

- Compliance with the sound insulation criteria required by the relevant Australian codes
- Installation using building materials and practices typically employed in Australia
- A comparison of the test results for multiple CLT products to determine variability between manufacturers
- Compilation of comprehensive data for future acoustic assessments. This was achieved using a staged testing approach.

New Zealand's Auckland University Acoustic Laboratory (Auckland Laboratory) was selected as the location for the acoustic testing as it fulfilled the following principle requirements:

- Laboratory conforms to ISO 10140-5 (2010) *Acoustics – Laboratory measurement of sound insulation of building elements – Part 5: Requirements for test facilities and equipment*
- Laboratory equipment calibrated by an accredited association
- Ability for wall (airborne) and floor/ceiling (airborne and impact) acoustic testing
- Accessibility for chief CLT supplier Xlam, whose factory is based in New Zealand
- Affordable laboratory hire and reporting costs due to comprehensive testing regime

The acoustic testing was conducted over the period 26th April to 10th June 2016. The results were collated from over 107 separate airborne/impact tests involving 25 walls and 41 floor/ceiling configurations.

One of the aims was to determine the variability of multiple CLT products. It was deemed impractical, in both cost and time, to construct replica system configuration 4 times over only to change the CLT panel type. Instead, the test schedule was conducted entirely from one CLT supplier. The remaining CLT products were then tested in a round robin fashion as a floor panel.

PKA has assessed the variance between CLT products, in terms of both airborne and impact sound insulation performance, and provided the CLT suppliers with individualised assessments relating to each system configuration tested.

This report does not contain the individualised data sets due to protection of individual supplier's intellectual property. Instead this report provides generic test results derived from the range of CLT acoustic performance relating to each system configuration tested.

PKA notes that although the system configurations were designed with assistance from TDA to be practically buildable, the aim of this CLT research program relates to acoustic considerations only. Qualified personnel should be consulted with regard to specific requirements for other non-acoustic considerations.

5 Explanation of Acoustic Terms

The following definitions have been simplified to convey a practical meaning of the technical acoustic terms used throughout this assessment.

Sound Level

A perceptible sound level is a result of a pressure variation in the air generally between the source and the ear. Sound levels are expressed as decibels (dB).

Sound pressure level (SPL) is effectively the loudness of a sound from a particular sound source measured at a particular distance.

Decibel (dB)

A unit of measurement that represents sound levels. The human ear can perceive a large range of sound levels, however it responds to the change in sound levels in a non-linear fashion, therefore for convenience the decibel is a logarithmic unit of measurement.

The table below sets out the subjective effect of changes in sound level:

Table 5.1: Subjective effect of changes in sound level.

Change in Sound Level	Change in Acoustic Energy	Change in Loudness
3 dB	2 times	Just Perceptible
5 dB	3 times	Clearly Perceptible
10 dB	10 times	Double/Half the Loudness
20 dB	100 times	Much Louder/Quieter

For example a 1-2dB change is unlikely to be perceptible, however a change of 5-10dB will be a significant increase or decrease in loudness.

Airborne Sound Insulation (R_w, $D_{nT,w}$, STC)

Airborne sound is a sound source that originates in the air such as a person talking or a loudspeaker, as opposed to impact sound which strikes a surface such as footsteps.

Airborne sound insulation is the difference in sound pressure level between the sound entering and sound leaving a building element. The higher the value the better the sound insulation.

The **R_w** rating, defined as a "Weighted Sound Reduction Index", is an acoustic laboratory measurement that determines the effectiveness of a building element's airborne sound insulation over a range of frequencies (100Hz to 3150Hz) in a single number quantity. The Rw value, expressed in dB, is corrected for room volume and reverberation time but does not take into account sound flanking paths associated with in-situ installations.

The **$D_{nT,w}$** rating, defined as "Weighted Standardised Level Difference", is similar to R_w but is measuring the building element's airborne sound insulation in-situ rather than an acoustic laboratory. The $D_{nT,w}$ is more indicative in determining the actual airborne sound insulation between spaces as it accounts for sound flanking paths and construction quality.

The **STC** rating, defined as "Sound Transmission Class" adopted in North America and New Zealand, is an acoustic laboratory measurement similar to Rw, however the range of assessing frequencies is shifted upwards to 125Hz to 4000Hz. For typical building elements the STC and R_w ratings are largely similar except where there is a significant decrease in performance at 100Hz which results in the R_w rating decreasing from the STC.

Spectrum Adaption Term (C_{tr})

The C_{tr} term is an airborne low frequency adjustment factor that helps quantify the low frequency performance of the building element from sources such as traffic, music, televisions etc. C_{tr} is a negative value that is added to an R_w or a $D_{nT,w}$ rating.

Impact Sound Insulation ($L_{n,w}$, $L_{nT,w}$, IIC)

Impact sound is a sound source that typically originates by striking a floor surface such as footsteps or moving furniture. Impact sounds can be transmitted through walls via lifts, plantrooms, washing machines, service pipes etc. however there is no functioning criteria for assessing these structure-borne sounds except for adopting discontinuous constructions.

Impact sound insulation is the resultant sound pressure level measured in the receive space when an ISO standard tapping machine is placed in the source space on the separating floor/ceiling. The lower the value the better the sound insulation.

The $\mathbf{L_{n,w}}$ rating, defined as "Weighted Normalised Impact Sound Pressure Level", is an acoustic laboratory measurement to determine the effectiveness of a building element's impact sound insulation over a range of frequencies (100Hz to 3150Hz) in a single number quantity. The $L_{n,w}$ value, expressed in dB, is corrected for room volume and reverberation time but does not take into account sound flanking paths associated with in-situ installations.

The $\mathbf{L_{nT,w}}$ rating, defined as "Weighted Normalised Impact Field Sound Pressure Level", is similar to $L_{n,w}$ however measures the building element's impact sound insulation in-situ rather than an acoustic laboratory. The $L_{nT,w}$ is more indicative in determining the actual impact sound insulation between spaces as it accounts for sound flanking paths and construction quality.

The **IIC** rating, defined as "Impact Insulation Class" adopted in North America and New Zealand, is an acoustic laboratory measurement that is derived from the sound pressure levels measured as described above. The IIC rating is not related to the $L_{n,w}$ rating as a higher IIC equates to a better sound insulation, however an approximate conversion is typically applied whereby the $L_{n,w}$ = 110 - IIC.

Spectrum Adaption Term (C_i)

The $\mathbf{C_i}$ term is an impact adjustment factor which is typically a negative value effectively improving the impact rating of the floor/ceiling system. The C_i term was included in the BCA 2004 following its adoption in the UK building code, but was soon discarded in the UK when it was found that known deficient constructions were achieving compliance. The C_i term was finally removed in the BCA 2016 due to pressure from the Association of Australian Acoustical Consultants (AAAC). The detailed test data state the C_i term for information purposes only.

6 Sound Insulation Criteria

6.1 NCC Criteria For Assessment in an Acoustic Laboratory

The NCC 2016, in Volume 1 Section F5 "Sound Transmission and Insulation" states that walls and floors separating places of occupancy *"must provide insulation against the transmission of airborne and impact generated sound sufficient to prevent illness or loss of amenity to the occupants".*

The following summarises the acoustic laboratory design requirements, brevity necessitates detail in the NCC taking precedence over the tables below.

Table 6.1: NCC sound insulation acoustic laboratory criteria for walls.

Wall Description	BCA Reference	Sound Insulation Requirements	
		Airborne	Impact
Separating sole-occupancy units (SOUs) habitable areas	F5.5(a)(i)	$R_w + C_{tr} \geq 50$	
Separating SOUs wet to habitable areas	F5.5(a)(i) F5.5(a)(iii)	$R_w + C_{tr} \geq 50$	Discontinuous Construction
Separating SOUs with corridor, stairway, lobby or different classification	F5.5(a)(ii)	$R_w \geq 50$	
Separating SOUs with plantroom or lift shaft	F5.5(a)(ii) F5.5(a)(iii)	$R_w \geq 50$	Discontinuous Construction
Separating Class 9c aged care SOU generally	F5.5(c)	$R_w \geq 45$	
Separating Class 9c aged care SOU with kitchen or laundry	F5.5(c)	$R_w \geq 45$	Discontinuous Construction
Separating SOU habitable area with services from another SOU	F5.6(a)(i)	$R_w + C_{tr} \geq 40$	
Separating SOU wet area with services from another SOU	F5.6(a)(ii)	$R_w + C_{tr} \geq 25$	

The NCC denotes "Discontinuous Construction" as follows:

Wall Type	Reference	Discontinuous Construction Requirement
Masonry	F5.3(c)(i)	Wall having a minimum 20mm cavity between the 2 separate leaves, with resilient wall ties if necessary
Other than masonry	F5.3(c)(ii)	Wall having a minimum 20mm cavity with no mechanical linkage except at the periphery

Table 6.2: NCC sound insulation acoustic laboratory criteria for floors.

Floor Description	BCA Reference	Sound Insulation Requirements	
		Airborne	Impact
Separating sole-occupancy units (SOUs)	F5.4(a)(i)	$R_w + C_{tr} \geq 50$	$L_{n,w} \leq 62$
Separating SOUs with plantroom, lift shaft, corridor, stairway, lobby or different classification	F5.4(a)(ii)	$R_w + C_{tr} \geq 50$	$L_{n,w} \leq 62$
Separating Class 9c aged care SOU generally	F5.4(b)	$R_w \geq 45$	
Separating SOU habitable area with services from another SOU	F5.6(a)(i)	$R_w + C_{tr} \geq 40$	
Separating SOU wet area with services from another SOU	F5.6(a)(ii)	$R_w + C_{tr} \geq 25$	

6.2 NCC Criteria for Assessment In-Situ Verification

The aim of the NCC acoustic laboratory design requirements is to allow compliance with the verification criteria FV5.1 and FV5.2 when measured in-situ. In most cases the verification criteria allows for a 5dB reduction in sound insulation performance between an acoustic laboratory test and a field test on site. This is due to sound flanking paths and variation of quality workmanship in construction.

Table 6.3: NCC sound insulation in-situ verification criteria: walls.

Wall Description	NCC Reference	Sound Insulation Requirements	
		Airborne	Impact
Separating sole-occupancy units (SOUs)	FV5.2(a)	$D_{nT,w} + C_{tr} \geq 45$	
Separating SOUs with plantroom, lift shaft, corridor, stairway, lobby or different classification	FV5.2(c)	$D_{nT,w} \geq 45$	

Table 6.4: NCC sound insulation in-situ verification criteria: floors.

Floor Description	NCC Reference	Sound Insulation Requirements	
		Airborne	Impact
Separating sole-occupancy units (SOUs)	FV5.1(a) FV5.1(b)	$D_{nT,w} + C_{tr} \geq 45$	$L_{nT,w} \leq 62$

6.3 NCC Criteria Discussion

The goal of the NCC is to *"enable the achievement of nationally consistent, minimum necessary standards"*. PKA considers the various airborne sound insulation criteria in the NCC to be of a high standard, which accordingly achieves a 4 Star Rating by the Association of Australian Acoustical Consultants (AAAC) in their document *"Guideline of Apartment and Townhouse Acoustic Rating 2010".*

However, along with the vast majority of Australian acoustic consultants, PKA considers the floor impact sound insulation verification criterion of $L_{nT,w} \leq 62$ to be of a poor standard resulting in a AAAC 2 Star Rating.

In PKA's experience, a typically non-carpeted floor/ceiling installation that simply complies with the NCC impact sound insulation criterion often leads to noise complaints from adjoining occupants. Impact generated sounds, such as footsteps and moving furniture, result in disturbances that cause significant anguish to occupants often leading to costly legal battles in consumer tribunals.

Therefore it is PKA's opinion that the NCC is in direct conflict with its own performance requirements to *"provide insulation against the transmission of airborne and impact generated sound sufficient to prevent illness or loss of amenity to the occupants".*

The following table demonstrates the AAAC Star Rating improvement over the NCC floor impact criterion and provides comments as to PKA's typical recommendation for floor/ceiling constructions involving hard surfaces:

Table 6.5: AAAC Star Rating improvement over the NCC floor impact criterion.

AAAC Star Rating	AAAC Criteria	Improvement over NCC Criterion of $L_{nT,w} \leq 62$	PKA Advice For Hard Surface Floor/Ceiling Constructions
3	$L_{nT,w} \leq 55$	7dB	Recommended as minimum for standard residential apartments
4	$L_{nT,w} \leq 50$	12dB	Recommended as minimum for luxury residential apartments
5	$L_{nT,w} \leq 45$	17dB	Recommended for luxury residential apartments
6	$L_{nT,w} \leq 40$	22dB	Recommended where hard floor is to have comparable high performance to a carpeted floor. This is generally not feasible

We note certain Councils require superior impact sound insulation beyond the NCC criteria. In PKA's experience this can range from 3 Star to 5 Star Ratings. When designing floor/ceiling systems, acoustic advice should be sought to determine the appropriate criteria taking into account location and intended quality of the development.

7 Acoustic Laboratory Testing Program

The acoustic research program was conducted at New Zealand's Auckland University Acoustic Laboratory (Auckland Laboratory). As listed in Section 4 Assessment Methodology the Auckland Laboratory was selected as it fulfilled the following principle requirements:

- Laboratory conforms to ISO 10140-5 (2010) *Acoustics – Laboratory measurement of sound insulation of building elements – Part 5: Requirements for test facilities and equipment*
- Laboratory equipment calibrated by an accredited association (Electroacoustic Calibration Services (ECS), an International Accreditation New Zealand (IANZ) registered laboratory)
- Ability for wall (airborne) and floor/ceiling (airborne and impact) acoustic testing
- Accessibility for chief CLT supplier Xlam, whose factory is based in New Zealand
- Affordable laboratory hire and reporting costs due to comprehensive testing regime

The acoustic testing was conducted over the period 26th April to 10th June 2016. The results were collated from over 107 separate airborne/impact tests involving 25 walls and 41 floor/ceiling configurations.

A representative of TDA attended the Auckland Laboratory for the duration of the acoustic testing program to ensure that the CLT system configurations were being installed correctly by the resident tradespeople and to document the building material properties.

A representative of PKA, Joel Parry-Jones, attended the Auckland Laboratory during the first few days of acoustic testing to inspect the CLT panel constructions and provide advice regarding installation of specific acoustic products.

The testing was conducted by Gian Schmid of Auckland University who has over 15 years of experience in acoustic laboratory testing.

The following standards are adopted by the Auckland Laboratory:

Table 7.1: Auckland acoustic laboratory standards.

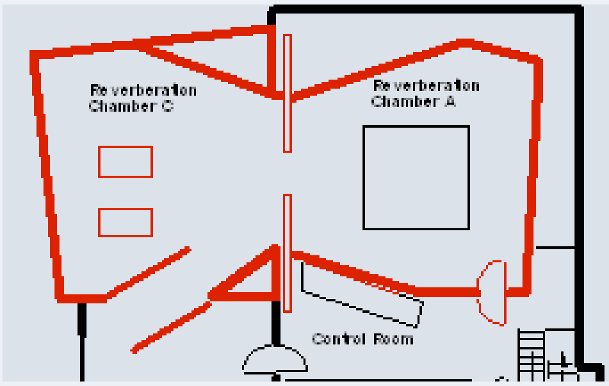
Type	Standard	Description
Airborne testing	ISO 10140-2:2010	Laboratory measurement of sound insulation of building elements Part 2: Measurement of airborne sound insulation
Impact testing	ISO 10140-3:2010	Laboratory measurement of sound insulation of building elements Part 3: Measurement of impact sound insulation
Airborne testing	ISO 717-1:2013	Acoustics – Rating of sound insulation in building and of building elements Part 1: Airborne sound insulation
	ASTM E413	Classification for Rating Sound Insulation
Impact testing	ISO 717-2:2013	Acoustics – Rating of sound insulation in building and of building elements Part 2: Impact sound insulation
	ASTM E989	Standard Classification for Determination of Impact Insulation Class (IIC)

The Auckland Laboratory has three large interconnected reverberation chambers. The chambers are shaped as asymmetrical hexagonal prisms that employ fixed and rotating diffusers to evenly distribute the sound field.

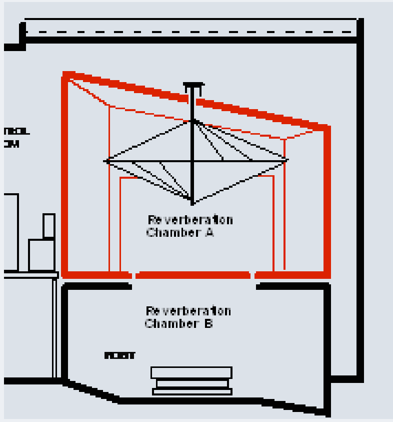
Figure 7.1: Reverberation Chamber A with bare CLT wall and floor installed.

Figure 7.2: Reverberation Chamber B with CLT floor and direct fixed plasterboard installed.

The reverberation chambers details and arrangements are as follows:



Plan View



Section View

Figure 7.3: Auckland Laboratory Reverberation Chamber details.

Element	**Test Type**	**Source**	**Receive**
Floor/Ceiling	Airborne and Impact	Chamber A	Chamber B
Wall	Airborne	Chamber C	Chamber A

Description	**Location**	**Volume**
Chamber A	Ground	$202m^3 \pm 3$
Chamber B	Basement	$153m^3 \pm 2$
Chamber C	Ground	$209m^3 \pm 4$

8 Summary of Acoustic Assessment

As detailed in Section 4 Assessment Methodology, PKA have assessed the range of acoustic performance expected in each CLT system configuration based on the Auckland University test data which incorporated a round robin of four different CLT products.

The following tables present the CLT system configurations organised into each BCA sound insulation criteria listed in Section 6.

8.1 Complying CLT Wall Systems

The complete descriptions of the building materials and system configurations tested are provided in Appendix A Detailed Test Data. For this summary the system configurations have been abbreviated as follows:

Table 8.1: Abbreviations and their meaning.

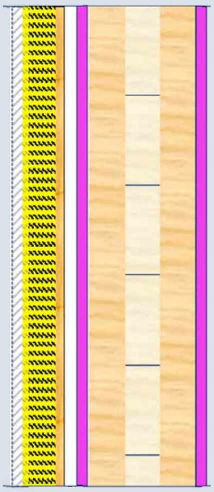
Abbreviation	Detailed Description
90mm Cross-Laminated Timber (CLT)	90mm Cross-Laminated Timber (CLT) wall 3 ply (40.0 – 42.5kg/m^2)
13mm standard plasterboard	13mm GIB standard plasterboard (min. 8.6kg/m^2)
13mm sound-rated plasterboard	13mm GIB Noiseline sound-rated plasterboard (min. 12.5kg/m^2)
16mm fire-rated plasterboard	16mm GIB Fyreline fire-rated plasterboard (min. 13.7kg/m2) *
9mm fibre cement sheet	9mm fibre cement sheet (min. 13.5kg/m^2)
50mm glasswool insulation	50mm DCT URSAcoustic glasswool insulation R1.4 (min. 18kg/m^3) or 50mm Bradford Acoustigard glasswool insulation R1.3 (min. 14kg/m^3)
50mm glasswool insulation	50mm Bradford Acoustigard glasswool insulation R1.3 (min. 14kg/m^3)
75mm glasswool insulation	75mm DCT URSAcoustic glasswool insulation R1.8 (min. 17kg/m^3) or 75mm Bradford Acoustigard glasswool insulation R1.8 (min. 14kg/m^3)
70mm timber stud	70mm x 45mm timber stud (cc 600mm)
64mm steel stud	64mm Rondo steel stud 0.50BMT (cc 600mm)
28mm furring channel	28mm Rondo 129 furring channel (cc 600mm)
Adjustable clip	30mm Rondo BETAGRIP1 BG01 adjustable clip (cc 1200mm)
Resilient mount	Rondo STWC resilient mount (cc 1200mm)

* The 16mm fire-rated plasterboard available for the acoustic laboratory testing in New Zealand is manufactured by GIB and is approximately 1kg/m^2 heavier at 13.7kg/m^2 than the leading Australian fire-rated plasterboard products averaging at 12.5kg/m^2. For the research program fire-rated plasterboard was exclusively used direct fix to the CLT panel and resulted in a maximum of 3dB improvement compared to the bare CLT panel. It is PKA's opinion that substituting Australian fire- rated plasterboard at nominal 12.5kg/m^2 as a direct fix lining would not deteriorate the acoustic performance of the wall beyond expected tolerances.

We note that any other deviations from the specific material properties tested may affect the acoustic performance of the CLT system. Advice must be sought from a qualified acoustic consultant as well as specific requirements for other non-acoustic considerations.

Figure 8.1: $R_w + C_{tr} \geq 50$ Discontinuous Construction

Configuration

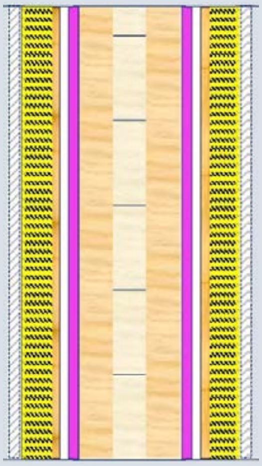


Lining
Stud frame
75mm glasswool insulation
Gap
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Stud Frame	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W02-05	T1617-89	2x13mm standard plasterboard	70mm timber stud 20mm gap	238	58 - 59	**52**	58 - 59
W03-04	T1617-44	2x13mm standard plasterboard	64mm steel stud 20mm gap	232	59 - 60	**52**	60 - 61
W03-05	T1617-46	1x13mm sound-rated plasterboard	64mm steel stud 20mm gap	219	58 - 59	**50**	59 - 60
W03-06	T1617-47	2x13mm sound-rated plasterboard	64mm steel stud 20mm gap	232	60	**53 - 54**	60 - 61

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

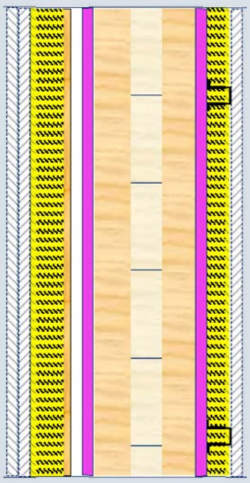
Figure 8.2: $R_w + C_{tr} \geq 50$ Discontinuous Construction

Configuration	
	Lining Side 1 70mm timber stud 75mm glasswool insulation 20mm Gap 16mm fire-rated plasterboard 90mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 20mm Gap 75mm glasswool insulation 70mm timber stud Lining Side 2

System	Test	Lining Side 1	Lining Side 2	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W07-03	T1617-91	1x13mm standard plasterboard	1x13mm standard plasterboard	328	64 - 65	**52 - 53**	66
W07-04	T1617-90	2x13mm standard plasterboard	1x13mm standard plasterboard	341	68 - 69	**58 - 59**	69 - 70
W07-05	T1617-19	2x13mm standard plasterboard	2x13mm standard plasterboard	354	69 - 70	**63**	70 - 71

Note:
Services are permitted in both cavities when separating habitable areas
Services are permitted in both cavities when separating wet areas

Figure 8.3: $R_w + C_{tr} \geq 50$ Discontinuous Construction

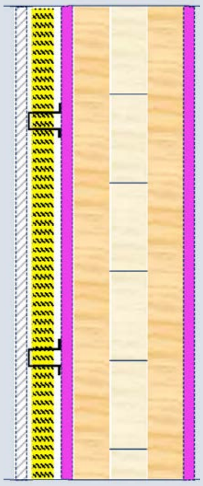
Configuration	
	Lining Side 1 70mm timber stud 75mm glasswool insulation 20mm Gap 16mm fire-rated plasterboard 90mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 50mm glasswool insulation Adjustable clip (45mm total cavity) 28mm furring channel Lining Side 2

System	Test	Lining Side 1	Lining Side 2	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W08-02	T1617-93	2x13mm standard plasterboard	1x13mm standard plasterboard	296	65 - 66	**52 - 53**	67

Note:
Services are permitted in both cavities when separating habitable areas
Services are permitted in both cavities when separating wet areas

Figure 8.4: $R_w \geq 50$

Configuration



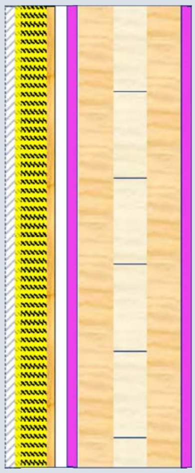
Lining
28mm furring channel
Clip or mount (min. 45mm cavity)
50mm glasswool insulation
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Connection	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W05-02	T1617-94	1x13mm standard plasterboard	Adjustable clip	180	**52**	**42**	53 - 54
W06-01	T1617-37	1x13mm standard plasterboard	Resilient mount	180	**51 - 52**	**42 - 43**	53
W06-02	T1617-38	2x13mm standard plasterboard	Resilient mount	193	**56**	**47**	57 - 58

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.5: $R_w \geq 50$ Discontinuous Construction

Configuration



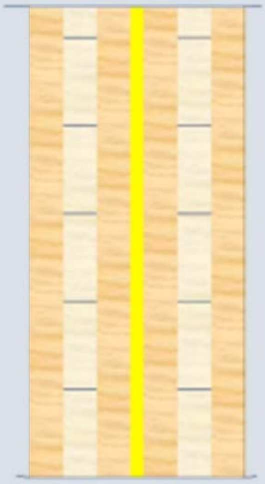
Lining
Stud frame
75mm glasswool insulation
Gap
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Stud Frame	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W02-02	T1617-88	1x13mm standard plasterboard	70mm timber stud 20mm gap	225	**56**	**48**	56 - 57
W04-02	T1617-43	1x13mm standard plasterboard	64mm steel stud 20mm gap	219	**58**	**48 - 49**	58 - 59

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.6: R_w ≥ 50 Discontinuous Construction

Configuration



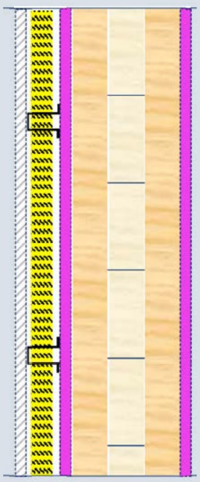
90mm Cross-Laminated Timber (CLT)
20mm gap
50mm glasswool insulation
90mm Cross-Laminated Timber (CLT)

System	Test	Lining Side 1	Lining Side 2	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W09-01	T1617-97	Nil	Nil	200	54 - 55	**47**	55 - 56

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.7: R_w ≥ 45 Class 9c - Aged Care

Configuration



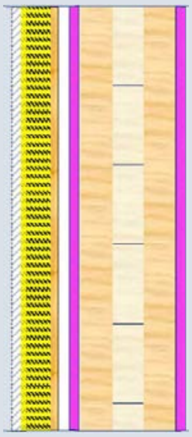
Lining
28mm furring channel
Clip or mount (min. 45mm cavity)
50mm glasswool insulation
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Connection	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W05-02	T1617-94	1x13mm standard plasterboard	Adjustable clip	180	**52**	42	53 - 54
W06-01	T1617-37	1x13mm standard plasterboard	Resilient mount	180	**51 - 52**	42 - 43	53

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.8: $R_w \geq 45$ Discontinuous Construction, Class 9c - Aged Care

Configuration



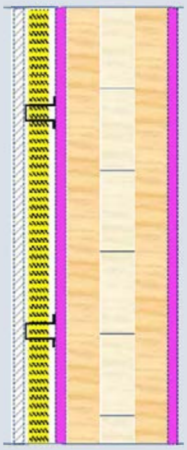
Lining
Stud frame
Cavity insulation
Gap
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Stud Frame	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W02-01	T1617-07	1x13mm standard plasterboard	70mm timber stud 20mm gap No cavity insulation	225	**47 - 48**	41	48 - 49
W02-02	T1617-88	1x13mm standard plasterboard	70mm timber stud 20mm gap 75mm glasswool insulation	225	**56**	48	56 - 57
W04-02	T1617-43	1x13mm standard plasterboard	64mm steel stud 20mm gap 75mm glasswool insulation	219	**58**	48 - 49	58 - 59

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.9: $R_w + C_{tr} \geq 40$

Configuration



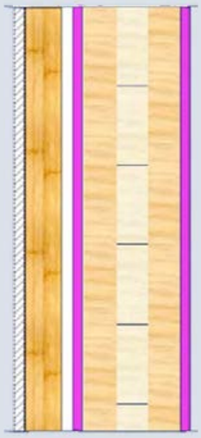
Lining
28mm furring channel
Clip or mount (min. 45mm cavity)
50mm glasswool insulation
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Connection	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W05-02	T1617-94	1x13mm standard plasterboard	Adjustable clip	180	**52**	42	53 - 54
W06-01	T1617-37	1x13mm standard plasterboard	Resilient mount	180	**51 - 52**	42 - 43	53

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.10: $R^w + C_{tr} \geq 40$

Configuration



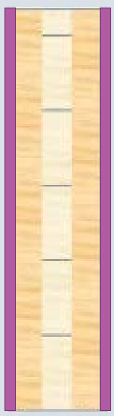
Lining
Stud frame
Gap
No cavity insulation
16mm fire-rated plasterboard
90mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard

System	Test	Lining	Stud Frame	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W02-01	T1617-07	1x13mm standard plasterboard	70mm timber stud 20mm gap No cavity insulation	225	**47 - 48**	41	48 - 49

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

Figure 8.11: $R_w + C_{tr} \geq 25$

Configuration



Lining Side 1
90mm Cross-Laminated Timber (CLT)
Lining Side 2

System	Test	Lining Side 1	Lining Side 2	Thickness (mm)	R_w	$R_w + C_{tr}$	STC
W01-01	T1617-86	Nil	Nil	90	**33 - 34**	30 - 31	33 - 35
W01-02	T1617-95	16mm fire-rated plasterboard	Nil	106	**36 - 38**	33 - 34	37 - 38
W01-03	T1617-87	16mm fire-rated plasterboard	16mm fire-rated plasterboard	122	**36 - 38**	34 - 35	36 - 38

Note:
Services are **not** permitted in the cavity when separating habitable area
Services are permitted in the cavity when separating wet area

8.3 Complying CLT Floor/Ceiling Systems

The complete descriptions of the building materials and system configurations tested are provided in Appendix A Detailed Test Data. For this summary assessment the system configurations have been abbreviated as follows:

Table 8.2: Abbreviations and their meaning.

PKA Abbreviation	Detailed Description
140mm Cross-Laminated Timber (CLT)	140mm Cross-Laminated Timber (CLT) wall 5 ply (62.2 - 65kg/m^2)
13mm standard plasterboard	13mm GIB standard plasterboard (min. 8.6kg/m^2)
16mm fire-rated plasterboard	16mm GIB Fyreline fire-rated plasterboard (min. 13.7kg/m^2)*
9mm fibre cement sheet	9mm fibre cement sheet (min. 13.5kg/m^2)
20mm strand board floor	20mm Strandboard floor (min. 14.2kg/m^2)
40mm screed	40mm sand-cement screed (min. 80kg/m^2)
25mm DCT SolidEX Screed	25mm DCT SolidEX screed (min. 59kg/m^2)
20mm DCT URSA TerraSol T70P mineral wool	20mm DCT URSA TerraSol T70P Terra Sol T70P mineral wool insulation (7kg/m^3)
50mm glasswool insulation	50mm DCT URSAcoustic glasswool insulation R1.4 (min. 18kg/m^3) or 50mm Bradford Acoustigard glasswool insulation R1.3 (min. 14kg/m^3)
75mm glasswool insulation	75mm DCT URSAcoustic glasswool insulation R1.8 (min. 17kg/m^3) or 75mm Bradford Acoustigard glasswool insulation R1.8 (min. 14kg/m^3)
185mm suspended ceiling with Resilient mounts	185mm Rondo suspension ceiling with Rondo STSU resilient mounts (cc 1000mm x 600mm) See test data in Appendix A for detailed description
67mm furring channel ceiling with Resilient mounts	67mm Rondo furring channel ceiling with Rondo STSL resilient mounts (cc 1000mm x 600mm) See test data in Appendix A for detailed description
Adjustable clip	50mm Rondo Betagrip2 BG02 adjustable clip (cc 1000mm)
10mm rubber underlay	10mm Embleton Impactamat rubber acoustic underlay

* As discussed in Section 8.1, it is this report's opinion that the 16mm fire-rated plasterboard with a mass of 13.7kg/m^2 can be substituted for Australian fire-rated plasterboard at nominal 12.5kg/m^2.

We note that any other deviations from the specific material properties tested may affect the acoustic performance of the CLT system. Advice must be sought from a qualified acoustic consultant as well as specific requirements for other non-acoustic considerations.

Figure 8.12: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 62$ (BCA) - Not recommended (see page 15)

Configuration

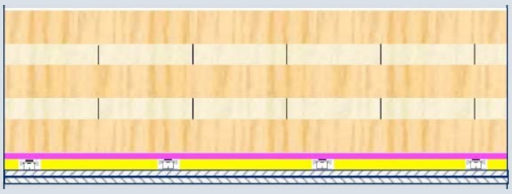
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
185mm suspended ceiling with Resilient mounts
75mm glasswool insulation
Lining

System	Test	Floor Covering	Lining	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F04-02	T1617-08	Bare CLT	1x13mm standard plasterboard	354	57 - 59	**51 - 53**	58 - 59	**59 - 61**	49 - 51
F06-01	T1617-09	Bare CLT	2x13mm standard plasterboard	367	59 - 61	**53 - 55**	59 - 61	**57 - 58**	52 - 53

Note:
If CLT floor panels traverse between SOUs without a resilient floor, it is most likely that compliance will not be achieved with BCA sound insulation criteria horizontally due to sound flanking.

Figure 8.13: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 62$ (BCA) - Not recommended (see page 15)

Configuration

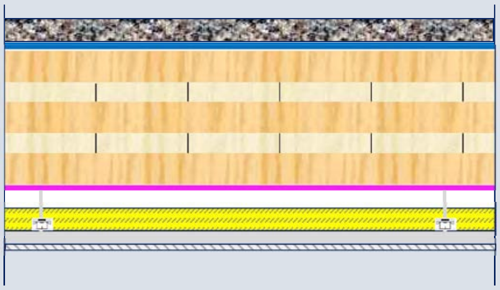


140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
67mm furring channel ceiling with Resilient mounts
50mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F12-01	T1617-68	Bare CLT	249	57 - 58	**50 - 52**	57 - 59	**61 - 62**	48 - 49

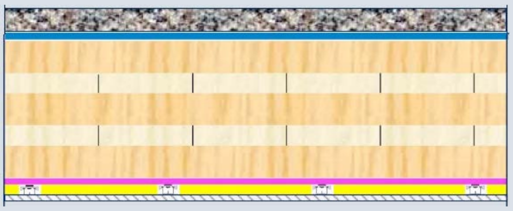
Note:
If CLT floor panels traverse between SOUs without a resilient floor, it is most likely that compliance will not be achieved with BCA sound insulation criteria horizontally due to sound flanking.

Figure 8.14: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 55$ (AAAC 3 Star)

Configuration	
	40mm screed 10mm rubber underlay 140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 185mm suspended ceiling with Resilient mounts 75mm glasswool insulation 13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F05-01	T1617-42	Bare screed	404	61 - 62	**54 - 56**	61 - 62	**50 - 51**	59 - 60

Figure 8.15: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 55$ (AAAC 3 Star)

Configuration	
	Floor 40mm screed 10mm rubber underlay 140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 67mm furring channel ceiling with Resilient mounts 50mm glasswool insulation 13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F11-01	T1617-52	Bare screed	286	60 - 61	**52 - 54**	60 - 62	**53 - 55**	55 - 57
F11-03	T1617-53	7mm laminate timber floor 3mm foam underlay on Screed	296	60 - 61	**51 - 54**	60 - 61	**52 - 53**	54 - 57

Figure 8.16: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 55$ (AAAC 3 Star)

Configuration	
	Floor 40mm screed 10mm rubber underlay 140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard Clips or mount (min. 67mm cavity) 50mm glasswool insulation 2x13mm standard plasterboard

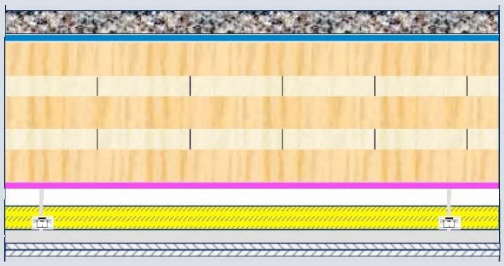
System	Test	Floor Covering	Connection	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F14-01	T1617-49	Bare screed	Adjustable clip	299	60 - 61	**51 - 54**	60 - 62	**54 - 55**	55 - 56
F13-01	T1617-56	Bare screed	Resilient mount	299	61 - 62	**54 - 56**	61 - 62	**50 - 52**	58 - 60
F13-04	T1617-62	10mm ceramic tiles 8mm adhesive bed Screed	Resilient mount	317	61 - 62	**55 - 56**	61 - 62	**50 - 51**	59 - 60

Figure 8.17: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 55$ (AAAC 3 Star)

Configuration	
	Floor 140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 67mm furring channel ceiling with Resilient mounts 50mm glasswool insulation 2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F12-03	T1617-65	7mm laminate timber floor 3mm foam underlay 2x9mm fibre cement 10mm rubber underlay	287	59 - 60	**53 - 54**	59 - 60	**50 - 51**	58 - 59
F12-04	T1617-66	7mm laminate timber floor 3mm foam underlay 2x9mm fibre cement 9.5mm Proctor Q-Silence C40 underlay	286.5	59 - 61	**52 - 53**	60 - 61	**51 - 52**	57 - 58

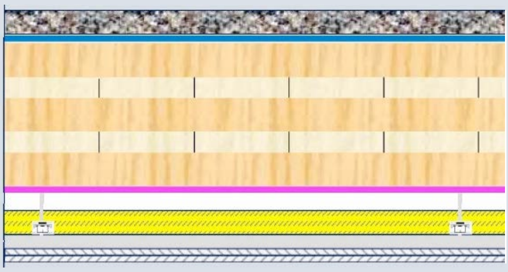
Figure 8.18: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 50$ (AAAC 4 Star)

Configuration	
	Floor 40mm screed 10mm rubber underlay 140mm Cross-Laminated Timber (CLT) 16mm fire-rated plasterboard 185mm suspended ceiling with Resilient mounts 75mm glasswool insulation 2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F07-01	T1617-34	Bare screed	417	61 - 62	**55 - 56**	61 - 62	**49 - 50**	60 - 61
F07-03	T1617-36	7mm laminate timber floor 3mm foam underlay on Screed	427	61 - 62	**55 - 56**	61 - 62	**46 - 48**	62 - 64

Figure 8.19: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 50$ (AAAC 4 Star)

Configuration



Floor
25mm DCT SolidEX screed
7.5mm Proctor Q-Silence P80 underlay
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
185mm suspended ceiling with Resilient mounts
75mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F08-01	T1617-25	Bare DCT SolidEX screed	399.5	60 - 61	**54 - 55**	60 - 61	**48 - 49**	61 - 62
F08-03	T1617-23	7mm laminate timber floor 3mm foam underlay on DCT SolidEX Screed	409.5	60 - 61	**54 - 55**	60 - 61	**47 - 49**	61 - 62

Figure 8.20: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 50$ (AAAC 4 Star)

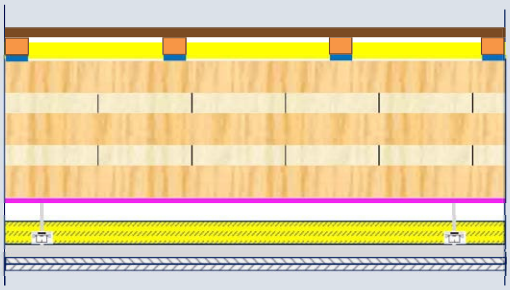
Configuration

Floor
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
185mm suspended ceiling with Resilient mounts
75mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F06-03	T1617-15	7mm laminate timber floor 3mm foam underlay 2x9mm fibre cement 10mm rubber underlay	405	60 - 62	**54 - 55**	60 - 62	**46 - 48**	62 - 64
F08-04	T1617-12	7mm laminate timber floor 3mm foam underlay 2x9mm fibre cement 7.5mm Proctor Q-Silence P80 underlay	402.5	60 - 62	**54 - 55**	60 - 62	**47 - 49**	62 - 63

Figure 8.21: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 50$ (AAAC 4 Star)

Configuration



Floor
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
185mm suspended ceiling with Resilient mounts
75mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F09-01	T1617-16	20mm strand board floor 56mm Proctor Q-Silence Dynamic Batten 20mm DCT URSAcoustic mineral wool in cavity	443	61 - 62	**55 - 56**	61 - 62	**44 - 46**	60 - 63
F09-04	T1617-26	20mm strand board floor 30mm Proctor Q-Silence Thin Batten 20mm DCT URSAcoustic mineral wool in cavity	417	60 - 62	**52 - 54**	61 - 62	**46 - 47**	60 - 62

Figure 8.22: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 50$ (AAAC 4 Star)

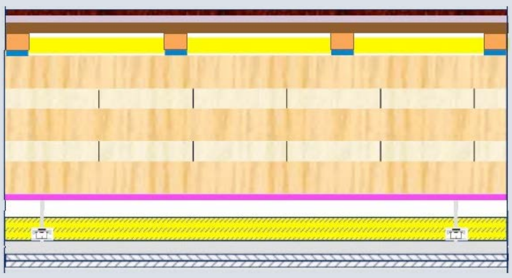
Configuration

Floor
40mm screed
10mm rubber underlay
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
67mm furring channel ceiling with Resilient mounts
50mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F13-03	T1617-54	7mm laminate timber floor 3mm foam underlay on Screed	309	61 - 62	**54 - 55**	61 - 62	**48 - 49**	59 - 62

Figure 8.23: Airborne $R_w + C_{tr} \geq 50$, Impact $L_{n,w} \leq 45$ (AAAC 5 Star)

Configuration



Floor
20mm strand board floor
56mm Proctor Q-Silence Dynamic Batten
20mm DCT URSAcoustic mineral wool in cavity
140mm Cross-Laminated Timber (CLT)
16mm fire-rated plasterboard
185mm suspended ceiling with Resilient mounts
75mm glasswool insulation
2x13mm standard plasterboard

System	Test	Floor Covering	Thickness (mm)	R_w	$R_w + C_{tr}$	STC	$L_{n,w}$	IIC
F09-03	T1617-20	7mm laminate timber floor 3mm foam underlay on Strand board	453	61 - 62	**56**	61 - 62	**41 - 43**	63 - 66

45

WoodSolutions™
design and build
woodsolutions.com.au

This guide was revised in Feb 2020

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

Code of Practice

Fire Retardant Coatings Applied to Wood Products

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: TREET, 14-storey timber-framed building, Bergen, Norway
Interior staircase photo: David Valldeby

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-67-6

Researchers
Paul England EFT Consulting,
PO Box 1288, Upwey, 3158 VIC
paul.england@eftconsult.net

First Published: May 2018

Revised: February 2020

Contents

Contents

1 Introduction

The primary purpose of this Code of Practice (CoP) for Fire-retardant Coatings Applied to Wood Products is to supplement the National Construction Code[1] (NCC) and relevant State and Territory Legislation by addressing issues relating to the supply, evidence of suitability, installation and maintenance of fire-retardant coatings used to modify the reaction to fire performance of wood products in greater detail. If there is any conflict between this CoP, the NCC or other relevant building legislation, the order of precedence is:

- **State and Territory Legislation**
- **National Construction Code**
- **The Code of Practice**

Notwithstanding the above, other legislation such as Workplace Health and Safety and Common Law obligations also apply to individuals and companies involved in the supply chain, design, and construction of buildings and components within buildings and the subsequent operation and maintenance of buildings.

These organisations must ensure that they adequately discharge their duty of care to ensure the building and its components are fit for purpose, comply with relevant legislation and do not pose an unacceptable level of risk to the health and safety of people.

Typical fire-retardant coatings are:

- Paints and varnishes
- Intumescent coatings
- Encapsulation coating systems
- Surface treatments with liquids

Fire-retardant coatings form part of a system comprising a wood product substrate, the fire-retardant coatings, and primers, undercoats and top coats if used in conjunction with the fire-retardant.

Parts of this Guide are also relevant to other fire-retardant treatments such as impregnation.

Compliance with the National Construction Code (NCC)

In order to comply with the NCC it must be demonstrated that the Governing Requirements of the NCC and the Performance Requirements have been satisfied.

The Governing Requirements of the NCC are documented in Section A of the NCC and provide rules and instructions for using and complying with the NCC including:

- Interpreting the NCC
- Complying with the NCC
- Application of the NCC in States and Territories
- Applying documents referenced in the NCC
- Documenting the suitability of the design, construction and/or use of materials to comply with the NCC
- Classifying buildings by their characteristics and intended use.

The performance requirements can be satisfied by means of a Performance Solution or a Deemed-to-Satisfy Solution or a combination as shown schematically in Figure 1.

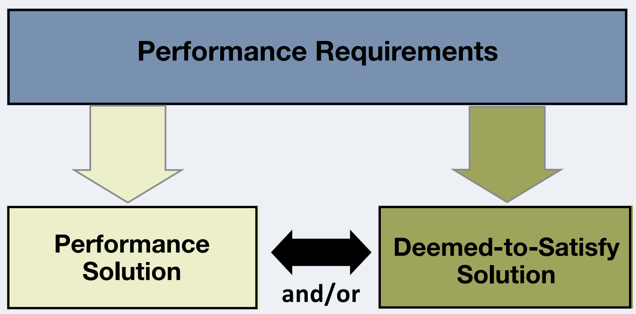


Figure 1: Pathways for demonstrating compliance with NCC Performance Requirements.

Compliance of a Performance Solution can be achieved by demonstrating compliance with all relevant performance requirements or demonstrating the solution is at least equivalent to the Deemed-to-Satisfy Provisions.

The NCC 2019 Deemed-to-Satisfy Provisions prohibit the use of fire-retardant coatings to achieve compliance with the fire hazard properties. Specifically, Clause C1.10 (b) states:

"Paint or fire-retardant coatings must not be used to achieve compliance with the required fire hazard properties".

Clause C1.10(a) identifies the following internal linings, materials and assemblies within a Class 2 to 9 building to which the fire hazard properties apply:

- Floor linings and floor coverings
- Wall linings and ceiling linings
- Air-handling ductwork
- Lift cars
- fixed seating in the audience area or auditorium and some proscenium curtains in Class 9b buildings used as a theatre, public hall or the like
- Escalators, moving walkways and non-required non-fire-isolated stairways or pedestrian ramps subject to Specification D1.12
- Sarking-type materials
- Attachments to floors, ceilings, internal walls, common walls, fire walls and to internal linings of external walls
- Other materials including insulation materials other than sarking-type materials.

It should be noted that there are a number of State variations to Clause C1.10a of the NCC and these take precedence for buildings that are constructed within these jurisdictions.

If the above prohibition applies, a Performance Solution will need to be developed if fire-retardant-coatings are to be used to achieve compliance with the required fire hazard properties unless specifically excluded by State or Territory variations.

Guidance on the development of performance solutions is available in other WoodSolutions documents and on the ABCB web site (abcb.gov.au)

The NCC does however permit the use of other fire-retardant treatments to achieve compliance with the fire hazard properties such as impregnation and also allow fire-retardant coatings to be applied to timber products to achieve compliance with certain Bushfire Provisions of AS 3959.

Typical Roles & Responsibilities for the Design, Construction and Operation of Buildings

Figure 2 is an indicative schematic showing typical roles and responsibilities for the design, construction and subsequent operation of buildings to provide a context for this Design Guide. The flow chart also identifies key documentation in order to facilitate the design and specification of fire-retardant treatments that are fit for purpose and provide evidence of suitability.

Four main processes are identified in Figure 2.

- Building Design
- Construction
- Compliance Checking
- Building Use and Occupation

For simplicity, Figure 2 shows a linear process but in reality, design modifications may be necessary throughout a building project and numerous iterations may occur. ***It is essential that all design changes are fully documented, and the modified design checked for compliance with appropriate evidence of suitability obtained.***

State government bodies and / or local councils have responsibilities for ensuring industry compliance with relevant building regulations; generally through accreditation or licensing of practitioners and audits of typical projects.

Building Design Process

The Building Design Process will typically be undertaken by a design team led by a consultant such as the project architect. The design team composition will depend up the needs of the project. For projects where fire related performance solutions are being considered, a fire safety engineer would form part of the design team and a Performance Based Design Brief (PBDB) Committee containing relevant stakeholders will normally be established. Refer to the above section 'Compliance with the National Construction Code (NCC)' for further information.

The outcome of the design process will be plans and technical specifications that should, amongst other things, define the required performance and evidence of suitability for various components, such as fire-retardant coatings, in order to satisfy the NCC Performance Requirements and other critical design objectives. Some key considerations are identified in the following section 'Typical Design Considerations' with more detailed information provided in Sections 2 to 5 of the Guide.

The design documentation is then submitted to the relevant authority (e.g. building surveyor / certifier) for approval.

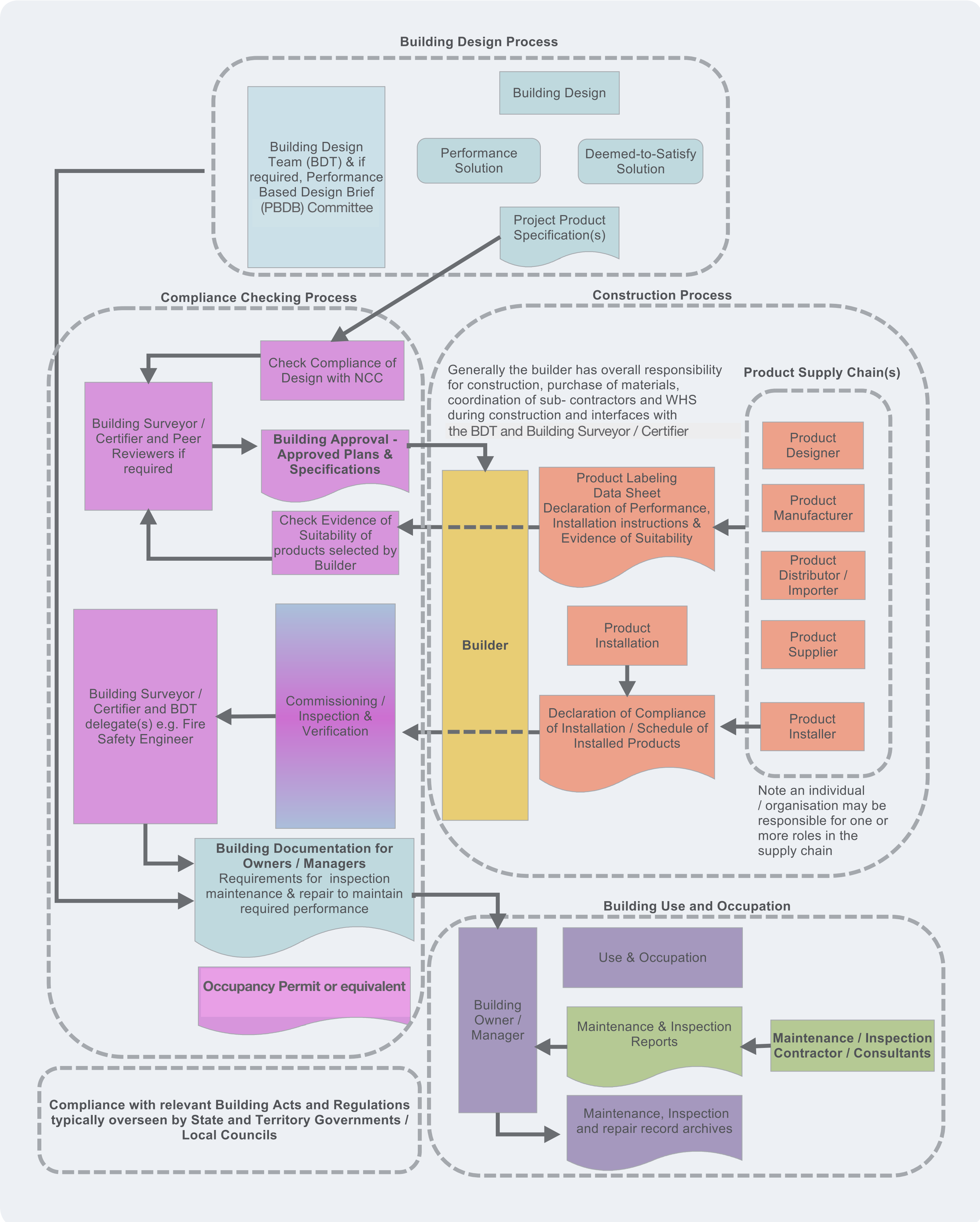


Figure 2: Typical roles & responsibilities for the design, construction and operation of buildings.

Compliance Checking Process

Compliance checking must be undertaken throughout the project including the design and construction phases through to hand over and involves close liaison with the building design team. Commissioning inspection and verification of final installations form a critical part of the process.

It is important that roles and responsibilities are clearly defined at the start of the project to ensure critical inspections / verification processes and necessary design checks are undertaken.

Building Acts and Regulations in the States and Territories specify administrative procedures for determining compliance and granting approval for building works to be undertaken and subsequently for buildings to be occupied.

Construction Process

A builder is generally responsible for implementing the design including purchase of materials but may rely on specialist sub-contractors to undertake installation of systems such as fire-retardant coatings. On large or complex projects, the builder will have to manage multiple product supply chains and act as a conduit for evidence of suitability and other compliance documentation to the design teams and compliance checkers notwithstanding information from the supply chain may have been provided to designers during the design phase independently.

The supply chain for fire-retardant coatings for wood products is the main focus of this design guide with critical documentation to facilitate compliance checking including;

- Product labelling
- Product Data Sheets
- Declaration of Performance
- Installation Instructions
- Evidence of Suitability
- Declaration of Compliance of Installation and Schedule of Installed products.

This is addressed further in Section 6.

Building Use and Occupation

Information relating to requirements for maintenance and inspection to ensure that the performance is maintained at the required level throughout the life of a building and if damaged rectification works can be undertaken. This is addressed in more detail in Section 7.

Typical Design Considerations

General Considerations

When considering the use of a fire-retardant coating in order to comply with the NCC, it is necessary to determine amongst other things:

- how the coating can be installed / constructed safely without unnecessary disruption to other site activities
- how the substrate will be prepared, and the coating applied to achieve the required performance
- how the correct application of the coating will be verified
- what is the minimum design life of the fire-retardant system?
- what measures are required to ensure that the performance of the substrate and coating are maintained throughout the design life of the system (inspection and repair)?
- measures to be taken to ensure the coating does not present a hazard during renovation / modification or demolition
- measures to be taken to ensure that the coating is not compromised during the renovation / modification process
- what evidence of suitability is required?
- how the performance of the coating can be re-instated, or the design life extended at the end of the specified coating design life particularly if it is less than the design life of the building?

Many of these matters are not specifically addressed in the NCC but they are inferred under Clause A5.0 of the NCC which states:

"A5.0 Suitability

(1) A building and plumbing or drainage installation must be constructed using materials, products, plumbing products, forms of construction and designs fit for their intended purpose to achieve the relevant requirements of the NCC."

Responsibilities for Safe Design

In addition, Workplace Health and Safety Legislation is also applicable which requires safe design principles to be applied. A Code of Practice on the safe design of structures has been published by Safe Work Australia[2] which provides guidance to persons conducting a business or undertaking who design structures that will be used, or could reasonably be expected to be used, as a workplace. It is prudent to apply these requirements generally since most buildings will be a workplace for people undertaking building work, maintenance, inspections even if the building is not primarily used as a workplace.

The Safe Design of Structures Code of Practice defines Safe design as;

"the integration of control measures early in the design process to eliminate or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed"

For further details on how to address WHS requirements refer to Model Code of Practice: Safe Design of Structures published by Safe Work Australia.

It indicates that safe design begins at the start of the design process when making decisions about:

- the design and its intended purpose
- materials to be used
- possible methods of construction, maintenance, operation, demolition or dismantling and disposal
- what legislation, codes of practice and standards need to be considered and complied with.

The Code of Practice for Safe Design of Structures also provides clear guidance on who has health and safety duties in relation to the design of structures and lists the following practitioners:

- architects, building designers, engineers, building surveyors, interior designers, landscape architects, town planners and all other design practitioners contributing to, or having overall responsibility for, any part of the design
- building service designers, engineering firms or others designing services that are part of the structure such as ventilation, electrical systems and permanent fire extinguisher installations
- contractors carrying out design work as part of their contribution to a project (for example, an engineering contractor providing design, procurement and construction management services)
- temporary works engineers, including those designing formwork, falsework, scaffolding and sheet piling
- persons who specify how structural alteration, demolition or dismantling work is to be carried out.

In addition, WHS legislation places the primary responsibility for safety during the construction phase on the builder.

From the above, it is clear that the design team in conjunction with owners / operators and the builder have a responsibility to document designs, specify and implement procedures that will minimise risks to health and safety throughout the life of the structure being designed.

Other Design Considerations

Designers need to take account of a broad range of design considerations to ensure that a building is fit for purpose and complies with all requirements of the NCC. Issues such as durability and weatherproofing may also apply to fire-retardant coatings particularly if used in external applications.

Clients may require specific issues to be addressed such as the impact on the environment.

Checking Interpretations of Regulations and Standards

Whilst this CoP focusses of NCC compliance of wood products protected by fire-retardant coatings, it should be noted that the NCC provides a uniform set of technical provisions for the design and construction of buildings and other structures throughout Australia. The NCC does not regulate matters such as the roles and responsibilities of building practitioners and maintenance of fire safety measures which fall under the jurisdiction of the States and Territories.

State and Territory Building legislation is not consistent in relation to these matters with significant variations with respect to:

- registration and licencing of practitioners,
- mandatory requirements for inspections during construction, and
- requirements for maintenance of fire safety measures.

In addition, changes to legislation are not necessarily linked to the NCC revision cycle. Therefore, current Common Law, State and Territory Building Acts and Regulations and Workplace Health and Safety Legislation should be checked for the specific jurisdiction that will apply to a building or product.

2 Fire Tests for Wood Products Protected by Fire-retardant Coatings Relevant to the NCC

This Section provides an overview of typical fire tests / classification standards specified by the NCC that may be used to evaluate wood products or systems protected by fire-retardant coatings.

NCC Classification Requirements for Fire Hazard Properties

The requirements for Fire Hazard Properties are prescribed in Specification C1.10 of the NCC in relation to linings, materials and assemblies. The applications most relevant to fire-retardant coated wood products are:

- Wall and Ceiling Linings classified in accordance with AS 5637.1[3]
- Floor linings classified in terms of the critical heat and smoke developed rate determined in accordance with AS ISO 9239.1[4]
- Miscellaneous materials / assemblies and attachments classified in terms of Smoke Developed and Spread of Flame indices determined in accordance with AS 1530.3[5]
- Building elements in Bushfire Prone Areas in accordance with AS 3959[6].

Further details of the applicable fire test methods and criteria are provided below.

Wall and Ceiling Linings – AS 5637

The fire hazard properties of wall and ceiling linings are determined in accordance with AS 5637.1. Materials are classified in terms of a *group number* determined by undertaking a large scale test in accordance with AS ISO 9705—2003[7] or, if the material has a confirmed correlation in accordance with AS 5637.1, the *group number* can be predicted based on data from bench scale tests performed at an imposed irradiance in the horizontal orientation of 50 kW/m^2 in accordance with AS/NZS 3837[8] or ISO 5660-1[9].

Supplementary smoke production criteria are specified in terms of the *smoke growth rate index* or *average specific extinction area*, depending on the test methods adopted.

Acceptable correlations exist for wood products and bench scale tests are normally sufficient for most homogeneous wood products.

Figure 3 shows a whole room assembly lined with the system to be evaluated being tested in accordance with AS ISO 9705. Figure 4 shows a specimen being subjected to a bench scale test in accordance with AS/NZS 3837.

Figure 3: AS ISO 9705 fire tests.

Figure 4: AS/NZS 3837 Cone Calorimeter test.

Wall and Ceiling linings are classified on the following basis.

Group 1—material that does not reach flashover when exposed to 100 kW for 600 s followed by exposure to 300 kW for 600 s.

Group 2—material that reaches flashover following exposure to 300 kW within 600 s after not reaching flashover when exposed to 100 kW for 600 s.

Group 3—material that reaches flashover in more than 120 s but within 600 s when exposed to 100 kW. Most unprotected wood products achieve this level of performance.

Group 4—material that reaches flashover within 120 s when exposed to 100 kW.

In addition, the following smoke production criteria apply depending upon the test method adopted unless the building is protected by a sprinkler system complying with Specification E1.5 (other than a FPAA101D or FPAA101H system).

AS ISO 9705: a *smoke growth rate index* not more than 100; or

AS/NZS 3837: an *average specific extinction area* less than 250 m^2/kg.

Most wall and ceiling linings of solid timber 19 mm thick are expected to achieve Group 3 performance. Some timber species of 9 mm thickness have also achieved Group 3 performance - Refer the WoodSolutions “fire test reports” webpage.

Fire-retardant treatments can be used to improve the performance of wood products in order to achieve Group 1 and Group 2 classifications.

Floor Linings and Coverings - AS ISO 9239.1

The fire hazard properties of floor linings and coverings are determined by undertaking tests in accordance with AS ISO 9239.1[10] to determine the *critical radiant flux* at extinguishment and the *smoke development rate*.

The test apparatus and a typical fire test are shown in Figure 5.

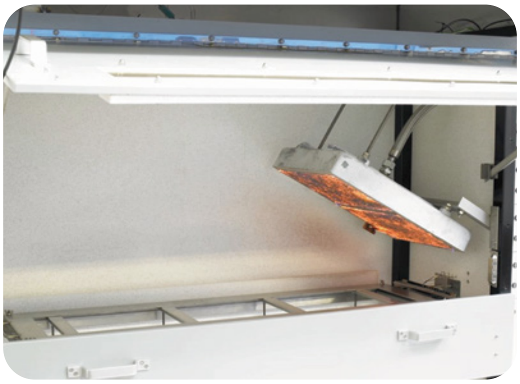


Figure 5: AS ISO 9239.1 fire testing.

Three levels of *critical heat flux* are prescribed in Specification C1.10 of the NCC

- 1.2 kW/m^2 applies to the most hazardous materials
- 2.2 kW/m^2 is an intermediate level of performance
- 4.5 kW/m^2 applies to the least hazardous.

Most wood products achieve a *critical heat flux* greater than 2.2, which satisfies the NCC Deemed-to-Satisfy requirements for most applications except for fire isolated exits and fire control rooms in some healthcare-buildings and patient / general areas in some healthcare-buildings that are not provided with automatic fire sprinkler systems – refer NCC Specification C1.10 for further information.

For most applications, fire-retardant coatings will not be required to enhance the performance of wood products to satisfy the NCC Deemed-to-Satisfy requirements.

A maximum *smoke development rate* limit of 750 percent-minutes is also required to be satisfied in buildings not protected by a sprinkler system complying with Specification E1.5 (other than an FPAA 101D or FPAA 101H). Most wood products will satisfy this criterion.

Attachments and Miscellaneous Applications – AS 1530.3

AS 1530.3[5] is used to classify the fire hazard properties of attachments and materials used for other miscellaneous applications – refer NCC C1.10 for a full list of applications to which this standard is applied.

For materials other than sarking, there is separate range of fire performance indicators termed Early Fire Hazard Indices: The *Spread-of-Flame Index* and *Smoke-Developed Index* are used by the NCC for classification of purposes.

Typical NCC deemed-to-satisfy requirements are summarised in Table 1.

Table 1: AS 1530.3 indices applied to attachments and other materials.

Material or assembly location	Element	Spread-of-Flame Index	Smoke-Developed Index
Fire control rooms[1] and fire-isolated exits	-	0	2
Class 9b buildings used as a theatre, public hall or similar	Fixed seating in the audience area or auditorium.	0	5
	A proscenium curtain[2]	0	3
Escalators, moving walkways or non- required non- fire-isolated stairways or pedestrian ramps[3]	-	0	5
Insulation materials other than sarking-type materials	-	9	8 if the Spread-of-Flame Index is more than 5
All other materials or locations	-	9	8 if the Spread-of-Flame Index is more than 5

Note:
1. Required by Specification E1.8
2. Required by Specification H1.3.
3. Required by Specification D1.12.

Most unprotected wood products are expected to achieve a *Spread of Flame Index* between 3 and 9 and a *Smoke Developed Index* less than 5. A notable outlier is unprotected Western Red Cedar with a *Spread of Flame Index* of 10. Further test data is provided on the WoodSolutions site – search "fire test reports".

AS 3959 Requirements for Bushfire-Resisting Timber

AS 3959[6] allows the use of bushfire-resisting timber for BAL levels up to BAL-29, subject to compliance with Appendix F of the standard.

Bushfire-resisting timber may be solid, laminated or reconstituted form and may be 'bushfire-resisting' by means of one or more of:

- the inherent properties of the material itself
- being impregnated with fire-retardant chemicals
- the application of fire-retardant coatings or substrates.

Where the timber has been altered by chemicals or protected by a coating, the test samples are required to be exposed to accelerated weathering prior to fire testing unless the timber is protected from the weather, as described in AS 1684.2[11] and AS 1684.3[12].

External wood products are deemed to be protected from the weather if they are covered by a roof projection (or similar) at 30° or greater to the vertical and they are well detailed and maintained (painted or stained and kept well ventilated).

AS 3959 also allows elements of construction to be tested to AS 1530.8.1[13] and AS 1530.8.2.[14] to provide evidence of suitability for use in accordance with the NCC when exposed to appropriate bushfire attack levels. Fire-retardant coated wood products may form part of these systems.

Fire Testing of Bushfire-Resisting Timber – AS/NZS 3837

In order to determine if a protected or unprotected wood product satisfies the requirements for bushfire-resisting timber, samples are fire tested in accordance with AS/NZS 3837 and are required to satisfy the following criteria:

- the maximum heat release rate shall be not greater than 100 kW/m^2
- the average heat release rate for 10 min following ignition shall be not greater than 60 kW/m^2 when the material is exposed to an irradiance level of 25 kW/m^2.

AS 3959 Requirements for Accelerated Weathering Prior to Fire Testing Bushfire-Resisting Timber

If an element protected by a fire-retardant coating is likely to be fully exposed to weather, accelerated weathering is required before testing to AS/NZS 3837. AS 3959 specifies the following accelerated weathering regime:

The fire-retardant-coated substrates need to be subjected to the ASTM D2898[15] Method B weathering regime, but with the water flow rate modified to be the same as that within ASTM D2898 Method A.

Fire Testing Elements of Construction Exposed to Bushfire Attack AS 1530.8.1

AS 1530.8.1 is a large-scale test method to evaluate the performance of elements of construction exposed to simulated bushfire attack.

The principle of the test is that a representative element of construction is subjected to an imposed radiant heat flux in conjunction with small flaming sources. The test method allows for different radiant heat exposure levels to enable the method to be applied to different bushfire attack levels (BALs). The radiant heat flux is varied with time to simulate the passage of the flame front.

During the test, a pilot ignition source is applied to exposed combustibles and volatiles on the exposed face simulating ember attack. Burning cribs are also applied on surfaces where there is potential for debris accumulation. Conditions are monitored during exposure to radiant heat and for a further period of 60 minutes following radiant heat exposure to identify potentially persistent hazardous conditions.

If all the performance criteria are satisfied a BAL rating is assigned based on the incident radiant heat applied during the test. Available BAL ratings are BAL 12.5,19, 29 and 40.

BAL–FZ applies to elements of construction potentially exposed to full flame engulfment from the fire front and specimens can be evaluated using AS 1530.8.2 which uses similar conditions to a standard fire resistance test and therefore use of fire-retardants is not common.

Combustible External Cladding Systems AS 5113

The NCC 2019 edition Deemed-to-Satisfy requirements generally require cladding systems for external walls to be non-combustible. Where combustible cladding systems are intended to be used a Performance Solution will be required.

AS 5113 Fire propagation testing and classification of external walls of buildings[16] specifies test methods, performance criteria and classification procedures that can be applied to combustible cladding systems.

The nominated test methods include ISO 13785.2[17] and BS 8414 Parts 1 and 2[18].

Figure 6: ISO 13785.2 test configuration.

AS 5113 is referenced in the NCC Volume One Amendment 1 Verification Method CV3, and enables industry to verify the fire performance of external cladding systems against the relevant Performance Requirements of the NCC. This seeks to improve compliance, promote innovative solutions and ensure the required fire performance is achieved.

Therefore if timber external cladding systems are intended to be used AS 5113 is the most appropriate method of evaluation of a façade system forming part of a Performance Solution.

3 Selection of Fire Test Configurations

End use of Fire-retardant Coatings

While fire-retardant coatings may be supplied independently of a wood product substrate, the coating forms part of an element of construction that includes a wood product substrate, the fire-retardant coatings, and primers, undercoats and top coats.

In many applications the reaction to fire performance of elements of construction also depends on fixings (including adhesives), the thickness of the substrate and backing details such as air cavities or insulation within the element.

Fire-retardant coatings should be fire tested in a manner that reflects their intended end use. This may require more than one test to cover a broad range of applications. Typical variations include:

- a range of wood products (timber species / type engineered products)
- substrate thicknesses
- cavity detail / backing materials
- different levels of performance / end use application
- fire-retardant coatings application rate
- primers types and application rates
- undercoats and application rates
- top coats and application rates
- presence of cavity barriers (when testing full scale facades).

ISO 14697[19] provides guidance on the choice of substrates for testing including specification of a 'standard' non-fire-retardant treated particleboard substrate.

Field of Application

It is recommended that fire-retardant suppliers define a target field of application for their fire-retardant products and liaise with a registered testing authority to define a cost effective test program prior to undertaking any significant fire testing. At the end of the test program an assessment from an Accredited Testing Laboratory confirming the field of application for the product based on referenced test data should be obtained in a format that is consistent with the evidence of suitability required by the NCC.

4 Durability

Relevant NCC Deemed-to-Satisfy Provisions

The National Construction Code 2019 provides limited guidance and Deemed-to-Satisfy requirements in relation to durability. With respect to fire-retardant coatings, the NCC Deemed-to-Satisfy requirements relating to bushfire specify compliance with AS 3959[6], which explicitly addresses the durability of fire-retardant coatings through the specification of accelerated weathering tests prior to fire testing of bushfire-resisting timber if the element is to be used in a position fully exposed to weather. This provides a useful benchmark.

NCC Clause 5.0 - Suitability provides a general fit-for-purpose statement that effectively requires matters such as durability to be addressed.

General advice relating to the durability of timber is provided in WoodSolutions Technical Design Guide #5 *Timber service life design - Design guide for durability*[20].

This CoP considers durability with respect to the fire properties of timber components with fire-retardant coatings applied.

ABCB Handbook - Durability in Buildings including Plumbing Installations

The Australian Building Codes Board has published a Handbook on the Durability in Buildings Including Plumbing Installations[21] to provide construction industry participants with non-mandatory advice in general terms to be used to develop solutions relevant to specific situations. The Handbook indicates that industry is expected to develop specific solutions relevant to specific materials using the principles and criteria within the handbook.

However, the Handbook indicates that it is not intended to:

- override or replace legal rights, responsibilities or requirements
- provide users with the specifics of the NCC
- address the issue of durability in terms of consumer protection.

Design Life For Building Components

The Handbook states that the minimum design life for a building or plumbing installation and the components of their subsystems should be as shown in Table 2 and that the design life of buildings shall be taken as ‘Normal’ for all building importance categories unless otherwise specified. The content of this CoP is based on a building design life of 50 years.

Most applications for which fire-retardant coatings are used are expected to be readily accessible or moderately accessible. For example, a corridor lining within a building would generally be considered readily accessible whereas timber components forming part of an external façade of a mid-rise building may be considered to have a moderate ease of access (unless the building design provides ready access).

Table 2: Design life of components and sub-systems from ABCB Handbook[16].

Design life of building (*dl*) (years)		**Design life of components or sub-systems (years)**		
		Category		
Category	**No. of years**	**Readily accessible and economical to replace/ repair**	**Moderate ease of access but difficult or costly to replace or repair**	**Not accessible or not economical to replace or repair**
Short	$1<dl<15$	5 or dl (if $dl<5$)	dl	dl
Normal	50	5	15	50
Long	100 or more	10	25	100

With respect to fire-retardant coatings, the design life of a coating may vary from that of the timber element forming the substrate.

If the fire-retardant coating is treated as a component in its own right then an appropriate design life for the coating should be between 5 and 15 years based on Table 2 depending on ease of access and cost or repair (reinstatement).

If the fire-retardant coating is treated as an integral part of the substrate then the reinstatement of the performance of the coating would be considered a maintenance activity. In these circumstances, if it was considered uneconomical to replace or repair the timber element forming a substrate (e.g. a loadbearing column), a design life of 50 years may apply to the column assembly, but reinstatement of the fire-retardant coating should be undertaken as a routine maintenance task and it is reasonable to base the periods for this activity on Table 2 (i.e. 5 or 15 years).

In most instances the minimum design life required for a fire-retardant coating system should be either 5 or 15 years depending on the accessibility and cost of reinstatement of the required performance of the fire properties of the coated element.

Factors Affecting Durability

The ABCB Handbook identifies a broad range of factors that may have a bearing on durability that should be considered when determining whether a component will have adequate durability; which are summarised below together with additional items relevant to fire-retardant coatings.

Environmental agents

- temperature
- solar radiation
- humidity
- rainfall
- wind and airflow
- soil type
- exposure to airborne salt
- pollutants
- saline environment
- biological agents
- chemical effects, etc.

Specific conditions

- condensation
- cyclic changes (e.g. from hot to cold or wet to dry)
- agents due to usage, (e.g. aggressive, inappropriate maintenance or agents)
- ground contact.

Actions by users

- direct use caused by heavy use (e.g. foot traffic on floors)
- accidental impacts
- spilled chemicals
- internal processes (e.g. laundries).

Design, Detailing and Workmanship

- prevention or reduced exposure to elements (e.g. containment of corrosive agents)
- overhanging eaves to reduce exposure to weathering
- detailing to avoid pooling and run off over surfaces
- detailing to avoid abrasion and impact damage
- surface preparation and condition of substrate at the time of application of a coating
- treatment at joints and connections
- compatibility with other treatments.

The ABCB Handbook identifies three strategies to achieve the required performance through the design life of a structure as shown in Figure 7.

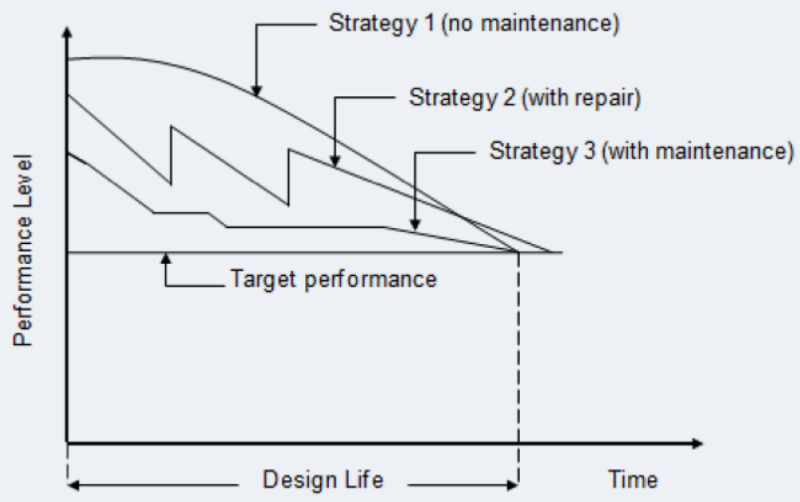


Figure 7: Strategies to achieve the required performance from ABCB Handbook[16].

To achieve the fire hazard properties required by the National Construction Code (NCC) a maintenance process is required in most jurisdictions. Many jurisdictions reference AS 1851[22], which requires annual inspections of fire and smoke barriers and fire-resistant structural members protected by coatings. It is appropriate to apply the same requirements to fire-retardant coatings for timber elements. Therefore Strategy 2 or Strategy 3 should be adopted for fire-retardant coatings applied to timber.

When designing for durability the ABCB Handbook indicates the following factors should be considered:

- intended use of the structure or system
- required performance criteria
- expected environmental conditions
- composition, properties and performance of the materials
- structural system
- shape of the members and the structural detailing
- quality of the workmanship and level of control
- particular protective measures
- maintenance during the design life.

For fire-retardant coatings used within the scope of this CoP the required performance criteria relate to fire properties which are expected to be determined in accordance with the NCC based on the intended use of a structure and to the degree necessary the structural system.

Shape of the members and structural detailing is addressed to the degree necessary in relation to fire properties through the performance criteria and referenced fire tests which vary to some extent with the orientation and use of the lining / cladding system being considered.

Typically environmental conditions for fire-retardant coatings can be classified as:

- internal conditions (e.g. fire-retardants applied in situ after structure is watertight)
- protected external conditions (semi exposed) (i.e. protected from contact with rain by horizontal projections and reduced exposure to UV radiation)
- external conditions (full exposure to weather).

Additional criteria may apply depending on the application such as exposure to chemicals, salt spray, abrasion (e.g. flooring), contact with external ground.

The composition, properties and performance of specific fire-retardant coatings will vary between products and the suppliers / manufacturers will need to consider, issues such as quality of the workmanship and level of control; supplementary protective measures; and maintenance / reinstatement procedures for the products they manufacture and / or supply.

Assessment of Durability

A general procedure for the assessment of durability is specified in the ABCB Handbook requiring the following tasks to be undertaken applying sound engineering / scientific principles with some assessment of the reliability of the proposed solution.

(a) identify the service conditions;

(b) identify the relevant environmental agents and specific locations that contribute to the problem (see Section 4.1 of this Handbook);

(c) identify the deterioration and damage mechanisms;

(d) identify the relevant factors to be considered (see Section 4.2 of this Handbook);

(e) identify the relevant limit states associated with the functional failures for the intended use;

(f) estimate the deterioration-time relationship; and

(g) determine whether the anticipated deterioration is acceptable or the building components or assemblies need to be maintained, repaired or replaced within the design life of the building.

Tasks (a) and (b) should be undertaken as part of the design / specification process of a typical building or structure.

Task (e) is inherently addressed through the specification of NCC Deemed-to-Satisfy fire safety related provisions that prescribe test methods and related performance criteria that components / elements of construction need to satisfy. If a Performance Solution is adopted relating to the fire safety provisions of the NCC, then as part of the analysis the probability and consequence of system failures should be considered.

Normally the product supply chain, comprising the product designer, manufacturer, distributor / importer and supplier will be responsible for ensuring the remaining tasks are adequately addressed; since they have responsibilities to ensure the product is fit for purpose and also have access to historic data, formulation details and evaluations of durability undertaken for other markets.

Appropriate evidence of suitability should be provided by the supply chain to the relevant building designer(s) and building certifier using one or more of the following assessment methods, specified in the ABCB Handbook:

- historical record
- modelling
- testing
- specialist expertise.

Established Test Procedures for Pre-fire Test Weathering of Specimens to Assess Durability

The following describes accelerated weathering test procedures that may be used as part of the determination of the durability of fire-retardant coated wood products. It is not written in mandatory terms since details of the historic record and previous evaluations, design exposure conditions and knowledge of materials used in the formulation of a coating system will vary and may be more reliable than accelerated weathering tests.

Notwithstanding the above, for unprotected external use it is mandatory to undertake accelerated weathering tests prior to fire testing when evaluating bushfire-resisting timber that relies on fire-retardant coatings to achieve the required performance in accordance with AS 3959.

It is important for the exposure of an element during accelerated weathering tests to reflect the end use. Advice on the preparation of specimens is provided in Appendix B.

AS 3959 Requirements for Accelerated Weathering Test

Appendix F Para F2(b) of AS 3959 states the following:

"Where the timber has been altered by chemicals, the test samples shall be subjected to the regime of accelerated weathering described in Paragraph F3 except that where the timber is protected from the weather, as described in the AS 1684 series (for example, cladding protected by a veranda), accelerated weathering of the test samples is not required before being tested to AS/NZS 3837.

External timbers are deemed to be protected if they are covered by a roof projection (or similar) at 30 degrees or greater to the vertical and they are well detailed and maintained (painted or stained and kept well ventilated)."

Appendix F Para F3 of AS 3959 states: "Where accelerated weathering is required before testing to AS/NZS 3837, external fire-retardant coated substrates shall be subjected to the ASTM D2898 Method B weathering regime, with the water flow rate modified to be the same as that within ASTM D2898 Method A."

This provides a benchmark in that accelerated weathering tests are not required for semi-exposed external conditions. Only fully exposed external conditions and an external weather testing protocol have been specified.

Since no design life is specified in AS 3959 it should be assumed that the design life of fire-retardant coatings evaluated under these conditions is the minimum recommended in the ABCB Handbook (i.e. five years unless additional evidence for the specific coating system is available).

General Accelerated Weathering and Fit for Purpose Assessment Methods

ASTM D2898

The accelerating weathering tests specified in ASTM D2898[18] are commonly used internationally and, since they are adopted by an NCC referenced standard for accelerated weathering tests on bushfire-resisting timber, it would be reasonable to adopt this method as part of an assessment of durability for fire-retardant coatings used externally based on the regime described in AS 3959 and summarised below:

Specimens should be subjected to the ASTM D2898 Method B weathering regime, with the water flow rate modified to be the same as that within ASTM D2898 Method A.

ETAG 028

ETAG 028[23] is a Guideline for the European Technical Approval of Fire-retardant Products (including fire-retardant coatings). It provides the basis for the preparation of European Technical Approvals, which are defined as technical assessments of the fitness for use of a construction product and technical specification of assessed products. ETAGs serve as the basis for CE marking when and where harmonised standards are not yet available.

While the fire test standards and related fire performance criteria specified in ETAG 028 are not directly applicable to the NCC, the ETAG includes detailed procedures for the assessment of the working life and other criteria relating to fitness for purpose of fire-retardant coatings. The ETAG assumes a working life of five years when the product is installed, subject to appropriate installation use and maintenance. This criterion is consistent with the durability criteria provided in the ABCB Handbook. The provisions in the ETAG were based on the current state-of-the-art knowledge and experience and form a useful basis for assessment of durability.

In summary: Annex B of ETAG 028 provides details of test and assessment methods to determine the durability of reaction to fire performance.

If fire-retardant coatings are applied to flooring, the performance is required to be verified after being subjected to abrasion testing.

Cone calorimeter testing is used to compare the performance of the coating before and after exposure to the durability tests and, if appropriate, abrasion tests even though cone calorimeter method is not used for the initial classification. The irradiance levels are specified depending upon the application. The total heat released is calculated over a 10 minute period. Average values are then calculated for the samples not exposed to durability testing and samples exposed to durability testing. The total heat released for the samples after durability testing must not increase by more than 20% above the value obtained from the specimens not exposed to durability testing. In addition the rate of heat release rate averaged over a 30s periods is calculated and maximum limits are specified, depending on the application. These heat release rate limits are independent of the initial test series.

Use of Cone Calorimeter Tests for Evaluation of Performance after Weathering / Durability Testing and Repair

Direct Determination of Classification after Weathering / Durability Testing

The requirements for bushfire-resisting timber in AS 3959 include procedures for evaluating the performance of coating systems after exposure to accelerated weathering using cone calorimeter testing in accordance with AS/NZS 3837.

In many instances the *group number* for wall and ceiling linings is permitted to be derived from cone calorimeter tests performed in accordance with AS/NZS 3837 or ISO 5660-1. In these cases, it is possible to predict the results directly from specimens cut from samples exposed to accelerated weathering if external exposure is expected. Refer Appendix B for preparation of specimens.

Determination of Acceptable Levels of Performance after Weathering / Durability Testing

It is impractical to undertake the accelerated weathering / durability testing of large or intermediate scale samples in order to evaluate the fire properties of fire-retardant coated wood products. In these circumstances comparison of the performance of the coating applied to a substrate before and after weathering / durability testing using cone calorimeter test methods provides a practical solution. This approach is adopted in ETAG 028.

Variability in results is expected and the standard cone test procedures require testing of more than one specimen in recognition of this variability. In addition most of the test / classification methods considered in this code specify pass/fail criteria that have to be met or exceeded and therefore in most instances there will be a safety margin. This is addressed to some extent in the ETAG procedure where the results from different samples are averaged and a 20% increase in the total heat released is permitted.

Irradiance levels of 25kW/m^2 and 50kW/m^2 are commonly used in Australia and ETAG 028 uses values of 30kW/m^2 and 50kW/m^2. The following irradiance levels are suggested to be used for comparative purposes and have been selected based on the commonly used levels.

Table 3: Suggested Cone Calorimeter irradiance levels for comparison of fire-retardant coating before and after durability testing

Classification test method	Comparative test irradiance level – kW/m^2
AS ISO 9705	50
AS ISO 9239.1	25 or 30
AS 1530.3	50
AS 1530.8.1 BAL 12.5,19 and 29	25 or 30
AS 1530.8.1 BAL 40	50

Overpainting Repair and Reinstatement

Cone calorimeter testing may be undertaken to evaluate options for overpainting repair or reinstatement of the performance of a coating at the end of the design life.

A test program should be developed in conjunction with the accredited testing laboratory to investigate the impact of events such as overpainting, damage and subsequent reinstatement to form the basis for the manufacturer's instructions.

Evaluation of Options at the End of the Design Life

At the end of the stated design life of the coating, there are a number of options that may be appropriate subject to verification of performance. Typical examples include:

- sanding back and reapplication of the entire system
- application of additional coat(s) of fire-retardant over existing coatings
- extending the life of the coating based on natural aging tests.

Cone calorimeter testing may be undertaken to evaluate options for extending the life of the design life of the coating system or repair / reinstatement to bring the performance up to required levels of performance.

5 Building Design and Product Specification

The design team is responsible for the specification of the components of a building to ensure the building achieves the design objectives, complies with relevant legislation and can be constructed and maintained safely. While product suppliers and companies and individuals associated with the product supply chain also owe a duty of care to ensure a product is fit for purpose, the specification provided by the design teams needs to include such things as:

- Clear description of the extent of services to be provided
- Expected service conditions
- Design Life
- Maintenance and repair / re-instatement (including access for maintenance and inspection activities in the completed structure)
- Details of the substrate (wood product) and backing materials if provided
- Surface preparation and installation requirements
- Required finish to be achieved
- Conditions and potential constraints of the site
- Co-ordination of site activities
- Workplace Health and Safety
- Regulatory requirements (e.g. performance required to satisfy the NCC)
- Other technical requirements and applicable standards (e.g. VOC limits)
- Compliance with other applicable policies
- Verification of compliance
- Evidence of suitability
- Applicable Quality Control and Quality Assurance Provisions including requirements for a declaration of performance from the manufacturer / supplier and required site inspections during installation.

6 Product Design and Development, Manufacturing, Supply and Evidence of Suitability

Product Design and Development

Fire-retardant coatings should be designed to be fit for purpose and a statement should be developed clearly stating the design objectives and intended purpose, design life and conditions that have to be satisfied for the design objectives to be satisfied.

Analysis and testing should be undertaken to determine that the design objectives will be satisfied throughout the design life of the system and to provide appropriate evidence of suitability that support the stated claims in literature and declarations of performance.

Quality Assurance

Manufacturers of fire-retardant products need to implement and maintain quality control systems that monitor the production of their products to ensure that the performance of the supplied products is capable of achieving the same performance as specified in their literature and declarations of compliance.

The Quality System should include procedures to ensure that products are not substituted or modified prior to installation and that installation contractors have received adequate training and are competent in the application of the coating including preparation of the substrate.

Labelling and Identification on Product Containers

Labelling on the product containers should include the following in addition to information required by other relevant legislation such as WHS regulations. The labelling should be in English and clearly legible supported by pictograms where appropriate.

- a unique identifier of the product that clearly differentiates the product from similar materials provided by the manufacturer / supplier
- colour of product
- batch number and date of manufacture
- intended use or uses of the product
- performance of the product when applied to nominated substrates or reference to a data sheet containing this information
- application instructions including details of compatible primers and top coats, drying time, recoating time, required wet and dry film thicknesses, and theoretical coverage or reference to a data sheet containing this information
- use by date
- the name, Australian address and business telephone number of either the manufacturer or importer / supplier
- any information about the hazards, first aid and emergency procedures relevant to the product and reference to a Safety Data Sheet
- storage instructions.

Product Data Sheet

Product Data Sheets should be provided and contain as a minimum the following information in addition to information required by other relevant legislation:

- the name, Australian address and business telephone number of either the manufacturer or importer / supplier
- unique identifier of the product that clearly differentiates the product from similar materials provided by the manufacturer / supplier
- colour range of product
- intended use or uses of the product
- general physical properties
- general application, typical drying time and recoating times, required wet and dry film thicknesses, and theoretical coverage
- details of compatible primers and top coats
- surface preparation requirements
- general application requirements
- available finishes
- safety precautions (including reference to Safety Data Sheet)
- performance of the product when applied to nominated substrates or reference to the Declaration of Performance containing this information including reference to evidence of suitability as required by the NCC
- approximate VOC concentrations
- storage instructions
- design life of installed system
- inspection and maintenance procedures
- end of design life procedures.

Declaration of Performance

General Declaration of Performance

The manufacturer must provide a declaration of performance of their product clearly stating the performance and referencing appropriate evidence of suitability on which the statement is based.

The declaration should be on company letter head and signed on behalf of the company by a director or authorised delegate.

The declaration must contain as a minimum the following:

- the name, registered trade name or trade mark, contact address, business telephone number, of the manufacturer. If the product is supplied through an importer / supplier other than the manufacturer the same details should be supplied for the importer / supplier
- unique identifier of the product that clearly differentiates the product from similar materials provided by the manufacturer / supplier
- intended use or uses of the product
- performance of the product when applied to nominated substrates including reference to evidence of suitability as required by the NCC
- durability performance including design life for nominated exposure conditions including reference to supporting evidence
- requirements for primers / top coats and coating thicknesses to achieve the stated performance
- approximate VOC concentrations (if low VOC content is claimed)
- details of any known hazards associated with the use of the product and reference to the Safety Data Sheet.

Project Specific Declaration of Performance

For major and or critical projects, a project specific declaration may be requested from a manufacturer / supplier. In these instances the manufacturer/supplier will:

- review the schedule of the supplier works required
- nominate the coating system to be used to achieve the required performance and design life having regard for the exposure conditions
- undertake training of the applicators or determine by other means that the applicator is competent to undertake the installation in accordance with the manufacturer's instructions
- check that adequate materials have been supplied to the contractor to undertake the project
- undertake inspections to check installations
- provide a project specific declaration that includes the information required in the general declaration of performance plus the following:
  - a schedule of protected elements with a clear statement of the performance required for each element and the coatings applied
  - confirmation that training or a competency assessment has been undertaken of the applicators to ensure they have the necessary competencies to install the products in accordance with the instructions
  - sufficient materials have been supplied to the contractor to complete the project
  - details of the inspections undertaken and observations made
  - a statement that based on investigations undertaken it is considered likely that the performance listed on the schedule will be achieved for the nominated design life subject to maintenance of the system in accordance with the nominated maintenance procedures

If a manufacturer / supplier lacks sufficient knowledge and documentation to support the above claims they should obtain the necessary information from the product designer. For smaller projects, where this level of involvement may be impractical, the installation should be monitored by a nominated member of the building design team and the necessary checks undertaken by that person. The use of installers recommended by the manufacturer / supplier under these circumstance would be prudent.

Installation Instructions

Installation instructions should be provided and contain as a minimum the following information in addition to information required by other relevant legislation:

- product storage instructions
- checks to ensure correct materials are used
- environmental conditions for application
- surface preparation and surface condition required for application
- primer requirements
- fire-retardant coating installation requirements and methods.
- required coating thickness and methods of verification
- top coat requirements
- finishing requirements
- potential installation defects and rectification methods
- repair methods if coating system is damaged
- clean up procedures including disposal of excess materials
- workplace health and safety precautions including reference to a Safety Data Sheet.

Evidence of Suitability / Compliance

Notwithstanding the requirement to provide a general declaration of performance it is still necessary to provide evidence of suitability in accordance with the requirements of the relevant regulations and standards such as the National Construction Code to the satisfaction of the relevant regulatory authority. This information will also be required by Building Owners at some stage who have a responsibility to check the building complies with regulations and undergoes regular maintenance checks.

Typical evidence of suitability may include the following:

- report from an Accredited Testing Laboratory confirming fire performance
- report supporting the claimed design life
- Safety Data Sheet
- other supporting data relevant to claims made in the declaration of performance.

7 Installation of Fire-retardant System

General Requirements

Fire-retardant coatings must be installed by persons trained by the product manufacturer or their representatives, or a nominated training organisation with the necessary knowledge of the specific products.

The installer should be provided with a specification, schedule of elements to be treated and required performance and associated drawings. This should be reviewed by the building certifier or authority having jurisdiction and a design team member to confirm the specified performance satisfies the relevant regulatory requirements and client brief.

The installer should review the schedule and confirm a coating system that will achieve the required performance, if necessary checking with the manufacturer, design team and building certifier.

A quality plan should be developed and installation program established by the installer in conjunction with the appropriate building design team member(s) and the authority having jurisdiction and builder to ensure compliance with the manufacturer's instructions. The quality plan must include details of inspections and verification requirements for coating thicknesses.

The installer must also comply with site WHS requirements and dispose of excess materials in accordance with the manufacturer's instructions and relevant legislation.

The installation must comply fully with the manufacturer's instructions. If there are any variations, the matter should be referred to the manufacturer in the first instance who may need to refer the matter to a registered testing authority for confirmation.

Declaration of Compliance of Installation / Schedule of Installed Product

Upon completion, the schedule of fire-retardant protected elements and drawings should be updated to clearly identify the protected areas, coatings applied including thickness, batch number and date applied and required performance and design working life. This should be appended to a declaration of compliance providing a statement that the installation has been undertaken in accordance with the manufacturer's instructions with the product and product thicknesses nominated on the attached schedule.

The declaration should be on company letter head and signed on behalf of the company by a director or authorised delegate and include the name, registered trade name or trade mark, contact address, business telephone number, of the installer and any applicable company or individual registrations.

Copies of this declaration should be provided to the building certifier, builder and design team for subsequent inclusion in a maintenance schedule and fire safety manual for the building which should also include the manufacturers instructions regarding maintenance and any restrictions on overpainting. Copies of the maintenance schedule and fire safety manual should be provided to the building owner.

8 Inspection and Maintenance

The NCC and Building regulations applicable in the States and Territories apply different requirements for the maintenance of fire safety provisions in domestic construction (Class 1 buildings) compared to commercial buildings (Class 2 to 9 buildings).

Inspection and Maintenance of Class 1 buildings

For Class 1 buildings, the main application for fire-retardant coatings is for bushfire protection in accordance with AS 3959 and there are generally no mandatory requirements for annual inspection and maintenance of fire safety provisions. It is important that building owners are made aware of the need for annual inspection and maintenance of fire-retardant coatings and actions to take at the end of the design life of the coating.

It is recommended that a schedule of fire-retardant treated elements and maintenance procedures are provided and kept in the meter box or other safe storage position readily accessible so that they are available to future residents.

Inspection and Maintenance of Class 2 to 9 buildings

For Class 2 to 9 buildings a maintenance process is required in most jurisdictions. Many jurisdictions reference AS 1851 which requires annual inspections of fire and smoke barriers and fire resistant structural members protected by coatings and it is therefore appropriate to apply the same requirements to fire-retardant coatings for timber elements. Therefore annual inspections should be undertaken as a minimum in addition to any additional requirements specified by the manufacturer.

As a minimum, the annual inspection should undertake the following activities in addition to any other activities specified by the manufacturer:

- refer to the schedule / fire safety manual to determine the extent of fire-retardant wood products, coating requirements and design life
- check if the design life has been exceeded – if it has initiate the end of design life procedures
- inspect the elements for any damage unauthorised over-painting and modification. If damage or over-painting is identified, require repair in accordance with the manufacturer's procedures.

End of Design Life Procedures

At the end of the design life of fire-retardant coating it is necessary to either verify that the performance of the system still satisfies minimum regulatory requirements or repair / reinstate the performance in accordance with manufacturer's instructions, which should be based on evidence of suitability from an Accredited Testing Laboratory. Typically, the design life may be extended on the basis of:

- Accelerated weathering / durability testing justifying the extension in design life
- Field testing / experience in the use of the system
- Testing samples taken from the building that have been exposed to similar environmental conditions. These samples may be taken from elements of construction or from samples prepared at the time of installation that have been retained on the site and exposed to similar environmental conditions.

In most instances small-scale cone calorimeter testing will provide appropriate data. The results should be reviewed by a registered testing authority and a revised design life estimated. Confirmation of the extended design life should be recorded on the maintenance schedule and building fire safety manual in the form satisfying the NCC requirements for evidence of suitability (e.g. a report from an Accredited Testing Laboratory).

Appendix A: Definitions

Fire-retardant coating: A coating supplied in liquid, paste or powder form that when applied to a substrate, improves one or more of the fire performance characteristics of the substrate.

Intumescent coating: A coating that is specifically formulated to provide a chemical reaction upon heating such that the physical form changes into an expanded foam and, in so doing, provides protection to the underlying surfaces from fire.

Encapsulation coating system: A coating system that completely encases a surface.

Surface treatments: A product in liquid or paste form that, when applied to a substrate, penetrates below the surface and, on drying or curing, deposits substances that impart fire-retardant properties to the substrate.

Durability: The capability of a building or plumbing installation to perform its function over a specified period of time.

Design life: The period for which a building or component of a building is expected to fulfil its intended function assuming regular maintenance will be carried out and that there will be no unusual events such as a large earthquake.

Maintenance: The total set of activities performed during the design life to retain a building installation in a state in which it can fulfil its intended function. Routine inspection activities are included within this definition of maintenance.

Product Supply Chain: includes individuals and entities responsible for any of the following; design, manufacture. importation, distribution / supply and installation of a product.

Repair: activities performed to return a building or building component to an acceptable condition. The activities may include raising the performance level, extending the design life, and making good any damage that impairs the original functioning and design life of a component. Maintenance may lead to repair.

Reinstatement (of fire properties): activities performed to ensure that the relevant fire properties of the element comply with the performance specified at the time of design and construction in order to comply with the National Construction Code, other relevant legislation and performance specifications.

Appendix B: Specimen Preparation for Accelerated Weathering Tests

The accelerated weathering specimen size must be the same as for the subsequent fire testing or preferably larger but with a minimum size along the grain of 800 mm.

The application of fire-retardant coating must be representative of the end use (i.e. if only one face is treated with a fire-retardant coating then only one face of the specimen should be treated with the coating).

The exposure to weathering must be representative of the end use (i.e. if only one face is exposed to weathering then only one face of the specimen should be exposed to accelerated weathering).

If edges perpendicular to grain are not intended to be exposed to weather the edges are to be sealed. A suitable seal consists of a thin coat of alkyds primer and a thick top coat of silicon sealer. If the rear face is not to be exposed to weathering, it may be covered by an impermeable membrane adhered to the specimen such as polyethylene.

The timber thickness should be representative of the end use.

Where small-scale fire testing is performed e.g. in the cone calorimeter (AS 3837), cut the specimen for fire testing ≥100 mm from the sealed edge of the exposed board, after accelerated weathering.

References

1. ABCB, *National Construction Code 2019 Volume One Amendement 1, Building Code of Australia Class 2 to 9 Buildings.* 2019, Australian Building Codes Board: Canberra.
2. Safe_Work_Australia, *Safe Design of Structures - Code of Practice* 2012, Safe Work Australia.
3. Standards_Australia, *AS 5637.1 Determination of Fire Hazard Properties.* 2015: Sydney Australia.
4. ISO, *AS ISO 9239.1-2003 (R2016) Reaction to fire tests for floor coverings - Determination of the burning behaviour using a radiant heat source* 2003, Standards Australia.
5. Standards_Australia, *AS/NZS 1530.3:1999 (R2016) Methods for fire tests on building materials, components and structures - Simultaneous determination of ignitability, flame propagation, heat release and smoke release* 1999, Standards Australia: Sydney.
6. Standards_Australia, AS *3959 Construction of Buildings in Bushfire-Prone Areas.* 2018, Standards_Australia: Sydney.
7. ISO, *ISO 9705 Fire tests - Full-scale room test for - surface products.* 1993, International Standards Organisation: Geneva.
8. Standards_Australia, *AS/NZS 3837:1998 Method of test for heat and smoke release rates for materials and products using an oxygen consumption calorimeter* 1998, Standards Australia: Sydney.
9. ISO, *ISO 5660-1:2015 Reaction-to-fire tests - Heat release, smoke production and mass loss rate - Part 1: Heat release rate (cone calorimeter method) and smoke production rate (dynamic measurement)*. 2015, ISO.
10. *AS ISO 9239-1:2013 (R2016) Reaction to fire tests for floorings - Part 1: Determination of the burning behaviour using a radiant heat source.*
11. *AS 1684.2 2010 Residential timber-framed construction - non-cyclonic areas*, 2010 Standards Australia 2010.
12. *AS 1684.3 2010 Residential timber-framed construction cyclonic areas.* 2010 Standards Australia 2010.
13. *AS 1530.8.1:2018 Methods for fire tests on building materials, components and structures. Tests on elements of construction for buildings exposed to simulated bushfire attack - Radiant Heat and Small flaming Sources,* Standards Australia 2018.
14. *AS 1530.8.2:2018 Methods for fire tests on building materials components and structures. Tests on elements of construction for buildings exposed to simulated bushfire attack - Large flaming sources* Standards Australia 2018.
15. ASTM, *ASTM D2898 - 10 Standard Practice for Accelerated Weathering of Fire-Retardant-Treated Wood for Fire Testing.* 2010, ASTM: US.
16. Standards_Australia, *AS 5113:2016 Fire propagation testing and classification of external walls of buildings.* 2016, Standards Australia: Sydney.
17. International_Standards_Organisation, *ISO 13785-2 Reaction-to-fire tests for façades – Part 2: Large-scale test.* 2002, International Standards Organisation: Geneva.
18. *BS8414 Parts 1 and 2 Fire Performance of External Cladding Systems.*
19. International_Standards_Organisation, *ISO 14697 Reaction-to-fire tests - Guidance on the choice of substrates for building and transport products.* 2007, International_Standards_Organisation: Switxerland.
20. MacKenzie, C., et al., *Design Guide #5 Timber service life design - Design guide for durability*. 2013.
21. ABCB, *Handbook - Durability in Buildings including plumbing installations*. 2015, Australian Government and States and Territories of Australia: Canberra.
22. Standards_Australia, *AS 1851 2012 Routine service of fire protection systems and equipment.* 2012, SAI Global: Sydney.
23. EOTA, *ETAG 028 Guideline for European Technical Approval of Fire-retardant Products.* 2012, European Organisation for Technical Approvals: Brussels.

46

This guide was published in Sept 2018
Please check woodsolutions.com.au/publications to ensure you have the latest edition

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.

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-44-7

Prepared by:

Gregory Nolan M.Arch, B.Arch (Hons)

Director, Centre for Sustainable Architecture with Wood
School of Technology, Environments & Design, College
of Sciences and Engineering, University of Tasmania

Dan Tingley Ph.D., P. Eng.(Canada), MIEAust, CPEng, RPEQ

Senior Wood Technology/ Structural Engineer, Timber Restoration
Systems Pty Ltd./Wood Research and Development
Caboolture, Queensland

Additional contributors
Robert Keller, PE, Senior Timber Design Engineer,
Chris Legg, Structure Designer and Senior CAD,
Patrick Bigg, B Eng., Timber Design Engineer,
Caleb Hurst, PE, Timber Design Engineer.

Wood Research and Development

First Published: September 2018

Contents

Contents

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.

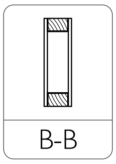
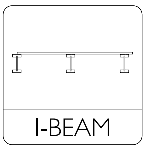
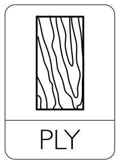
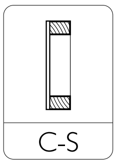
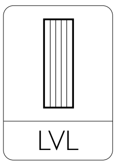
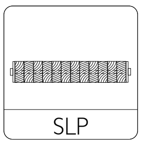
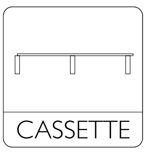
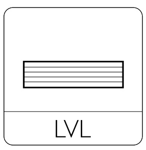
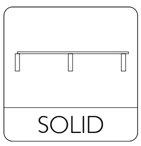
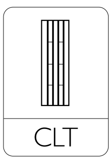
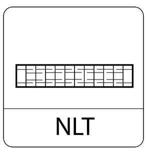
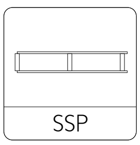
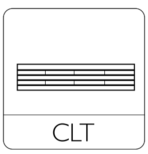
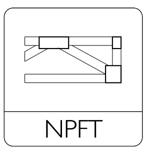
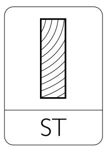
Acronyms and icons

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.

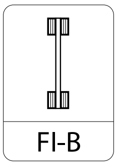
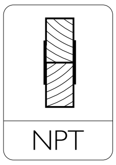
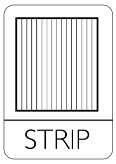
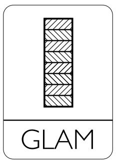
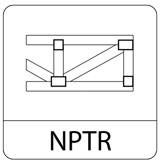
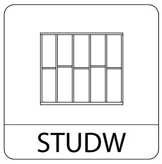
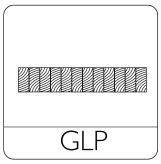
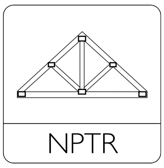
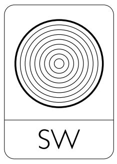
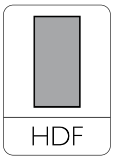
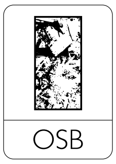
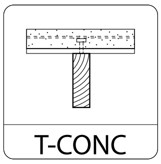
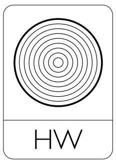
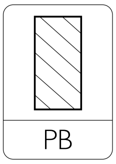
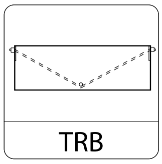
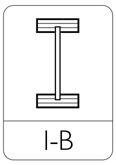
General acronyms

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		

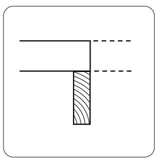
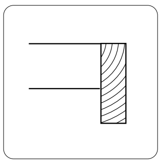
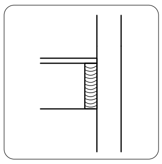
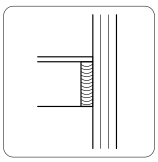
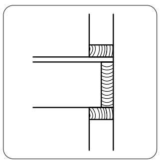
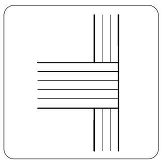
Material icons and acronyms

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

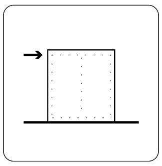
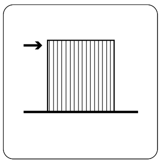
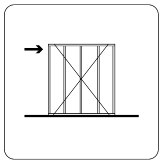
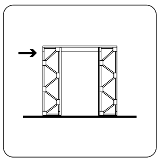
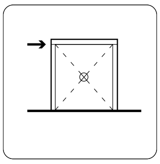
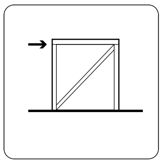
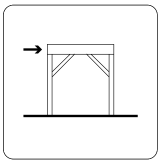
Material icons and acronyms (continued)

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				

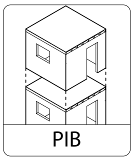
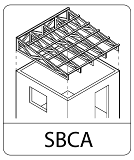
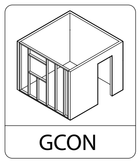
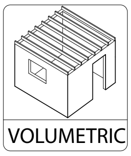
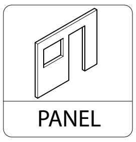
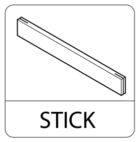
Construction arrangement icons

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

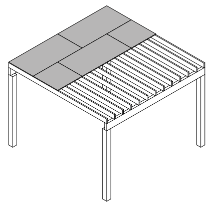
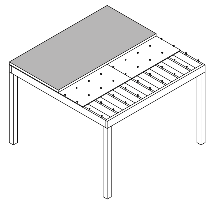
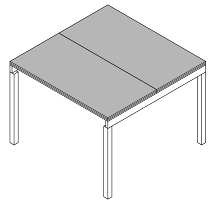
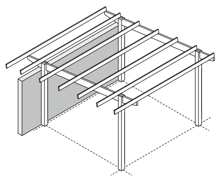
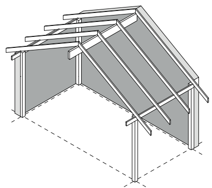
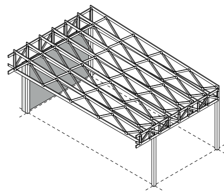
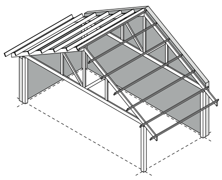
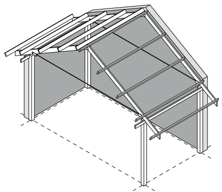
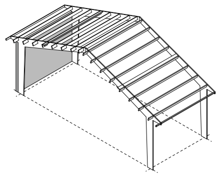
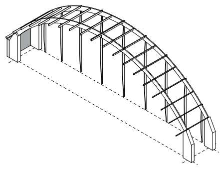
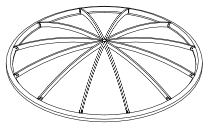
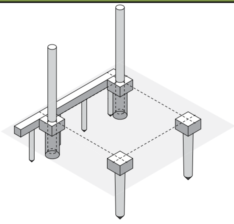
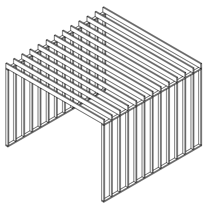
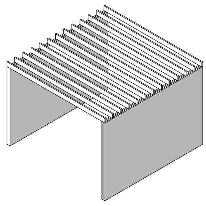
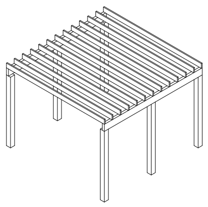
Timber contributes to design flexibility. A versatile product suite is available and these products can be configured into a broad set of construction options. These products are available off the shelf or they can be shaped or assembled to suit a particular project. This is discussed in Section 4.

Timber is clean and easy to work with accuracy

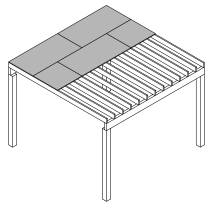
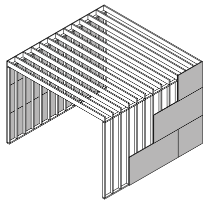
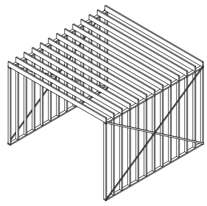
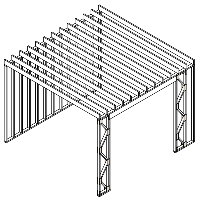
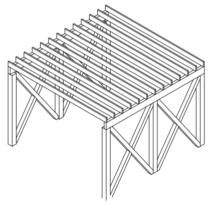
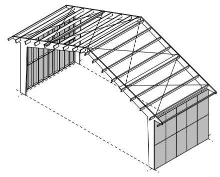
Timber elements are relatively light and clean to handle, and easy to work at either large or small scale. Timber is relatively soft and can be readily worked and shaped with simple hand and power tools or more sophisticated router and computer-controlled robots. It is easy to join. A child can nail 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.

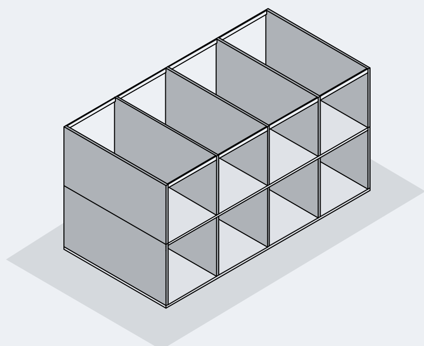
Building typologies

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.

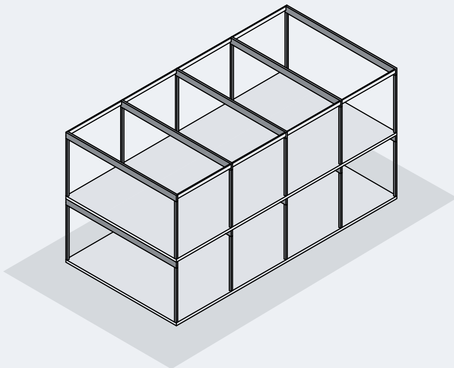
This system is particularly relevant to Class 2, 3 and 4 residential buildings and Class 9 health care buildings. In these building classes, the spanning, support and lateral resistance systems may have to comply with stringent regulatory requirements for fire and sound separation. These can influence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

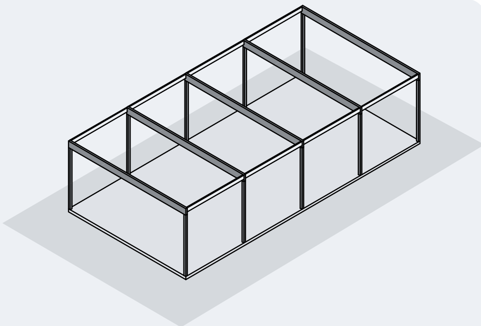
This system is particularly relevant to Class 5 and 6 buildings and Class 9 educational buildings. In these buildings, the columns, beams and floor plates must comply with stringent regulatory requirements for fire resistance while market requirements may dictate sound separation requirements. These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

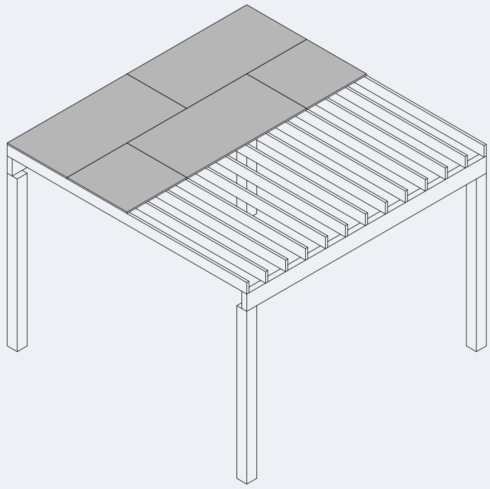
The spanning systems often have a greater impact on the cost and architectural character of a structure than the support or lateral systems. Generally, the critical factors to consider when selecting a spanning system for the span length, the applied loads and the required stiffness. Other important 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:

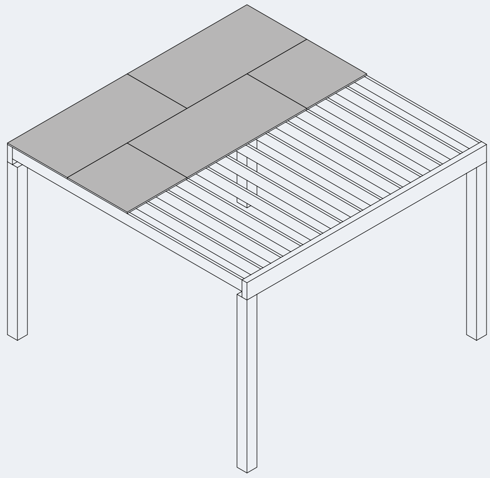
- joists and beams on columns or joists supported on walls and covered with a flooring material
- massive timber plates and beams on columns or plates supported on walls.

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

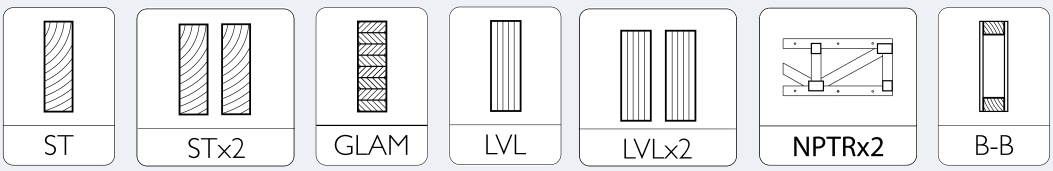
Description

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.

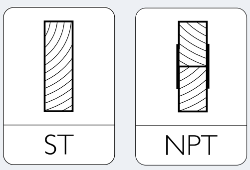
Regular beam options



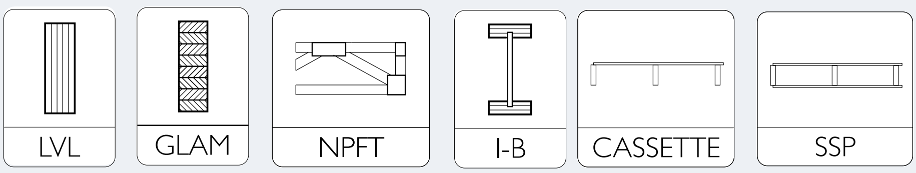
Notes
Glulam is available in wide sections.

Regular joist options by span and configuration

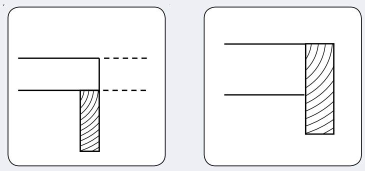
<5 m



5 m+

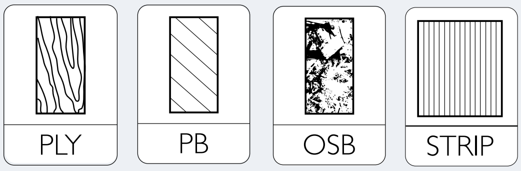


Assembly options



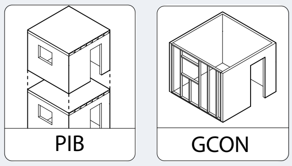
Assembly notes
When joists hang from the beam's sides, metal joist hangers typically secure them. The hangers can also assist resist racking.

Regular flooring options

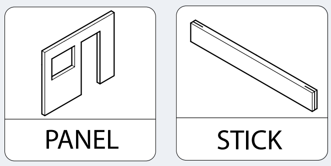


Notes
Panel systems can resist shear stress and serve as part of the lateral resistance system. Plywood can generally handle heavier concentrated loads than other floor panels.

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

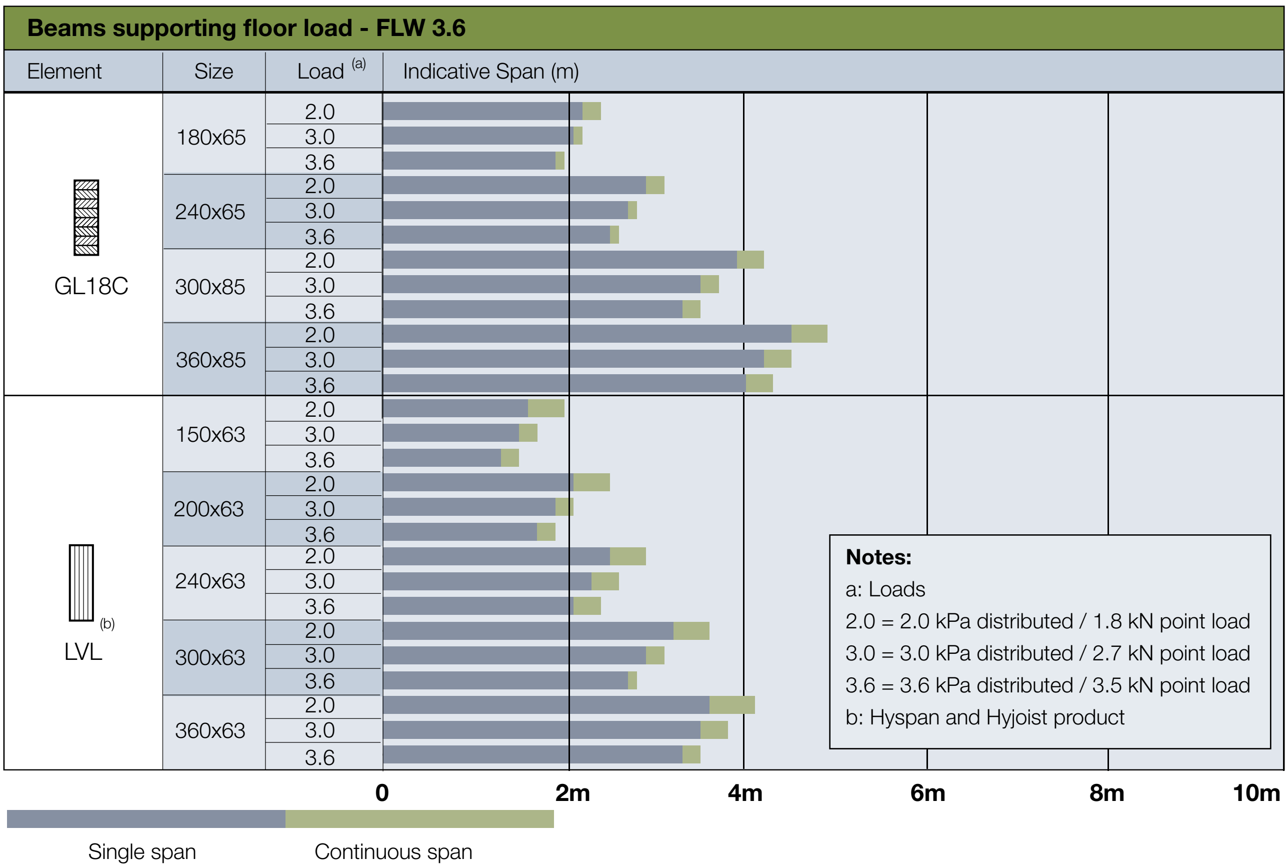


Table 3: Indicative span – Beams supporting floor load – FLW 4.5

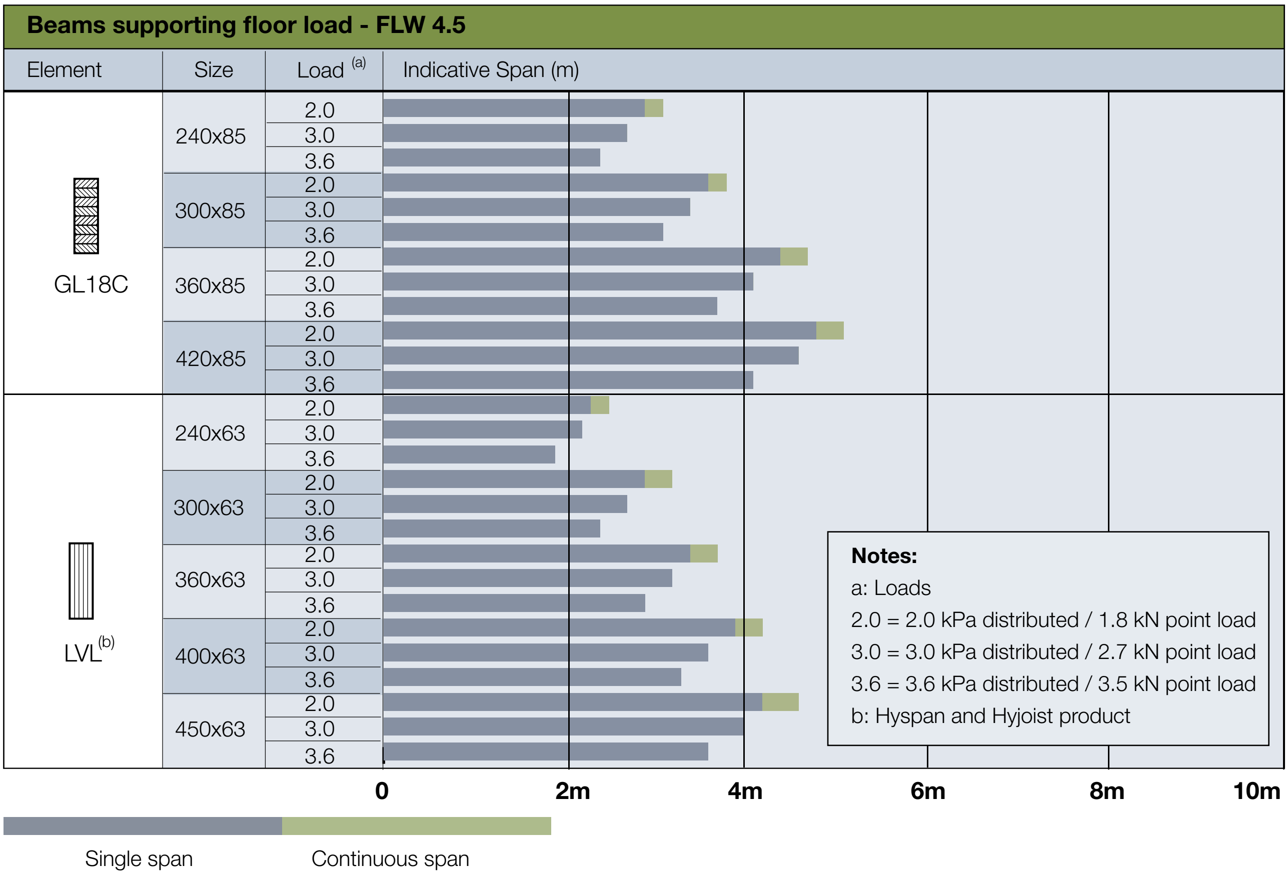


Table 4: Indicative span – Beams supporting floor load – FLW 5.4

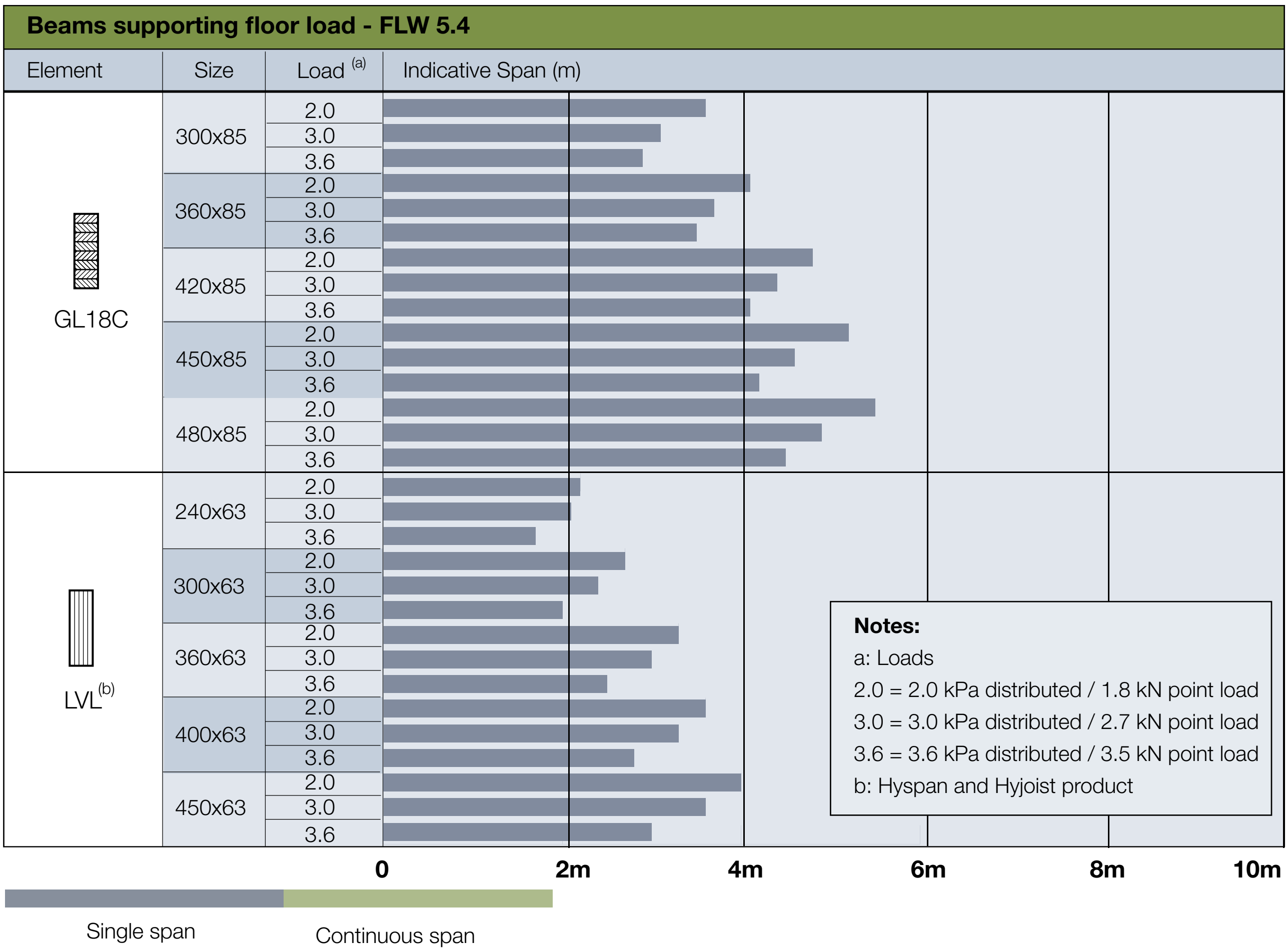


Table 5: Indicative span – Floor joist: 150 nominal depth at 450 centres

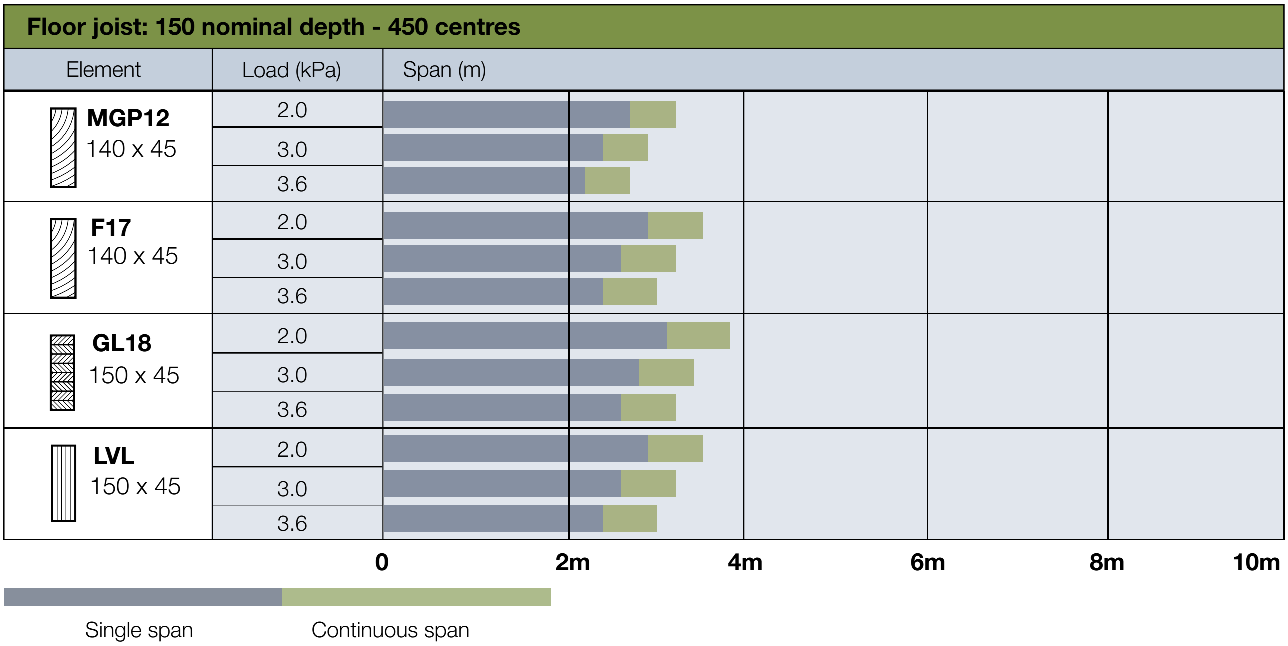


Table 6: Indicative span – Floor joist: 200 nominal depth at 450 centres

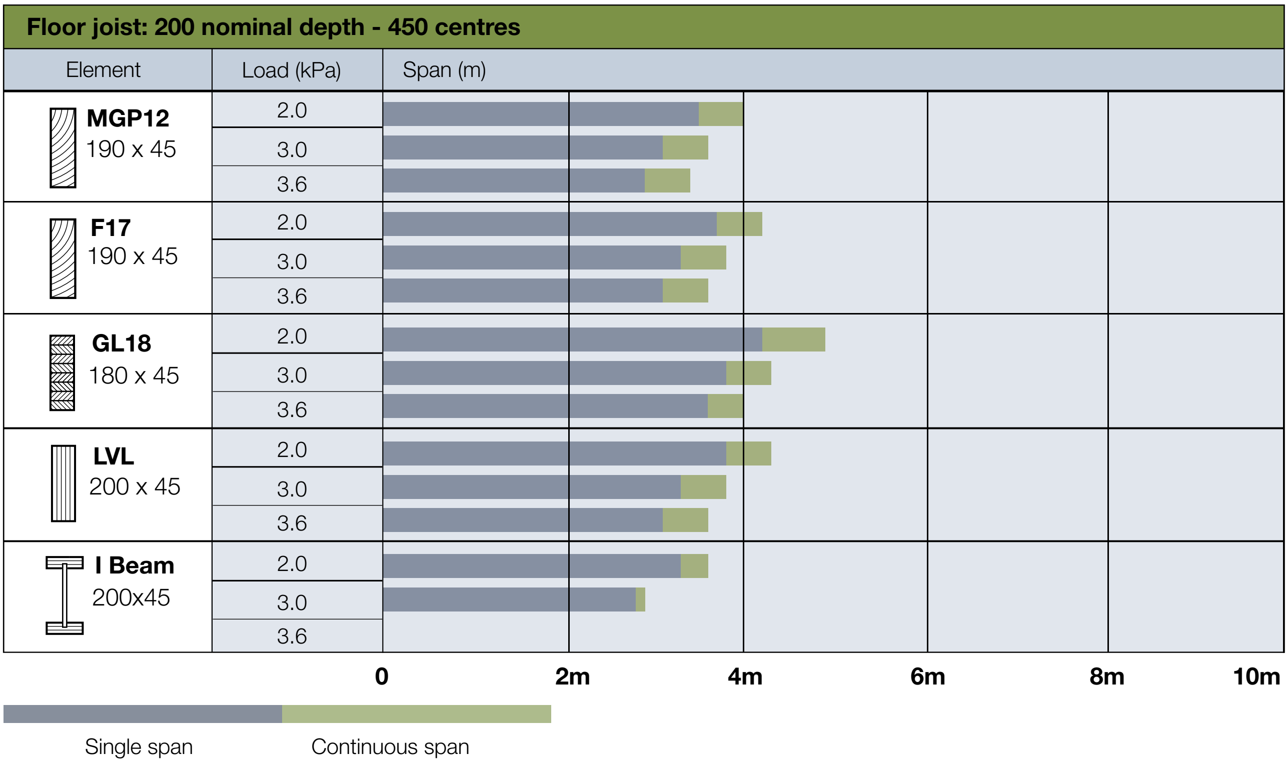


Table 7: Indicative span – Floor joist: 240 nominal depth at 450 centres

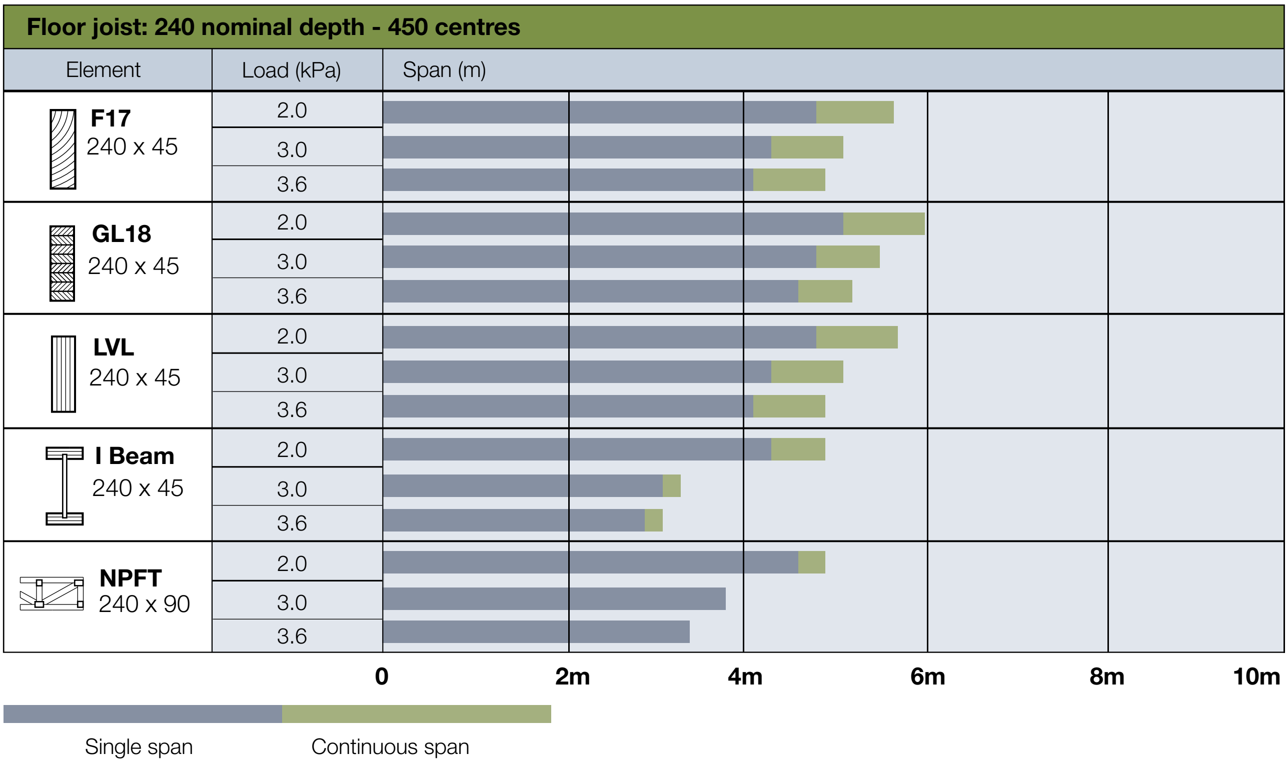


Table 8: Indicative span – Floor joist: 300 nominal depth at 450 centres

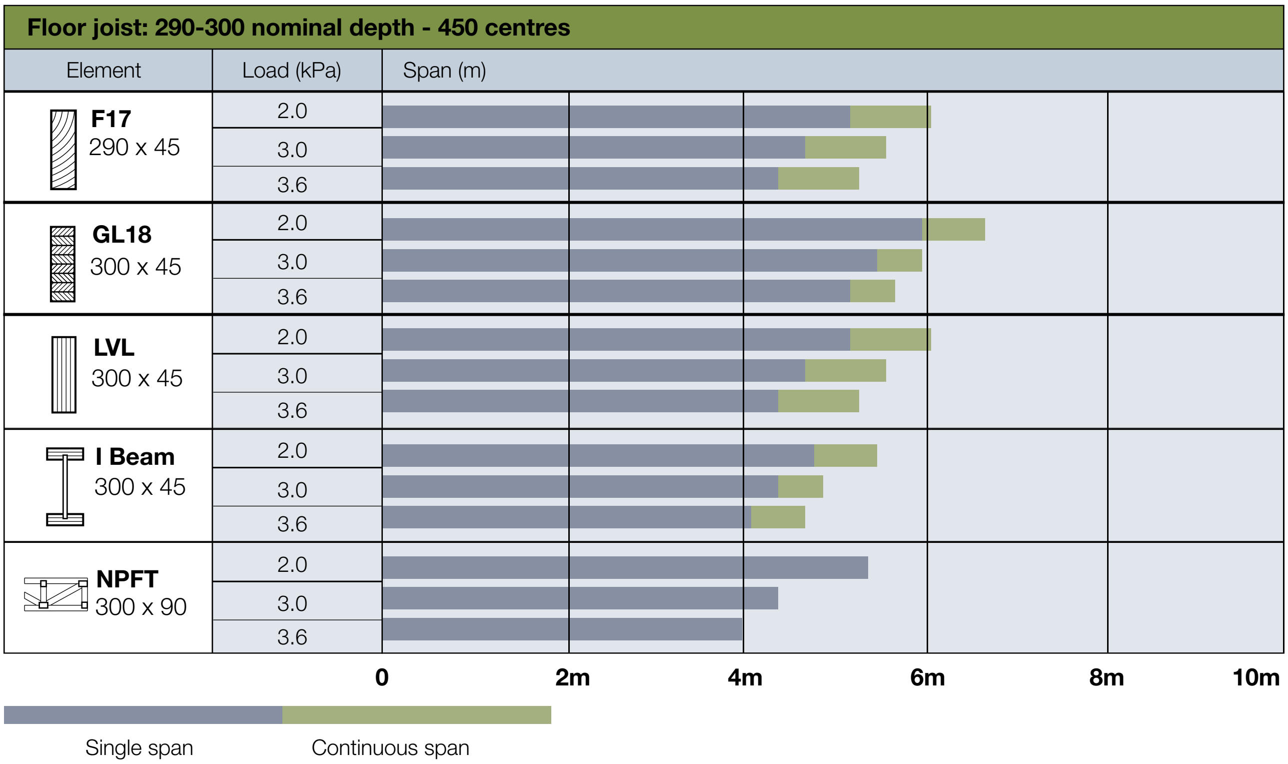


Table 9: Indicative span – Floor joist: 360 nominal depth at 450 centres

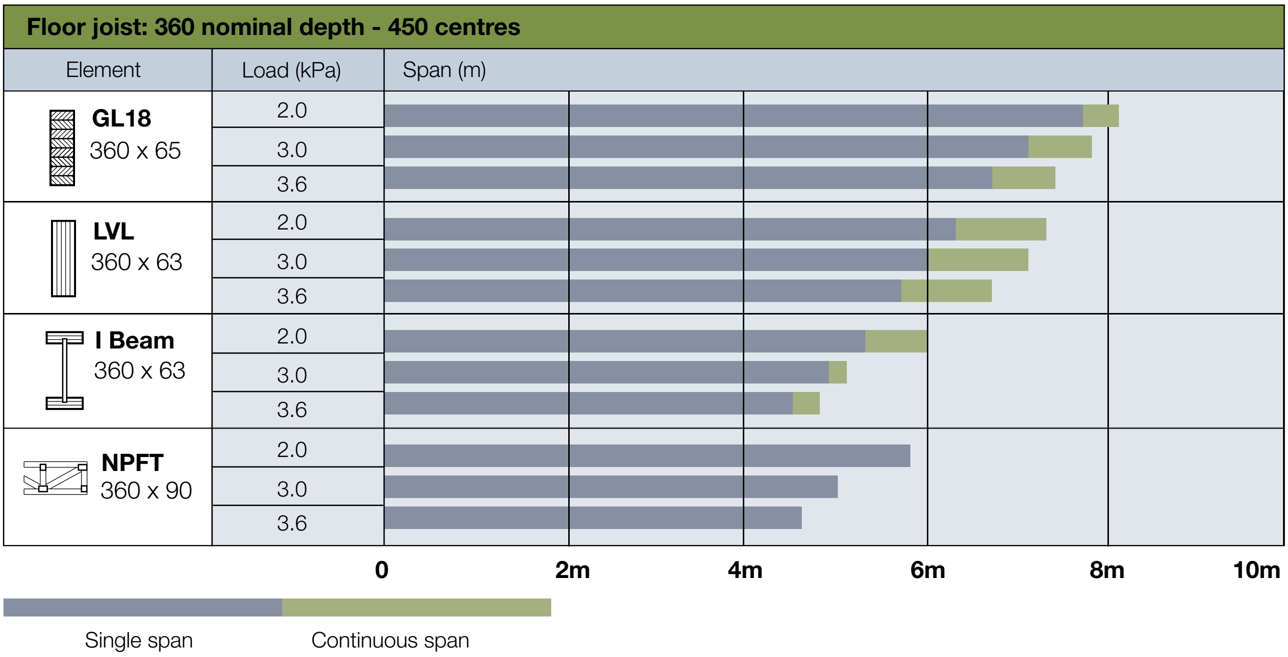


Table 10: Indicative span – Floor joist: 400 nominal depth at 450 centres

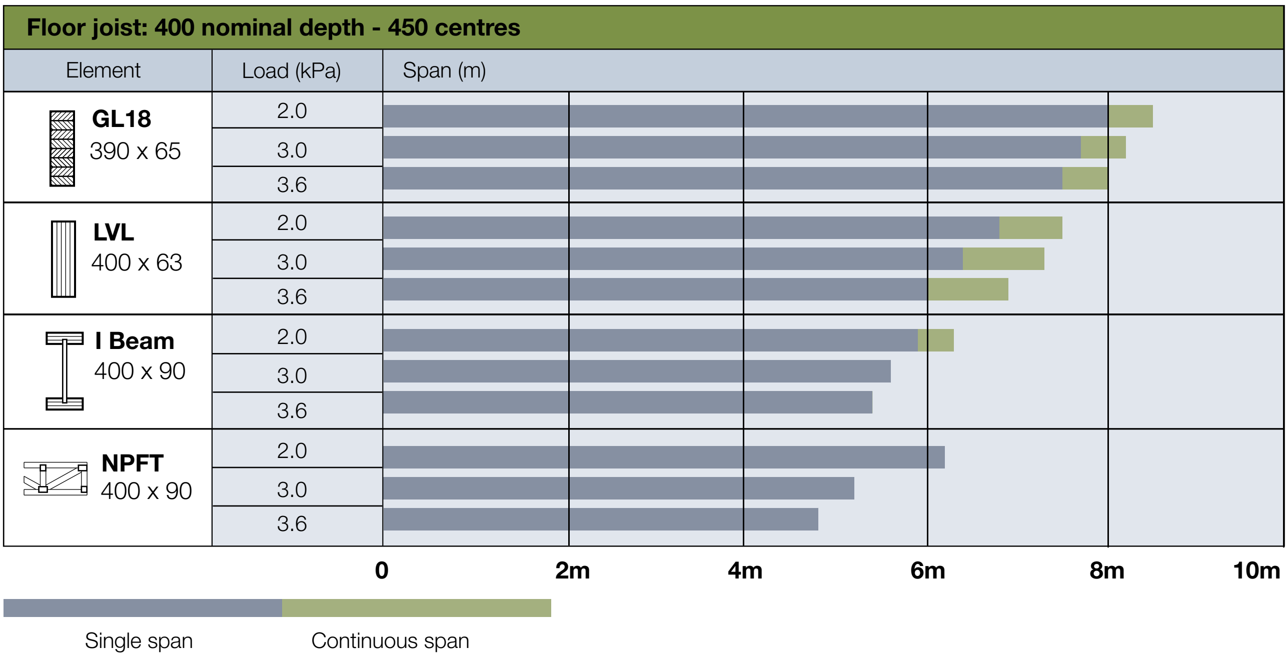


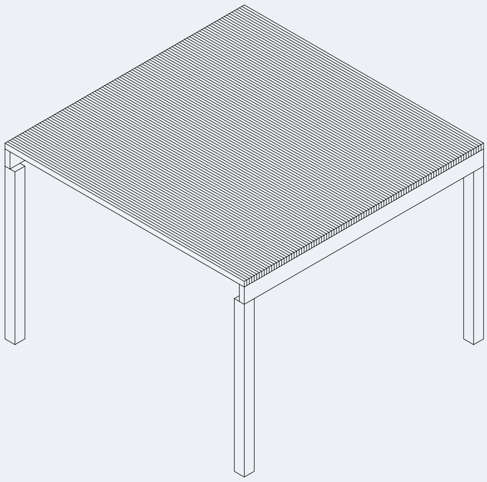
Table 11: Guide to maximum joist spacing for butt-jointed T&G flooring under domestic loads

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

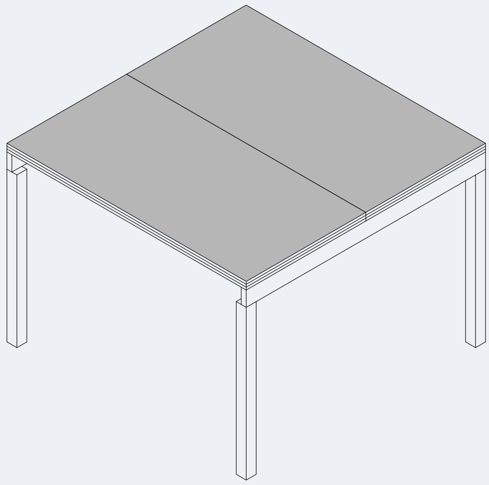
Table 12: Guide to maximum joist spacing for plywood flooring under domestic loads

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

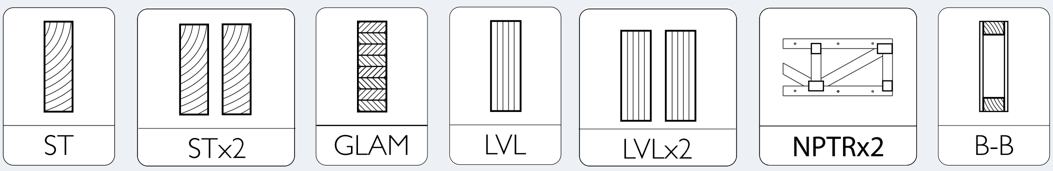
Panel span to depth ratio: 25 to 1

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.

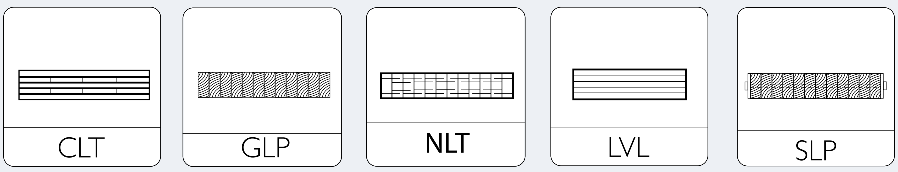
Massive timber systems are well suited to longer spans, heavier loads and larger structures. Compared to traditional joist floors, massive timber floors can have a shallower depth for a given span and provide excellent resistance to concentrated loads. They also have excellent fire-resistance. The panels 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



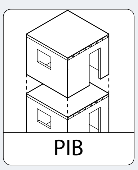
Notes
Glulam is available in wide sections.

Regular panel options

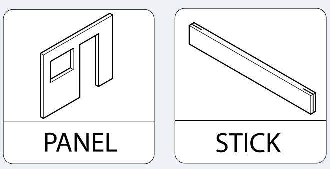


Notes
Stress laminated panels are generally used in civil construction such as bridge decks.

Regular construction and prefabrication options

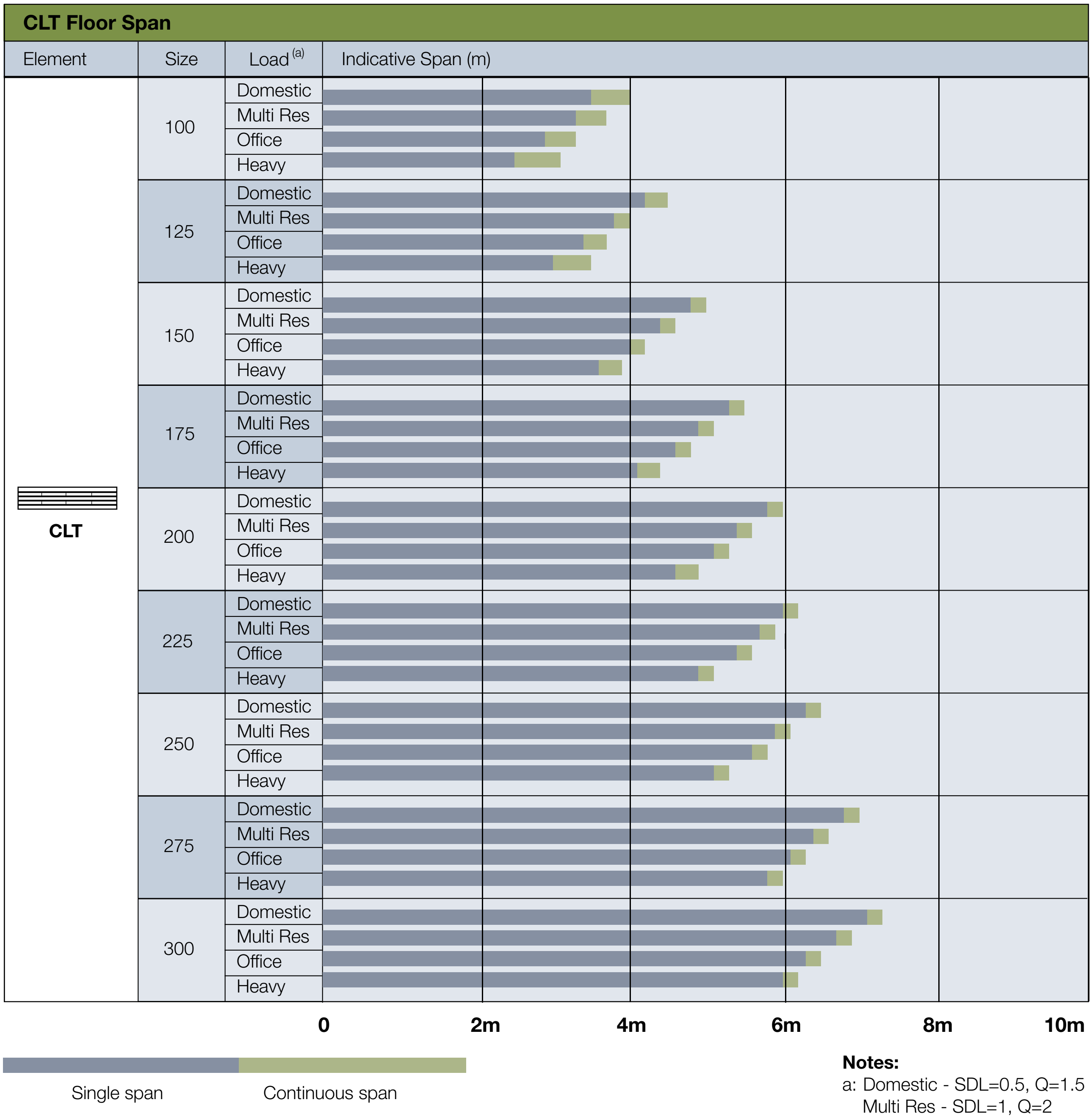


Construction notes
Beams and all floor panel systems can be fully prefabricated.



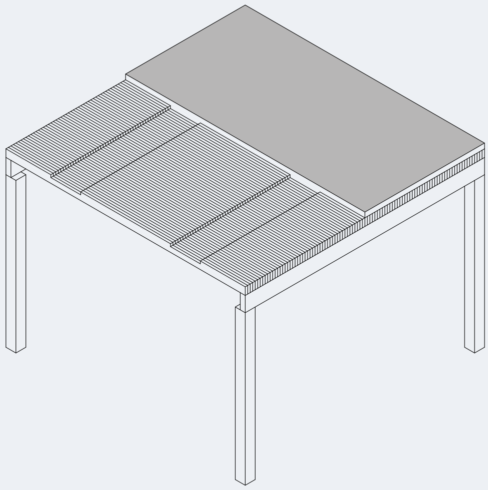
Prefabrication notes
Panels and beams can be fully prefabricated for the project.

Table 13: Indicative span – CLT floor panels

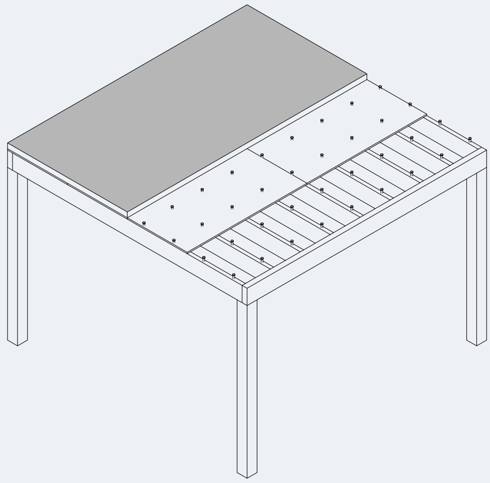


Notes:
a: Domestic - SDL=0.5, Q=1.5
Multi Res - SDL=1, Q=2
Office - SDL=1.5, Q=3
Heavy - SDL=2, Q=5

Source: Xlam



Composite massive timber floor



Composite joist system floor

Floor span range: up to 8 m

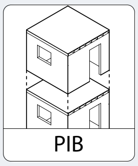
Description

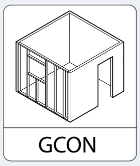
Timber components can be combined with a concrete slab to form a composite floor, with the concrete acting in compression and the timber acting in tension. To achieve composite action, shear connections are required between the timber and concrete. This may be achieved by cutting keyways into the top 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.

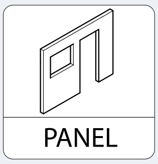
Creating a composite floor system can be an effective way of increasing the load-bearing, thermal or acoustic capacity of an existing wood floor system. The concrete increases the system's mass, reducing vibration and noise transmission, but places higher loads on the support system and foundation.

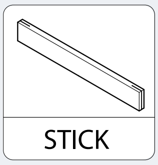
Regular construction and prefabrication options





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





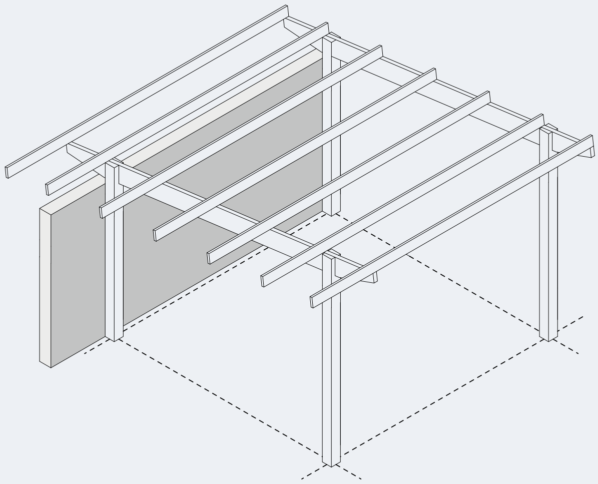
Prefabrication notes
Beams and floor plates can be prefabricated and installed with or without the concrete layer or built with generic products.

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.

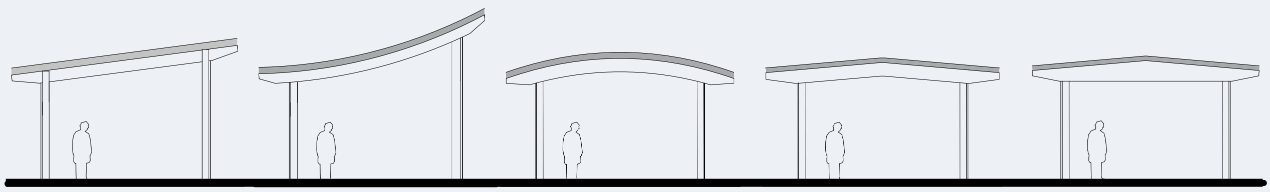
The required span is the primary design consideration for the roof's form and materials while other factors to consider include material availability, fabrication capacity, the potential for prefabrication, and appearance. Often, simple flat or pitched roofs are the most economical choice because they 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



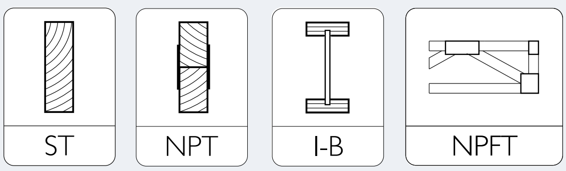
Description

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.

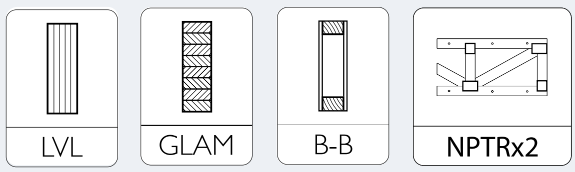
Most timber beams are rectangular in cross section and deep, slender beams are the most efficient at resisting bending loads. However, these may require additional lateral bracing to resist buckling or racking. Purlins set between the beams, instead of running continuously over them, can often provide 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

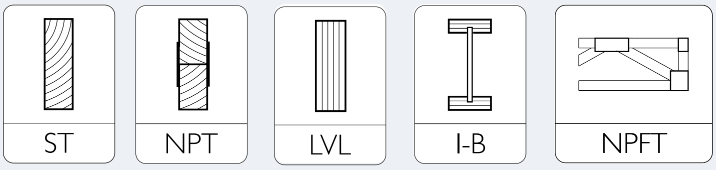
<5 m



5 m+

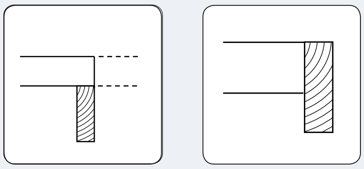


Regular purlin options and configuration



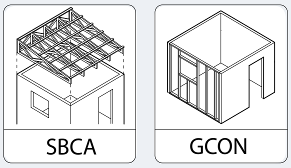
Notes
Purlin span to depth ratio is nominally 24 to 1. Options reduce considerably for spans over 4.8 m. See Table 19.

Assembly options

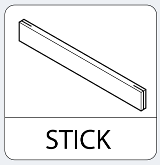


Assembly notes
Purlins set between beams provide them with lateral restraint while continuous spans allow for a smaller purlin section.

Regular construction and prefabrication options

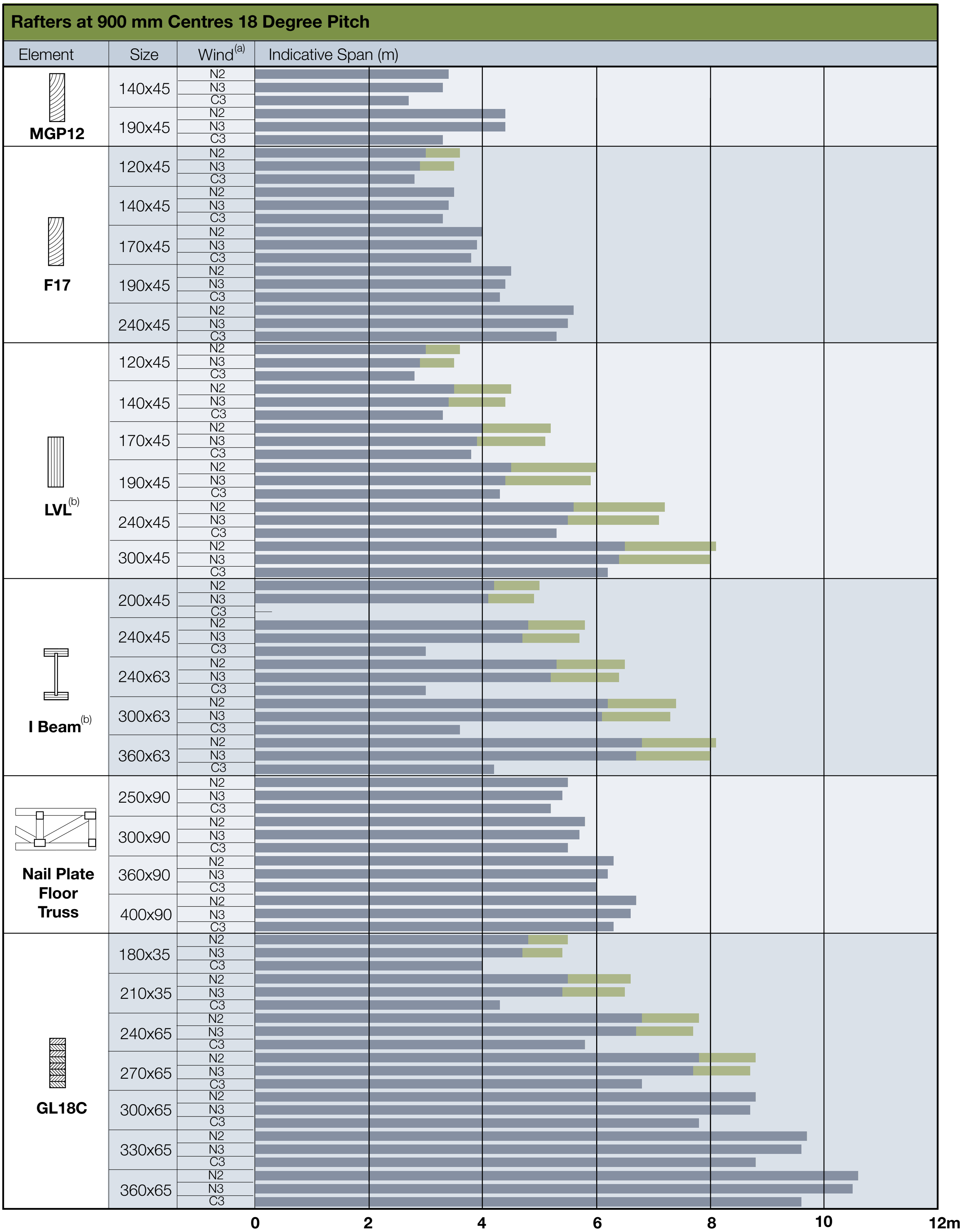


Construction notes
Systems are often site assembled but roof beams and purlins can be assembled as a module on the ground and lifted into place.



Prefabrication notes
Rafter and purlins can be optimised and shaped.

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



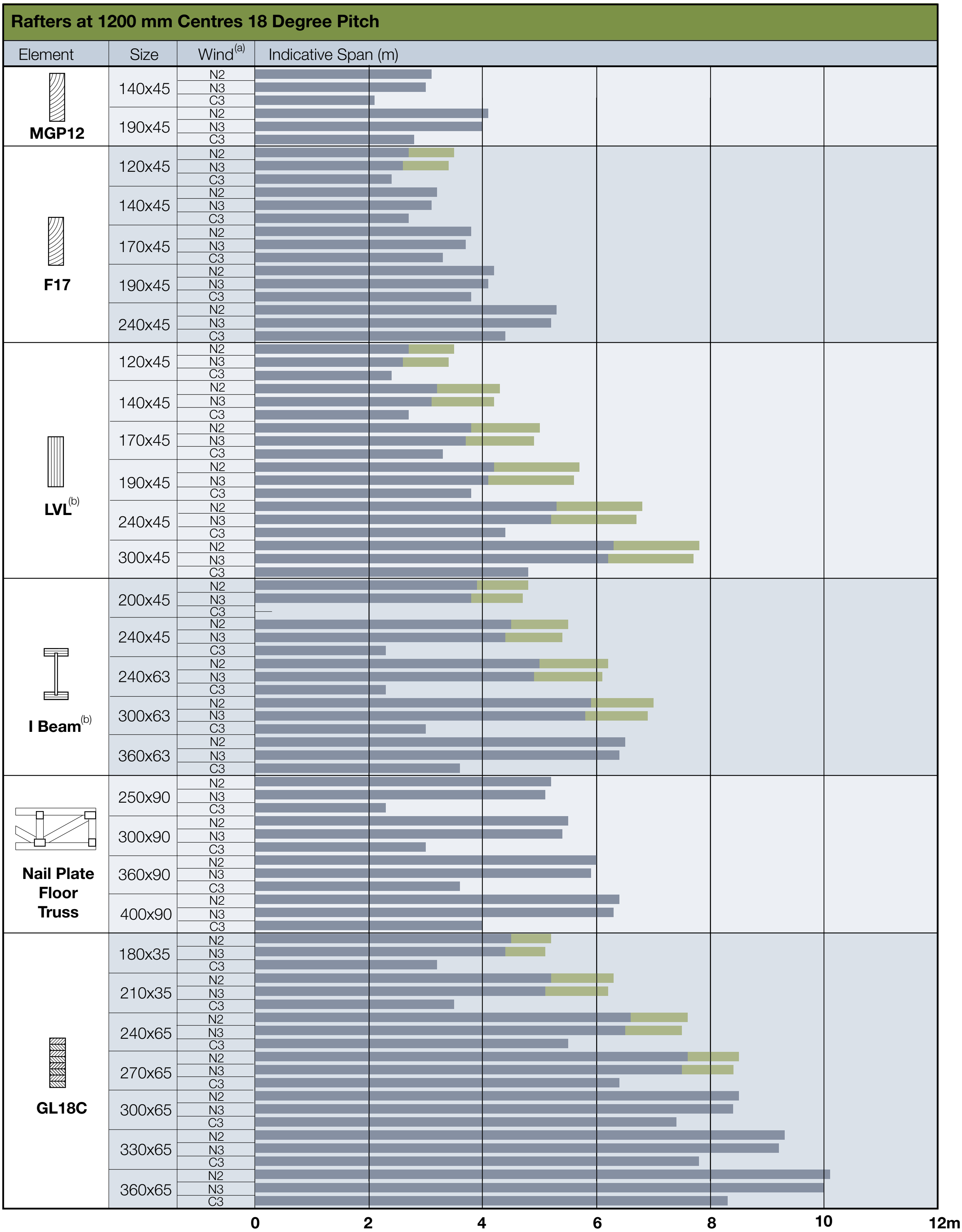
Notes:

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



Single span Continuous span

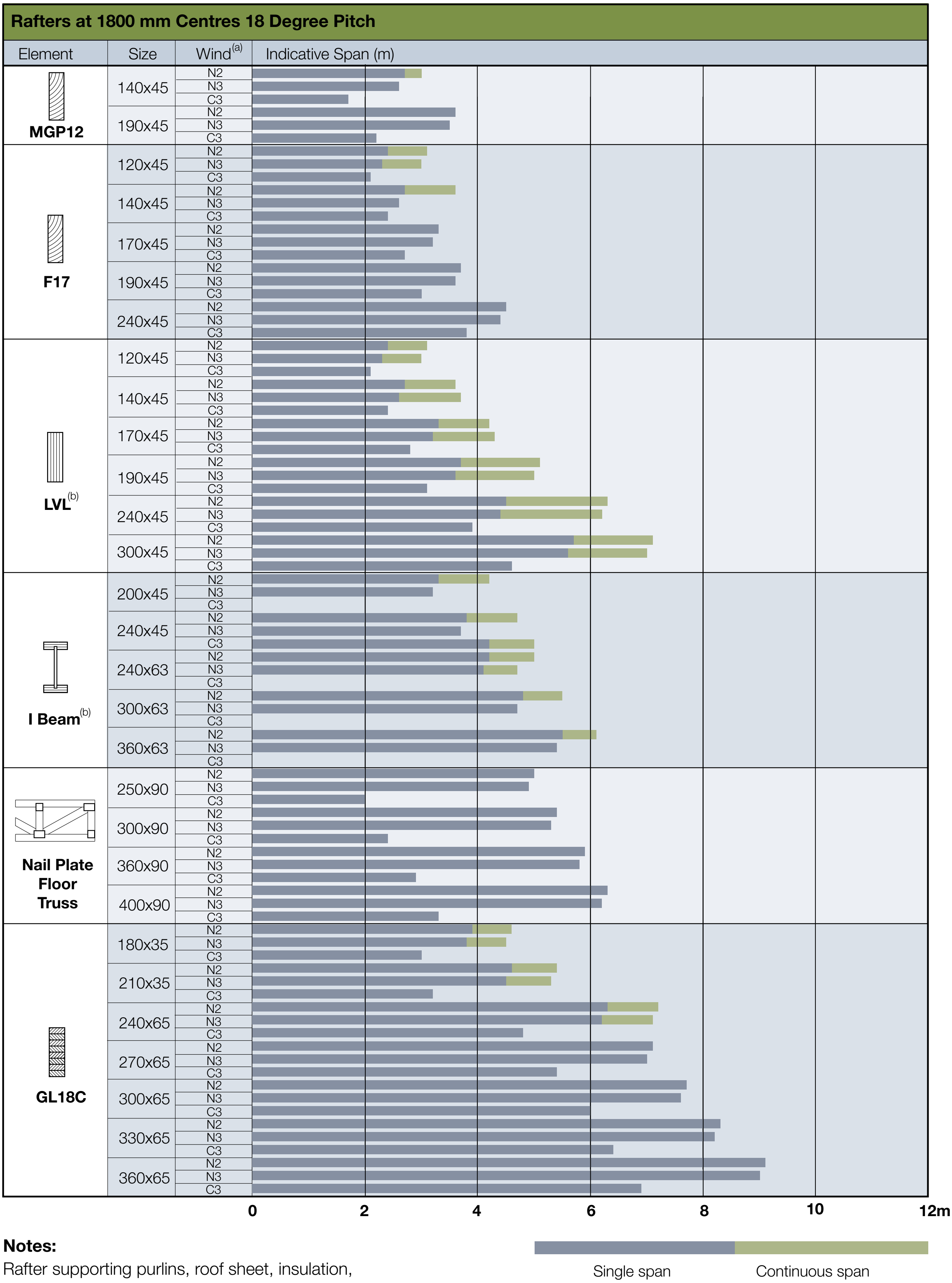
Notes:

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



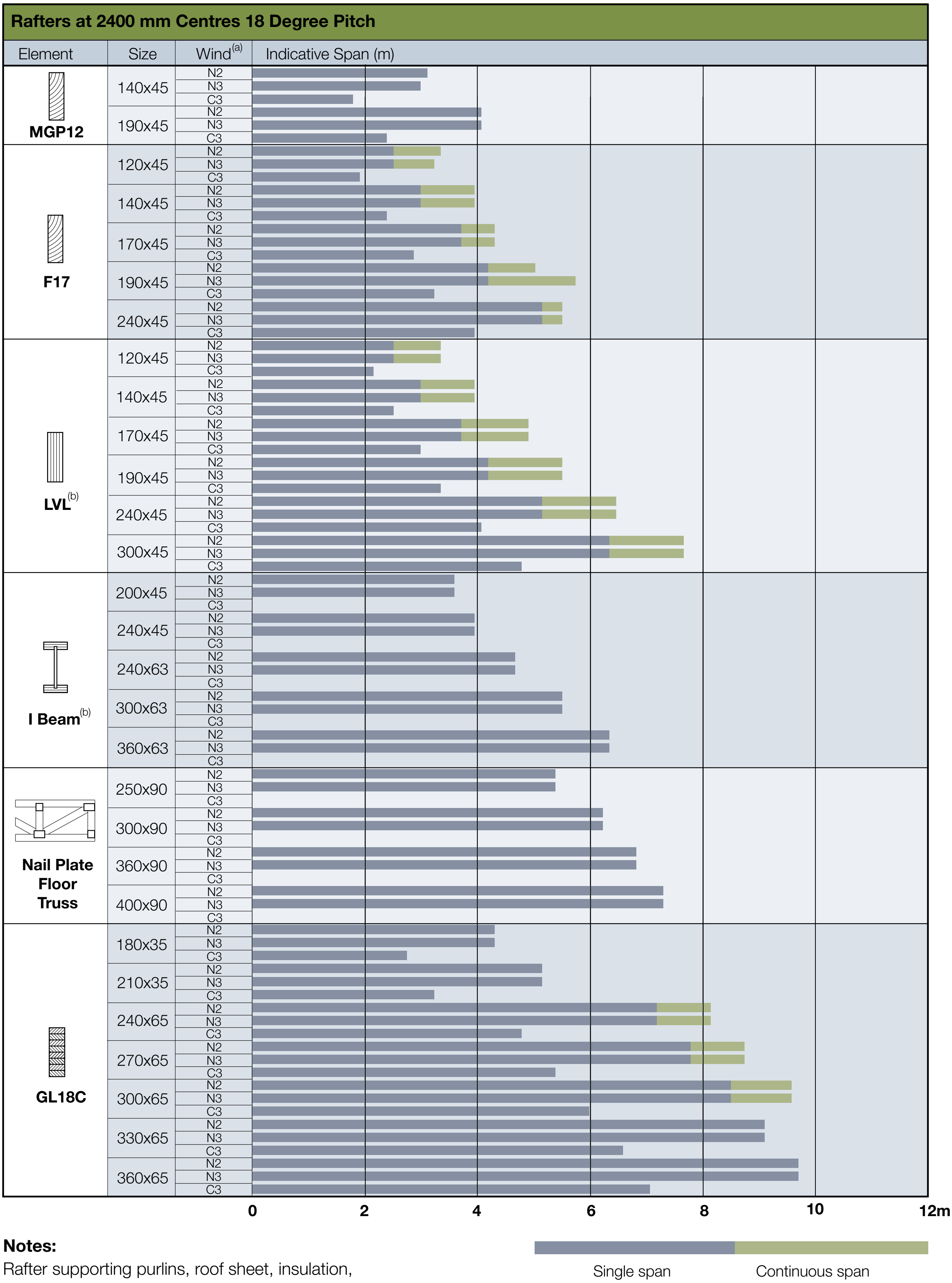
Notes:

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



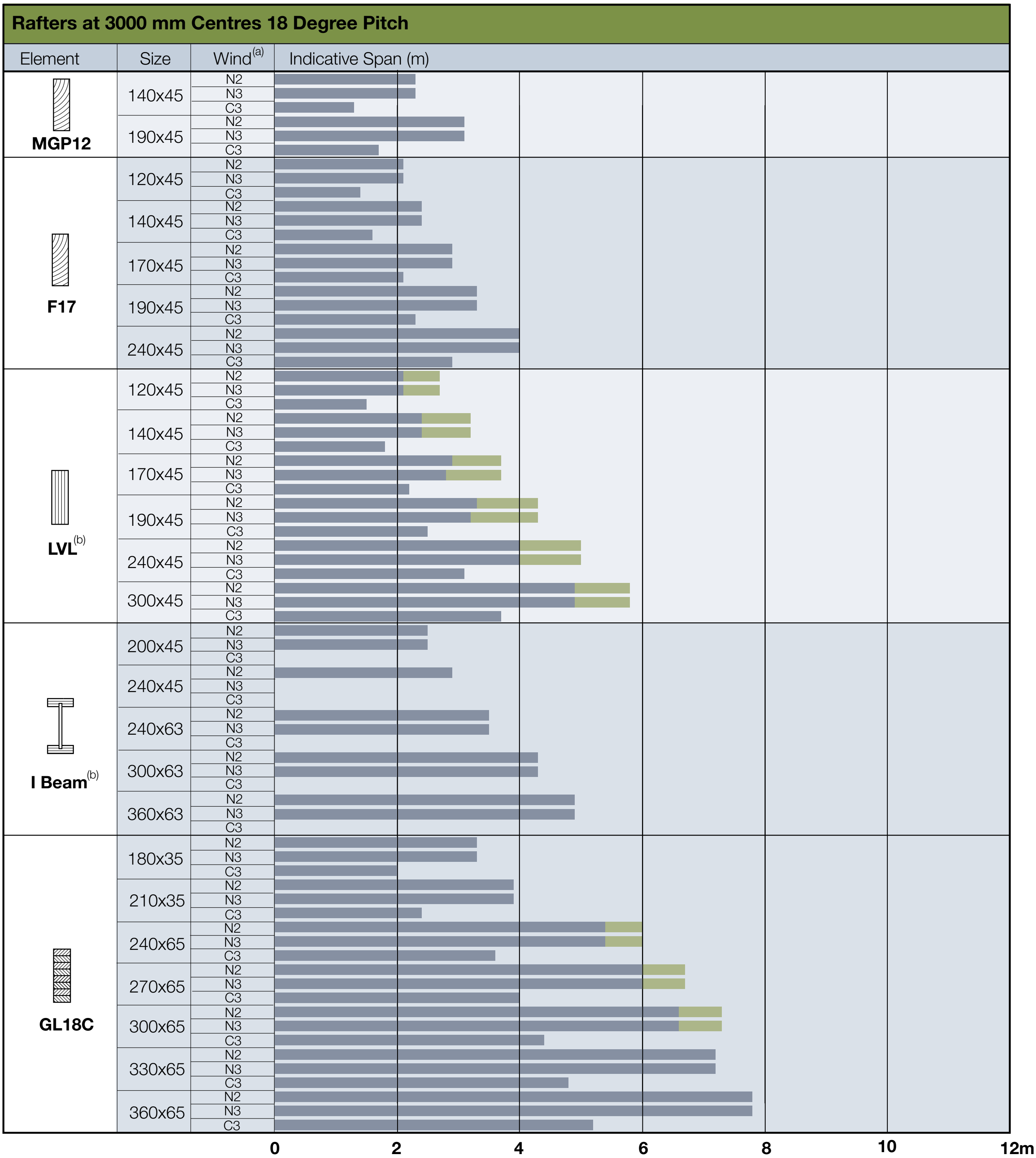
Notes:

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:

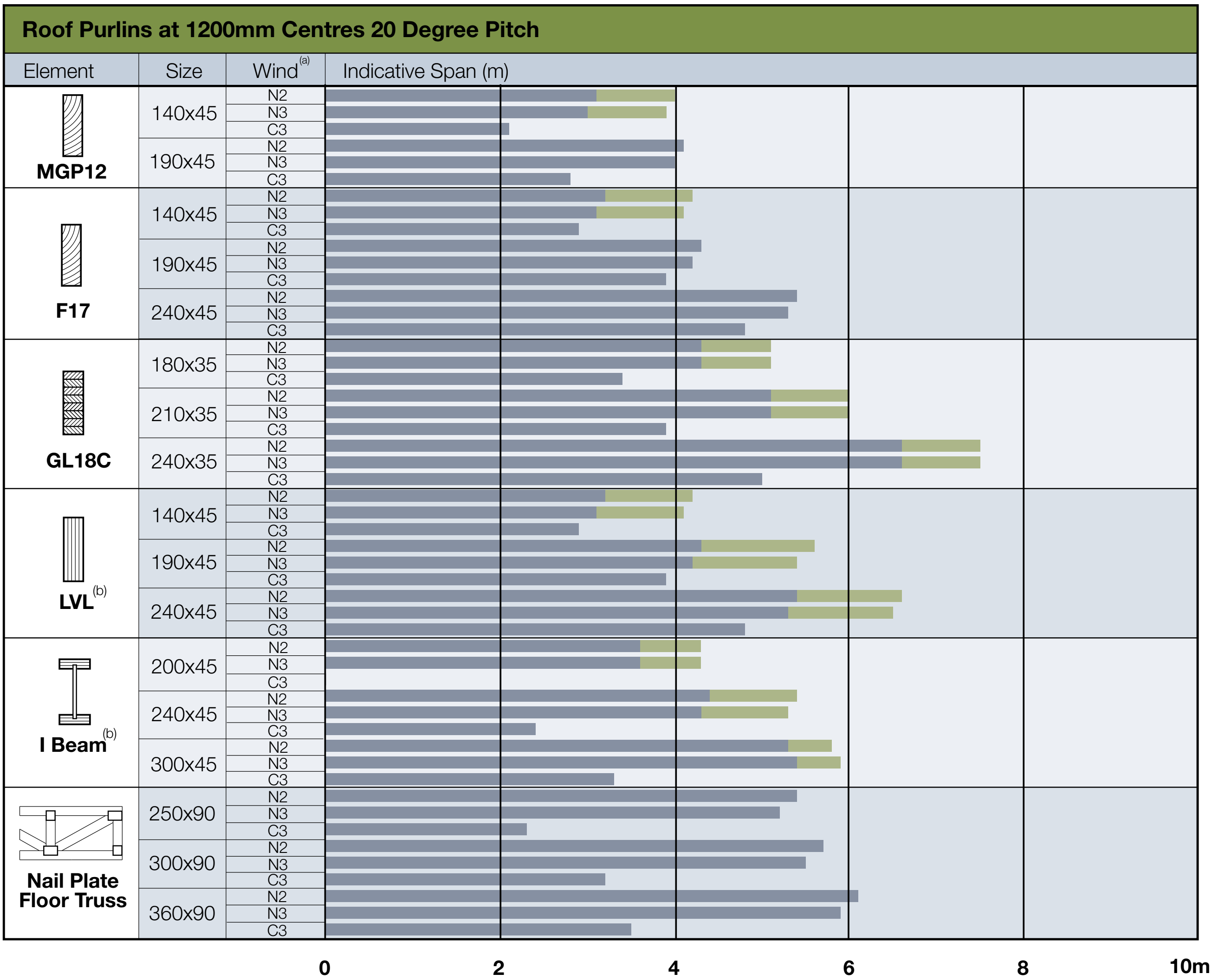
Single span   Continuous span

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 19: Indicative span – Roof purlins at 1200 centres

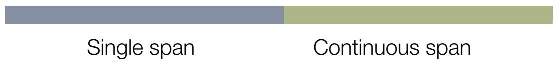


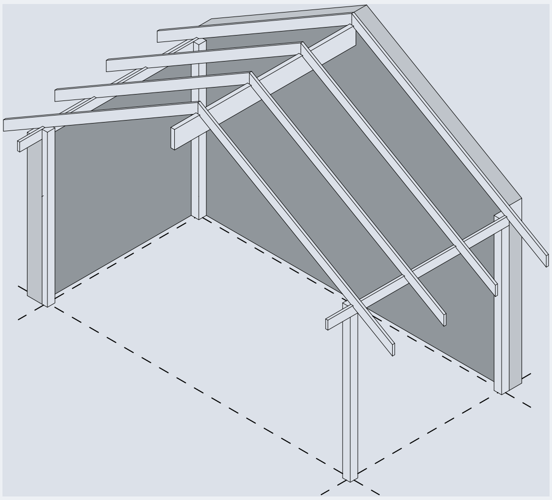
Notes:

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

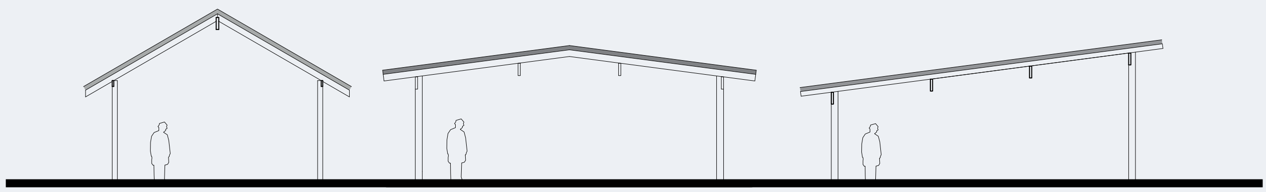




Rafter span range: Up to 8 m

Rafter span to depth ratio: 24 to 1

Form options

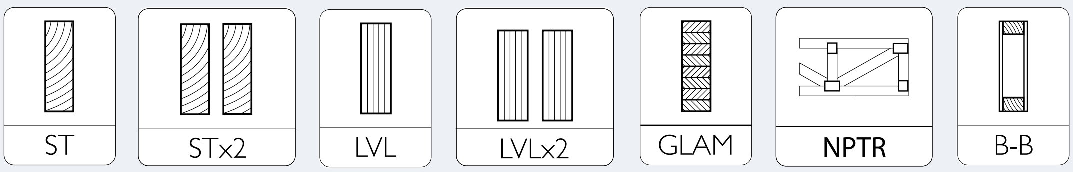


Description

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.

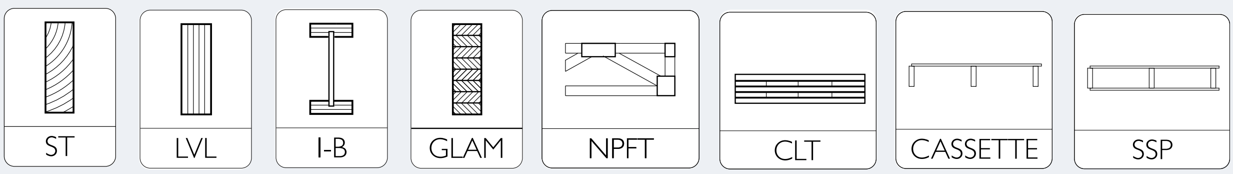
Numerous options exist for the supporting beams depending on the architectural intent, the element option selected, and the beam spacing. More limited options exist for widely spaced beams as they carry heavy loads and tie-down requirements at the supports can be considerable.

Regular beam options

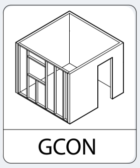


Notes
Glulam is available in wide sections while LVL and box beams do not have the depth limitations of sawn timber.

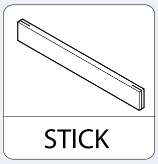
Regular rafter or roof panel options



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.

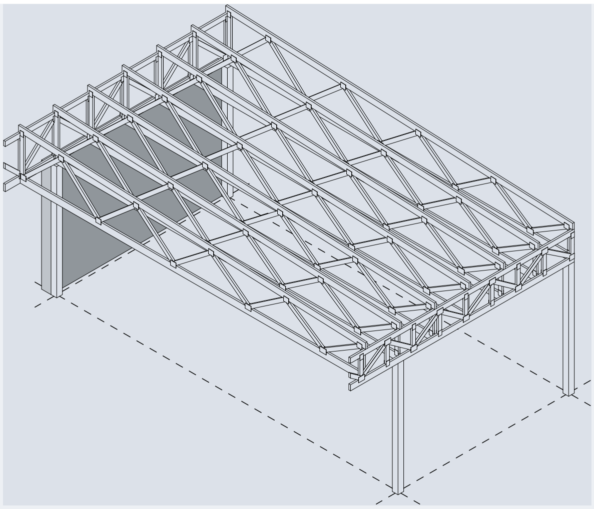
2.3.3 Roofs: Trusses

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

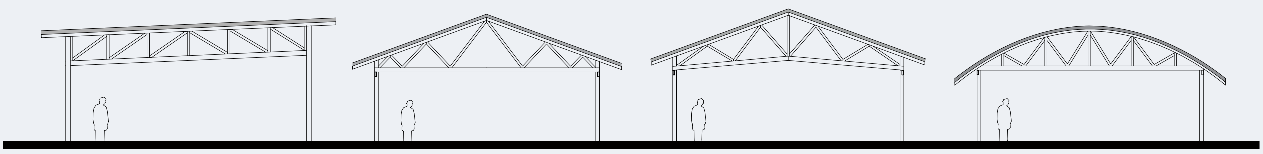


Truss span range: 25 m+

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

Truss spacing: 0.6 – 1.2 m

Form options



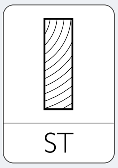
Description

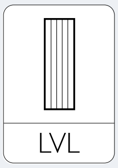
Light nailplate trusses are highly versatile components manufactured for the project by a licensed fabricator using proprietary nailplate connectors to make the truss node joint and generic timber sections for the webs and chords. The trusses and node connections are designed using proprietary software 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.

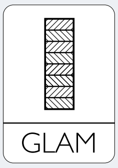
Light nailplate trusses are usually closely spaced and support battens, insulation and ceiling systems and the roofing material. To ensure economy, the trusses are often made from standard sawn timber with LVL or glulam used in high stress elements. Trusses are often 35 to 45 mm thick but can be grouped 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

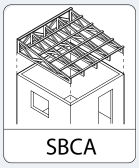


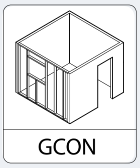




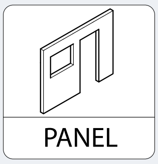
Notes
While economy drives most nailplate truss design, systems allow for the development of architectural solutions using more robust sections, painted nailplates or plywood cladding.

Regular construction and prefabrication options



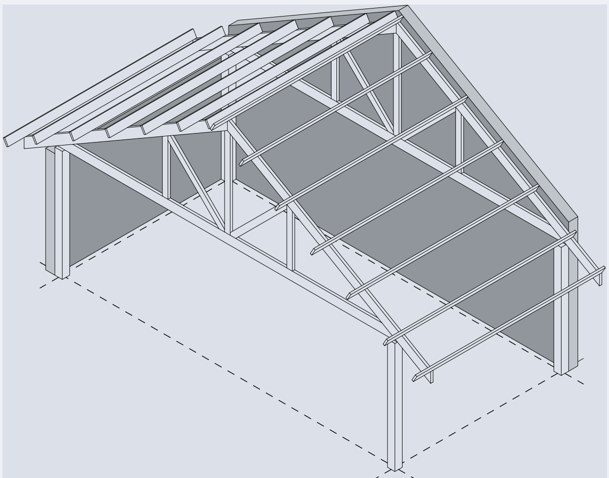


Construction notes
Systems are often assembled on site but trusses can be assembled as modules on the ground and lifted into place.



Prefabrication notes
Trusses are prefabricated as panels.

2.3.5 Roofs: Heavy Trusses

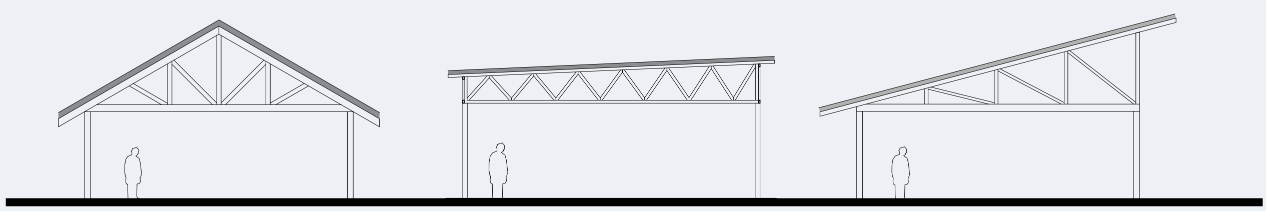


Truss span range: 30 m+

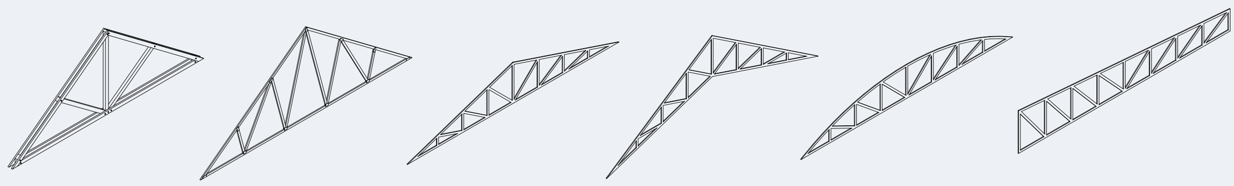
Truss span to depth ratio: 10 to 1 for parallel chord trusses

Truss spacing: 2.4 – 6.0 m

Form options



Additional configuration options

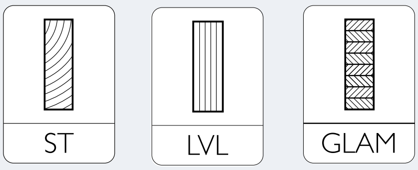


Description

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.

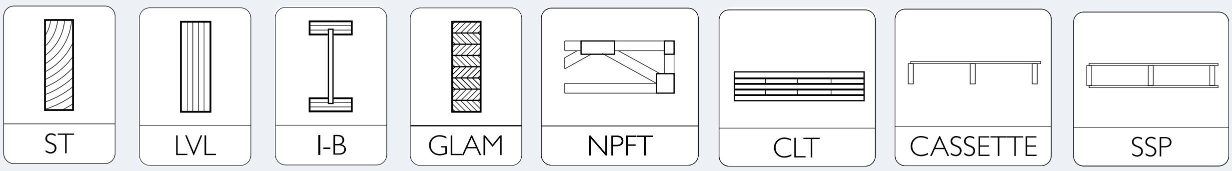
Heavy trusses are made from larger timber elements, which may be sawn timber or, more regularly, glulam. LVL is less common as it is not generally available in appearance grades. Members are most often joined at the nodes with bolted or dowel connections, often using steel gusset or fin plates. Plywood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

Regular truss chord and web options

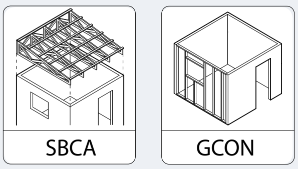


Notes
Architectural considerations can often influence the grade and size of the selected truss material.

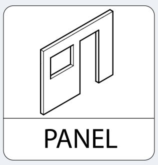
Regular purlin or roof panel options



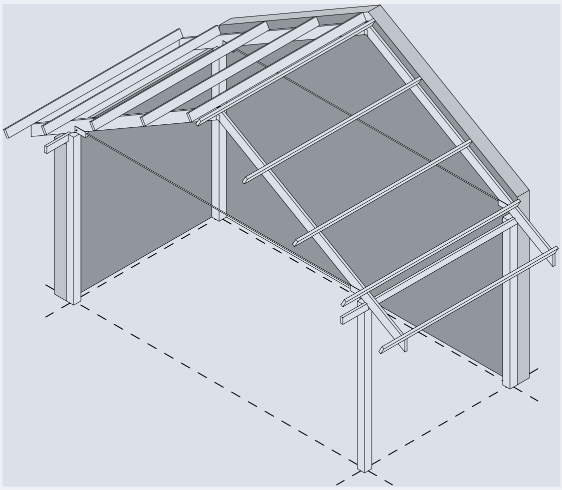
Regular construction and prefabrication options



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



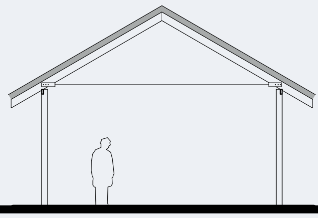
Prefabrication notes
Trusses are prefabricated as panels. Purlins can be optimised for single or multiple spans.



Frame span range: 20 m+

Frame spacing: 2.4 – 4.8 m

Form options

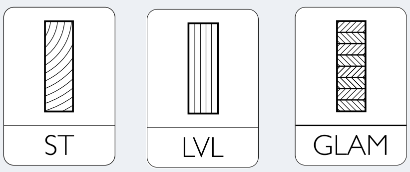


Description

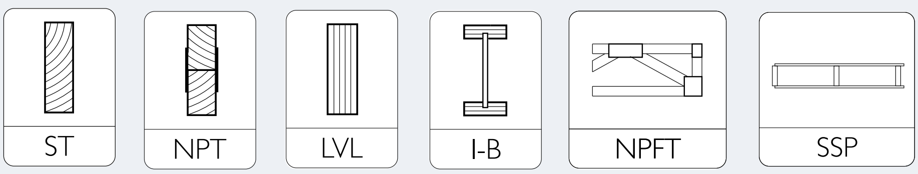
Tied frames are a simplified truss form that spans between walls or columns. Inclined rafters form the component's top chords and steel or timber tension members tie the base of the rafters together to resist horizontal loads and eliminate the need for a beam at the ridge.

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.

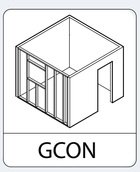
Regular rafter options



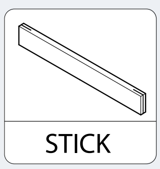
Regular purlin options



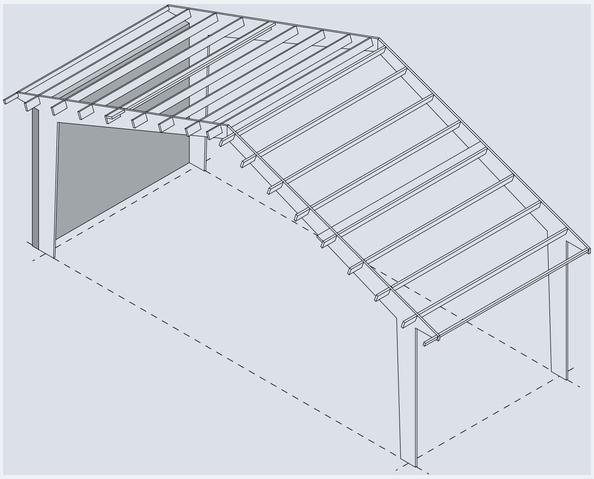
Regular construction and prefabrication options



Construction notes
Systems are generally assembled on site.



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

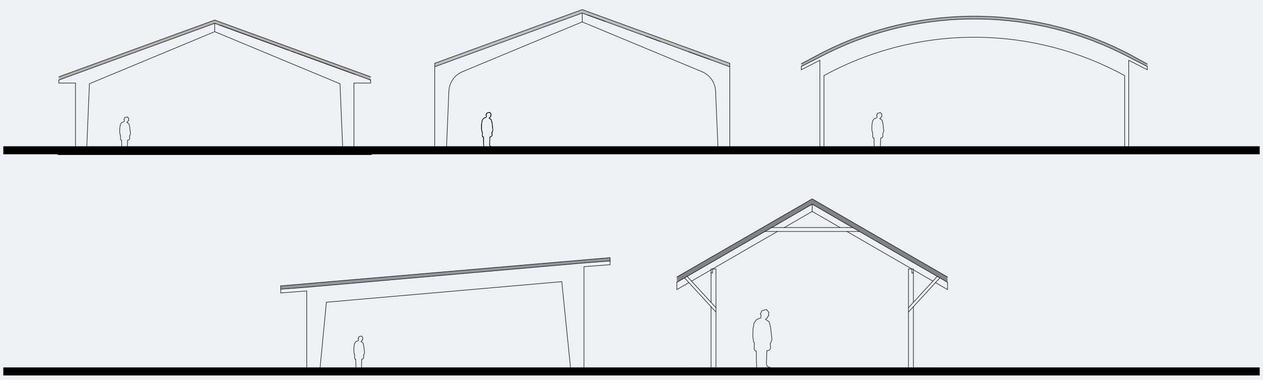


Truss span range: 30 m+

Frame spacing: 3.0 – 4.8 m

Frame span to rafter depth ratio: 35 to 1

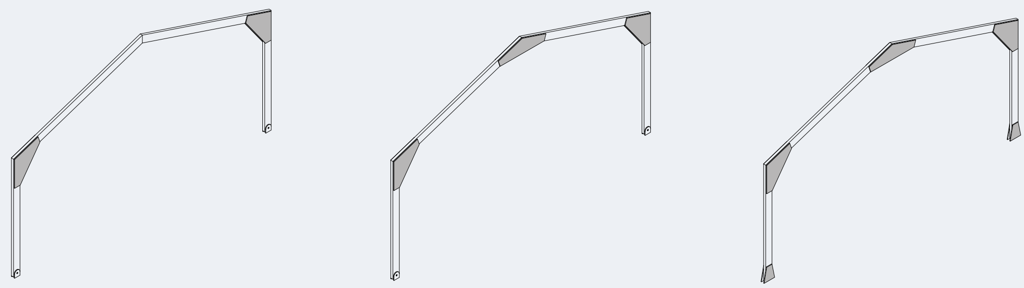
Form options



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.

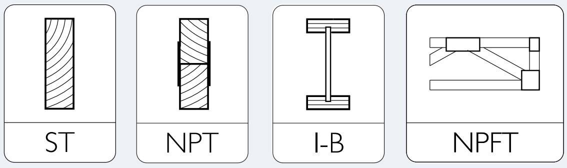
Portals can be 3-pin, 2-pin or rigid frames. 3-pin frames are the most common form for timber portals and have moment-resisting column to rafter connections but pinned connections at the bases and apex. 2-pin frames are where the column to rafter and the apex connections are moment-resisting and bases 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.



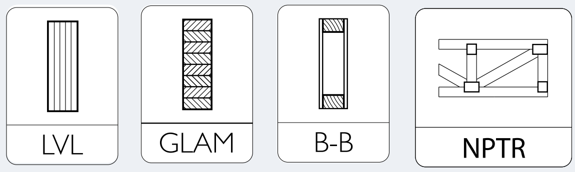
3-pin frames ***2-pin frames*** ***Rigid frames***

Regular column and rafter options by span

<6 m

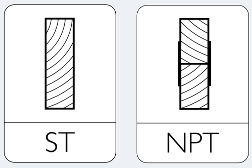


6 m+

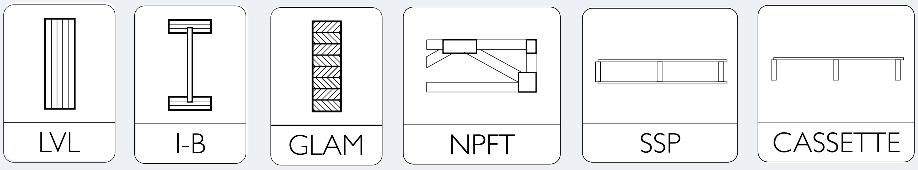


Regular purlin options by span

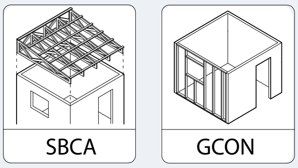
<3.6 m



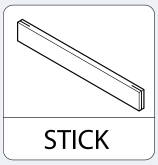
3.6 m+



Regular construction and prefabrication options

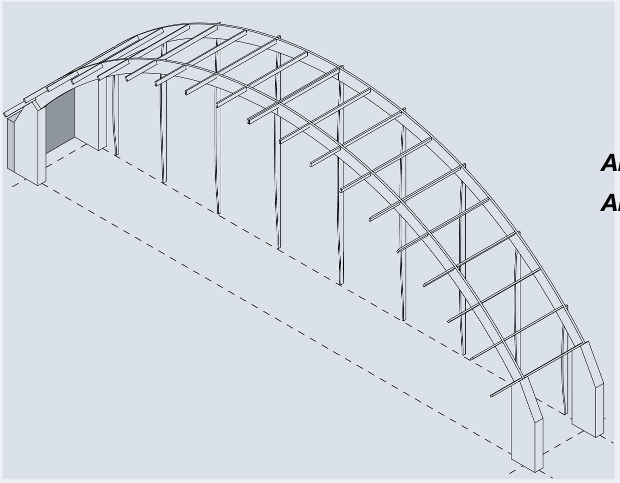


Construction notes
While systems are often assembled on site, rafter sets can be assembled with purlins and roof sheet as modules on the ground and lifted into place.



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

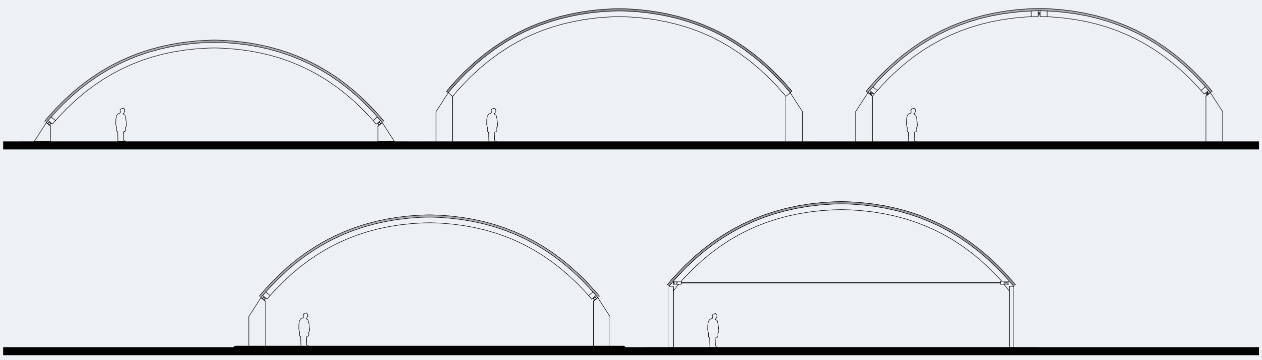
See WoodSolutions Design Guide 33: Quick Connect Moment Connection



Arch span range: 50 m+

Arch span to arch depth ratio: 40 to 1

Form options

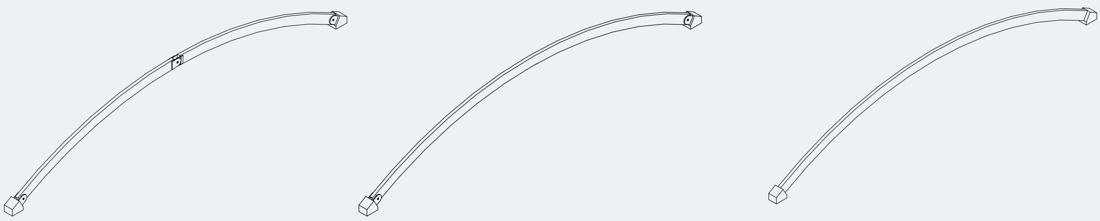


Description

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.

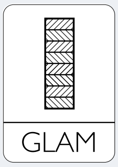
The compressive forces in an arch have to be resisted at the reaction points. This is easiest in foundations at ground level but this reduces the usable head height near the arch's base. Arches raised above the ground require large buttress foundations to resist the thrust or need to include a tension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

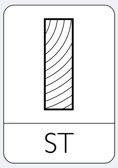
Regular arch options

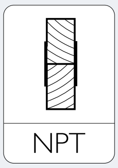


Notes
Glulam fabrication capacity can limit effective depth and length.

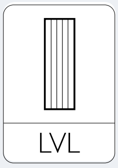
Regular purlin options by span

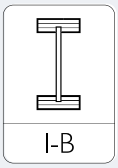
<3.6 m

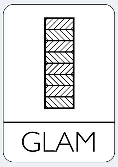


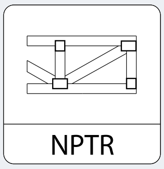


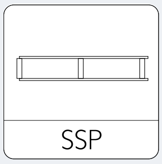
3.6 m+

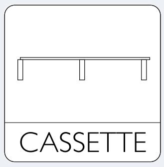




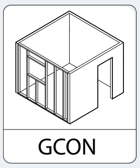




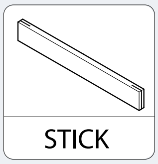




Regular construction and prefabrication options

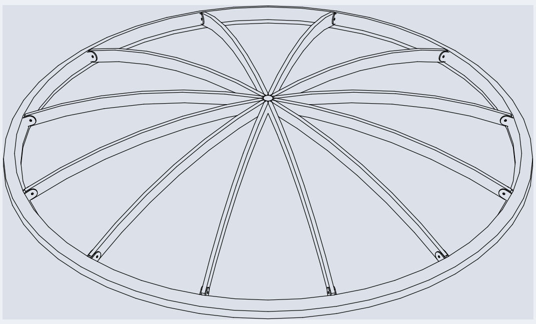


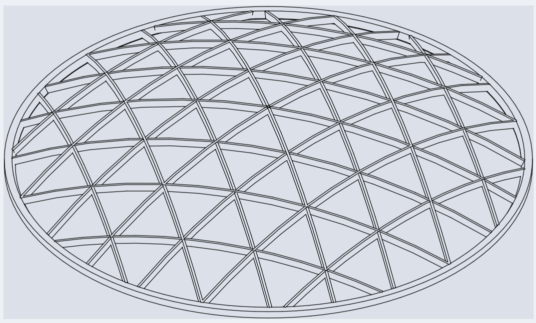
Construction notes
Systems are often assembled on site from prefabricated components.



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

2.3.9 Roofs: Dome



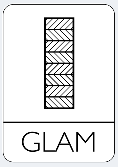


Beam span range: 50 m+

Description

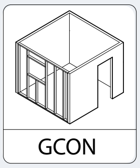
Many domes are essentially a 3-dimensional arrangement of arches that carry loads to a circular foundation. As with arches, domes act primarily in compression with minimal bending stresses in the members but create significant thrust at the foundation, typically resisted by a foundation tension ring 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

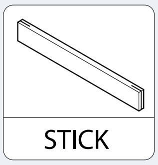


Notes
Glulam fabrication capacity can limit effective depth and length.

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.

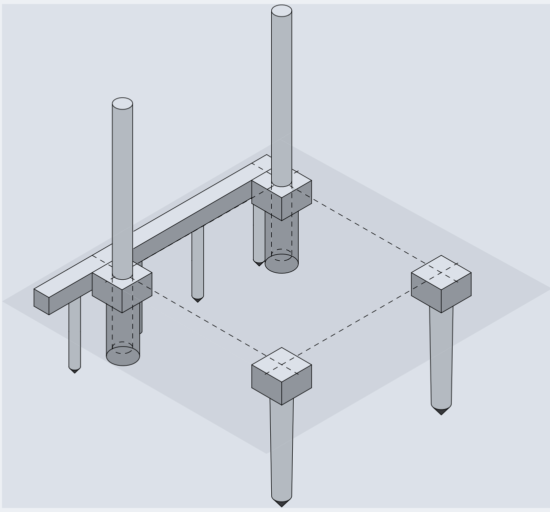
Generally, the critical factors considered when selecting a support system are the elements' required load and buckling resistance and other performance requirements such as fire resistance and acoustic separation. These are discussed in Sections 9.4 and 9.6.

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.

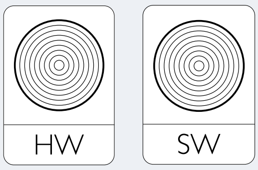
The spacing of the support lines that the walls and columns provide is an important consideration in the selection of the primary spanning and support systems. It affects the spanning members' required span and design, the sizing of support system elements, and often the functional efficiency of the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



Description

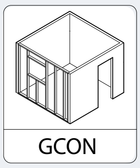
Treated and natural timber rounds used as poles and piles serve a range of foundation and support functions. Poles can be set into the ground to form cantilevered columns. Timber pile systems generally use driven treated hardwood rounds to transfer loads from the building structure, through a concrete 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

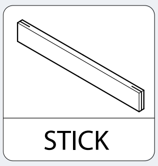


Construction notes
Treatment only effectively penetrates the sapwood. See Section 8.5.2

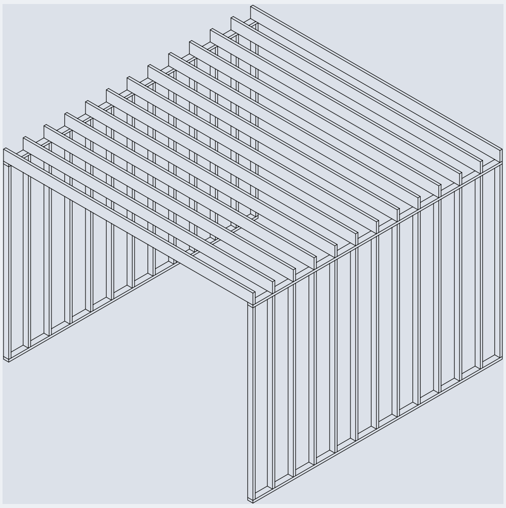
Regular construction and prefabrication options



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

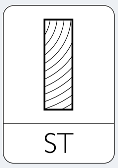
Timber stud frame walls form the primary support system for the majority of timber-framed building. Stud frames are highly versatile and economic components assembled from closely spaced vertical timber studs fixed between horizontal timber plates. The studs share the load applied to the wall between 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.

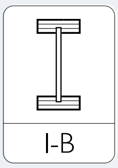
Timber stud frame walls are simple to construct and widely prefabricated. They are easy to handle and transport and can be installed without special equipment. While stud framing for residential construction is normally 70 or 90 mm thick, wider sawn timber or engineered wood products (EWPs) can be used 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.

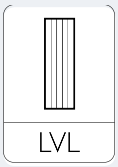
Most modern frame construction is platform construction, where the floor forms a platform on top of the wall frames. The next level of walls is then built off this platform. However, wall framings can be continuous through multiple storeys, with the floor plate hung from the side of the walls. This approach 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

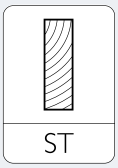
Studs

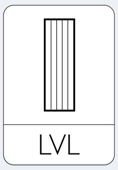


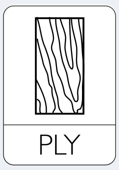




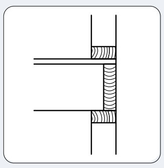
Plates

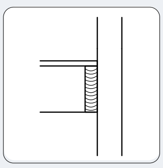






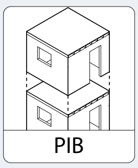
Assembly options

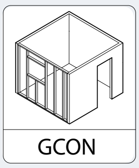




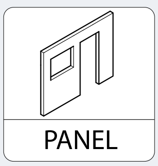
Assembly notes
In platform construction, the floor platform sits on the walls. In balloon construction, the studs run through with the floor supported from a ledger.

Regular construction and prefabrication options

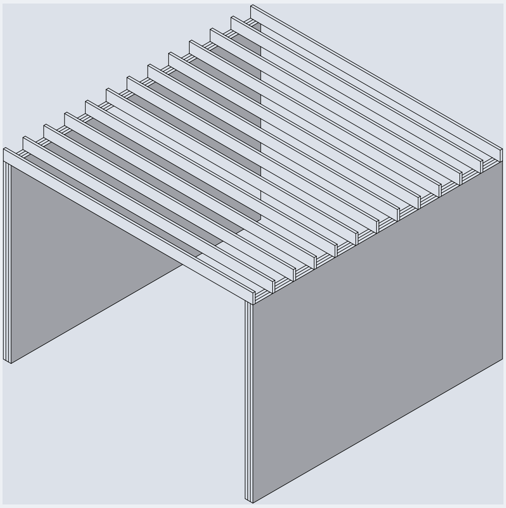




Construction notes
Wall framing can be fully prefabricated or built on site from generic products.



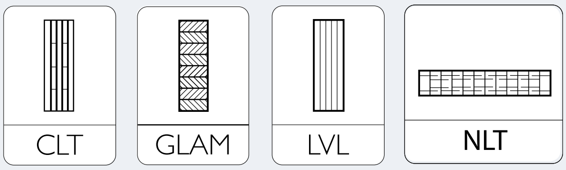
Prefabrication notes
Braced wall frames with or without cladding, lining and insulation can be prefabricated as panels.



Description

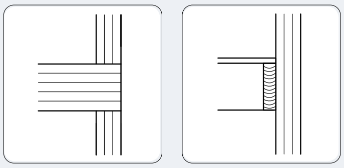
Massive timber walls are solid timber panels that carry forces from the spanning system to the foundation and provide lateral rigidity. Massive timber walls can be constructed from CLT, glulam and, occasionally, LVL. Massive timber walls have similar benefits to massive timber floors as panels are prefabricated 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



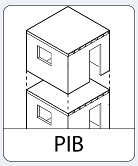
Notes
Systems may combine several materials to provide an efficient solution. For example, glulam and LVL may be used with CLT to increase the load bearing capacity of lintels.

Assembly options

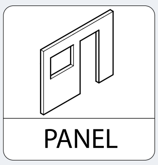


Assembly notes
CLT floor plates regularly sit on the wall frames or panel. However, panels can run through with the floor supported from a ledger.

Regular construction and prefabrication options

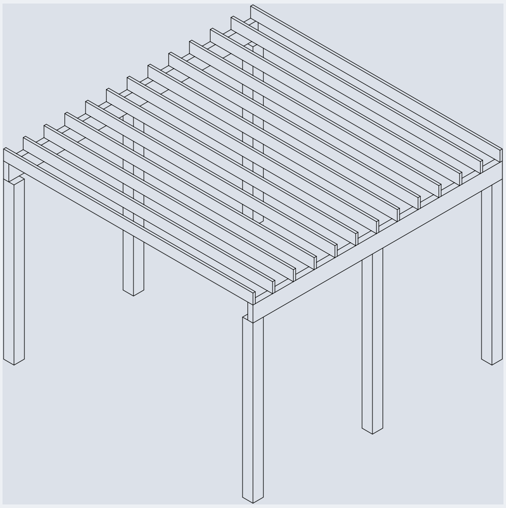


Construction notes
Mass timber walls are installed as prefabricated panels.



Prefabrication notes
Mass timber wall are generally fully prefabricated.

2.5.4 Support systems: columns



Description

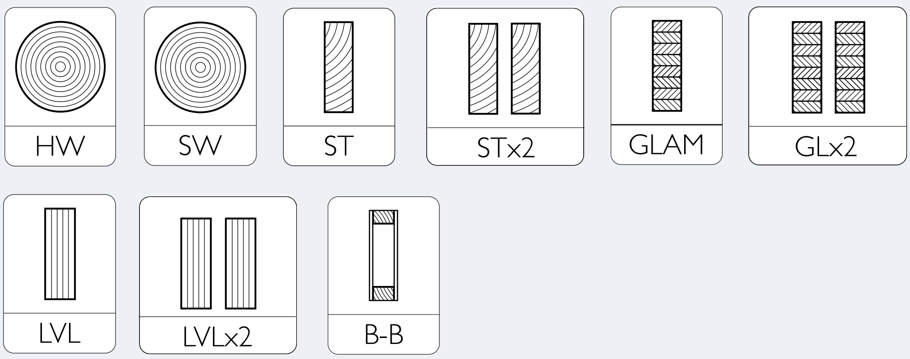
Columns transfer concentrated loads from spanning elements down to the foundations, ideally along a direct load path. Columns can form part of a post and beam system with the beams of the floor spanning system, or receive loads from major roof elements such as heavy trusses or modules of light nailplated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

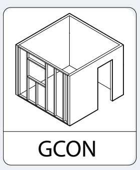
Column sizes must be designed to resist both compression and buckling. Tall slender columns may require bracing to resist buckling. Round, solid square or spaced columns are more efficient than narrow, deep rectangles, as they have equal capacity to resist buckling about both axes. An exception would 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.

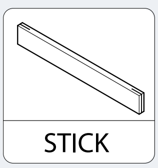
Regular column options



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

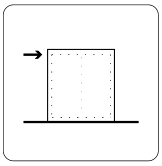
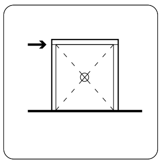
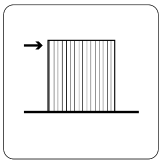
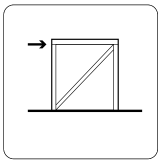
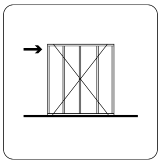
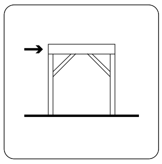
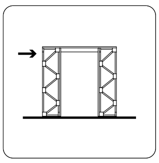
Description

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:

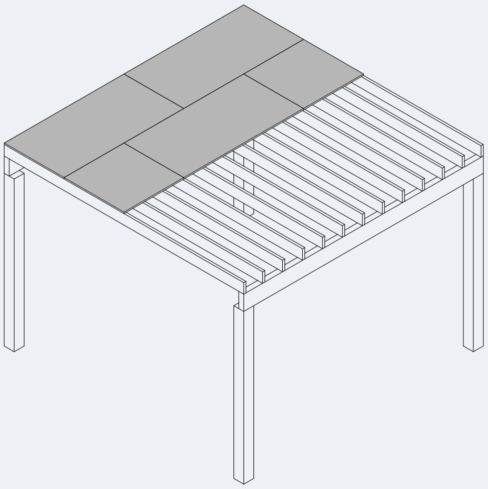
- diaphragms through the floor or roof planes or the walls
- bracing through light steel strap or folded sheet steel
- bracing through truss panels built into the wall, ceiling or roof plane
- bracing through solid timber braces or pairs of steel rod
- moment-resisting connections from columns to beams 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.

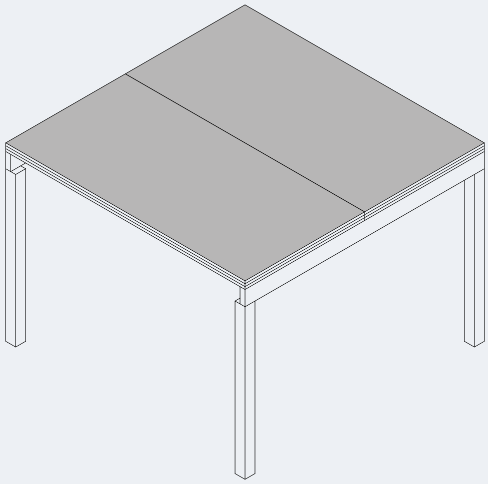
Table 20: Typical bracing systems

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.



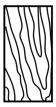
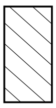
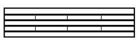
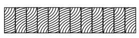
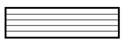
Sheet floor fixed to joists

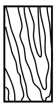
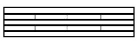
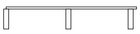
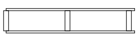


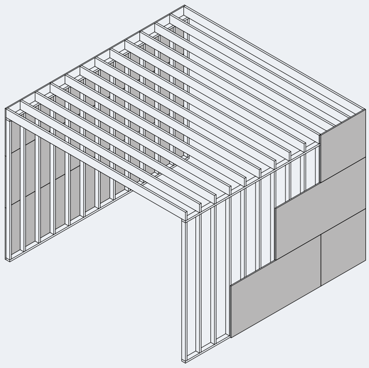
Mass timber fixed to beams

Description

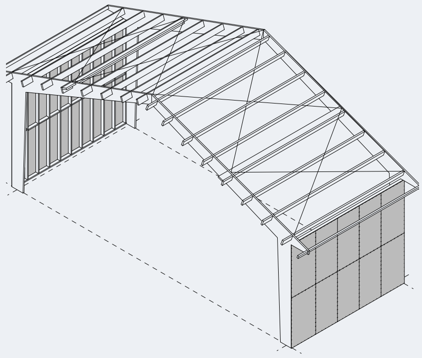
Floors often serve as diaphragms to transfer lateral loads horizontally while sheet bracing or panel systems can establish a diaphragm in the roof plane. Sheet or massive timber panel flooring can form a continuous plate to transfer loads through shear stress in the panels. Specific fixing requirements 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.

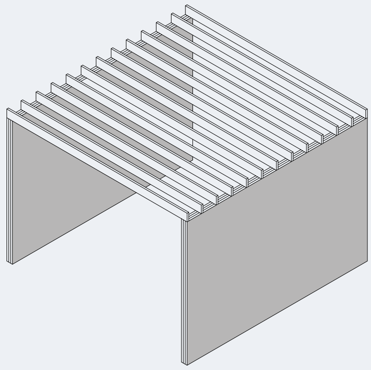
Regular roof diaphragm options					
 PLY	 OSB	 CLT	 CASSETTE	 SSP	**Notes** Specific fixing requirements may apply to ensure adequate shear transfer.



Sheet material bracing wall



Sheet material bracing wall



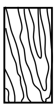
Massive timber wall

Description

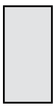
As well as carrying loads vertically to the foundation, stud frame walls clad with ply or similar sheeting material and massive timber walls can serve as bracing diaphragms or shear walls. Effectively anchoring the diaphragm to the surrounding structure or foundations is critical. This is especially 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



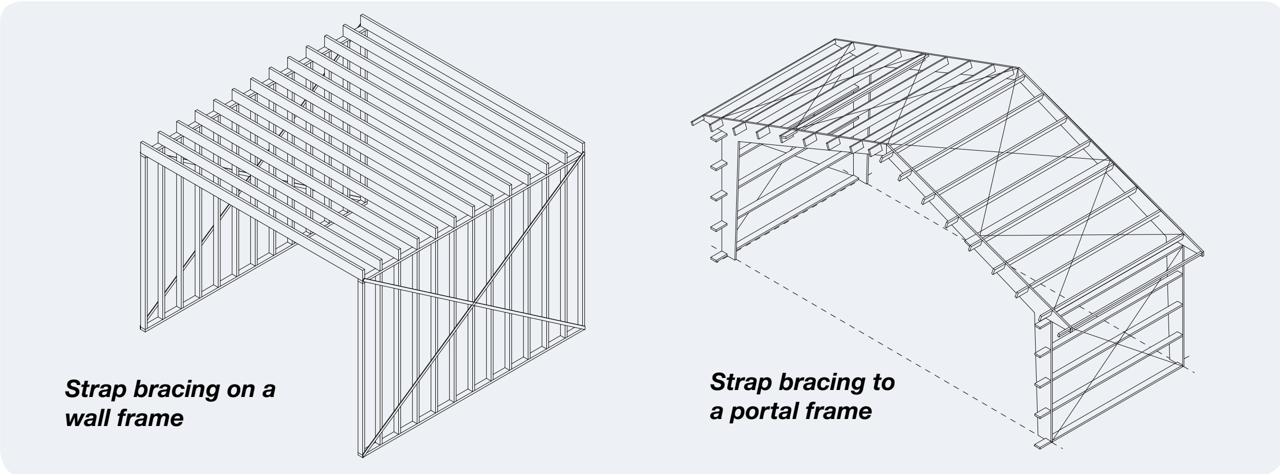
HDF

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

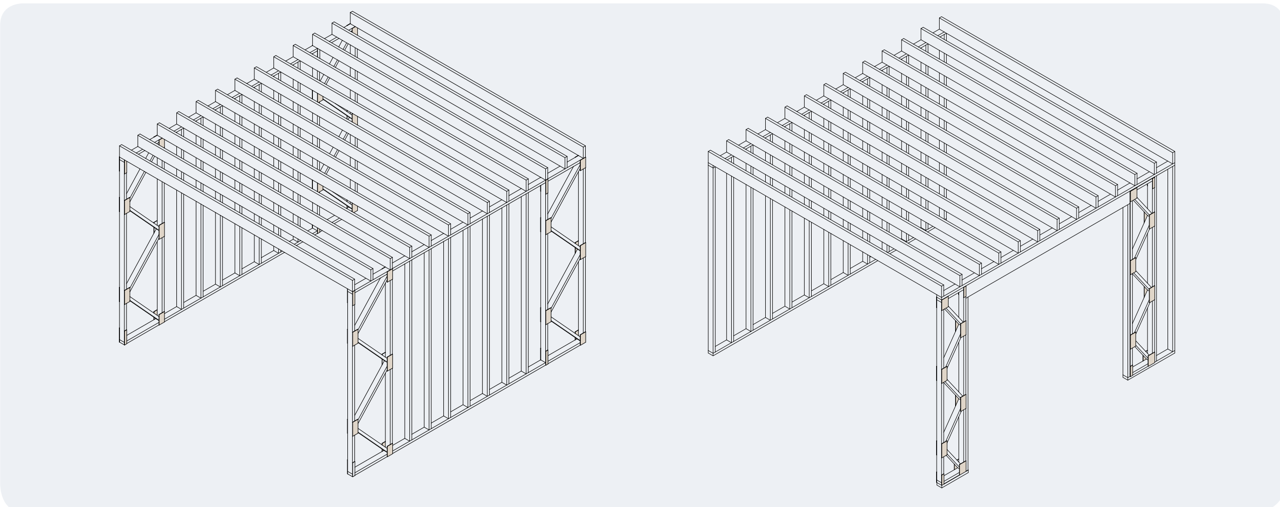


Description

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.

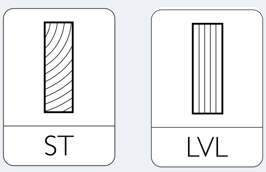
2.6.4 Lateral resistance systems: Truss panels



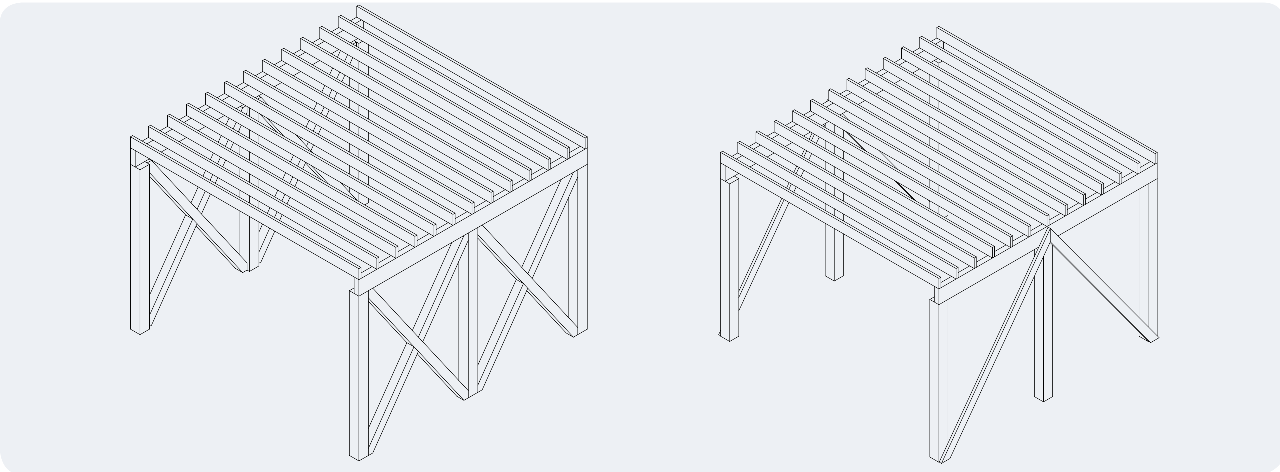
Description

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

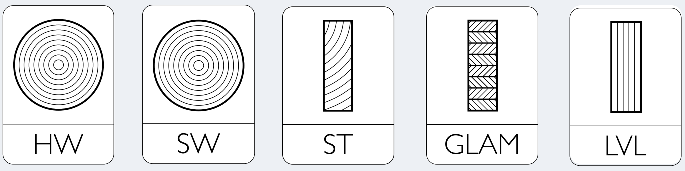


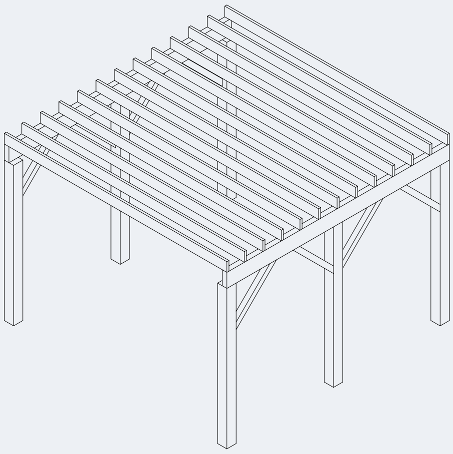
Description

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.

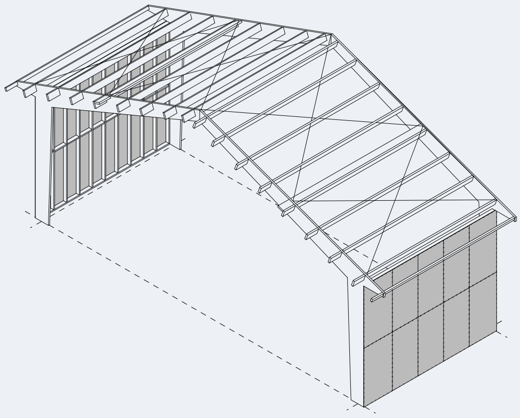
If the bracing only needs to act in tension, steel rods or cables are often used, especially for horizontal bracing in roofs. In these cases, they will be installed as cross-brace pairs to resist lateral loads coming from either direction, with only one half active under a given load condition.

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.

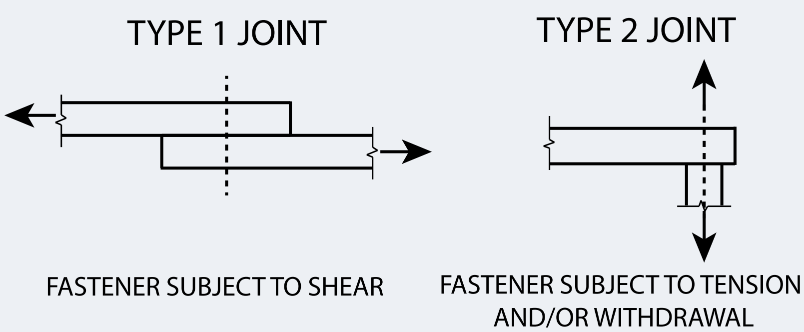


Figure 2: Joint type and direction of forces

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.

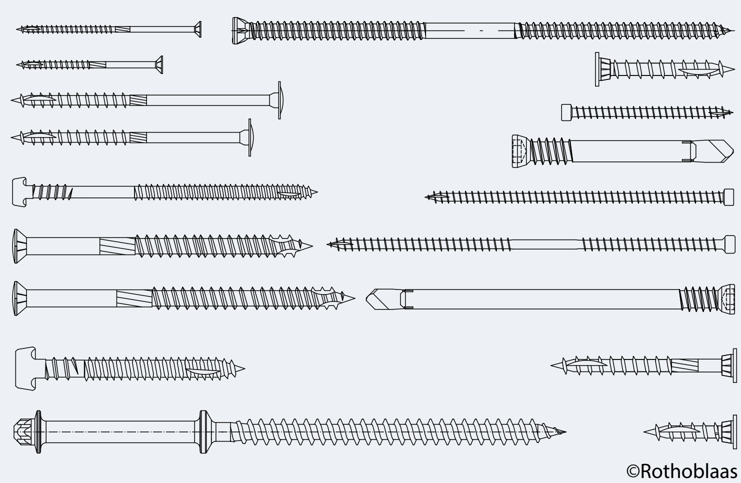


Figure 3: Variety of screw configurations (Courtesy of Rothoblaas SRL)

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.

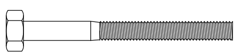
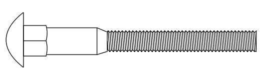
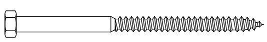
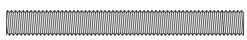
Under shear load, the bolt's shank presses against the timber's end-grain in the pre-drilled holes. In high loads, this can lead to localised crushing. As using bolts involves drilling holes in the wood, the timber's effective cross-section is reduced, impacting the member's strength at the connection. 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

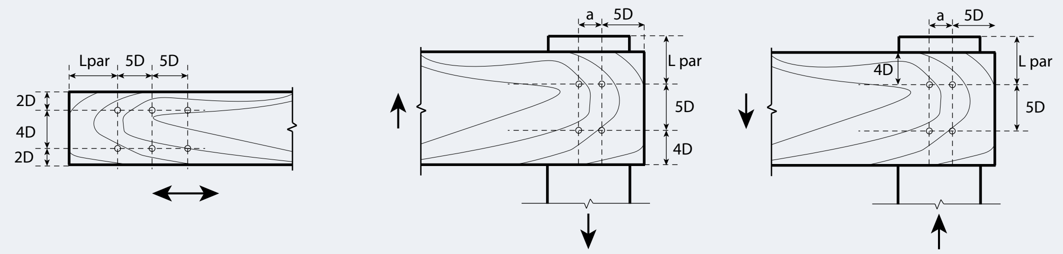


Figure 4: Bolt spacing to AS 1720.1 (Source: AS 1720.1 Fig 4.9 Bolt spacing)

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.

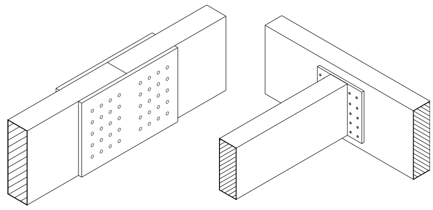
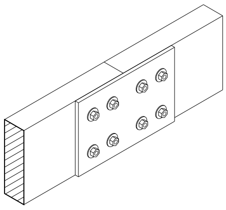
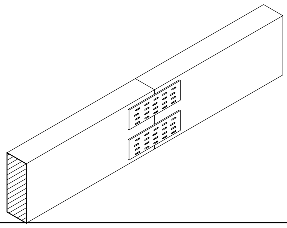
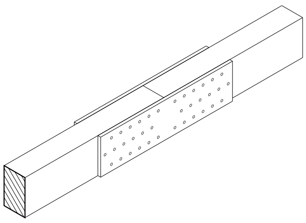
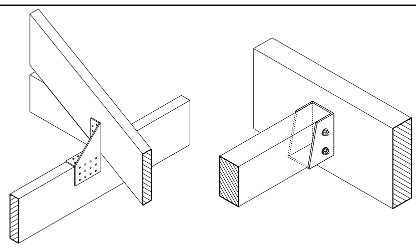
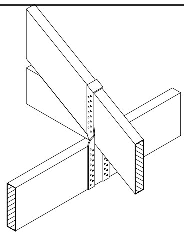
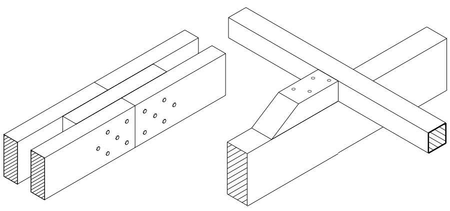
Notes:

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

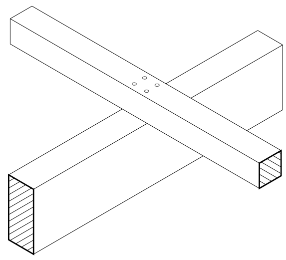
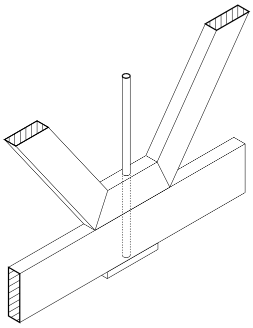
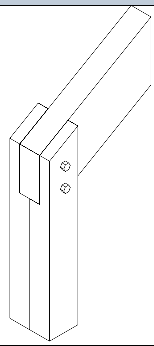
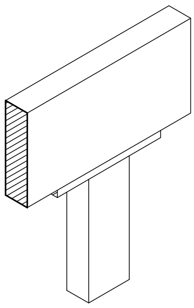
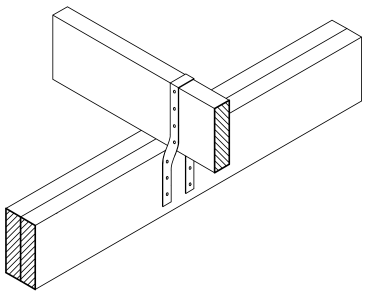
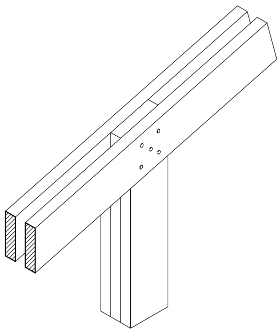
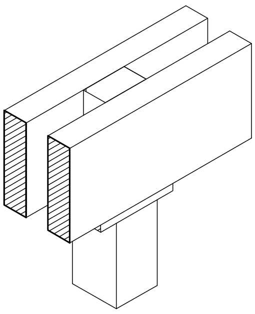
Illustration	Description
	**Wood gussets** Wood gussets are usually sections of plywood or occasionally LVL used with fasteners to join timber elements. Gussets can be applied parallel to the grain or on the end grain of one element and the face of another.
	**Metal gusset plates** Metal gusset plates 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.

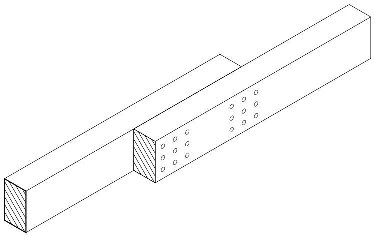
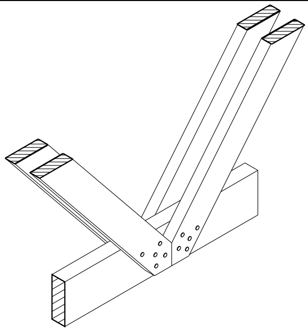
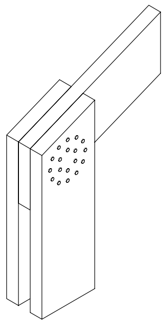
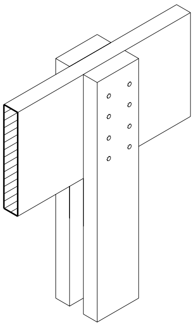
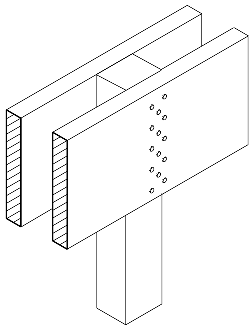
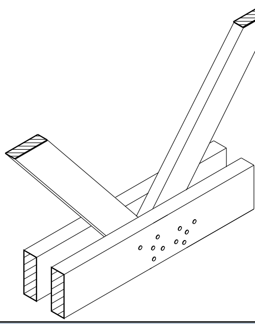
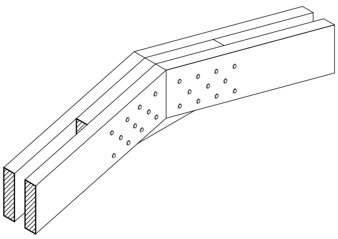
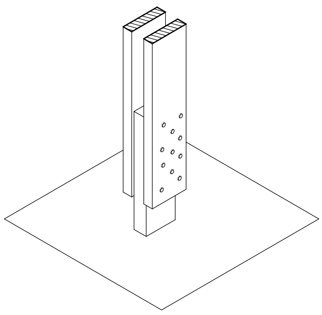
3.4.1 Contact transfer

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**	
		

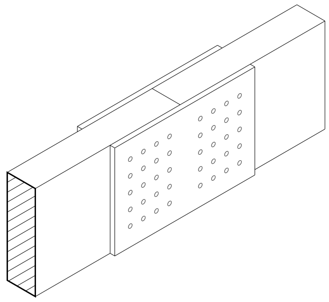
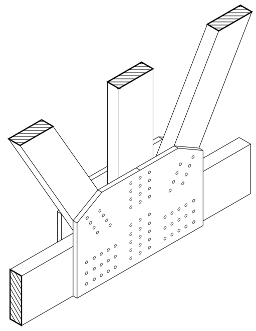
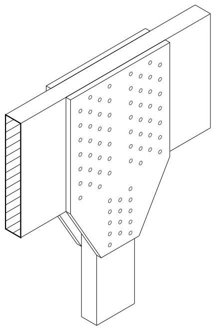
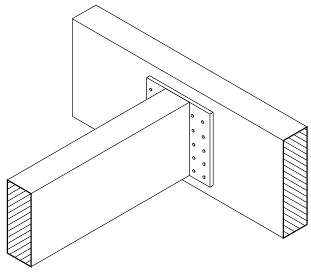
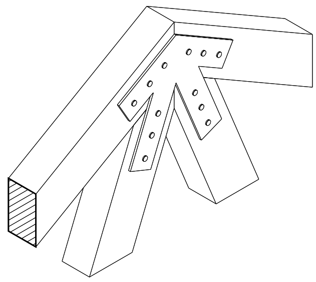
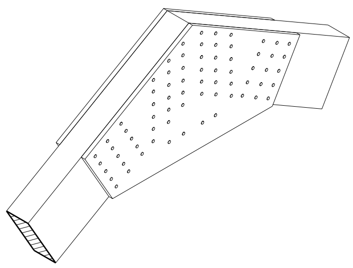
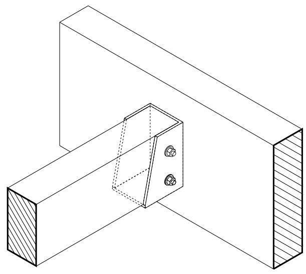
Description

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**	
		

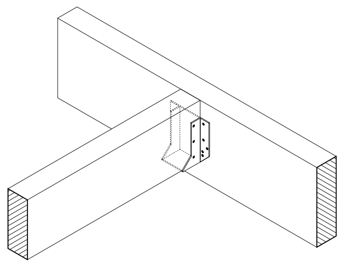
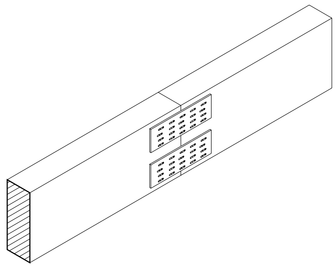
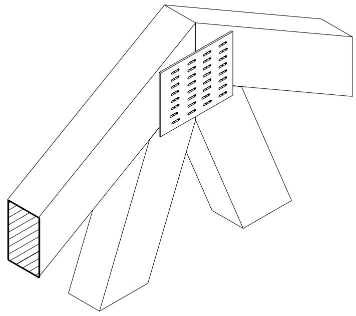
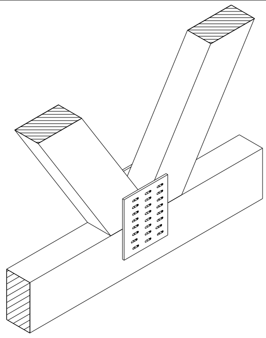
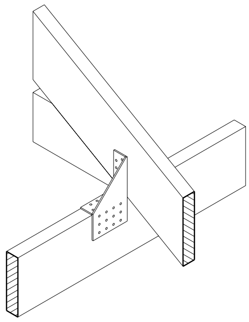
Description

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.

3.4.4 Nail and nail-on plates

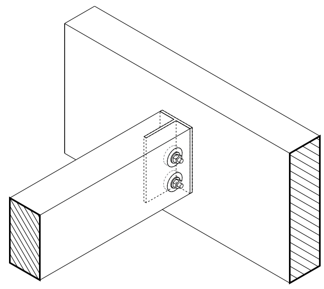
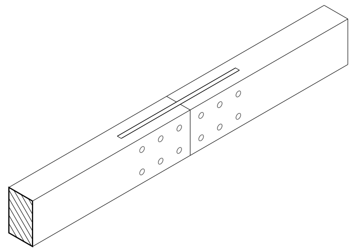
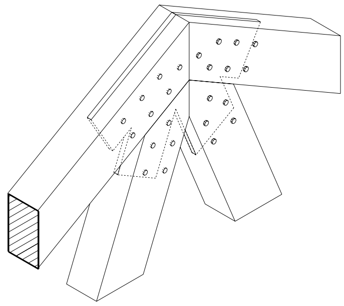
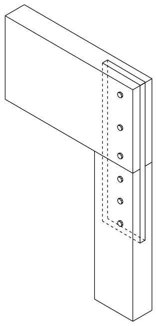
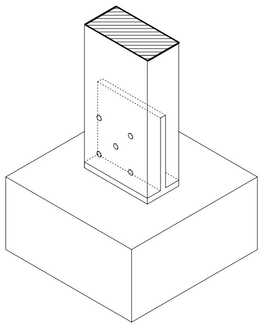
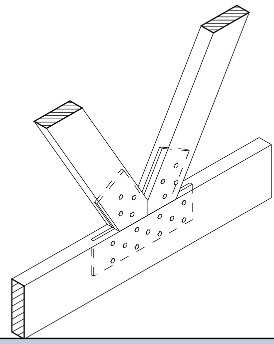
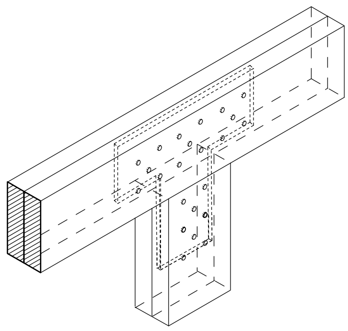
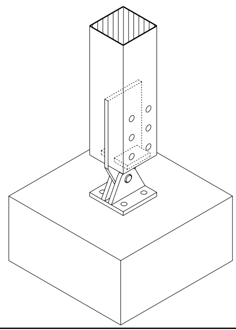
Joist to beam	In-line join	Truss node
		
Truss node	**Truss to beam**	**Post to the ground**
		

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.

3.4.5 Fin plates with fasteners

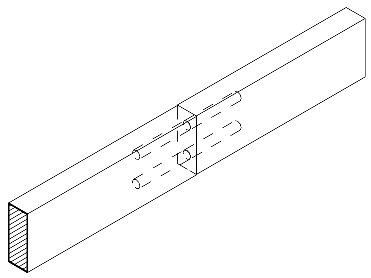
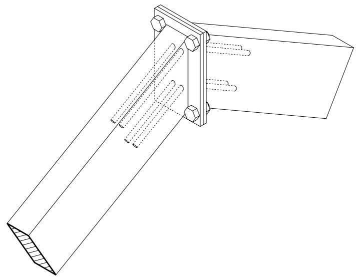
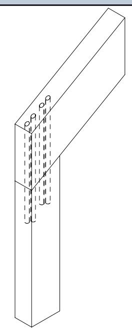
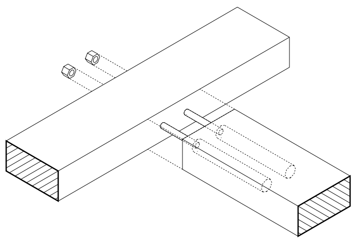
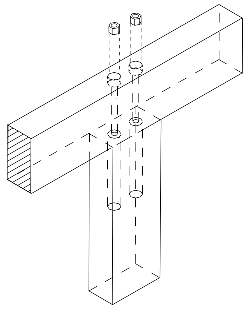
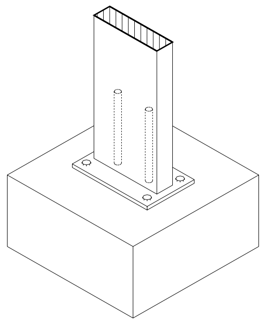
Rafter to beam	In-line join	Truss node
		
Beam to post	**Column to the ground**	**Truss node**
		
Beam to post	**Column to the ground**	
		

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

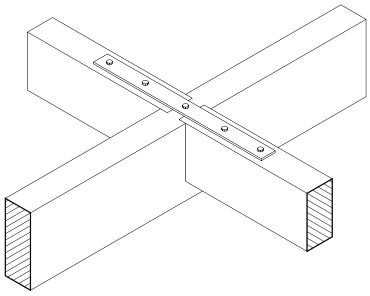
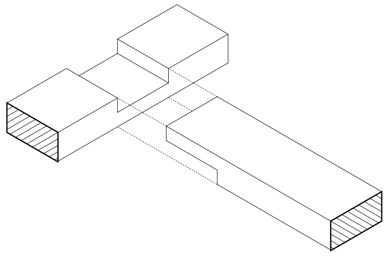
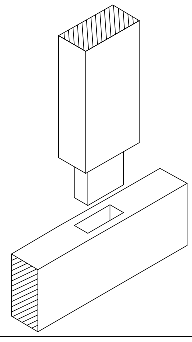
Rafter to beam	In-line join	Truss node
		
Beam to post	**Column to the ground**	**Truss node**
		

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.

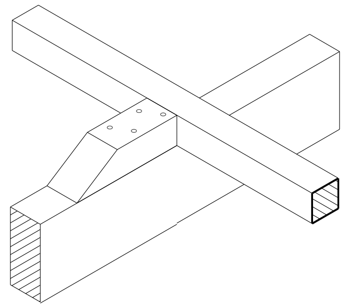
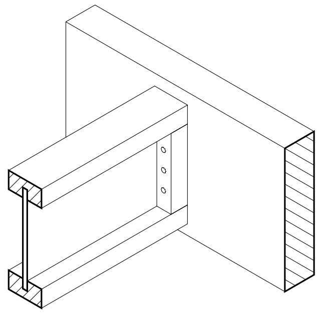
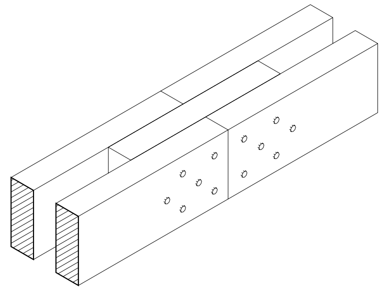
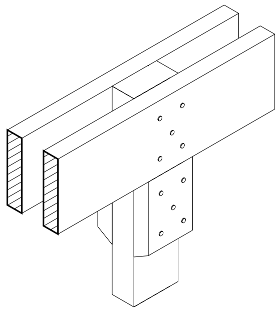
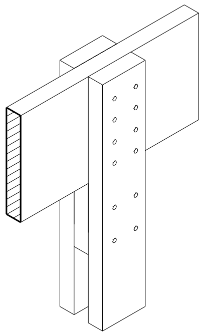
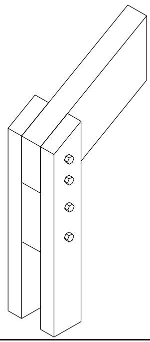
Epoxy dowel connections can form tension, compression and moment-resisting joints and allow timber elements to be arranged and joined in the same plane. The connection relies on the glue forming a full bond between the rod and the timber so workshop assembly is essential.

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	$$$	✗
Interlocking housing					
Various carpentry connections	✓	✓	✓ unless site made by specialists	$$$	✗

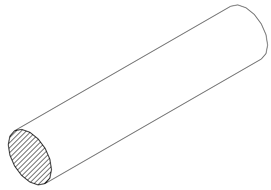
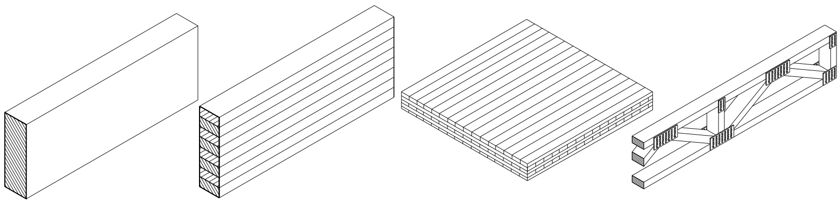
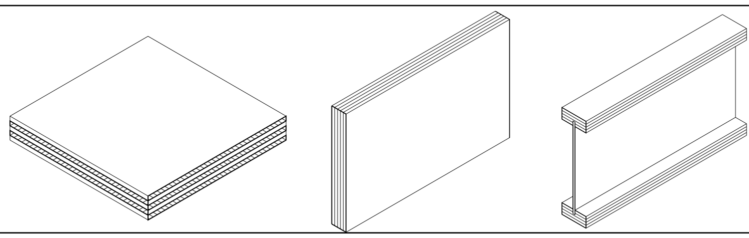
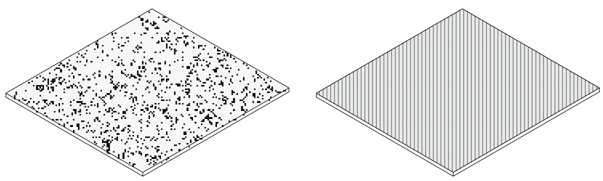
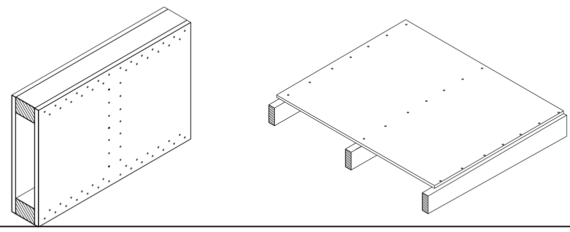
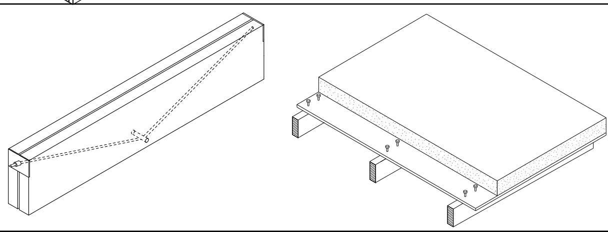
Notes:
Estimated price ranges take into account material and labour costs: $ = low cost, $$ = medium cost, $$$ = high cost

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.

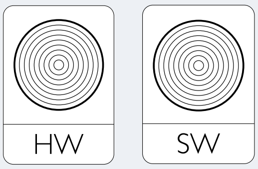
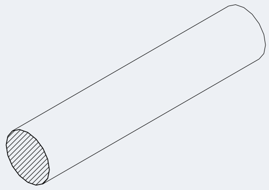
Table 28: Element groups and sections in which they are included

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	

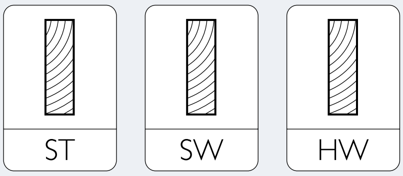
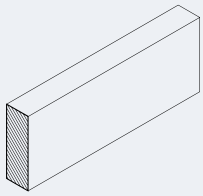
General element supply varies with the types of element. Three general supply options are common:

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

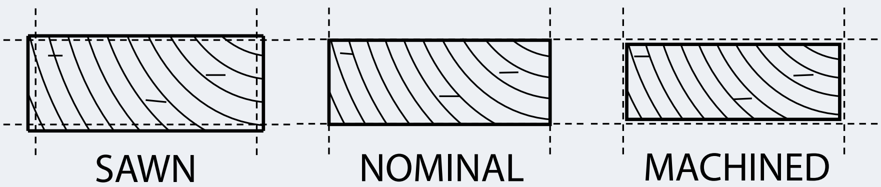


Figure 5: Sawn timber sizes: sawn, nominal and machined.

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

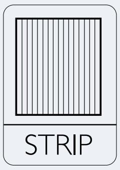
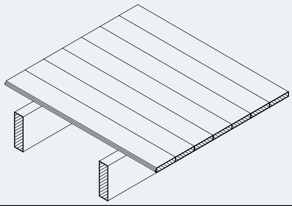
Width	Depth									
mm	50	75	100	125	150	175	200	225	250	300
25	■	■	■	□	■	□				
38	■	■	■	■	■	■	■	■	■	■
50	■	■	■	■	■	□	■	■	■	■
75		◘	■	◘	◘	◘	◘	◘	◘	◘

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

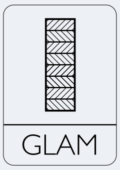
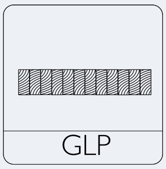
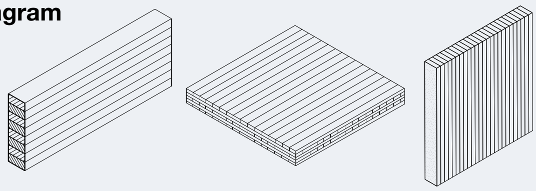
Glue laminated timber	
Icon  	**Diagram** 
Description	Glue laminated (GLAM) timber is sections of sawn timber glued together to form larger, more structurally reliable timber elements. Lamination increases the size, structural capacity and reliability of elements and produces a wide range of beams, 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 ☐

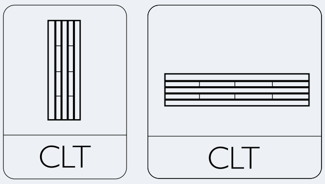
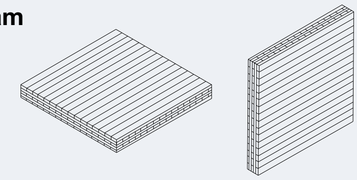
Notes: Hardwood glulam to standard machined, seasoned hardwood section sizes is also available.

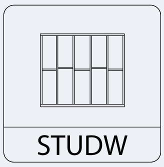
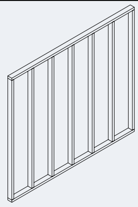
Table 35: Typical glulam sections – GL17 Softwood

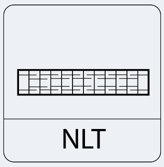
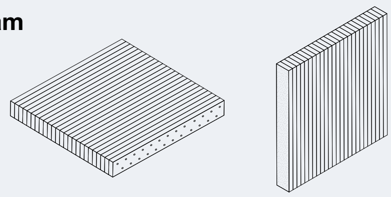
Width	Depth													
mm	**130**	**165**	**195**	**230**	**260**	**295**	**330**	**360**	**395**	**425**	**460**	**495**	**525**	**560+**
65	■	■	■	■	■	■	■	■	■	■	■	■	■	■
85	■	■	■	■	■	■	■	■	■	■	■	■	■	■
130	■	■	■	■	■	■	■	■	■	■	■	■	■	■

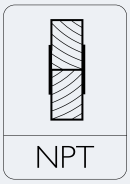
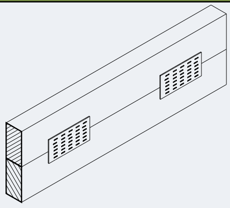
Legend: Commonly available ■; Available on order ◘; In limited supply ☐

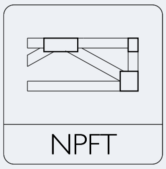
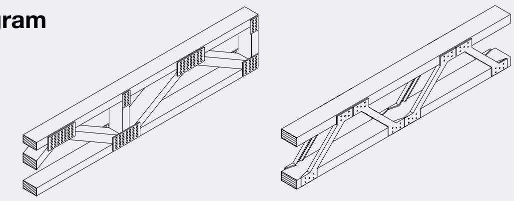
Note: GL21 is available up to 300 x 300 mm, GL18 up to 450 x 450 mm and GL17, GL13 and GL12 up to 600 x 600 mm. Larger sections up to 1200 mm deep are available on order.

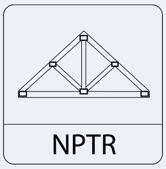
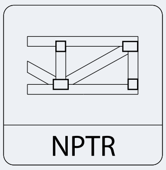
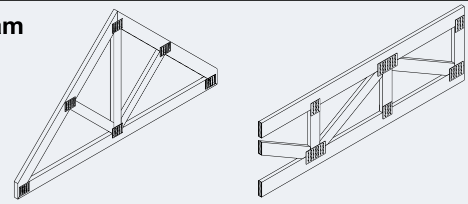
Cross-laminated timber	
Icon	 **Diagram** 
Description	Cross laminated timber (CLT) is engineered wood panels made by joining layers of timber together with the grain direction of alternating layers at right angles. Most CLT is glue laminated but some is assembled with mechanical fasteners, usually nails. 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

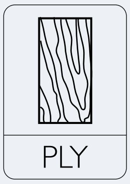
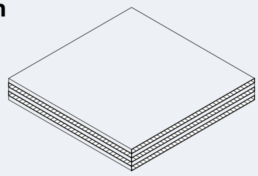
Plywood		
Icon		**Diagram** 
Description	Plywood (Ply) is a wood panel product manufactured from peeled softwood and hardwood veneers laminated into a sheet with the veneer's grain direction alternating between layers. The grain on the outside veneer runs along the sheet with the grain of alternate inner 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

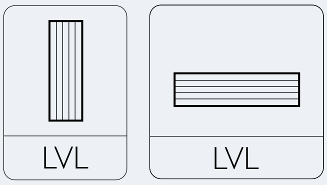
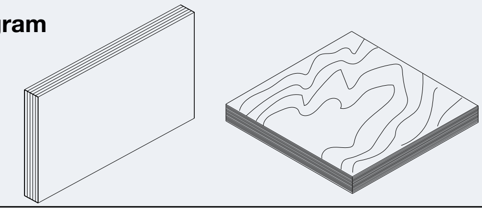
Laminated veneer lumber	
Icon 	**Diagram** 
Description	Laminated veneer lumber (LVL) is a structural wood panel product manufactured from peeled veneers laminated into a panel with the grain of most veneers running parallel to each other along the board. Cross veneers can be included and these increase 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

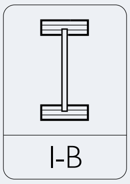
Table 40: Typical LVL sections

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

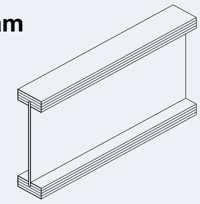
Legend: Commonly available ■; Available on order ◘; In limited supply □

I-Beams

Icon



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 ☐

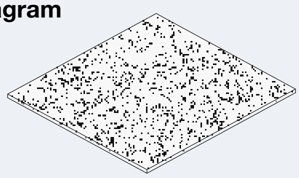
Oriented strand board	
Icon 	**Diagram** 
Description	Oriented strand board (OSB) is a reconstituted structural panel product made from thin wood flakes or strands combined with adhesive into a mat and pressed together. A wax may be added to the resin to increase the material's moisture resistance. Controlled 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 □

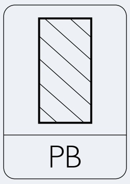
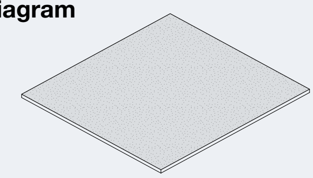
Particleboard	
Icon 	**Diagram** 
Description	Particleboard (PB) is a reconstituted wood panel product manufactured from wood particles combined with adhesive into a mat and pressed together. As the wood particles are randomly oriented, the finished panel has uniform properties in each direction. Particleboard 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 ☐

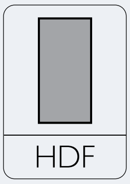
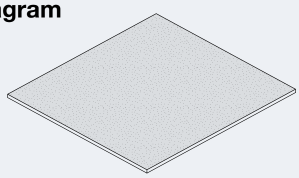
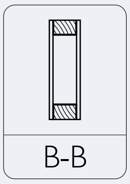
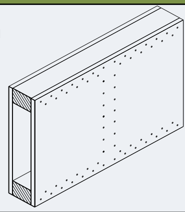
Fibreboards			
Icon		**Diagram**	
Description	Fibreboards are a diverse group of sheet products including low, medium and high-density fibre board manufactured from wood fibres arranged into a mat and pressed. They are generally very stable with uniform thickness and properties. High-density fibre (HDF) 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		

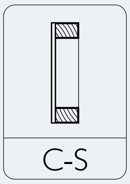
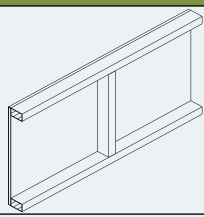
Table 44: Typical high-density fibreboard size range

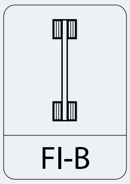
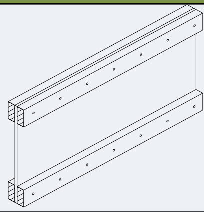
Width	Length		
mm	**2440**	**2745**	**3050**
460	■	■	◘
610	◘	◘	◘
900	■	■	◘
1200	■	■	◘
1350	◘	◘	◘

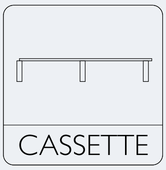
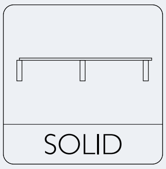
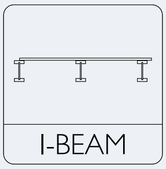
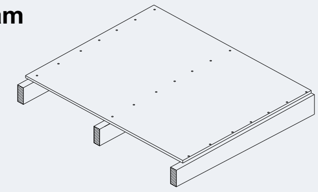
Legend: Commonly available ■; Available on order ◘; In limited supply □

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

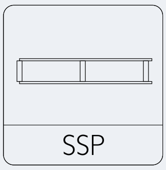
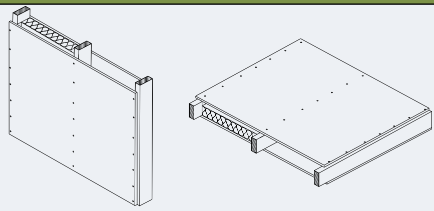
See WoodSolutions Design Guide 7: Plywood box beam construction

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

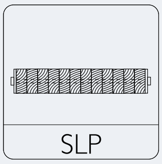
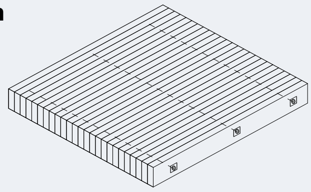
I-section beams			
Icon		**Diagram**	
Description	Fabricated I-section beams (FI-B) are lightweight, structurally efficient and engineered elements fabricated with solid timber or LVL flanges, and plywood, OSB or similar sheet material as the central web. While I-beams are available as generic off-the-shelf 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		

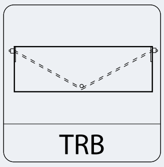
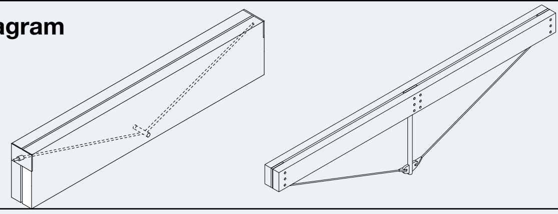
Cassette floor panels			
Icon	  	**Diagram**	
Description	Floor cassettes (Cassette) are prefabricated, engineered modules made with solid timber, LVL, I-beams or floor trusses as joists, and ply, OSB or particleboard as flooring. Workshop fabrication allows for precise assembly, increased quality control, 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		

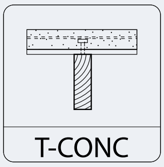
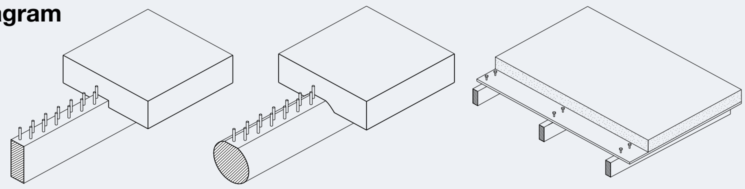
See WoodSolutions Design Guide 31: Timber Cassette Floors

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 	**Diagram** 
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.

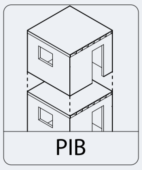
Icon	
Description	Prefabrication-intensive building (PIB) is the building's assembly from elements and components predominantly prefabricated off site and transported to it before being lifted into place. In this type of construction, most elements are designed and documented as volumetric modules or 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

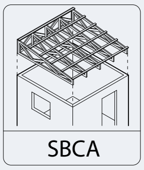
Icon	
Description	Site-based component assembly (SBCA) is the separate site assembly of significant building components from generic and fabricated elements. Once assembled, they are lifted into place in the structure. This approach overcomes the transport limitations of prefabrication-intensive building 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

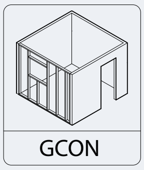
Icon	
Description	General construction (GCON) is a building's assembly on site from a combination of generic and readily available fabricated elements, such as prefabricated wall frames and nailplate floor and roof trusses. Generally, the work site is exposed to the elements and construction relies on 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

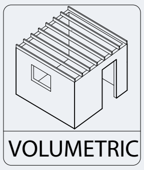
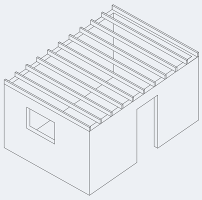
Icon		**Diagram** 
Description	Prefabricated volumetric modules (Volumetric) are three-dimensional components such as rooms assembled from wall, floor and other elements. They can be designed and assembled as whole buildings, as components to be stacked one on another, or as serviced or structural cores 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

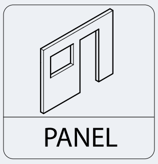
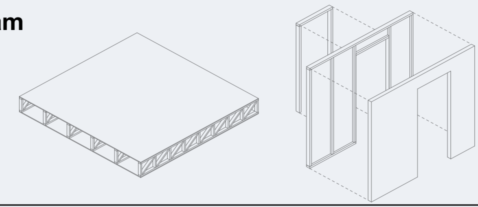
Icon	 **Diagram** 
Description	Prefabricated panels include preassembled wall frames, nailplate and other floor and roof trusses, floor cassettes, stressed skin panels, assembled post and beam units, and massive timber panels. In their simplest form, they are open frame items that are installed and fixed 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

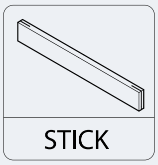
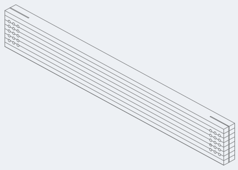
Icon	 **Diagram** 
Description	Prefabricated sticks are timber elements made or cut to size and prepared for projects. In their simplest form, these are simply cut to length, shaped and delivered to site as part of optimised, knock together component systems. They can also be generic products nailplated 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.

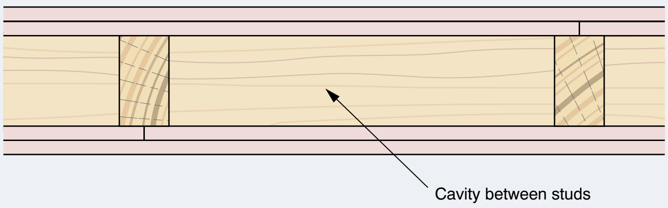


Figure 12: Plan – Fire-resistant lining on a stud framed wall

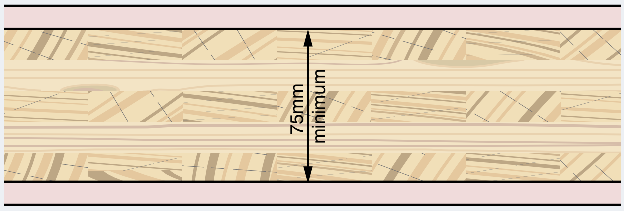


Figure 13: Plan – Fire-resistant lining on a massive timber wall

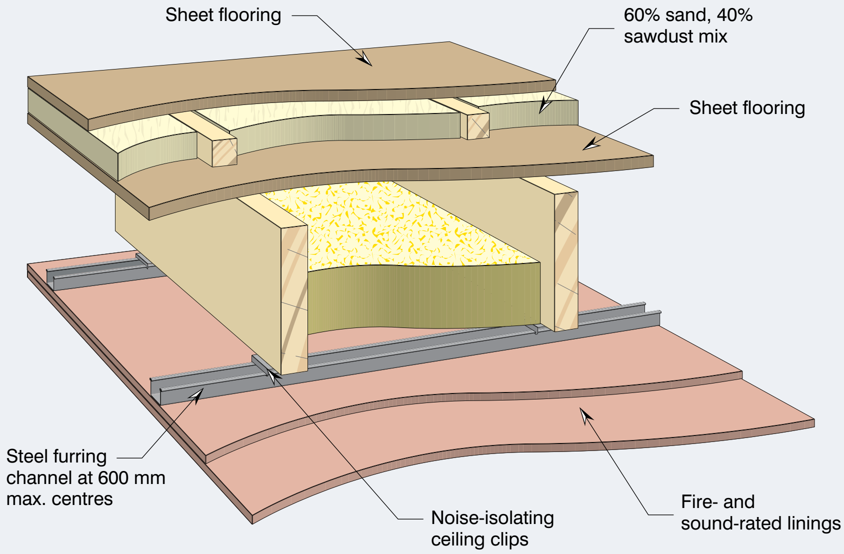


Figure 14: Section – Fire-resistant lining on a framed floor

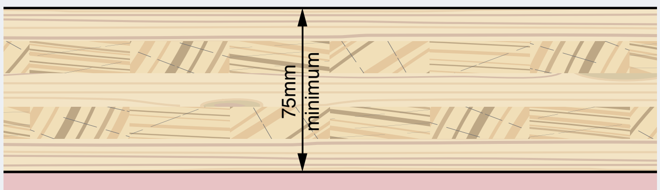


Figure 15: Section – Fire-resistant lining on a massive timber floor

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.

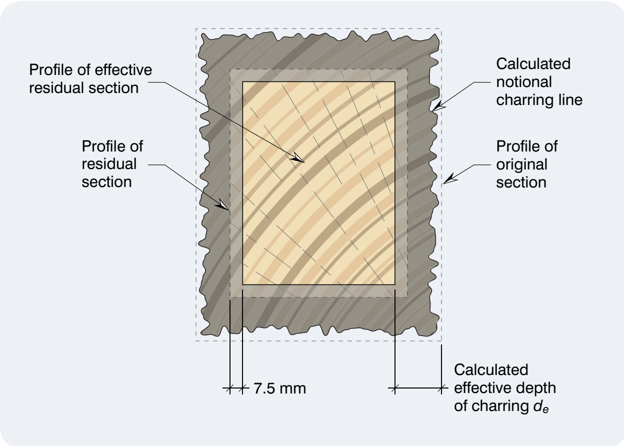


Figure 16: Charring rate and the effective section.

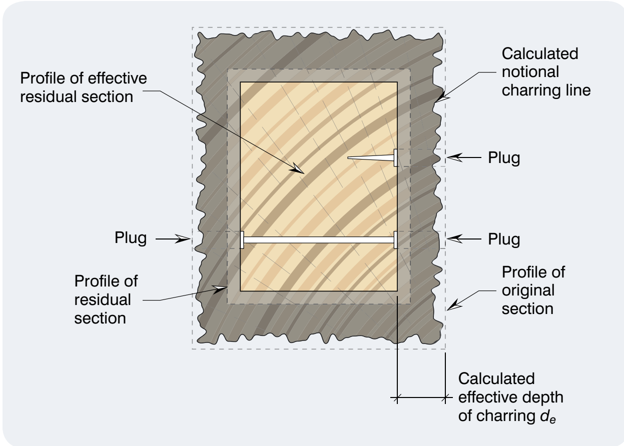


Figure 17: Embedded connectors in the timber.

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.

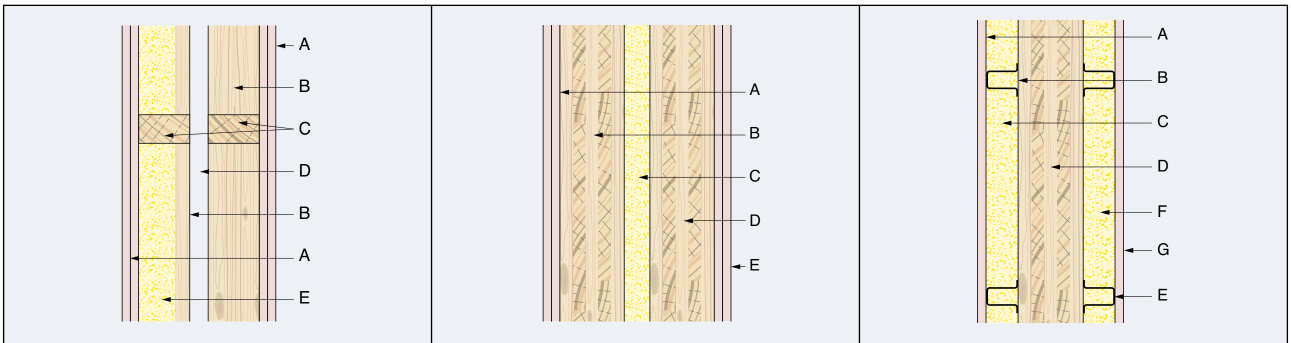
To inform conceptual design, this guide summarises the major methods for ensuring the structural system selected provides satisfactory acoustic separation in apartment buildings and between floors in office and similar buildings.

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.

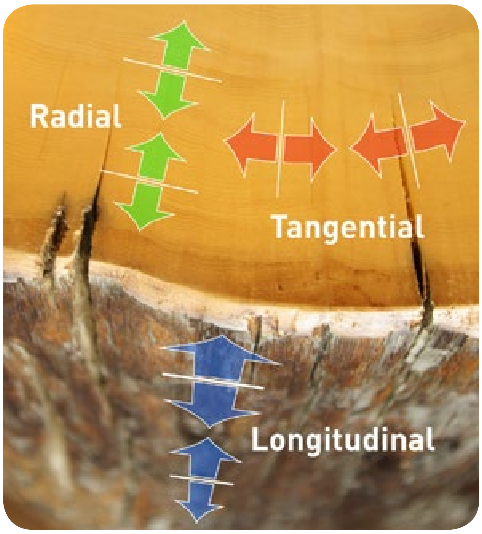


Figure 22: The three principal directions of wood grain

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.

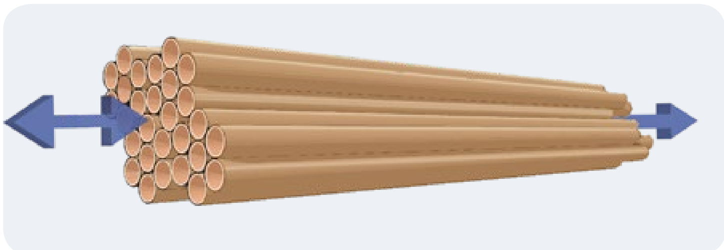


Figure 23: Strength along the grain

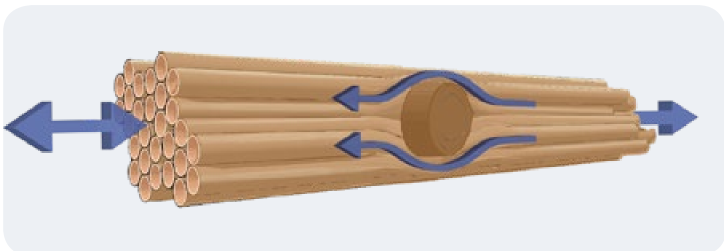


Figure 24: Strength and grain impact of knots

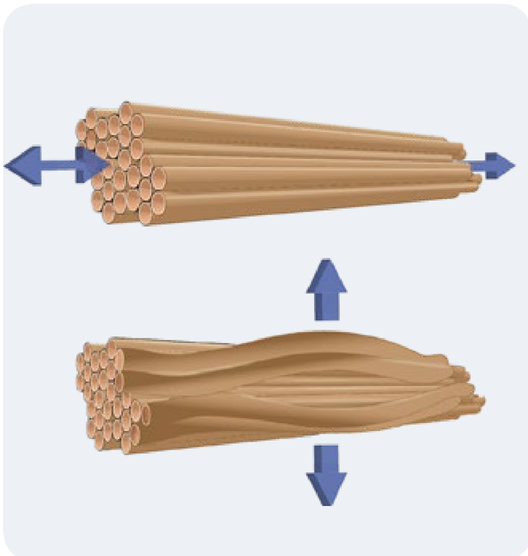


Figure 25: Strength perpendicular to grain

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%.

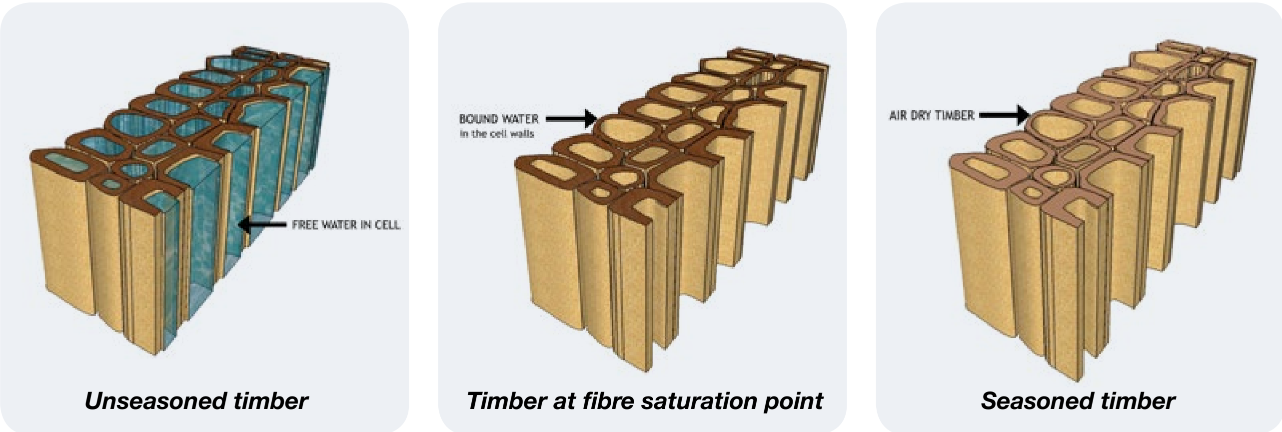


Figure 26: Types of water in timber

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.

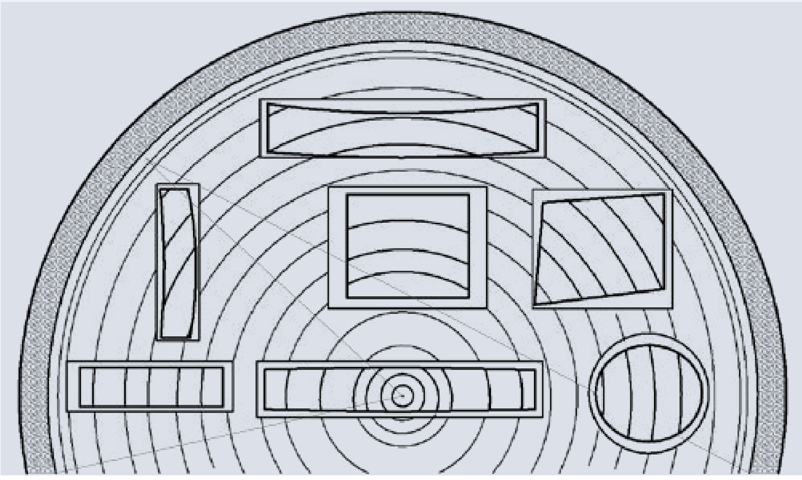


Figure 27: Characteristic shrinkage of boards with different growth ring patterns as the wood dries.

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

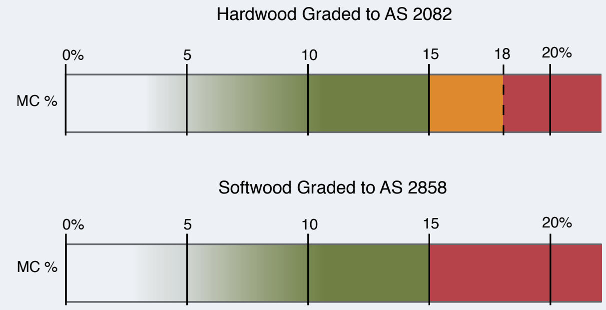
8.3 Moisture content

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.

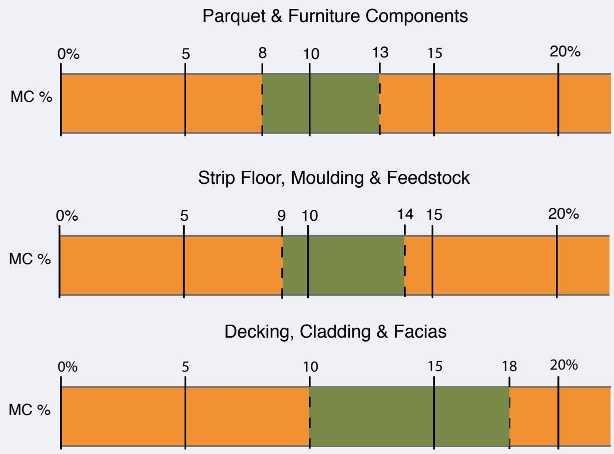
Standards-based MC requirements

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.

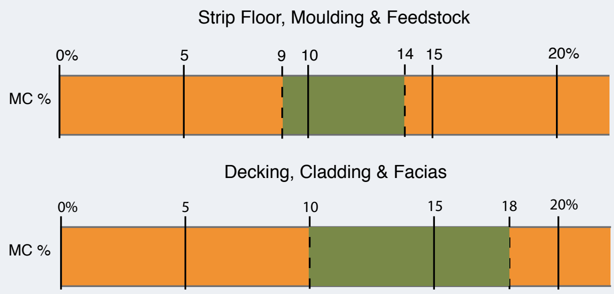
The required MC for appearance hardwoods graded to *AS 2796* varies with each of three product types, with the lowest MC required for parquetry and furniture products. The required MCs for appearance softwood graded to *AS 4785* also vary with product types, with lower and tighter limits set for material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



MC requirements for structural hardwood graded to AS 2082 and softwood to AS 2858



MC requirements for appearance hardwood graded to AS 2796



MC requirements for appearance softwood to AS 4785

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.

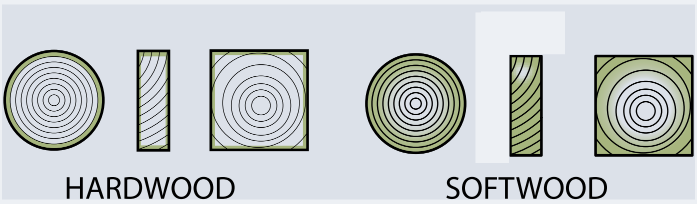


Figure 29: Likely preservative penetration after treatment for hardwood and softwood

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.

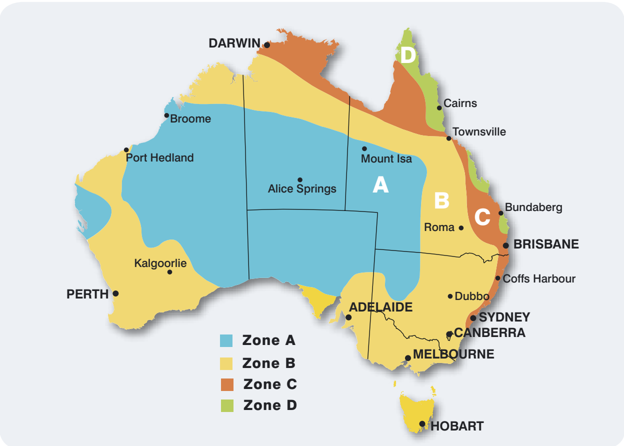


Figure 30: In-ground decay hazard zones for Australia

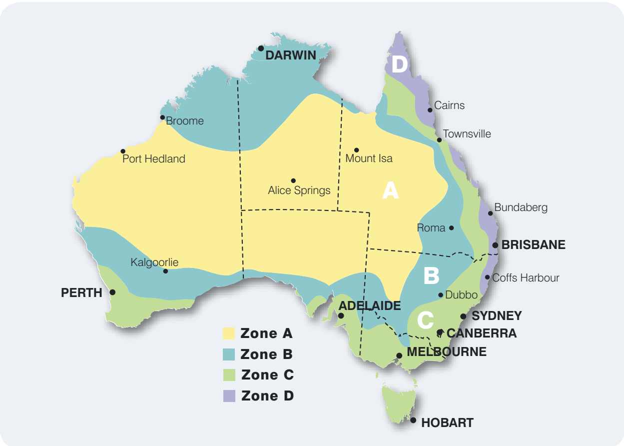


Figure 31: Above-ground decay hazard zones for Australia

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.

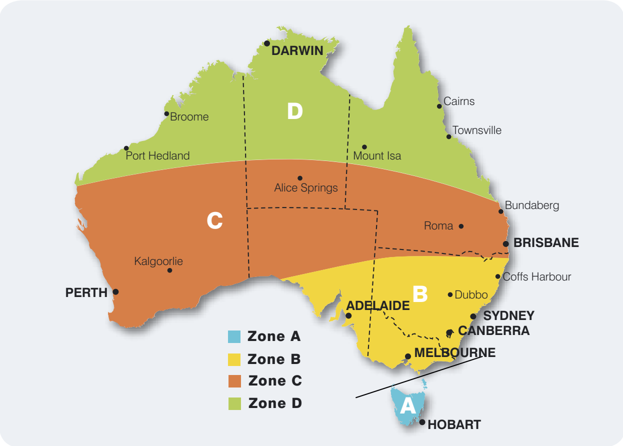


Figure 32: Termite hazard zones for Australia

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.

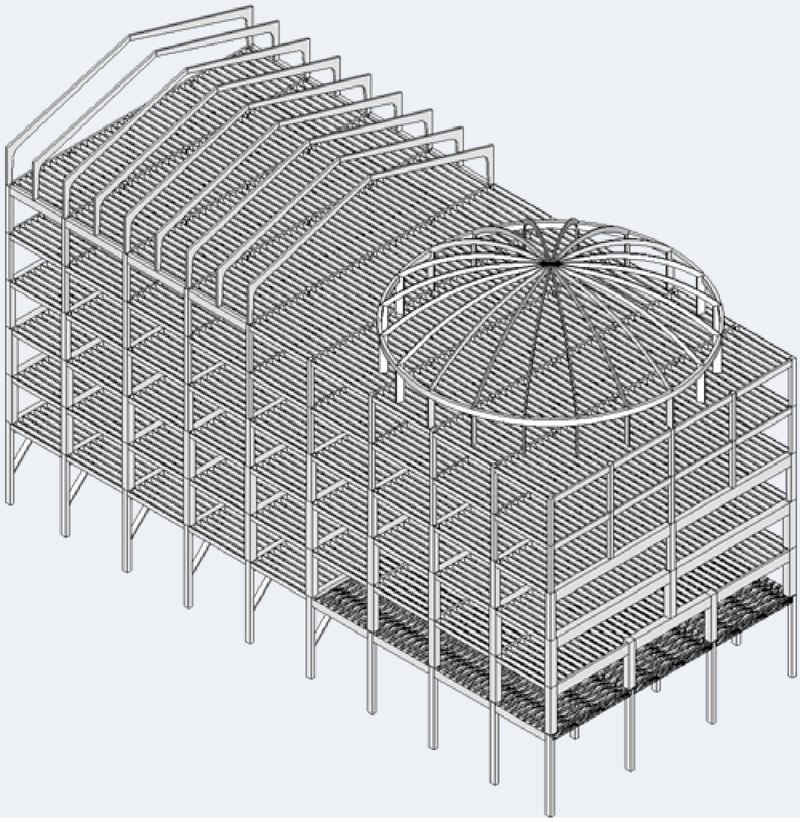


Figure 34: General conceptual design of Worked Example 1

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, $V_{des\vartheta}$ from the site wind speeds:

$V_{des\vartheta}$ =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;

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

$$F = \Sigma(p_z A_z) \qquad \text{… } \textit{Equation 3}$$

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.

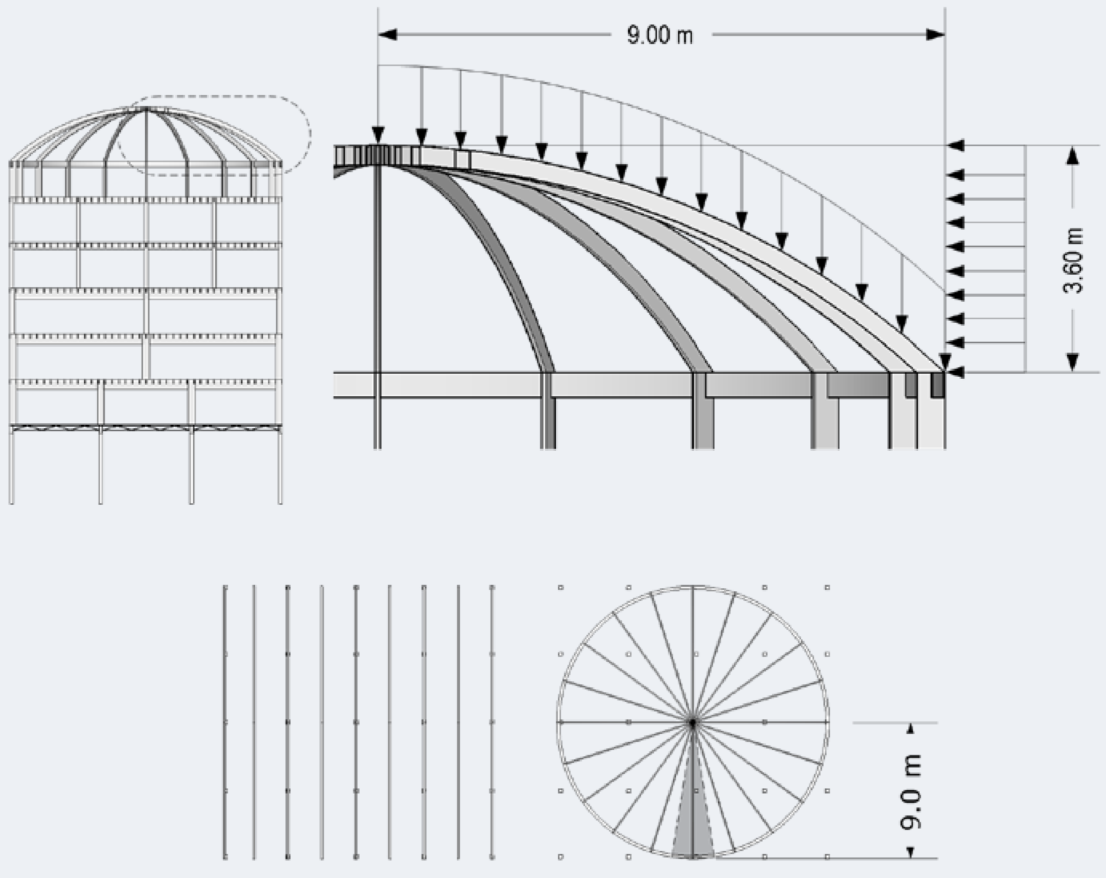


Figure 35: Curved glulam beam for ribbed dome

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).

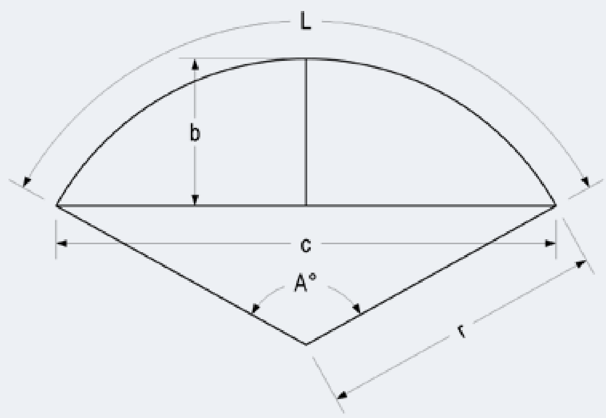


Figure 36: Properties of a circle

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.

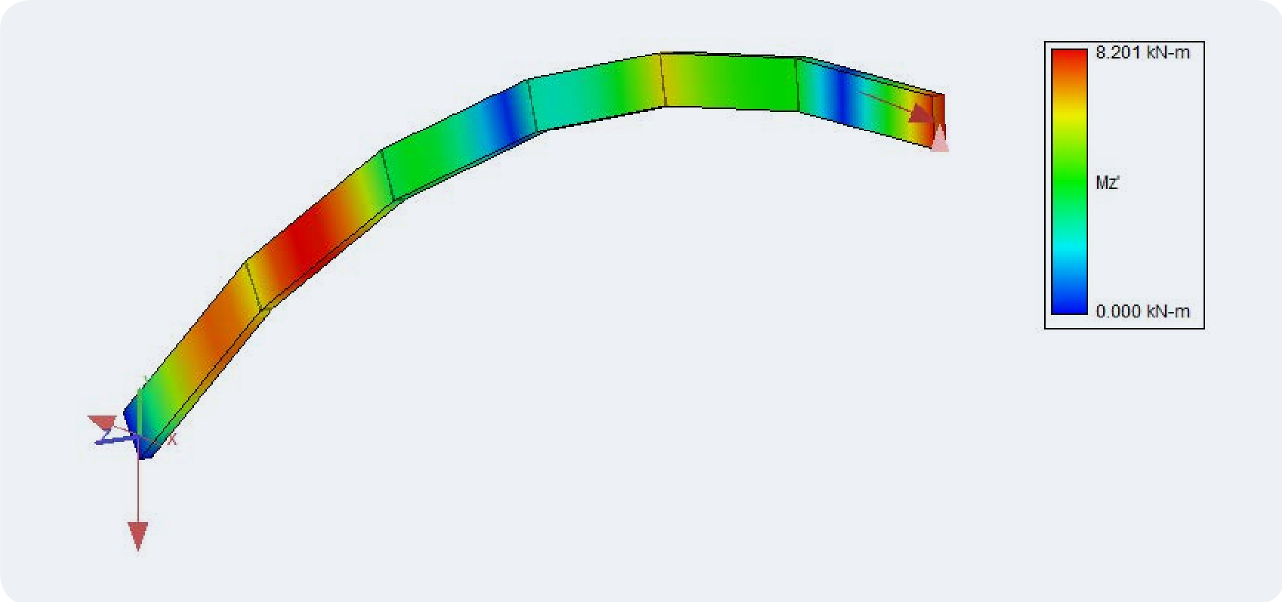


Figure 37: Cured glulam beam software output

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.

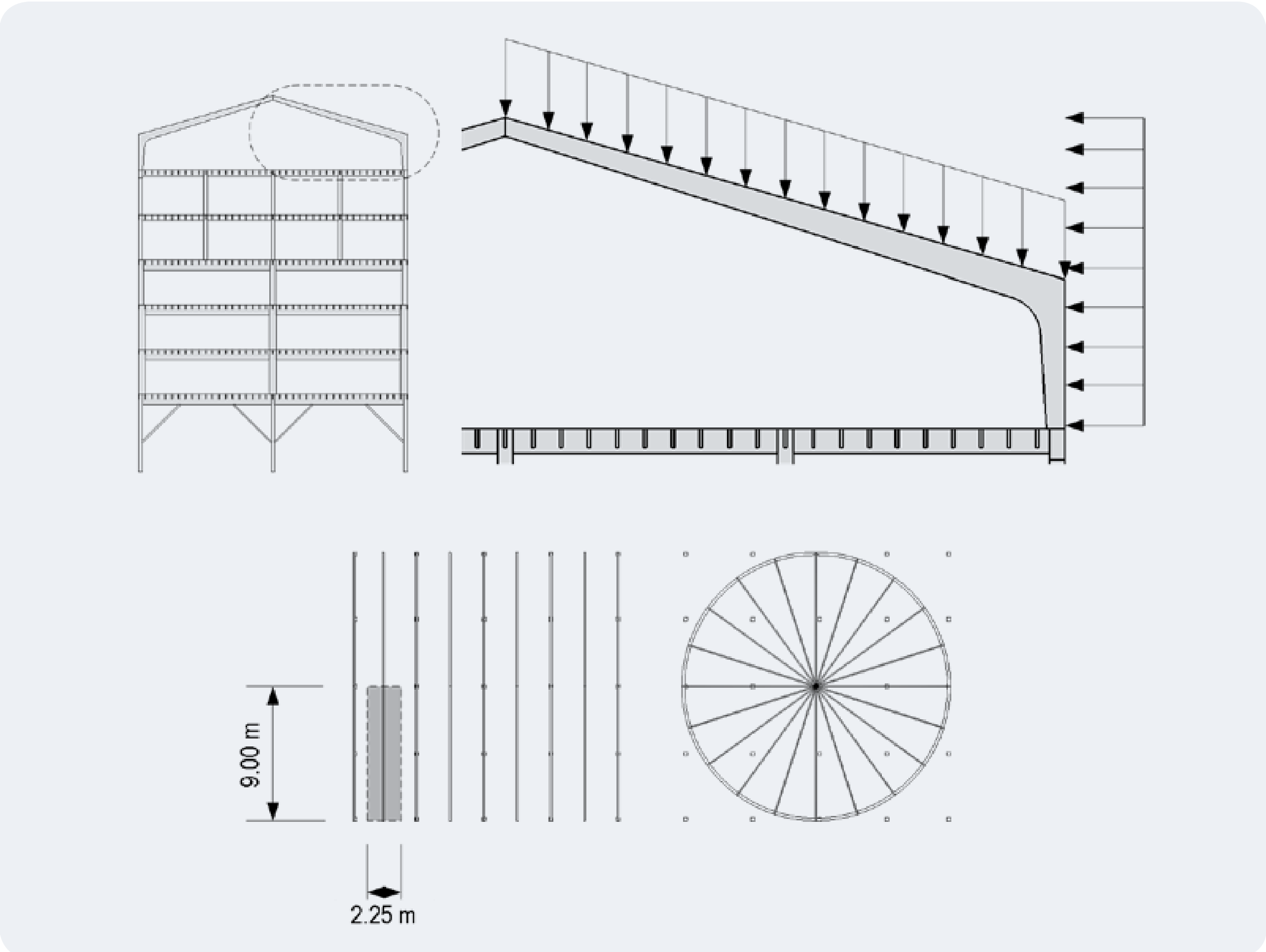
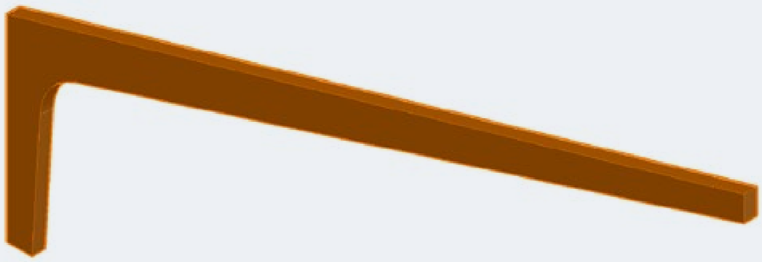


Figure 38: Illustration of the Tudor arch beam computed in this example.

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.

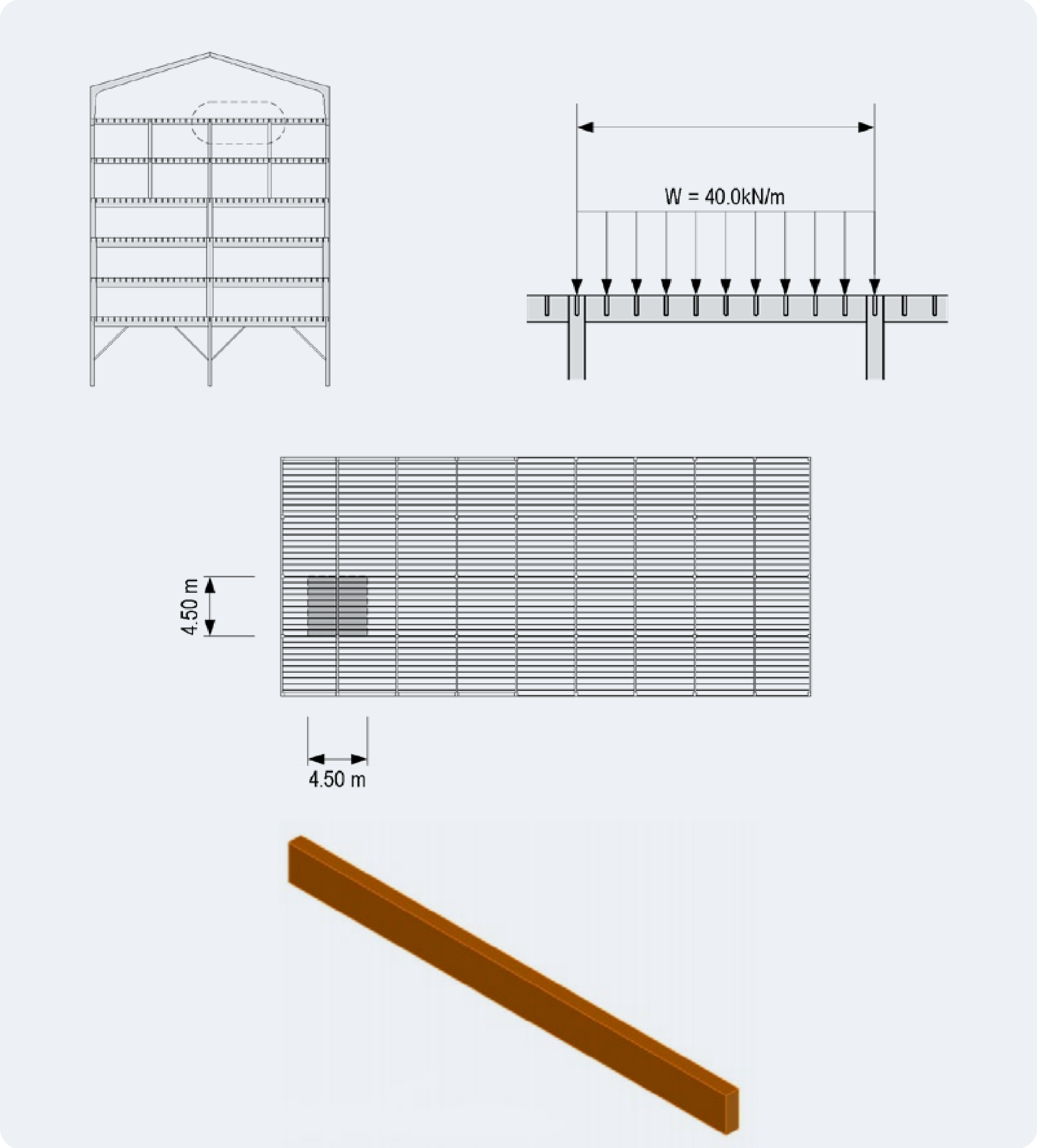


Figure 39: Structural location and configuration of the roof deck beam in this design example.

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	= 0.27 kN/m	… g = 9.81m/s^2
	Where density = 560 kg/m^3		
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.

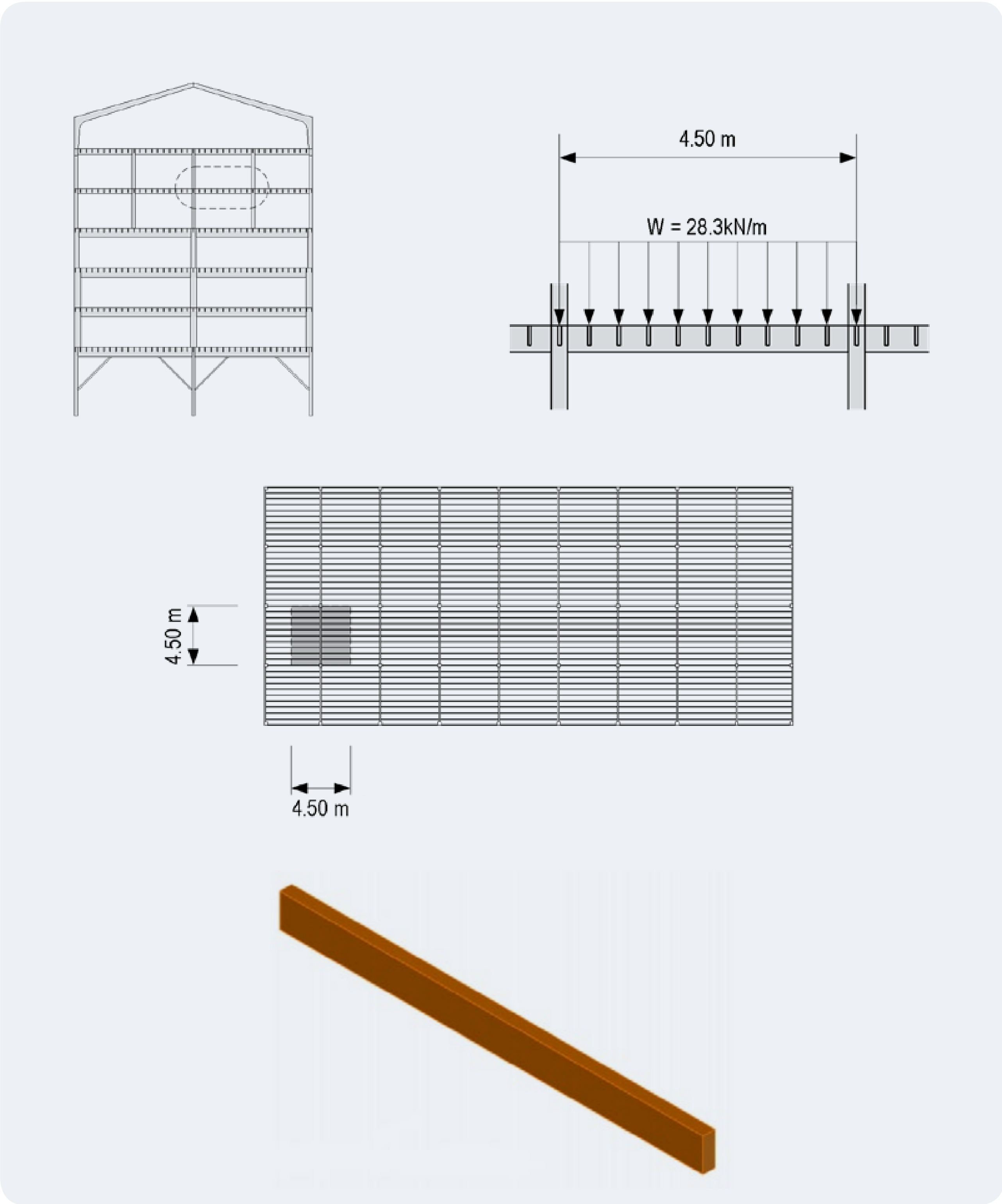


Figure 40: Structural location and configuration of the fifth level floor beam.

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.

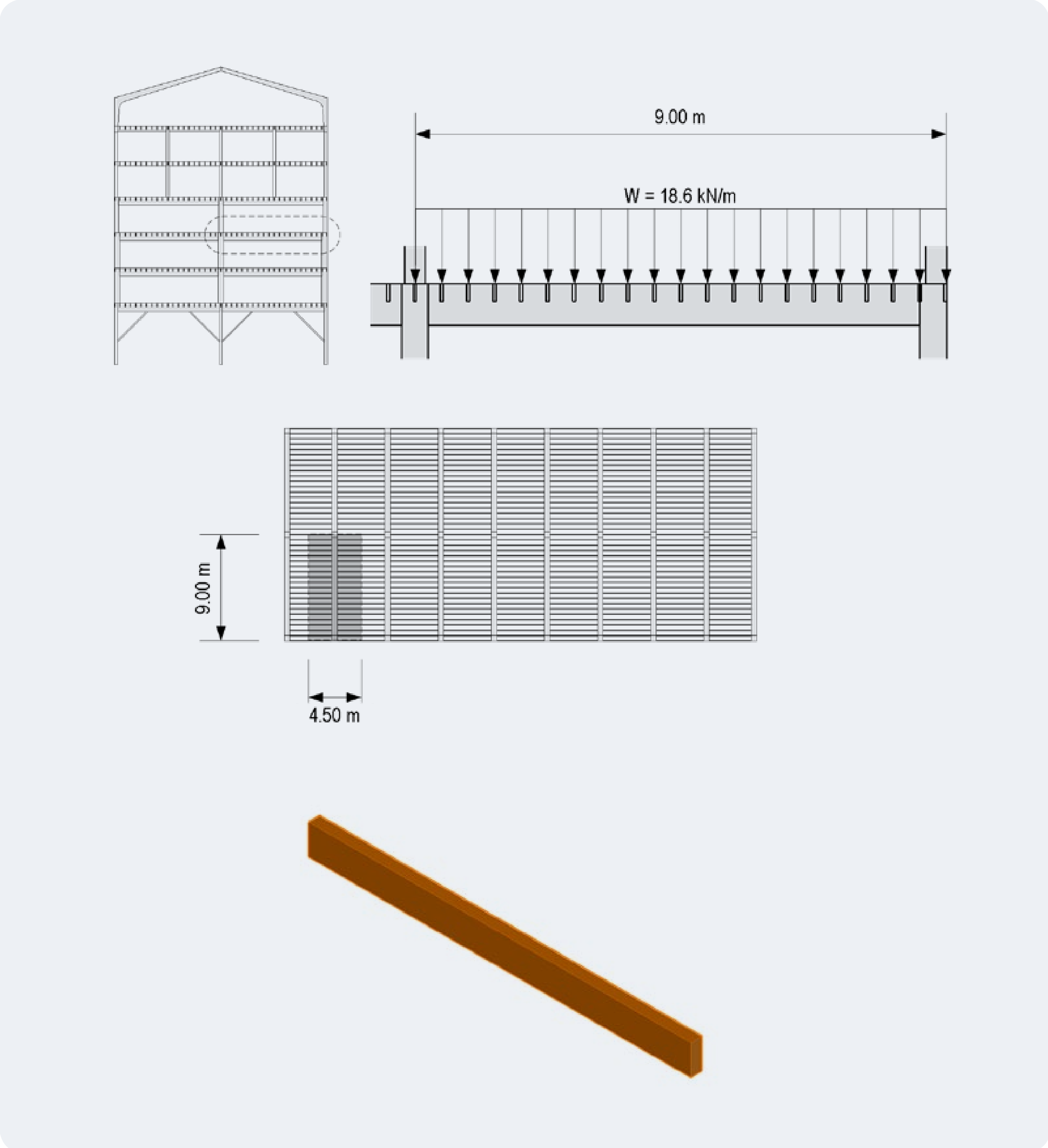


Figure 41: Structural location and configuration of the fourth level floor beam in this design example.

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.

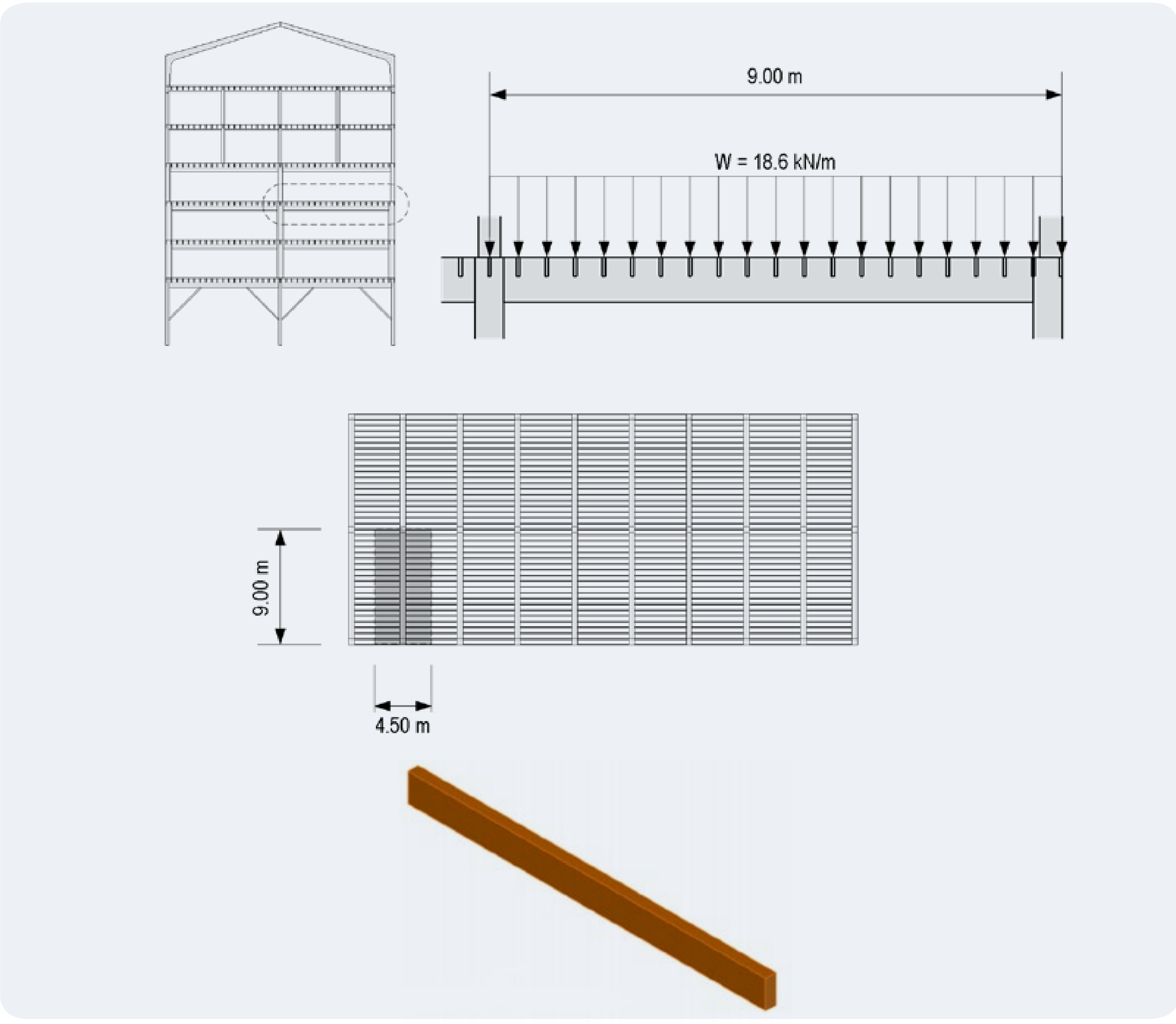


Figure 42: Structural location and configuration of the third level 4 hour fire resistant beam in this design example.

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.

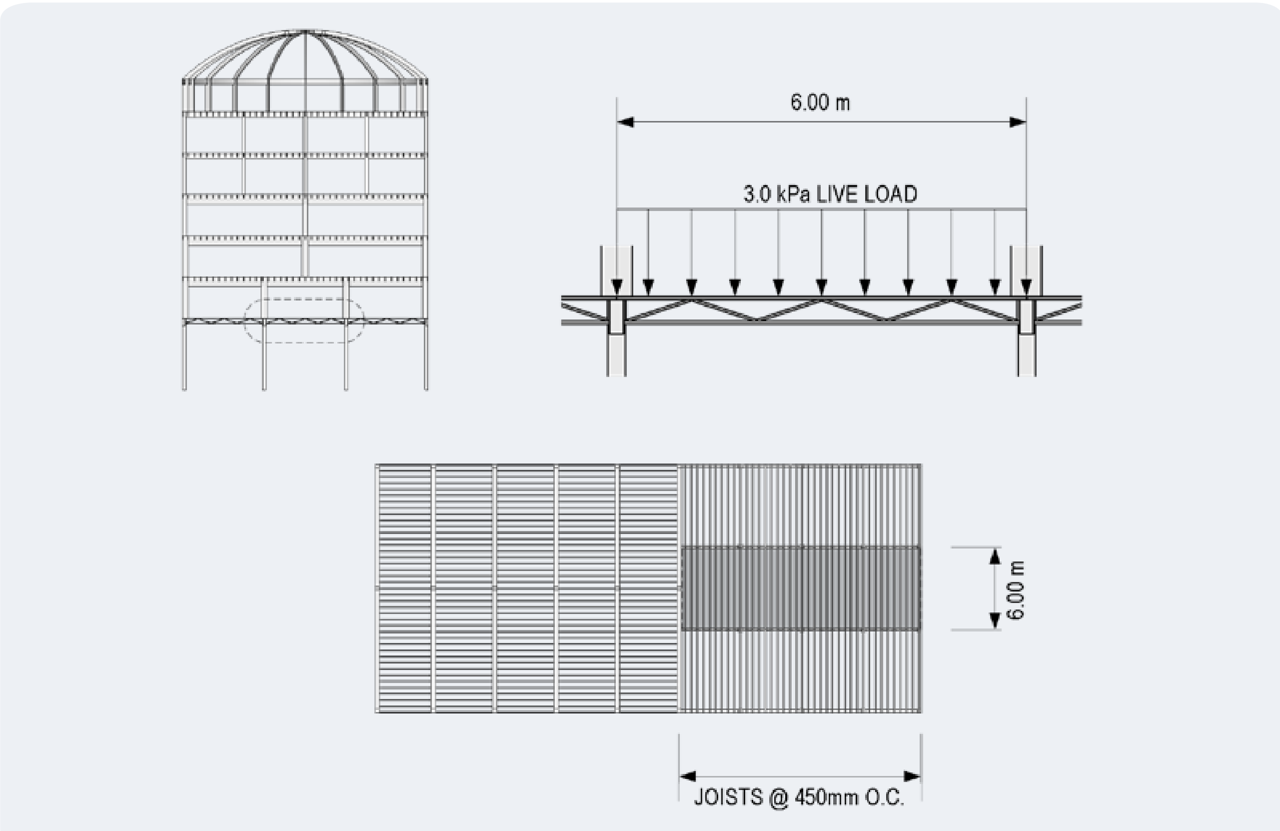


Figure 43: Structural location and configuration of the open web floor joist in this design example.

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

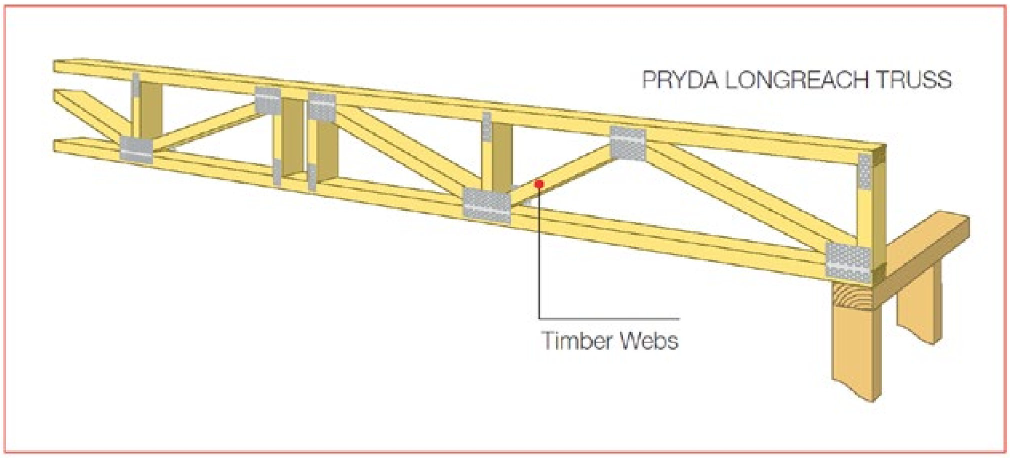


Figure 45: 3-Dimensional Sketch of a Pryda Longreach Truss

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.

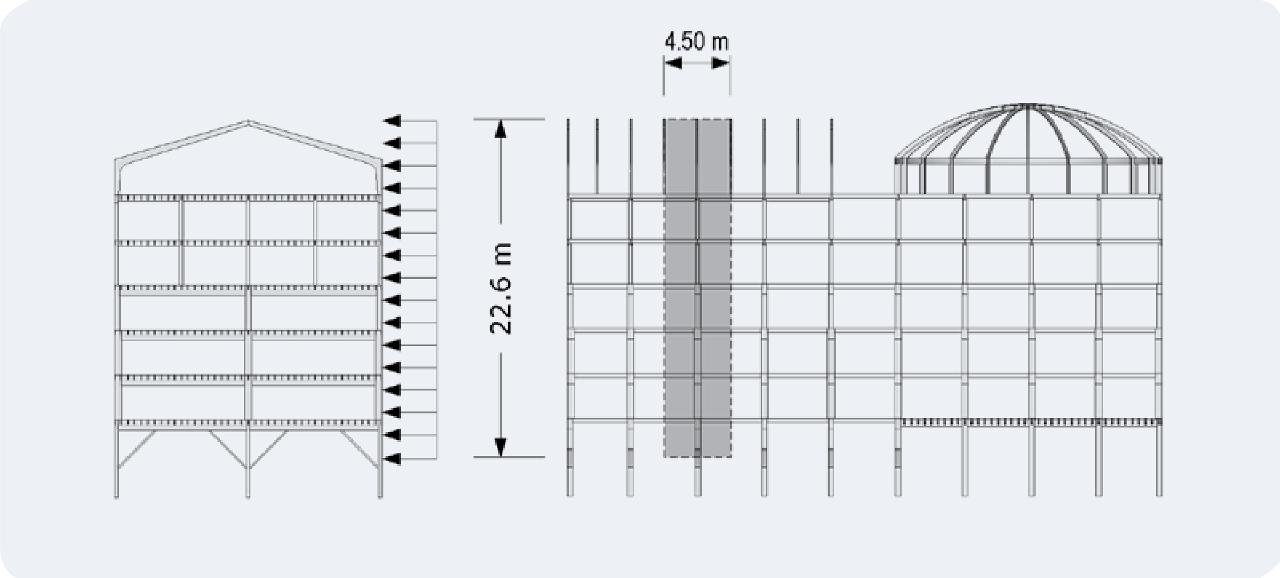


Figure 46: Illustration of the configuration of the portal frame in this design example.

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 … $1.2G + 1.5\psi_L Q$

N^*_c = 1.2 x (6.7 x 9 + 284.9) + 0.4 x (13.5 x 9 + 688.5) = 738.2 kN … $1.2G + Wu + \psi_c Q$

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	
ρbS_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.

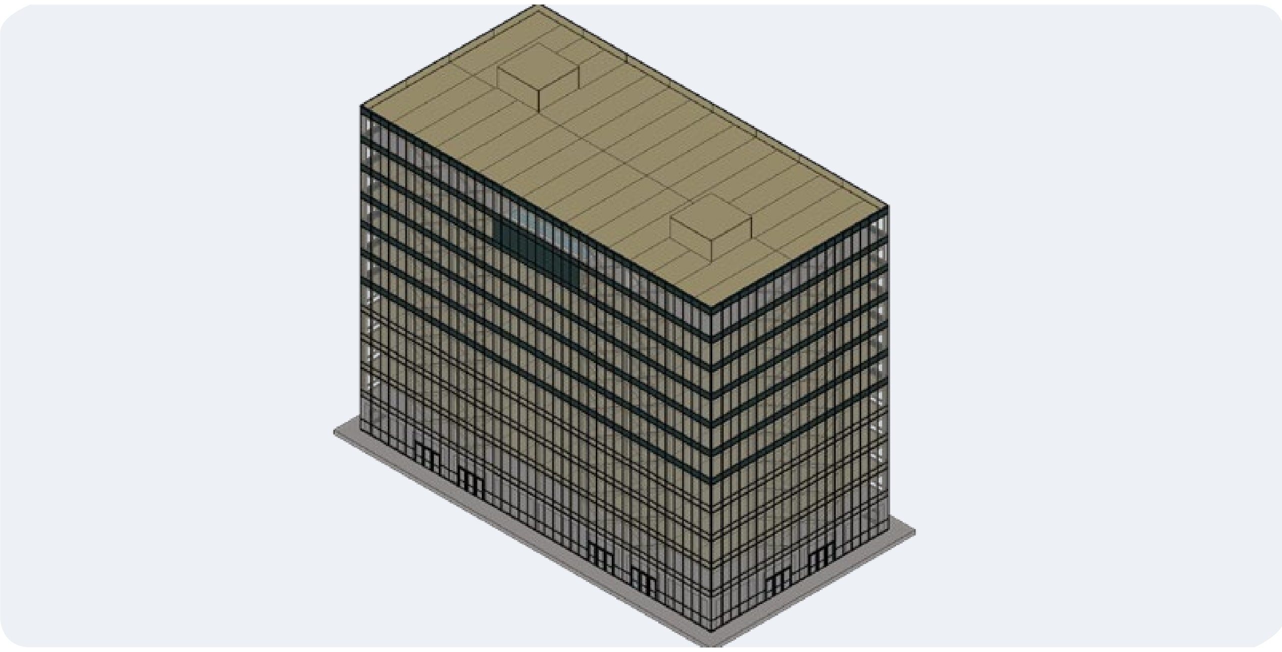


Figure 47: General conceptual design of Worked Example 2.

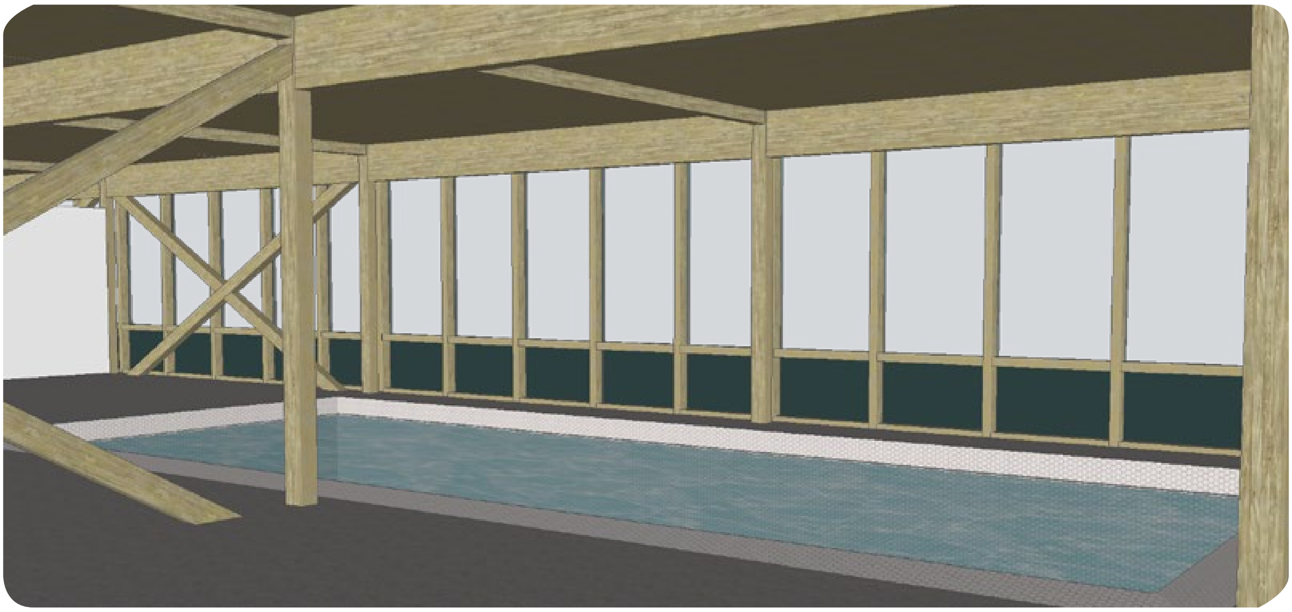


Figure 48: Conceptual design of inset pool on level ten

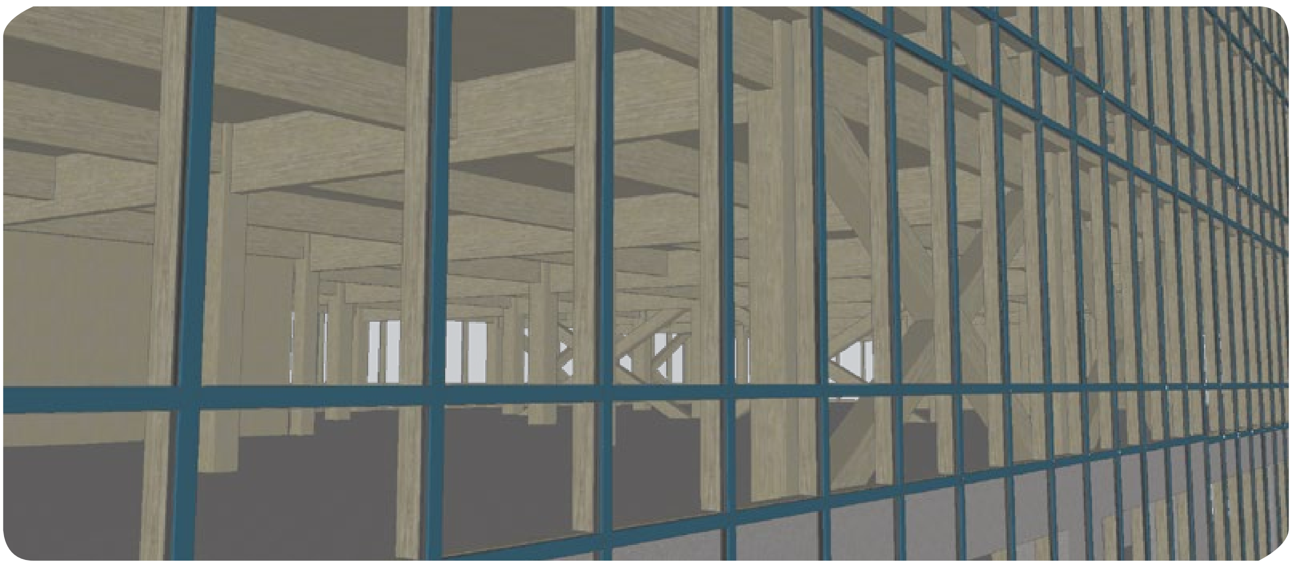


Figure 49: Conceptual exterior glass shell design.

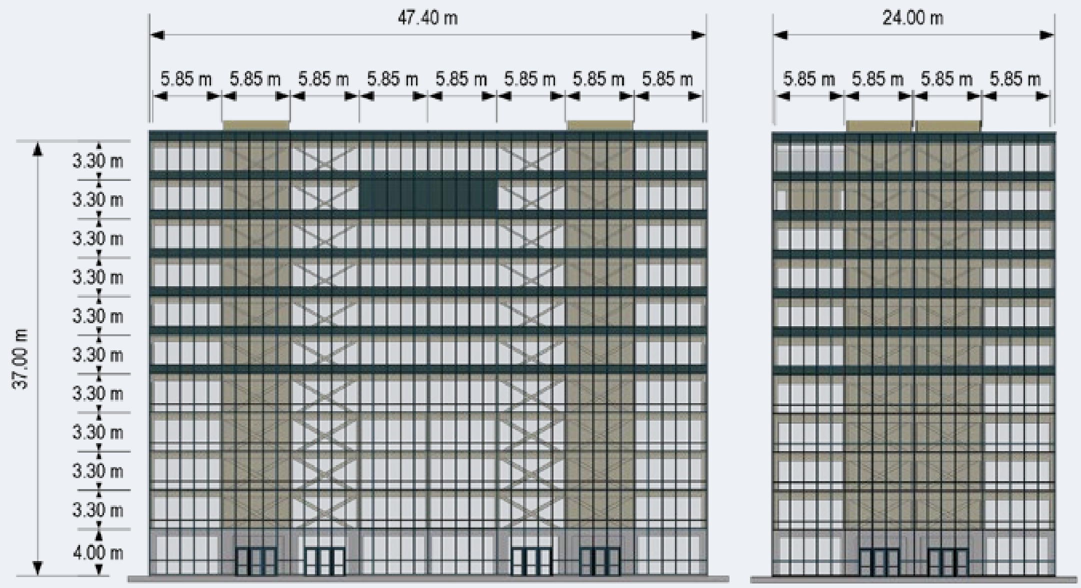


Figure 50: Conceptual design structure dimensions.

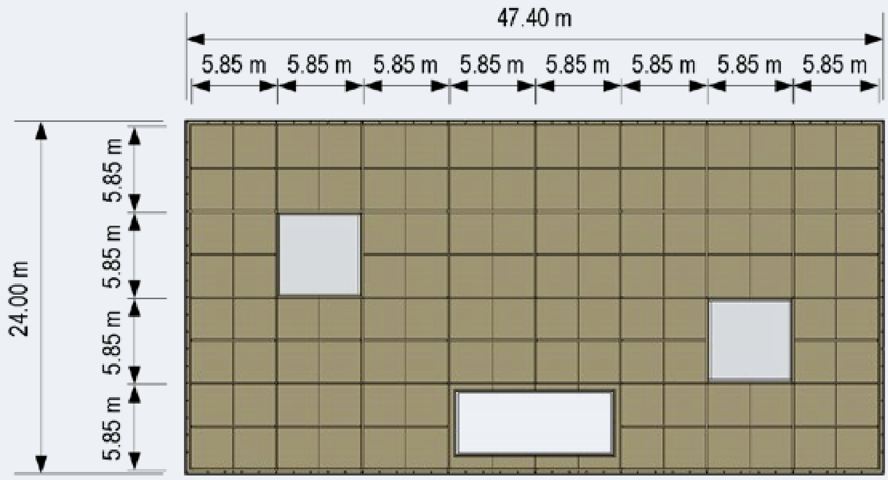


Figure 51: Conceptual design Level 10 framing dimensions.

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

   $g_v = 3.7$

   $\zeta = 0.01$

   $H_s = 2$ …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	

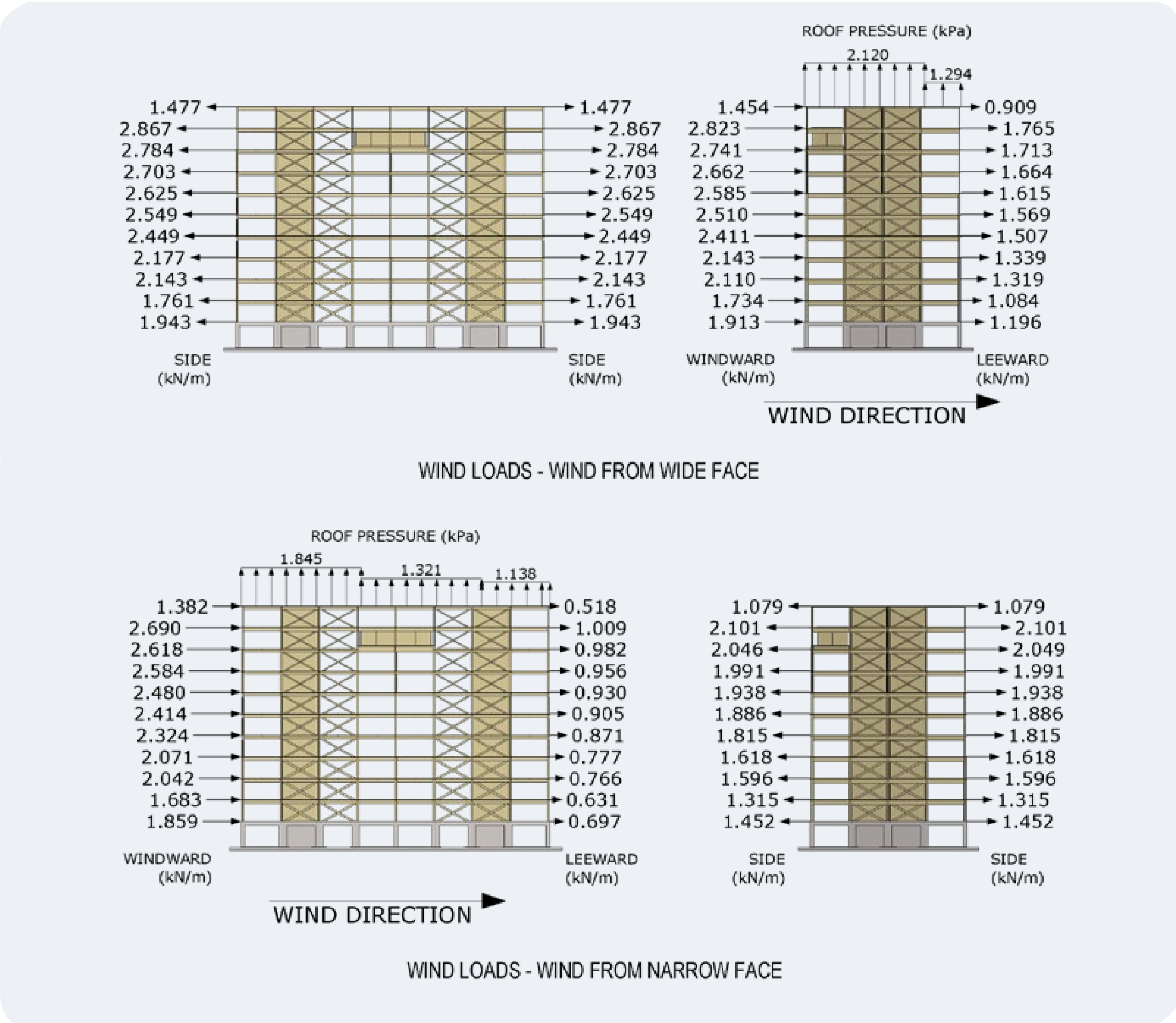


Figure 54: Illustration of the Wind Actions imposed on the structure.

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. $K_pz = 0.117$

      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.

   $V = ([k_pZC_h(T_1)S_p]/\mu)W_t$ … *Equation 4*

   a. C_hT_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

      $T_1 = 1.25k_th_n^{0.75}$ …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.

$$F_i = k_{F,i}V \qquad \ldots 6.3\ (1)$$

$$= \frac{W_i h_i^k}{\sum_{j=1}^{n}\left(W_j h_j^k\right)}\left[k_p Z C_h(T_1)\frac{S_p}{\mu}\right] \qquad \ldots 6.3\ (2)$$

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:

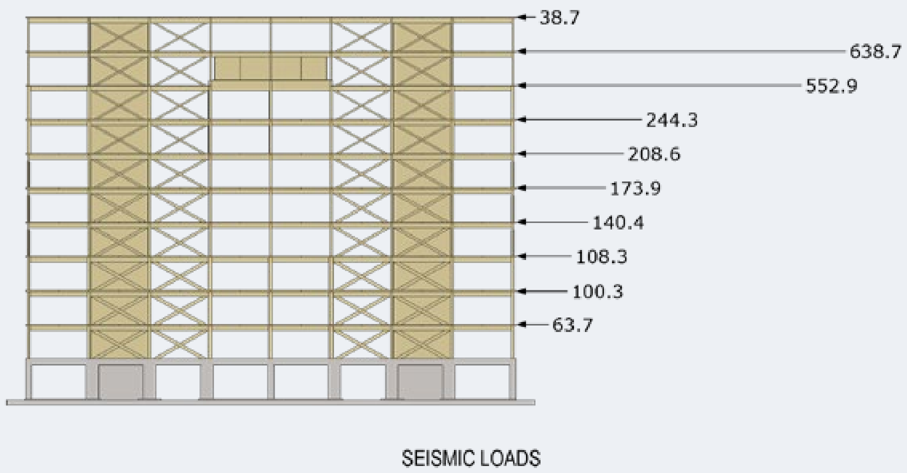
$$Scaling\ Factor = \frac{0.9}{(1-\theta)} \geq 1.0$$ *…Equation 6*

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.



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:

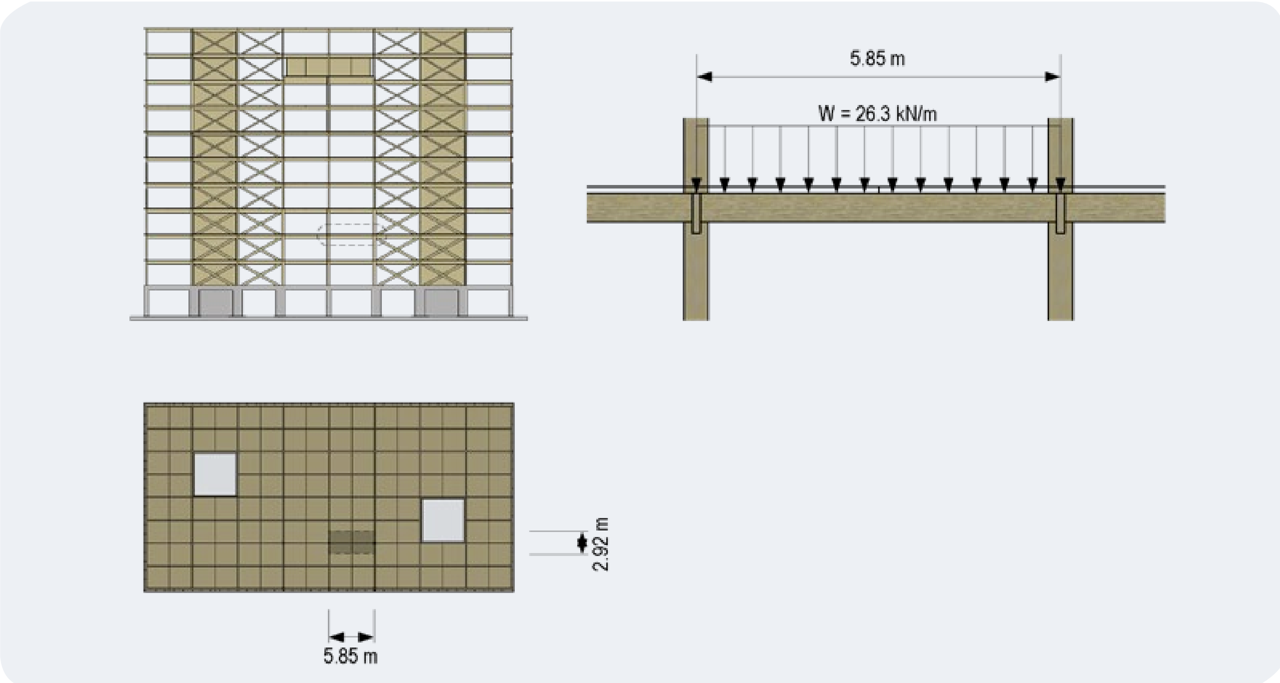


Figure 56: Structural location and configuration of the glulam floor beam in this example calculation.

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:

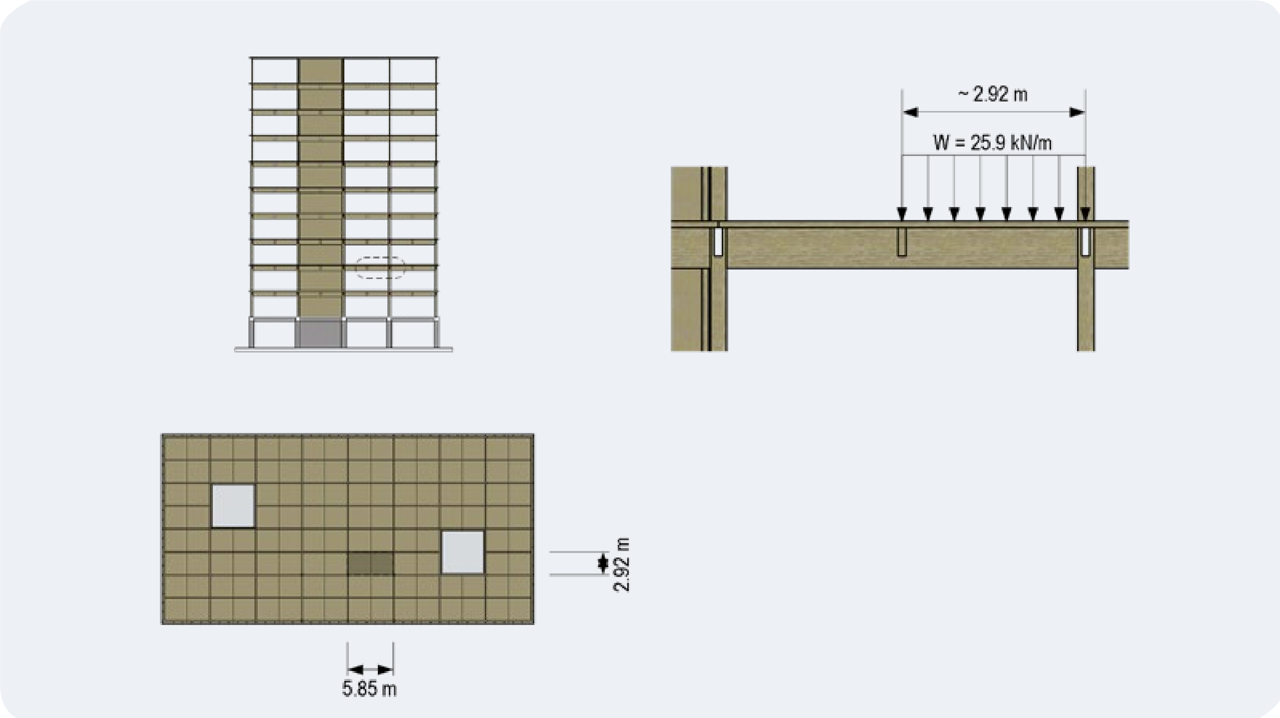


Figure 57: Structural location and configuration of the CLT floor panel.

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:

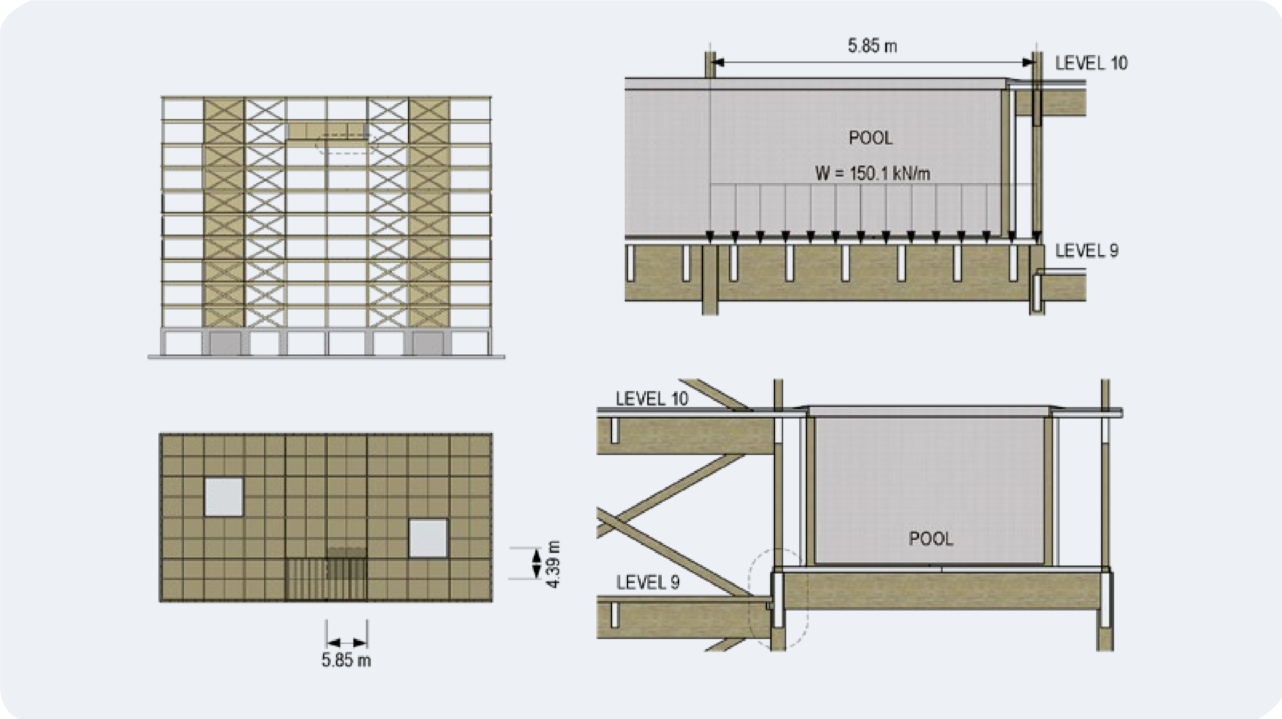


Figure 58: Structural location and configuration of the glulam floor beam supporting the pool.

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:

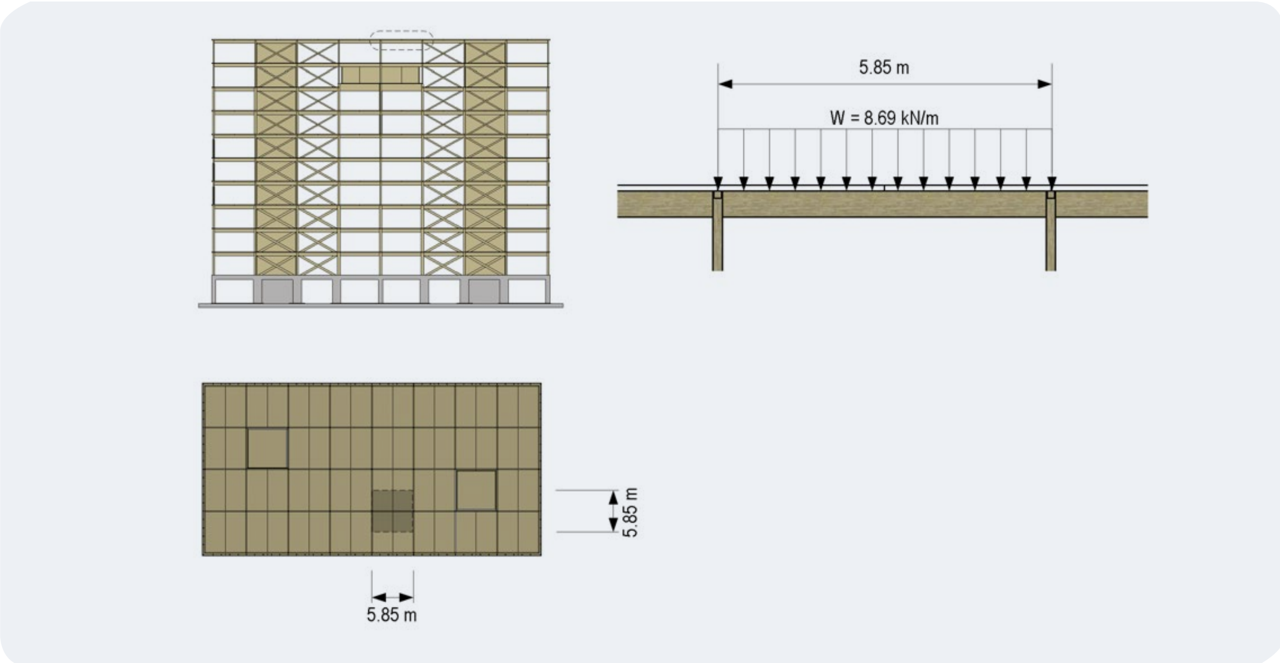


Figure 59: Structural location and configuration of the glulam beam supporting the roof.

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:

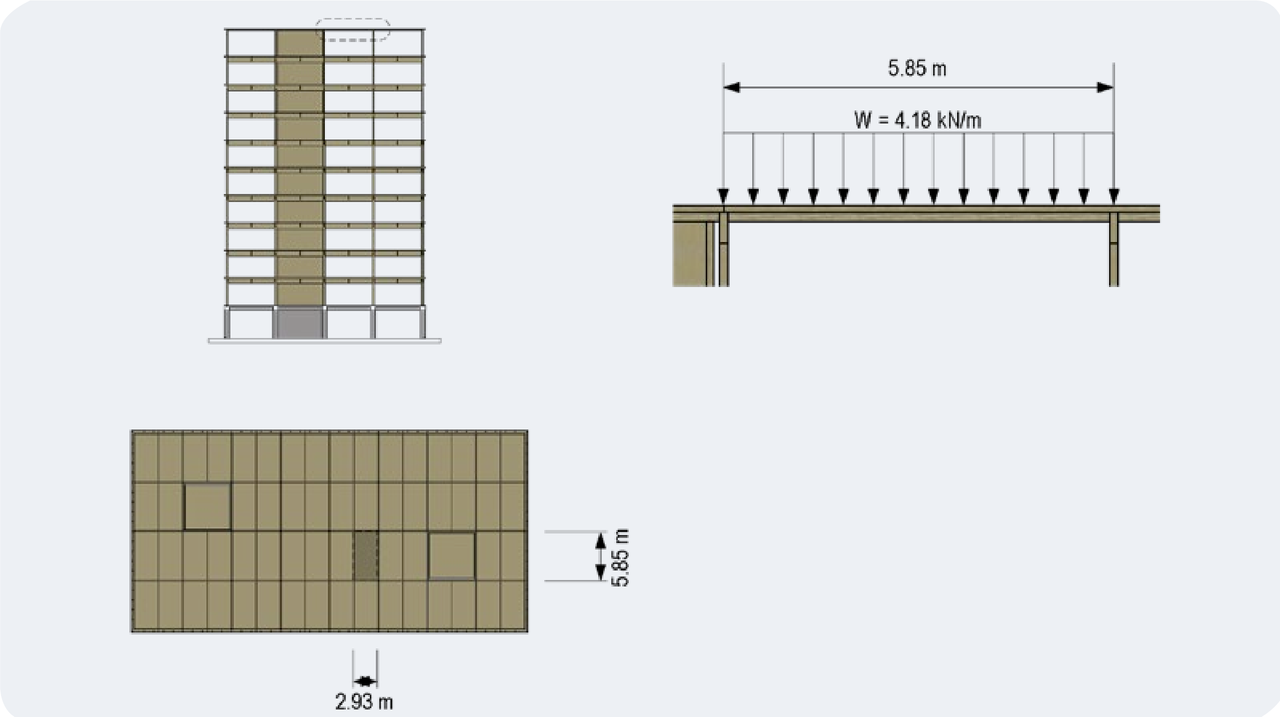


Figure 60: Structural location and configuration of the CLT roof panel.

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	

$\mathbf{N_{d,j}}$ **= 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	

$\mathbf{N_{d,j}}$ **= 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.

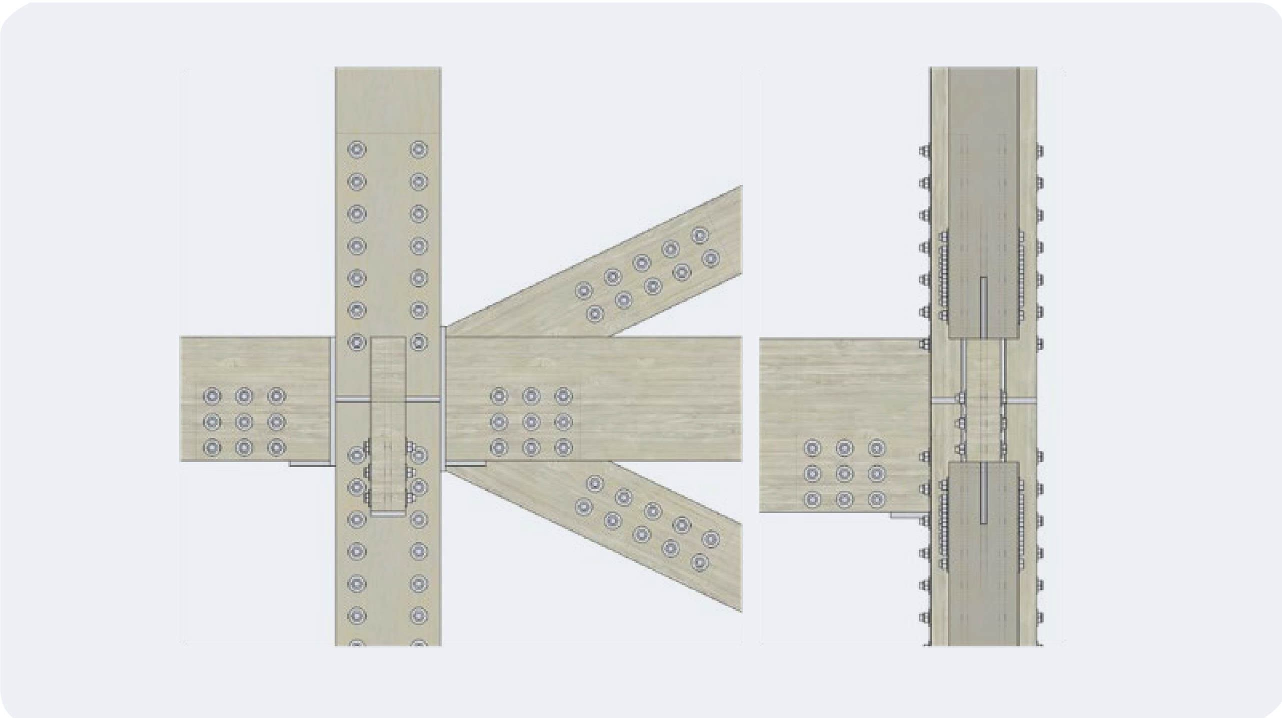


Figure 62: Connection design illustrating bolt pattern at level two.

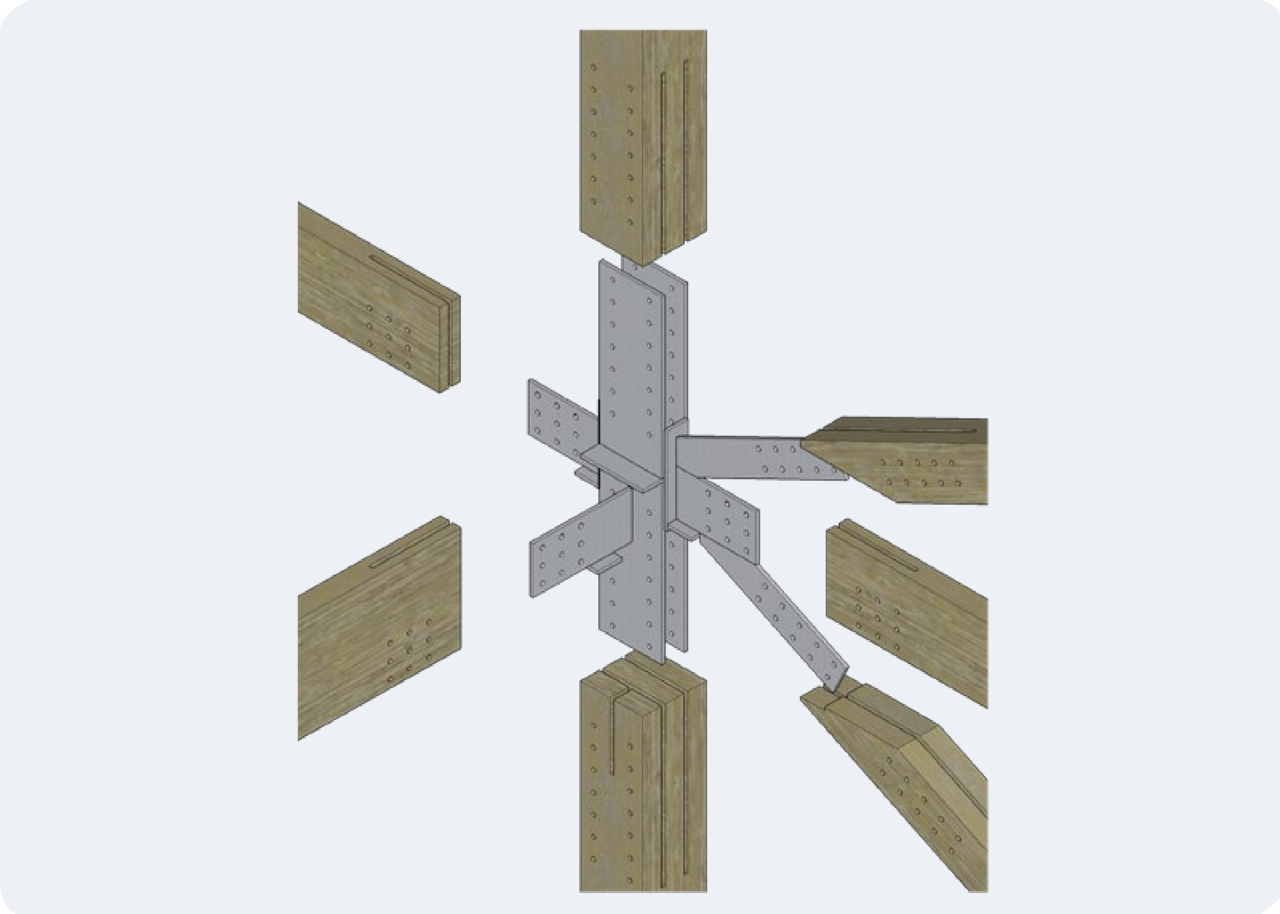


Figure 63: Steel connection design.

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:

$FRR=0.1 f b[4-(b/d)]$ *…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.

$FRR_r=0.1 f b[4-(b/d)]$ *…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.

Discover more ways to build your knowledge of wood

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

www.woodsolutions.com.au

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

Technical Publications

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

WoodSolutions Tutorials

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

Seminars and Events

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

What is WoodSolutions?

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

WoodSolutions™
design and build
woodsolutions.com.au

47

This guide was revised in Feb 2020

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

Timber Bollards

For protection and traffic control

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Aveo Group, Strongbuild - Bella Vista luxury retirement community (10-storey luxury retirement village build with cross-laminated timber - CLT).

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-50-8

Acknowledgments
Thanks to Tom Lenigas, Stephen Pate, Edgar Stubbersfield, Scott Willey

First Published: October 2018

Revised: February 2020

Contents

1 Introduction

Bollards can define space, control vehicles and people, and protect from unwelcome traffic. They also can be used for decorating and wayfinding.

As security awareness increases, bollards are being used to resist threats such as ram-raid burglaries, and even potential terrorist activity. External bollards provide one of the most weather-exposed uses of timber. They are rarely maintained and yet are expected to have long service lives.

The selection of appropriately durable timber species and grade are important – not only for the weather-exposed portion of bollards, but also that buried in-ground.

This guide details the considerations for better bollard design and specification so that common poor construction and inferior product substitution can be avoided.

Bollards are a permeable barrier used to control and guide pedestrians and vehicles. Their form and layout mediate the transition across both physical and perceptual boundaries. This control can be used to minimise conflict between different traffic types.

Bollards offer safety and refuge as a physical barrier, and provide visual clues to deliver a range of functions:

- **traffic flow** – visually define road and path boundaries
- **pedestrian control** – chains between bollards can create a form of balustrade
- **vehicle control** – can be closely spaced to restrict vehicle movement
- **impact protection** – act as security barriers to protect infrastructure from accidental and deliberate impact
- **guidance** – can include signage and wayfinding functions
- **decoration** – provide sculptural and memorial points of interest.

1.1 Bollard Types

Bollards have maritime beginnings as wharf-side posts for mooring ships, but are now common throughout the built environment. While hinged and machine-operated bollards are used for variable vehicle access, the focus of this guide is on fixed timber bollards that are:

- **surface mounted** – bolted to the ground plane *or*
- **embedded** – buried at one end to support the above-ground portion.

External bollards are fully exposed to weather variations. Their top edge is the most susceptible to degradation. When bollards are embedded in-ground the timber is also exposed to ground moisture.

The likelihood of maintenance for many bollards is low, and a 'set-and-forget' mindset demands an emphasis on good design and correct specification. This Guide sets out design parameters and core principles of bollard design. While the core function of bollards remains largely static, there is a trend for increasingly more complex designs that push the boundaries of both function and appearance.

1.2 Bollard Aesthetics

Beyond function requirements, bollard design needs to consider the following aesthetic considerations:

- **establishing context** – the bollard use categories described in this Guide steer aesthetic form
- **style** – while bollard uses vary, their design can reflect a project's individual design style
- **clear function** – detailing can be used to convey the intended bollard function (i.e. wayfinding, slowing traffic, protection, etc).

1.3 Bollard Use Categories

Using bollards in particular scenarios requires different bollard forms. If designed to protect a building from impact, a bollard may be quite squat in form. If used for wayfinding, a bollard may have a flat profile to be more visible and if it just denotes a pathway, it may be quite slender.

This Guide divides bollard use into three distinct classes that offer greater clarity in design and detailing:

- **park scenarios** – for open recreational spaces
- **urban scenarios** – used to define decision points and mark boundaries of abutting spaces
- **esplanade scenarios** – when bollards are seen in long vistas based on formality and repetition.

The design parameters for each installation type are outlined in the following sections. While variation of bollard function does not appear to be a strong design consideration in the outdoor furniture industry, allowing function to guide the bollard form offers major benefits in controlling both movement patterns and the sense of barrier and/or safety.

2 Parkland Environments

Park environments are predominately open and uncluttered. Park bollards are often experienced at a pedestrian scale, resulting in a more personal engagement between the pedestrian and the bollard. Bollards offer the following benefits to park environments:

- **add interest** – enliven an otherwise unobstructed expanse
- **pedestrian engagement** – encourage engagement with pedestrians and a sense of play
- **unobtrusive** – provide rail-less boundary fencing (typically these are less visually intrusive).

In the past, bollards have been used in parks as purely functional barriers. However, there are ways for bollards to be used in a way that is more engaging to park users.

Figure 2.1: Parks frequently use timber bollards to guide pedestrians and define borders.
Photograph: Dennis Clark Photography

The two main forms of bollards for park environments can be categorised as ‘mass’ and ‘feature’.

2.1 Mass Bollards

Bollards are often used en masse to define the boundaries of a space, much like fencing without rails. The long runs of bollards that can traverse a park landscape provide the opportunity to design for their mass visual effect rather than just their individual impact. The potential is to use bollards as a more sculptural item. Mass bollards tend to be:

- **simple** – of plainer detail
- **thinner** – smaller cross-section
- **inexpensive** – contributed to by the points above.

The market for mass park bollards will always be price sensitive, with high levels of competition from low-grade timber substitutions. Designers can respond with careful specifications and by inspecting supplied timber to ensure it conforms to the required grade prior to installation.

2.2 Feature Bollards

Feature bollards are located at points of entry, street intersections or activity centres. They tend to be:

- **taller** – more bollard height than mass to increase visibility
- **thicker** – increased size and bulk also aids visibility
- **visually obvious** – act as demarcations for entry points or a change in conditions
- **detailed** – often have a more detailed design to increase their visual emphasis.

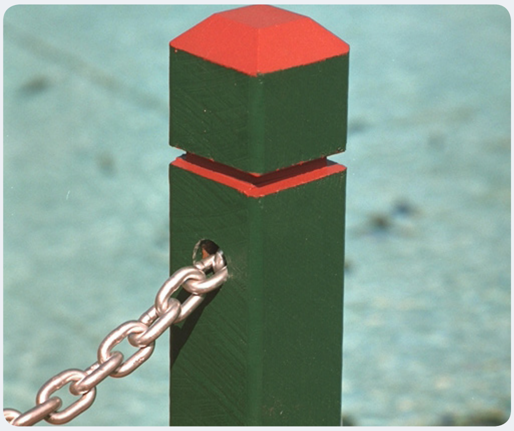
2.3 Bollard Design

Noting the simplicity of the park landscape, design trends generally focus on primarily simple forms. Detail is applied with banding or capping that acknowledge the dominate design form. Details can include:

- **circumferential banding** – for simple decoration
- **wave lines** – often used in riparian or coastal environments
- **other forms** – natural forms such as leaves or animals
- **places names** – reinforcing region or locale identity
- **signage and wayfinding** – direction arrows and symbols such as bikeways, etc.

Figure 2.2: When used for sculptural purposes, decorative timber bollards can use scale and design to create interest and leave a strong impression of place.
Photograph: Kevin Tostado; sourced under Creative Commons from wikipedia.com.

A: Path delineation –
Bollards give visual cues to direction.
Copyright Dennis Clark Photography



B: Vehicle control –
Functioning as physical barriers.

C: Mooring – Maritime use for mooring vessels.
Copyright Dennis Clark Photography

D: Decorative function – Simple and low-scale decorative function with longevity.

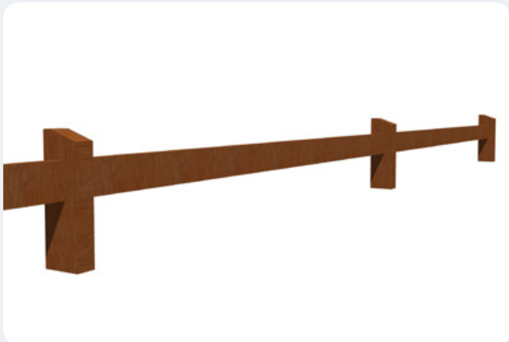
E: Impact protection – Protection for buildings and infrastructure.

F: Signage and wayfinding – Wider faces of rectangular profiles allow signage to be routed into the face or affixed.
Copyright Dennis Clark Photography

Figure 2.3: Bollards can be designed to perform a range of functions, including guidance, traffic control and protection.

A: Rope and chain – Linking bollards restricts vehicle movement and discourages pedestrian access with minimal visual intrusion. Copyright Dennis Clark Photography

B: Balustrade – Infill cables and balustrading provide protection from falling.



C: Rails – Rails between low-height bollards provide a unobtrusive barrier to vehicles.

D: Enclosure – Bollards can be in groups and linked with rails to provide protection to trees or structures.

E: Face-fixed rails – Simple construction with lower potential to harbour moisture.

F: Recessed rails – Rebates for rails offer better support, though greater potential for decay.

***Figure 2.4:** A range of external furniture forms occur when bollards become fences that are likely to be in the same design language as adjacent bollards. Much of their construction will be similar to that outlined for bollards.*

3 Urban Bollards

The use of bollards in urban environments can be quite distinct from those in park and coastal esplanade environments. In urban environments, bollards are primarily sited in key locations to define decision points and mark boundaries. Typical functions of bollards in urban locations include:

- **blocking streets** – particularly T-intersections at entries to malls or pedestrian ways
- **pedestrian protection** – at crossing areas and where pedestrians approach verges and potentially come in to conflict with vehicles
- **vegetation protection** – additional barrier fencing may be required in high-use pedestrian areas
- **building protection** – where structures are close to traffic zones (typical in older city areas where space is at a premium and not designed for modern traffic conditions).

3.1 Visual Impact and Legibility

Situations that require the long 'soldier' runs typical of park bollards rarely occur in urban environments. In the urban setting, ground-level vistas lose clear perspective lines due to fragmentation by both the activity of people and competing street furniture. Bollards in these locations require particular characteristics to enable them to compete for visual space in a cluttered environment and fulfil their intended role.

Urban bollards need to maximise visibility to ensure they are readable elements in the landscape. Mass, size and strong graphical design will help pedestrians read a bollard's intended purpose.

3.2 Thresholds

Rigid structure and signifiers of urban language inform and regulates pedestrian behaviour along controlled footpaths. Pedestrian behaviour in urban environments is more chaotic when pedestrian traffic is unrestricted – such as a market square.

In this environment, bollards can act as a semi-permeable boundary to mediate between pedestrian-only zones and zones shared with vehicles. Pedestrian transition through this permeable boundary creates a sense of safety and welcomes the pedestrian without the conventional constraint of kerbed lanes used to control vehicle movement.

3.3 Bollard Design

More visually conspicuous bollards offer an increased sense of protection for both pedestrian and driver. This can be achieved through greater visual mass, size and a strong graphical impact. Design considerations for urban bollards include:

- **fixing method** – bollards may be surface-mounted to existing paving rather than penetrate it; fixing mechanisms need to be considered as part of the overall bollard design
- **durability** – edges and joints need suitable resilience to weathering without degradation
- **context** – the design language should reference the environs and the shape; the form and the materials should be of an urban scale and design detail
- **adequate passage** – allow suitable threshold between bollards for the intended level of pedestrian flow; thresholds might be deliberately reduced where additional pedestrian caution is required.

4 Esplanade Bollards

'Pedestrianisation' has been used as a mechanism to revitalise city centres, industrial landscapes and waterfronts. Malls, boulevards and river walks have become increasingly popular. Even though many of these spaces have been vehicular streets, esplanades tend to be strongly pedestrian in focus. Long vistas result in an aesthetic language that is based in formality and repetition.

Unlike the urban market place, pedestrian movement along esplanades is characterised by the linear flow of the promenade. These are generally open carriageways that may be shared by vehicles and pedestrians. Bollards can be used as merely a visual indicator of traffic flow or be spaced close enough together to physically restrict vehicle access.

Typical functions of bollards on esplanade environments include:

- **visual dominance** – usually there is little competing street furniture
- **pedestrian control** – providing general directional guidance, and then used at key junctions to draw pedestrians' attention to key conflict points
- **vehicular control** – providing barriers to low-speed vehicular traffic ways
- **protection** – for existing features such as planting areas or protecting pedestrians by denoting hazards.

4.1 Designer Requirements

Because esplanade bollards are close to people and often with little competition from other outdoor furniture, they can be viewed more as furniture than being primarily a barrier. As such, their form and finish can be designed to be more tactile, approachable and interactive with the pedestrians.

Critical design issues include:

- **path width**: The threshold width on esplanades is often narrower than other pedestrian paths due to their consistent lineal nature. Narrowing path width at points of conflict can focus pedestrian attention and control walking speed.
- **bollard design**: The tendency for longer runs of bollards along esplanades increases their visual legibility. The bollards can be of a finer scale and simpler detail.

5 Bollard Design

The use categories for bollards discussed in earlier sections give the parameters for function and some visual demands. Design of the bollard itself needs to knit these visual and functional demands together with detailing that allows timber to perform at its best.

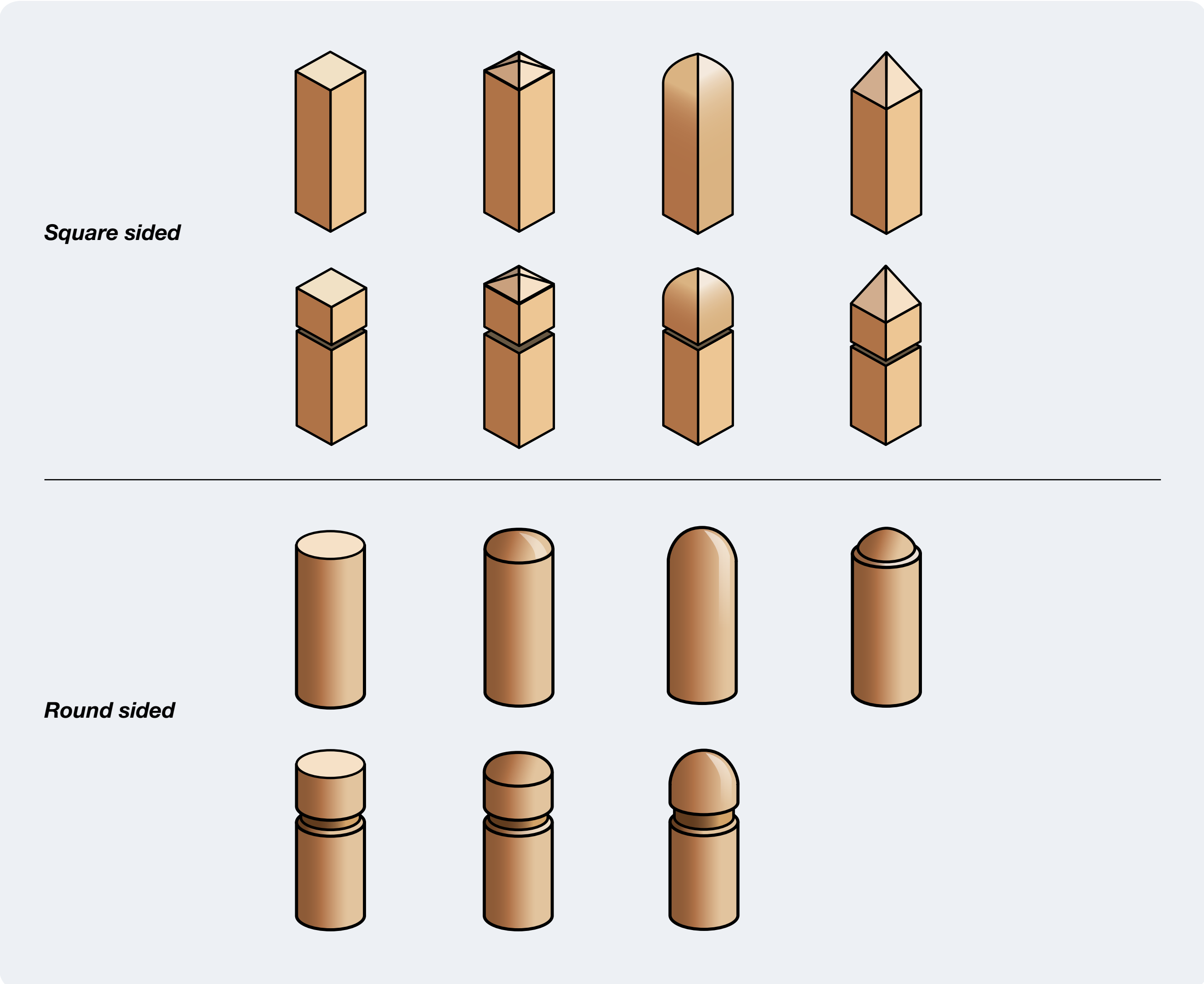


Figure 5.1: Uncapped bollards. The detail of bollard ends can be used to give a distinguishing style and, in many cases, minimise weathering.

5.1 Determining Size

Larger posts can be striking in appearance and have been seen in older buildings and wharfs, evoking a sense of tradition. However, large members need careful consideration of timber sourcing.

As with many materials used in the built environment, attention must be given to the effect on the natural environment from which the raw materials are drawn. As an example, the square post in Figure 5.2A has considerable width and is hewn from one piece of timber. The timber shown contains no heart (otherwise known as pith or corewood) and the growth rings suggest that it was cut from a log that was about two metres in diameter. A tree of sufficient size to provide such a log are no readily available.

A: Mass bollards – Mass bollards from single large timber members risk environmental consequences.

B: Composite alternative – Two wide members are fixed to a central steel fin, providing a mass appearance.

C: Composite alternative – Fixing multiple smaller members in conjunction with stainless steel fins and capping provides a sophisticated alternative to single mass timber elements. Those pictured provide crash protection at an airport.

Figure 5.2: Large saw logs come with higher environmental cost. Specifying larger members free of heart can require cutting very large trees .

5.1.1 Heart vs Heartwood

The centre of a tree is called ‘heart’; in sawn softwood it is commonly referred to as ‘pith’ or ‘corewood’. Designers often confuse the term ‘heartwood’ with wood that contains ‘heart’ due to the similar sounding terms, though they describe two different parts of the tree. Heartwood, and sometimes termed ‘truewood’, is that portion between the heart and the sapwood (see Figure 5.3).

In hardwood, this inner pith is frequently at least is 50 mm either side of the centre. It is important to note that hardwood that contains ‘heart’ (pith) is particularly vulnerable to splitting and therefore degradation.

Sapwood, heartwood and the tree heart need to be considered differently in hardwood, cypress and pine. The chemical preservation needed to adequately protect timber from decay due to weathering is generally only absorbed by sapwood, leaving any exposed heart vulnerable to weathering.

- **Hardwood** – While some naturally durable hardwoods will have highly durable heartwood, the sapwood itself will quickly decay if not treated.
- **Softwoods generally** – Sawn pine must be incised to allow adequate penetration of chemical preservation of the heart, though once treated, the heart (pith) can be considered structural. Pine ‘rounds’ can be used where the pith is shielded by treated sapwood; the sapwood will decay very quickly if not treated.
- **Cypress specifically** – While cypress heartwood offers good natural durability, cypress sapwood doesn’t absorb waterborne chemical treatment and so it can’t be preserved at all.

Table 5.1: Durability of timber for bollards.

Timber	Sapwood	Heartwood ('truewood')	Heart/pith
Hardwood	Not durable - can be treated	Sometimes durable - cannot be treated	Unstable - cannot be treated
Cypress	Not durable - cannot be treated	Durable - cannot be treated	Structural - durable, cannot be treated
Pine	Not durable - can be treated	Not durable - cannot be treated unless incised	Structural - not durable cannot be treated

When hardwood bollards contain heart it is important that it is shielded from weathering by capping the top of the bollard with a material that protects from moisture. Normally, a sloping top will suffice for pine and cypress. Thin aluminium plates are used to similar purpose to protect the top of steel electricity poles.

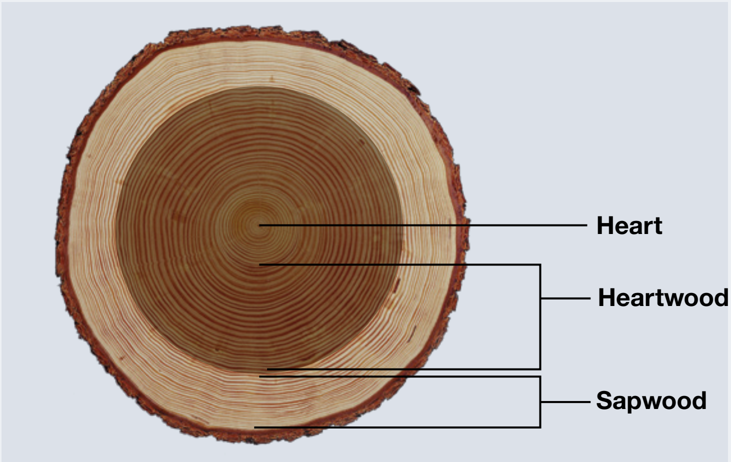


Figure 5.3: The very centre of a tree is where early growth took place and is known as the heart or pith. Wood that contains this 'heart' is quite different from the 'heartwood' that surrounds it. The heart is vulnerable to splitting, while the heartwood is the more durable wood.

Development of forestry practices in Australia has led to more responsibly harvested timber being available but generally in smaller stem (saw log) sizes. The durability required for external timber calls for hardwood members up to 175 mm x 175 mm to be free of heart (pith). Designers can encourage better practice by specifying alternative members such as:

- **smaller sizes** – without heart (pith)
- **allowing heart** – pith centres are permissible in larger sizes where these are suitably protected with a top cap (pith should be shielded by being at the centre of the member)
- **uniform size** – consistent member sizing makes fitting of protective caps easier
- **improved detailing** – see discussion below on expansions joints
- **sustainable resource** – consider selection of member size with realistic availability of plantation timber.

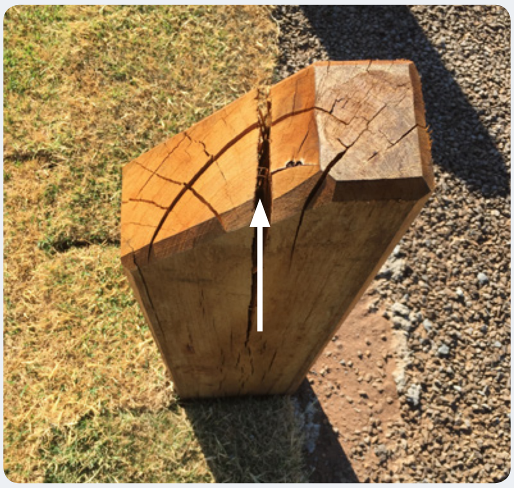
5.1.2 Protecting Pith Centres

The Australian Standard *AS 2082: Timber - Hardwood - Visually stress-graded for structural purposes* now allows smaller members to contain heart (pith) in many common species in all sizes. Previously, the Standard only allowed heart in 175 mm x 175 mm and larger, though it is common for even bollards as large as 200 mm x 200 mm to split lengthways when they contain heart. This can result in unsightly, random tears that many asset owners object strongly to and which can considerably hasten deterioration due to weathering.

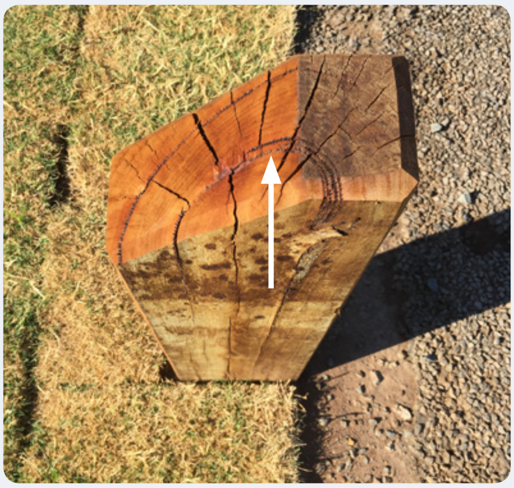
If the timber to be used contains heart, designers should consider using a sufficiently large size to encapsulate the pith and ensure this heart is more at the centre of the member. Splitting will not be avoided but can be controlled by expansion grooves.

A: Radial splitting through pith

B: Splitting in an arc on the heart and heartwood border



C: No connecting tissue



D: Gum vein with connective tissue

Figure 5.4: Exposed pith (heart) resulting in splitting. Timber heart should be excluded from the member or fully concealed. If present, exposed heart is likely to result in severe splitting leading to early degradation of the bollard.

5.1.3 Size Considerations

Uniform size

Dimensional tolerances for sawn timber mean that bollards ordered at a set size will potentially each have a slightly different dimension. The sawing tolerance of a standard rough-sawn 200 x 200 mm post used to be +8 mm and -3 mm. Although this has tightened to +/-3 mm, the author's experience is that the old standard more accurately reflected what will be supplied. This subtle inconsistency can make fitting premade protective end caps difficult. One solution is to specify dressed timber or consider smaller caps to allow for dimensional variation (refer Figure 5.7B for an example).

Appropriate width

Bollards used for impact protection need serious consideration of the nature and speed of any impact threat. Advice from a structural engineer is suggested. Beyond structural considerations, other size determinants are listed below.

Increasing a bollard's thickness will allow for considerable degradation from weathering to occur before affecting the bollard's performance.

Where the appearance of a larger bollard is desired, it is possible to gain bulk by forming a composite of smaller, and thus more readily available, members. Figure 5.7B shows an eloquent example of such designs.

Considerations for bollard size are:

- **function** – need for resisting vehicle impact, etc
- **visibility** – larger sizes are more visible (though using smaller members in series increases visibility)
- **proportion** – bollards used to support signage, safety reflectors, etc, may need to be wider
- **weathering** – greater width can give more protection to central pith and disguise warping that may occur from shrinkage.

See *Appendix A: Selection and Construction Notes* for more on particular-sized bollards.

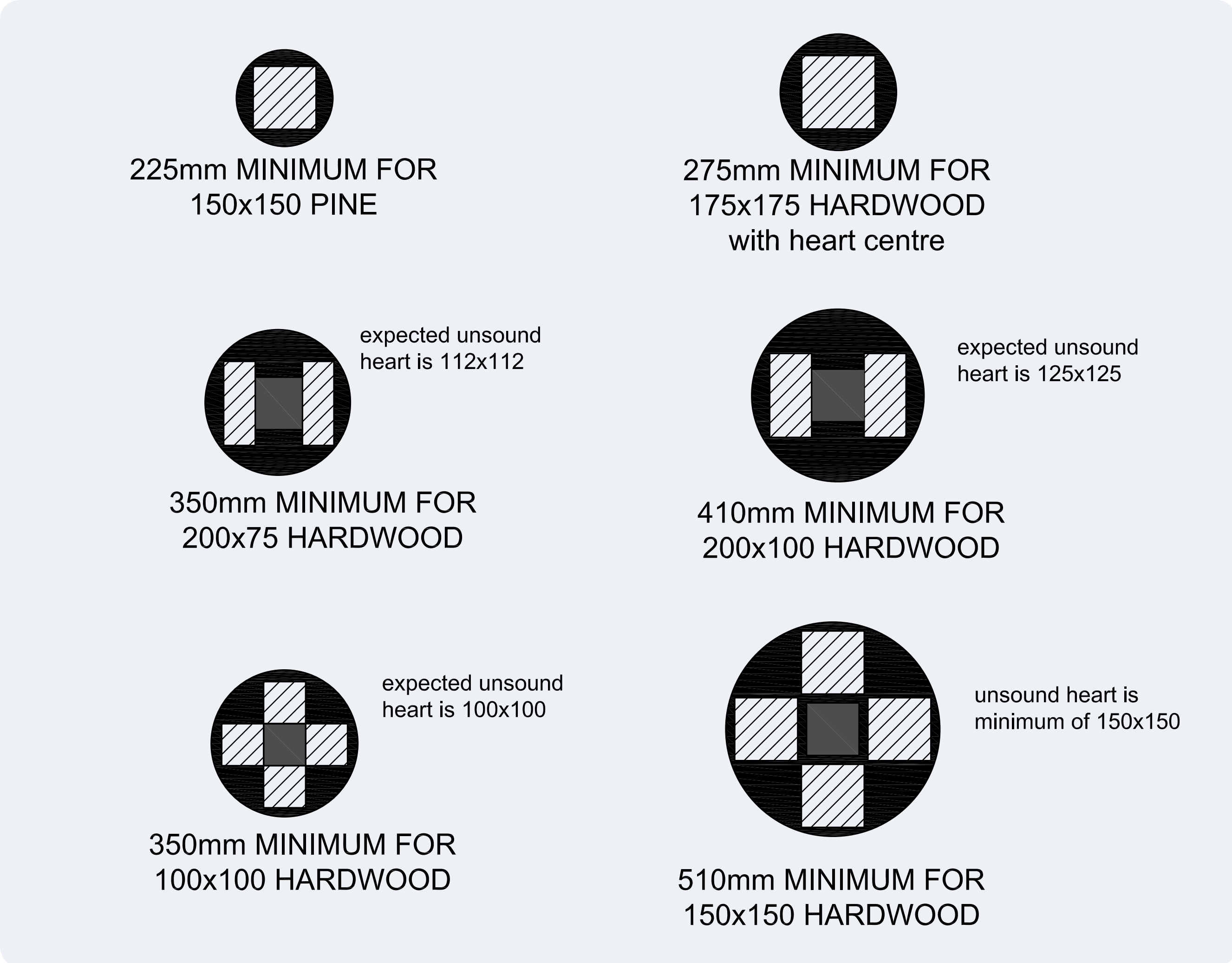


Figure 5.5: Wider bollards require wider saw logs. Using smaller sizes requires smaller sawlogs. This increases sourcing options, especially from more sustainable sources.

Bollard height

The length of bollard will depend on its in-ground length and desired height. For non-surface mounted bollards the in-ground depth would generally be 600 mm (see Section 6 Bollard Installation). The above-ground height will be controlled by a number of considerations:

- **protection** – if a bollard is merely to protect from vehicle collision it might only need to be low
- **safety** – the height needs to be sufficient to avoid trip hazards and conspicuous enough to be noticed amid the anticipated surrounding pedestrian/vehicular traffic
- **visibility** – consider the height of signage sight lines, undulation of ground surface and the needs of people who are visually impaired.

Some visually impaired people have only a narrow cone of vision that means they may not see a bollard they are approaching until they collide with it. The national transport standards document *Disability Standards for Accessible Public Transport* concerns itself with public transport and related infrastructure. This Standard prohibits the use of bollards in pedestrian spaces. The understanding of what is considered a pedestrian space can vary and the use of bollards within and at the borders of pedestrian spaces needs careful consideration. In some cases, bollards 1,200-1,500 mm high with very high luminance contrast have been used. Advice should be sought on the location and height of bollards in relation to disabled access and visibility.

5.2 Bollard Colour

Bollard design might involve the use of colours as part of aesthetic considerations or even to form sculptural feature bollards. Applied finishes such as penetrating oils and paints are added to protect the timber from weathering.

Apart from aesthetics, the choice of bollard design should consider safety concerns for visibility including those for vehicles and pedestrians. This includes the visually impaired. The previously mentioned Disability Standards for Accessible Public Transport 2002 says where 'poles and obstructions' might be considered obstacles they need to be made more apparent. This Standard says: "Obstacles that abut an access path must have a luminance contrast with a background of not less than 30%".

5.3 Bollard Detail

Timber will move over time as it seasons and some surface checking on the face of heart free members is likely. Extra consideration will need to be given to contain this movement in larger bollards with heart in the centre where splitting can occur (Figure 5.6BA). These larger bollards can be sawn longitudinally along the face (say 25 mm deep and 3 mm wide) and the edges of that cut arrissed as seen in Figure 5.6b. Bollards that are more exposed to close pedestrian traffic will benefit from pencil-rounded edges to avoid splinters. Bollard detail design considerations include:

- **expansion joints** – routed grooves can enhance aesthetics while providing a controlled and neat path for any surface cracking that occurs as the timber ages
- **pencil round** – rounding all edges increases safety (also consider rounding expansion grooves)
- **signage** – utilising at least one face to allow for engraving of wayfinding and signage symbols
- **end caps** – offer weather protection for timber containing heart, while enhancing aesthetics.

A: Uncontrolled splitting – Bollards that contain heart (pith) frequently split along their length.

B: Controlled movement – Using a router to provide a groove over a sawn line controls surface splitting and avoid splinters.

C: Deterioration at heart – Traditional steel banding to bollard ends does nothing to protect the post from weathering.

Figure 5.6: Avoidable failure in large hardwood bollards. Incorrect use of heart in timber bollards leads to splitting and deterioration.

Dressing considerations

Dressed horizontal timber surfaces deteriorate more rapidly than rough-sawn surfaces. In the case of bollards, however, the bulk of the exposed timber is vertical and so a rough-sawn finish is less valuable than it would with decking, for example. Durability aside, dressing offers benefits such as:

- **preservative treatment** – reveals timber grain after preservative treatment
- **machine worked** – allows for accurate indexing
- **multi-faceted cutting** – gives a more even alignment.

The decision to dress the timber used for bollards will be driven primarily by aesthetics. If the timber is to be treated with chemical preservatives before being dressed, the natural grain colour will be evident. If left rough-sawn, the colour of the treatment will be evident rather than the grain. If dressed then treated, the colour of the grain is visible

Creating bollards for signage or with complex shapes may require the timber to be processed in computer-controlled equipment such as CNC routers. In this case, the timber needs to be dressed to enable the machine to index accurately when working from two or more sides.

Multi-angled tops of bollards, such as pyramids, can be difficult to cut accurately on rough-sawn timber. The irregular size of a rough-sawn bollard makes getting a neat central diamond difficult. In this case, dressing 6 mm undersize rather than the normal 5 mm will assist accuracy by avoiding hit and miss on the dressing.

Rounded and arrissed edges

Providing a pencil rounded edge with a router provides a neat edge. It is more difficult to achieve a neat edge with a planer and splintering can occur on the two edges created when arrissing. If stainless is being used for a cap that the author recommends 316 grade to avoid tea staining.

5.3.1 Protecting Bollard Ends

Bollard caps act primarily as protection mechanisms of the underlying bollard structure and are essential for all timber bollards that contain heart. They also give the opportunity enhance the appearance of bollards.

Haptic design

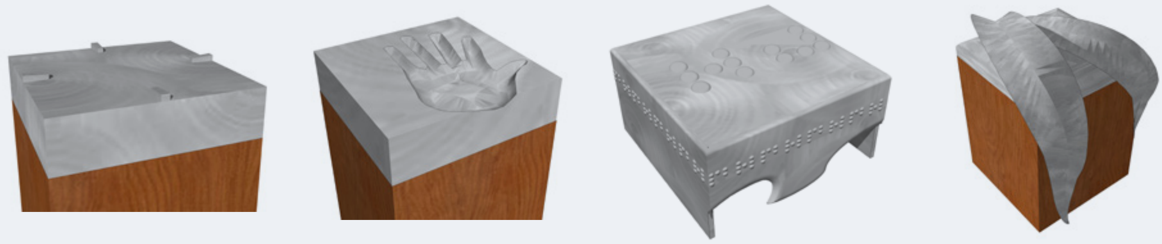
Although designed as barriers and to direct and protect, bollards also have a role as furniture. Social interactions in the environment tend to occur at points of transition. It is at these points that bollards are more likely to be touched by pedestrians, giving the opportunity for the bollard cap to communicate via touch. Bollard caps can be designed to generate a sensation in addition to enlivening the bollard's visual form. Through what is known as 'haptic' design, components designed to come in contact with people can be used to:

- **invite touch** – be visually identified by the use of contrasting material, form or texture
- **communicate** – braille can be used at key nexus to communicate to visually impaired people.

A: Over-fitted – Simple push-on end cap requires accurate symmetrical post cutting and ideally discrete fixing.

B: Flush-faced – Rebating bollard ends allows caps to fit flush with column sides, giving a more complex but neater finish.

Figure 5.7: Using metal bollard caps protects timber end grain from weathering and enhances aesthetics though it requires considerable accuracy in bollard cutting and cap fabrication to ensure a good fit. Caps can be chemically bonded (though with limited success) in place or mechanically fixed from the side. The use of security or one-way screws can avoid vandalism. Photographs: Dennis Clark Photography



A: Basic protection – Caps fold over edge of bollard ends to shield from weathering.

B: Inviting touch – When used in heavily pedestrianised areas caps invite touch.

C: Braille text – Adding braille can transform a bollard into an information point.

D: Sculpture – Identity and interest can be given by the addition of sculptured forms.

Figure 5.8: Bollard caps protect and communicate. Providing caps to protect the most vulnerable top face of bollards from weathering also creates an opportunity for tactile and visual design.

5.3.2 Practicalities of Cap and Strap Design

Finger entrapments and caps

For playground equipment, gaps between 8 mm to 25 mm are considered finger entrapments; the ACT government works on 6 mm to 25 mm. Bollards are not playground equipment but it is difficult to think of other standards that can be referenced. The response to finger entrapment risk is up to the asset owner but the issue should not be treated lightly. After less than a year of shrinkage, the bands around some bollards can loosen to a point that there are finger entrapment risks. Consider the advisability of using bands that can potentially develop finger entrapment.

Figure 5.9: Large gaps under straps.

Caps illustrated in Figure 5.9 are made from 5 mm stainless steel. They do not have a simple 90-degree bend but rather taper over the vertical faces. This more complex shape is likely to be the reason that the caps' internal measurement is not constant.

Cap variability, checking out the top and shrinkage has led to finger entrapments under some caps illustrated in Figure 5.10. This will worsen as the timber shrinks further.

Figure 5.10: Finger entrapment.

Some caps are installed using two screws on opposite faces. This allows the caps to rock (see Figure 5.11).

Figure 5.11: Misaligned caps.

For bollards, the author suggests the top not exceed say, 250 mm and rebate the top approximately 5mm to a depth of 50 mm. Use much thinner material for cap than shown in Figure 5:9, say 1.2 mm, fixed with four screws positioned away from any expansion grooves. This will flex with the continued timber shrinkage.

5.4 Bollard Layout

When bollards are designed as part of a landscape they may function singularly but frequently have impact through their numbers. Beyond the multiple functions discussed in the Park, Urban and Esplanade use categories there are general factors to consider in their placement, including:

- **in series** – visual route markers should be spaced close enough so as to be clearly read en masse and generally positioned on a smooth line – aligned with that of adjacent path or boundary
- **individual** – located at key junctions and may be used to obstruct pathways in order to be more obvious
- **protection** – closely spaced to physically disallow vehicles or to discourage pedestrians
- **universal access** – given the evolving awareness of needs of people with a disability and corresponding standards and legislation, gaining specific design advice from an appropriately qualified disability consultant is advisable.

6 Bollard Installation

As discussed in the Introduction, there are several common ways of mounting bollards:

- **surface mounted** – bolted to the ground plane
- **embedded** – base buried in ground
- **kinetic** – rebounding, retractable and removable bollards

Setting timber posts in concrete could almost be classed as universal practice due to its simplicity but this is bad practice in the case of hardwood and cypress, where trapped moisture can decay bollard bases.

Retractable bollards are generally proprietary items and their installation requires specialist detailing from manufacturers.

6.1 Surface-mounted Bollards

Surface-mounted bollards are commonly fixed with steel plates bolted to a paved ground plane. Surface mounting avoids major penetration of paved surfaces, avoiding deterioration that would otherwise be exacerbated by moisture penetration.

As a bollard's length allows considerable leverage to be exerted, base fixing brackets need enduring strong construction. Welded steel plate brackets provide this strength and can be readily protected by hot-dipped galvanising. A 6 mm steel blade welded to a 12 mm base with four M16 stainless steel chemical anchors is adequate for most standard applications. Where a specific load must be resisted, seeking advice from a structural engineer is advisable. Considerations for surface-mounted bollards include:

- **shrinkage** – to allow shrinkage of bollard timber the base can be constructed slightly smaller (say 6%)
- **hole size** – laser cut holes to support brackets allows greater accuracy, which is especially important if holes are to align with CNC-produced timber
- **base-plate purchase** – fixing base plates a minimum of 125 mm from the edge of the concrete reduces risk of pavement cracking
- **safety** – using 'captive nuts' with rounded heads avoids the danger of protruding bolts to pedestrians passing by.

6.2 Timber without Concrete Footings

Timber bollards can be founded in earth without footings and this is often done to avoid timber decay from any moisture that may become trapped in a concrete footing. Considerations for founding bollards in earth include:

- **timber durability** – consider timber durability and any preservative treatment is adequate to achieve required in-ground durability rating
- **natural earth** – consider bedding in the earth foundation material if it is suitably free draining and provides sufficient resistance to overturning
- **crushed rock** – if the foundation material is not free draining, fine crushed rock such as crusher dust can be used
- **sinking depth** – bollard bases should be buried 600 mm in ground to avoid overturning (less than this can lead to failure as shown in Figure 6.1)

Figure 6.1: Bollards buried in ground should aim for 600 mm deep to avoid overturning.

6.3 Timber in Concrete Footings

Cypress and hardwood bollards inserted into concrete footings are initially more secure, but as the timber shrinks, the developing gap between the post and the concrete can harbour moisture. A common cause of decay for timber bollards with concrete footings is irrigation and/or rainwater seeping down the gap around the bollard and being held by capillary action against the side of the bollard. Any fertilisers present will nitrify irrigation water, which further promotes timber decay.

Using timber of suitable natural durability will not compensate for inappropriate installation. Great care is needed when specifying concrete bollard footings as hardwood and cypress degrade more quickly when encased. To date, the author is not aware of decay problems related to correctly treated pine set into normal concrete (containing sand in the blend) footings. Considerations for using concrete footings with timber bollards include:

- **durable species** – with an 'in-ground' durability rating of 1 or a high 2 (the author has had considerable success with spotted gum, strictly a 'durability 2 in-ground' timber in tropical and subtropical climates)
- **permeable concrete** – use of 'no fines' concrete for footings as it is made without sand and thus allows moisture to drain through it
- **moisture cap** – rather than bringing a concrete footing to ground level, a clay cap (minimum 100 mm depth) can be added to prevent infiltration of surface water into any gap between bollard and footing that may develop as the timber shrinks
- **wrap in-ground timber** – suitable plastic pole bandages applied to lower durability species such as jarrah will offer some protection from decay
- **inhibiting biodegradation** – some products include pellets of slow-release active ingredients that inhibit biodegradation, but caution is needed as bandages can create a detrimental microclimate that can exacerbate timber decay.

WoodSolutions Design Guide No 41 *Timber Garden Retaining Walls* makes recommendations for construction of concrete footings for retaining wall design. The recommendations are also suitable for bollard applications, say: "No fines concrete shall be 10 mm maximum aggregate size, 450 kg cement per cubic metre and a water cement ratio of 0.55. The concrete shall be ready mixed or hand mixed manufactured to the requirements of AS 1379. For no fines concrete the concrete shall be well agitated immediately before placing to ensure a complete coating of the aggregate. The concrete shall be discharged directly into the holes and tamped without delay. All concrete shall be placed within one hour of batching. The no fines concrete shall not be reworked as this destroys the bond."

This clay capping method is more important for structural applications due to potential danger of failure. For large, free-standing, in-ground timber structures - such as totem type poles – seek advice from a specialist timber engineer.

Failure through decay – Decay in this timber bollard has been hastened by the post base being set in concrete. The bollard is under 20 years old.

Figure 6.2: Providing a free-draining foundation for bollards limits the potential for moisture build-up and thus decay.

7 Timber Durability

Timber is a natural product that, if left to weather, will in time break down to organic matter. There are many different situations and applications in which timber can be used, and bollards that are both buried and weather exposed are one of the most demanding. Timber may deteriorate through the action of insects, fungi and marine-boring organisms. This deterioration can be prevented if conditions are made unsuitable for these destructive agents.

Timber preservation uses chemicals that improve the natural durability of the timber, while rendering the material unpalatable to insects, fungi and marine borers. Correct specification of timber with appropriate treatment provides timber bollards that are durable and which endure and age well.

7.1 Natural Durability

The natural durability of timber is classified according to the heartwood's resistance to deterioration. The sapwood of all timbers, softwoods and hardwoods, is always non-durable and will rapidly deteriorate if not protected.

Natural durability refers only to mature outer heartwood. Resistance is given by the presence of special tannins, oils, resins and extractives in the heartwood that repel or kill insects and decay.

Australian Standards defines timber durability of in-ground and above-ground use and for treatment levels. AS 5604 *Timber – Natural durability ratings* gives four classifications of natural durability with a rating of one being the highest. Refer Table 7.1.

Table 7.1: Probable life expectancy for natural durable timber

Durability Class	Probable in-ground life expectancy (years)	Probable above-ground life expectancy (years)
1	Greater than 25	Greater than 40
2	15 to 25	15 to 40
3	5 to 15	7 to 15
4	0 to 5	0 to 7

Adapted from AS 5604: Timber – Natural durability ratings.

7.1.1 Hardwood

Sapwood is the only portion of the hardwood that can be treated with chemical preservatives as it contains open passages that allow chemicals to travel into the timber's cellular fabric. As mentioned previously, treating timber with preservatives does lift the durability of sapwood, but it offers no chemical protection to the remaining timber.

Sawn hardwood bollards invariably contain only a small amount of sapwood. Figure 7.1A illustrates the amount of sapwood that can be treated in a typical batch of hardwood timber. If the timber was not treated and the sapwood was allowed to completely decay – the bollard would still be within recognised structural limits. The Australian Standard *AS 2082: Timber – Hardwood – Visually stress-graded for structural purposes* allows up to 20% of the cross section to be missing from most grades. Timber with this extent of decaying sapwood is unattractive. The treatment process on sawn timber bollards serves an important task of stabilising the sapwood but it does not preserve the critical heartwood portion either above or in the ground.

Vague specifications for bollards that consist merely of a timber strength and treatment level are frequently seen. A request for example of "F14 treated to H5" will say nothing worthwhile for determining the durability of the timber. Treatment will not compensate for inappropriate specification and installation.

Species selection, grade and correct installation are far more critical for longevity than treatment alone. That said, treatment will prevent the sapwood decaying.

Table 7.2: In-ground durability for Australian species.

In-ground durability class	Description	Common Australian Species
Class 1	Timbers of the highest natural durability, may be expected to resist both decay and termite attack for at least 25 years and up to 50 years	Grey Box Grey Ironbark Red Ironbark Tallowwood White Mahogany Yellow Box
Class 2	Timbers of high natural durability, may be expected to have a life of about 15 to 25 years.	Jarrah Red Box River Red Gum Spotted Gum White Cypress Yellow Gum
Class 3	Timbers of moderate durability, may be expected to have a life of about 8 to 15 years.	Broad-leaved Peppermint Brush Box Southern Blue Gum Sydney Blue Gum Yellow Stringybark Western Red Cedar
Class 4	Timber of low durability, which may last about 1 to 8 years. These timbers have about the same durability as untreated sapwood, which is generally regarded as Class 4, irrespective of species.	Candle bark Douglas Fir (Oregon) Hoop pine Manna Gum Mountain Ash Radiata Pine Slash Pine

Adapted from AS 5604: Timber – Natural durability ratings, Table A1

7.1.2 Softwood Bollards

While correctly specified hardwoods can have greater durability than softwoods, it is possible to successfully use treated softwoods for bollards. Unlike hardwood, the chemical treatment of softwood is a must. Surface penetration of preservative treatment is a particular issue in softwoods, which is aided by incising. Additional to chemical treatment, timber durability can be optimised with the following considerations:

- **profile shape** – using round elements to avoid cutting through untreatable heart
- **incise timber** – softwoods should be incised to the correct depth prior to treatment
- **two-stage treatment** – steaming of timber prior to preservative treatment increases penetration.

Profile shape

Common softwood, such as pine, has very poor natural durability with an in-ground rating of 4.

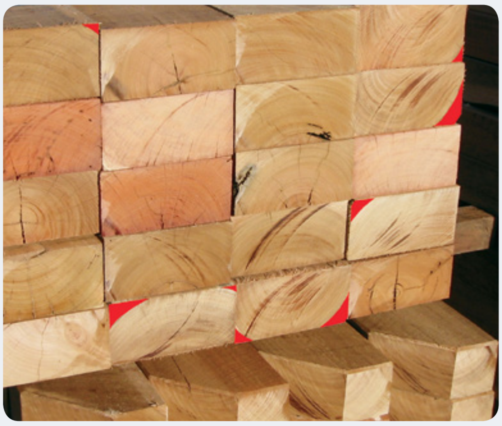
When round members are well treated, an outer unbroken band of sapwood can provide very effective envelope protection around the untreatable heart. Square-sawn members, on the other hand, particularly with a large portion of heart, can have large areas of the cross section untreated due to the limitations on treatment penetration of heartwood.

Incising

Scoring the face of softwoods aids preservative penetration, but the timber needs to be incised deeply enough. To achieve the in-ground H4 rating needed for large pine bollards, they require incising to a depth of 10 mm. Figure 7.1B shows an example of how too-shallow incising can limit the penetration of preservative into softwood.

Steaming

While steaming softwood prior to treatment can improve the preservative penetration, not all timber preservers do so. There is no substitute for incising and world-wide suppliers are increasingly seeing the cost of steaming and correct incising as a part of the cost of doing business. The ends of the packs can be easily inspected to confirm the level of preservative penetration. If more than 20% of the cross section of non-incised timber is untreated it should be rejected (the opposite to hardwood).



A: Extent of treatable sapwood in hardwood. Picture shows sapwood (highlighted in red) in untreated hardwood bollards. It is not possible to speak in a meaningful way of treating hardwood sapwood beyond H3.

B: Softwood with insufficient incising. Good penetration of softwood treatment is dependent on incising to the correct depth. An applied stain detects the level of penetration of the preservative treatment. This pine bollard was incised to about 3 mm rather than the 10 mm required resulting in a centre vulnerable to deterioration.

Figure 7.1: Providing a free-draining foundation for bollards limits the potential for moisture build-up and thus decay.

7.2 Specifying Preservatives and Hazard Levels

The treatment of timber with preservatives is concerned mainly with the protection of sapwood. The amount of preservative required is expressed as its Retention Level.

Australian Standard AS 1604: *Timber – Preservative-treated – Sawn and round* and often used in building structures. The standard provides strict guidelines for the amount of chemical preservative required in the sapwood of timber in order for the wood to perform as per its designated durability category. There are six categories with category six being the highest. Designers and constructors are best guided by AS 1604, which describes six main exposure and biological hazards as shown in Table 7.3.

In any particular charge (the term used to describe an individual batch of timber being treated) of treated timber there will be a range of preservative penetrations and retentions depending on the moisture content, sapwood to heartwood ratio, species, treatment schedule and inclusion of additives. Table 7.3 shows the treatment requirements considering the timber species natural durability.

Table 7.3: Exposure & biological hazard levels.

Hazard Level	Exposure & Biological Hazard	Typical Use
H1 – interior above-ground	Completely protected from weather and well ventilated. Beetles and borers only.	Susceptible framing, flooring, furniture, interior joinery
H2 – interior above-ground	Partially protected from wetting. To prevent termites and borers only.	Framing, flooring
H3 – exterior above-ground	Subject to periodic wetting. To prevent decay, termites and borers.	Weatherboard, fascia, window joinery, exterior framing and decking
H4 – exterior in-ground	Subject to severe wetting. To prevent severe decay termites and borers.	Fencing, greenhouses, above ground portions of pergolas and landscaping timbers
H5 – exterior in-ground	With or in fresh water. To prevent very severe decay, termites and borers.	Retaining walls, piling, house stumps, building poles, cooling tower fill
H6 – marine water exposure	To prevent marine borers.	Boat hulls, marine piles, jetty cross-bracing, landing steps

Taken from AS1604.1: Timber – Preservative-treated – sawn and round

8 Specification

Clear specifications can avoid undesirable outcomes. Providing greater clarity can require additional detail, which may not be warranted with quality manufacturers or contractors. However, in public tender scenarios, it is critical to ensure the desired outcome is delivered.

Consideration for the specification of timber bollards include:

- **species** – appropriately durable and commercially available species
- **treatment type** – timber preservative type
- **treatment method** – if pre-steaming or incising is required
- **treatment order** – timber may be processed then treated versus treated then processed
- **member size** – the size of member and format (round, square or rectangular) can have a direct effect on performance and ecology of sourcing
- **pith** – if heart is allowed, the required cover by durable shielding timber is import to avoid decay
- **composite construction** – detailing how multiple members are to be combined and fixed
- **timber finish** – dressed timber may be required for increased dimensional accuracy and different appearance as discussed above
- **control joints** – sawn grooves in faces can control splitting of heart in timber
- **edges** – a choice of arrissed or pencil-rounded edges will affect both splintering and weathering
- **fixings** – the fixing material and its finish impact its durability but also specification of sleeve nuts will provide better appearance than countersunk nuts for example (see Figure 8.1)
- **footing** – where natural earth is inappropriate, specifying a footing of no fines concrete will limit moisture retention (see notes about providing earth capping to footings above)
- **presentation** – achieving surfaces low in timber characteristics (knots and gum vein) and marks, including having any working annotations sanded off.

Figure 8.1 illustrates the different products that can be delivered under the same specification, if it does not contain sufficient detail.

Left: A: Well-supplied product – Note the use of 'captive nut' sleeved fixings, which avoid potential injury. This bollard is founded in a no-fines concrete footing to maximise drainage.

Right: B: Lower-price substitute – Manufacture marks give an unsightly finish and because the composite members were cut before assembly, the bollard has an uneven top face.

Figure 8.1: The illustrated composite bollards were built on the same project at different stages. The image A bollard has four spotted gum species members bolted together to gain a mass appearance.

9 Maintenance of Exterior Timber

While weathering primarily affects the appearance of timber, in the long term it can affect durability and performance. All timber will change when exposed to the sun and rain, irrespective of species, durability classification, or whether it is preservative treated or not.

Ultraviolet light and changing moisture exposure will cause timber to weather, resulting in a loss of natural colour over time. All timber will fade to a silver-grey and its surface will become rough, with the potential for cracks and spits to develop.

Weathering protection

Treating timber with preservatives to protect from decay and/or insects does not prevent timber from weathering. However, the application and regular maintenance of protective coatings will reduce weathering. Proper finishing also helps external timber fulfil its designed function. Protective finishes form a barrier between the weather and the timber, reducing water absorption on wetting, and slowing moisture loss on drying. To protect from UV light, finishes generally should contain a pigment (light colours preferred). The pigment reflects or absorbs the UV light, and shields the timber. Options for finishing timber bollards include:

- **natural unfinished** – consider if the species chosen weathers well and with an acceptable colour change
- **pigmented penetrating stain** – ensure that they have high UV blocking and water repellent ability
- **clear finish** – consider the potential high cost of maintenance for recoating and potential re-sanding over time
- **paint** – an oil-based primer followed by two coats of top coat generally offers the best result (ensure the paint chosen has Australian Paint Accreditation Scheme approval).

9.1 Penetrating Oils

A correctly detailed bollard of the correct species and quality does not need to be oiled to ensure longevity, but designers may wish to apply a penetrating oil to improve the initial aesthetics. The only oil that offers a preservation function is Copper Naphthenate (CN). However, it has the potential to stain clothes and skin on touch.

The prime benefits of using penetrating oil are that it repels water and blocks ultra-violet radiation. UV blockers are expensive and some products can include very little of them, and some offer very little water repellence. It is important that any penetrating oil includes these properties or it will not offer the protection sought.

Some confuse penetrating oil with film finishes as, when the oils are first applied, it does have some level of gloss finish but this is frequently short-lived, especially on unseasoned timber. A lack of an apparent surface gloss does not mean that an oil is not present and/or not working. A test of the efficacy of an oil treatment is to apply water to the surface. Once the surface stops repelling moisture it is time to reapply.

If reapplication is required, the timber does not need to be sanded. A simple wash will remove dust and application of a fungicidal, if needed, will kill any mould present – following which the oil can be re-applied. This simplicity and relatively low cost makes maintenance a possibility.

Unseasoned hardwood will not allow the oil to penetrate deeply so for best results apply a coat when the bollards are installed and then again prior to handover. The timber can be left to weather to a typical silver grey colour over time.

A good quality oil should contain ingredients such as:

- **suitable oil/resin** – often resins are modified to have characteristics that address the limitations of natural oils, including mould growth in linseed oil and lanolin
- **UV absorbers** – offer protection to timber substrate and resin/oil system
- **water repellents** – can vary significantly in type and quality; it is best to have repellents that are not prone to mould (issues with linseed oil and lanolin)
- **mould and algae inhibitors** – will not remove or prevent mould from pre-infected timber
- **solvent** – provide the carrier that helps the oil's ingredients penetrate the timber.

Solvent system

There are many types of solvent systems. Many penetrating oil formulations contain petroleum-based solvents of varying flash points (degree of flammability) and levels of aromatics as well as those that have surfactants to allow water to be incorporated.

The best penetrating oils contain a petroleum-based solvent system as these are more able to penetrate timber and less prone to facilitate movement of tannins contained in the timber to the surface. A product that incorporates a solvent with a high flashpoint and low aromatics would be preferable. A high flashpoint (>60.5°C) will mean the product will not be considered flammable, reducing risks for transport, storage and use. A low aromatic solvent will reduce odour and potential health risk often associated with using solvents. While these features do not necessarily add to the quality of a product, they do provide benefits that make oil-based, penetrating oils more amenable to use and hence get the best result in timber.

Figure 9.1: Timber durability is enhanced by applying appropriate paint and oil systems.
The bollard shown is protected with a penetrating oil that repels water and blocks ultra-violet radiation.

10 Learning from Case Studies

10.1 Park Perimeter Bollarding: Gatton Region

- **Function**: Bollards and low height barriers used to define the parks perimeter
- **Material**: Spotted gum and blackbutt hardwood species
- **Key learning**: Durable species selection needs to be paired with supply of timber with no heart

Figure 10.1: Original timber barriers at Lake Apex, Gatton, Queensland. The exposed hardwood shows some weathering but is still functioning after 30 years exposure, some areas experienced extended areas of submersion.

The Lake Apex Park in Gatton is a council park that was enclosed by a hardwood post and rail fencing when the park was first developed in 1984. The author was involved in providing the timber for this project and, as a local, has seen how it has endured. After more than 30 years, the hardwood bollards and fencing endure. Based on the performance, it appears the barriers could have had a 50-year service life but were replaced with fresh timber bollards in a redevelopment of the park in 2018.

In the interim, recycled plastic bollards have been added adjacent in the park. Recently, new hardwood bollards were installed in nearby parkland, which provide a useful comparison to the original hardwood construction. The original hardwood rails and posts installed were a mixture of spotted gum and blackbutt species.

Although the original hardwood has endured and performed well, there are still lessons to be gained on improved supply and construction methods.

Portions of the fences were submerged repeatedly in the adjacent lake for long periods as the lake levels fluctuated, which led to more sever deterioration and were replaced after approximately 30 years.

Performance of rails

Of the visible weathering, the greatest deterioration is on the horizontal rails, as would be expected. The deterioration became apparent on the top face of the 200 mm x 50 mm rails after about 15 years of exposure to the warm and humid South East Queensland climate. The deterioration detracts from the appearance of the rails but does not affect their function. Suggested improvement:

- **Current**: rough-sawn rails with arrissed edges
- **Improvement**: dress the top of the rail with a moisture shedding top with a 6 mm rounding to the edges.

In-ground performance

Even at this stage, decay at the ground line is only slight. This has been helped by the posts being installed with a natural earth as opposed to a concrete footing that could harbour moisture. Suggested improvement:

- **Current**: 200x100 mm posts buried to about 600 mm in natural earth footing
- **Improvement**: good performance – none required.

Termite resistance

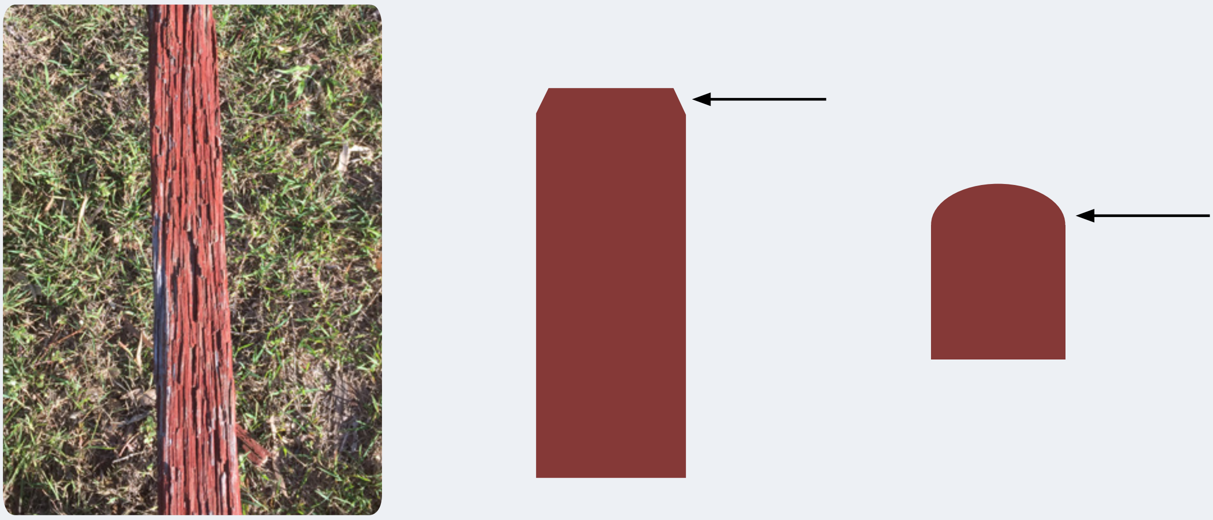
Because termite-resistant species were used, termite attack has been slight. Only two posts (i.e. less than 1% of the total) appear to have any termite damage. Suggested improvement:

- **Current**: naturally termite-resistant timber
- **Improvement**: good performance – none required.

Maintenance

The fencing was given a coat of protective stain in recent years, and a couple of rails have been replaced due to vehicle impact. Apart from that nothing has been done, nor needed to be done.

- **Current**: no finish was applied when installed, and one known application since
- **Improvement**: good performance – none required.



A: Top rail weathering – Some degradation apparent in more than 30 years of weathering.

B: Built top rail profile – A flat-topped member with arrissed edges was installed.

C: Improved profile – Providing water-shedding profile to top surfaces considerably enhances durability.

Figure 10.2: Although these hardwood rails have endured, water-shedding top edges would have further increased their life.

Plastic bollard comparison

A decade ago the local authority installed a recycled plastic bollard a few metres from the 1984 timber installation (figure 10.3B). Despite the poor weathering of this bollard, in 2015 a large number of plastic bollards were installed. Vehicle strikes have seen more than a dozen of the plastic balustrades sheered to the ground (Figure 10.3A). Figure 10.3 illustrates the comparative sturdiness of the three decade- old hardwood barriers, which have survived vehicle collisions.

When the council was queried about the choice of plastic as material for the new bollards, the author was told: "For several reasons, the use of timber bollards is diminishing over time, namely high maintenance costs, cracking timber, splinters, bollard weight and termite/borer attack protection. Recycled plastic bollards have a longer maintenance-free lifespan, are much lighter, don't splinter and because they are made from recycled plastic they reduce our environmental burden."

Maintenance demand was also linked to perceived need to oil spotted gum bollards that were less than two years old. In the author's experience such maintenance was completely unnecessary.

Hardwood specification

In a nearby car-park hardwood bollards have been used as part of a new installation. These are 200 mm x 100 mm in section, a common dimension. Although the species used appears to be spotted gum and ironbark, a lack of grading has seen timber supplied that has been cut indiscriminately, with no concern to avoid the more vulnerable heart (pith). The result has been severe splitting in the majority of these new bollards. This splitting exposes the heart to weathering and hastens the decay of the timber. Some are so split that they create the potential to trap fingers.

Figure 5.4 shows typical images of what are largely defective bollards installed in this parkland.

When supplied for use as landscaping sleepers, similar-sized members are often cut in an indiscriminate manner that exposes the vulnerable timber heart. Exposed heart is unsuitable for structural applications and quality control is needed to preclude it. Correct timber selection for durable performance requires not only correct specification of timber species, and how heart is to be avoided when cut, but also inspection of product after grading. Even when the specification is correct there is a danger that lower-priced material will be substituted.

A: Sheared bollard – This new recycled plastic bollard was sheared at the ground by vehicle impact.

B: Reinstated – The damaged bollard is about 10 years old and is shown reinstated with the aid of a steel picket.

C: Hardwood barrier – The 30-year-old hardwood barrier is shown here intact despite having rendered considerable damage to the vehicle.

Figure 10.3: Comparison of plastic to hardwood bollard. The occurrence of vehicle collisions illustrates hardwood endurance.

11 Glossary

Common industry terminology and abbreviations:

CUAZ:	Copper and Azole is a timber preservative.
ACQ:	Alkaline Copper Quaternary is a timber preservative.
CCA:	Copper Chrome Arsenic is a timber preservative
dressed:	Off-the saw timber that has been planed to provide smooth, even faces.
extractives:	The constitute liquids which seep from green timber.
ex:	‘Ex’ sizes are the original cut size from which shrinkage, and dressing reduce in size.
heart:	The centre of a tree is called ‘heart’, and in sawn softwood it is commonly referred to as ‘pith’ or corewood. Hardwood that contains ‘heart’ (pith) is particularly vulnerable to splitting.
heartwood:	That portion of a tree between the sap and the heart.
pith:	see ‘heart’
rough-sawn:	Timber left with a rough face, as it has been initially sawn.
rounds:	Timber that has not been sawn, can either be natural or parallel sided.
sapwood:	The outer growing zone of the tree.

12 References

Australian Standards

The following Standards have some coverage that affects the timber used to construct bollards or the nature of the design and placement of bollards:

Timber

AS 1604 Timber – Preservative-treated – Sawn and round

AS 2082 Timber—Hardwood—Visually stress- graded for structural purposes

AS 5604 Timber – Natural durability ratings

Concrete

AS 1379 Specification and supply of concrete

13 Appendix A: Timber Selection and Construction Notes for Bollards

<table>
<tr><th colspan="5">Species and Preservative</th></tr>
<tr><th colspan="5">Hardwood</th></tr>
<tr><td colspan="5">The best species performance can be achieved by using the readily available timbers traditionally known as ‘Royal species’. These include spotted gum, ironbark, tallowwood, etc. In some drier climates other in-ground durability class 1 and 2 timbers species, e.g. blackbutt, may provide adequate durability. New England blackbutt is not suitable due to lower durability and higher shrinkage (refer to Figure 5.9 and 5.10).</td></tr>
<tr><th>Nominal Size (mm)</th><th>Containing Heart?</th><th>Comment</th><th>Construction</th><th>Preservative</th></tr>
<tr><td>Smallest size <100 mm</td><td rowspan="5">No Heart</td><td>• An increasing trend to reduce size to 75 mm to save cost might save as little as $15 on a typical bollard length but not installation cost.
• Using smaller members could see a 25% reduction in service life.</td><td rowspan="5">• Free of heart.
• Grade is Structural Grade 2 for all species.
• Note on grading: Inspect each piece to ensure defect is placed in ground.
• Mark base with lumber crayon before processing and make available for inspection prior to processing.</td><td rowspan="5">• Sapwood treated to H3 with ACQ or Copper azole.
• CCA not acceptable</td></tr>
<tr><td>100 mm x 100 mm</td><td>• The standard size for bollards used for many years.</td></tr>
<tr><td>125 mm x 125 mm</td><td>• Easier to source, and fewer quality consequences than 150 mm.</td></tr>
<tr><td>150 mm x 150 mm</td><td>• A historically common size.
• Not recommended, as it can be hard to supply in suitable quality.
• As ‘free of heart’ material is much more expensive than ‘heart in’, be aware that ‘heart in’ is frequently substituted.
• Consider reducing to 125 mm for a more achievable size without loss of function.</td></tr>
<tr><td>200 mm x 100 mm</td><td>• Easily substituted with landscaping sleepers that may have low quality, low durability and quickly deteriorate.</td></tr>
<tr><td rowspan="2">175 mm</td><td>Contains Heart</td><td>• AS 2082 allows many common species to contain heart but this should not be done until the size is greater than 175x175 mm.
• More prone to splitting lengthways which can expose the heart and leads to deterioration.
• Consider increasing the specification to 200 mm.</td><td>• Sound heart in the centre.
• Structural Grade 2 required for all species.
• Cut expansion joints to at least two faces, (more is better) within one week of milling.
• Inspect each piece to ensure any defect is placed in-ground.
• Mark base with lumber crayon before processing and make available for inspection prior to processing.</td><td>• Sapwood treated to H3 with ACQ or Copper azole.
• CCA treatment is not acceptable.
• Note sapwood content will be small.</td></tr>
<tr><td>No Heart</td><td colspan="3">Not available without heart</td></tr>
</table>

Nominal Size (mm)	Containing Heart?	Comment	Construction	Preservative
200 mm	Contains Heart	• Much more dimensionally stable members. • Contains proportionally less heart and are less prone to splitting.	• Sound heart in the centre. • Grade is Structural Grade 2 for all species. • Expansion joints to at least two faces, more is better, ideally all within one week of milling. • Inspect each piece to ensure defect is placed in ground. • Mark base with lumber crayon before processing and make available for inspection prior to processing.	• Sapwood treated to H3 with ACQ or Copper azole. • CCA treatment is not acceptable. • Note sapwood content will be small.
	No Heart	Not available without heart		
>200 mm	Contains Heart	• Much more dimensionally stable members. • Contains proportionally less heart and are less prone to splitting.	• Sound heart in the centre. • Grade is Structural Grade 2 for all species. • Expansion joints to at least two faces, more is better, ideally all within one week of milling. • Inspect each piece to ensure defect is placed in ground. • Mark base with lumber crayon before processing and make available for inspection prior to processing.	• Sapwood treated to H3 with ACQ or Copper azole. • CCA treatment is not acceptable. • Note sapwood content will be small.
	No Heart	Not available without heart		
>300 mm	Either	Impractical to supply. Consider composite members.		
Natural Hardwood Rounds	Contains Heart	• Reasonably readily available and, when turned, striking in appearance	• Durability 1 in-ground timber with a small sapwood boundary, e.g. red ironbark. • Overgrowth of injury and 'dryside' not permitted above ground. • Place any irregular shape in the ground. • When grading natural rounds in their natural state the defects that are seen in the sawn are generally not evident but may be hidden behind a bump or irregularity on the surface. These should not be removed.	• Sapwood treated to H3. • All in Ground Durability 1 and 2 species are automatically H5 if there is less than the usual 20% sapwood. • If specifying H5, CCA will be supplied unless stated otherwise.
Cypress				
Sawn - all sizes and species	This product may be unsuitable for use as bollards as the sapwood cannot be treated. If used sapwood content, which can be considerable must be limited to a maximum of 20% of the cross section.			
Natural Cypress Rounds	Contains Heart	• If the sapwood is removed in a rounding machine similar to pine, it would be eminently suitable. • Install as above.	• Maximum amount of sapwood not to exceed 20%. • Structural grade 2. • Inspect each piece to ensure defect is placed in ground. • Mark base with lumber crayon before processing and make available for inspection prior to processing.	• Sawn and natural rounds of cypress can contain large amounts of sapwood. • The sapwood of cypress is resistant to treatment and should not be ordered treated.

Nominal Size (mm)	Containing Heart?	Comment	Construction	Preservative
Pine				
Sawn - all sizes and species	Either	• A significant quantity of treated pine does not meet the required standard. Inspection for heartwood compliance if not incised is critical	• Incised to a depth of 10 mm or maximum amount of sapwood not to exceed 20%. • Structural grade 2. • Inspect each piece to ensure the depth of incisions is 10 mm or heartwood does not exceed 10%. • Ensure the defect is placed in ground. • Mark base with lumber crayon before processing and make available for inspection prior to processing.	• Sapwood treated to H4 with ACQ or Copper azole. • CCA treatment is not acceptable. • Incising is very important.
Pine Rounds	Contains Heart	• It would be rare to find natural rounds available on the landscaping market in any quantity.	• The product is now machined to give a true diameter and form along its length and is pre-graded. • Install as you would sawn pine.	• Sapwood treated to H4 with ACQ or Copper azole. • CCA treatment is not acceptable. • Incising is not an option.
Mounting and Footings				
Surface Mounted		• Galvanised or stainless steel. • Note stainless fasteners into the concrete give less trouble.	• Provide fixings 125 mm from paving edge.	
Earth		• Set in natural earth if suitable. • An alternative is fine crushed rock.	• Consider use of a pole bandage to assist lower durability species.	
Concrete	No-fines	• Use where possible to avoid potential moisture holding in footing.	• Consider whether it really is needed.	
	Normal	• Consider its use only with pine.	• Where possible, provide a 100 mm thick earth cap to top of footing to limit ground water entering any gap around between the footing and bollard base.	

48

This guide was published in Nov 2018

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

Slip Resistance and Wood Pedestrian Surfaces

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

WoodSolutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-46-1

Prepared by:
Hunarch Consulting
Author: Rodney A Hunter
Photographs and diagrams: Hunarch Consulting

Acknowledgements
Sub-editor: Bylund Enterprises, Sue Bylund

First Published: November 2018

Contents

Contents

1 Introduction

This guide is intended for designers, specifiers, producers, installers and end-users of pedestrian trafficable wood products—such as flooring, decking, boardwalks, steps, stairs, ramps and bridges.

The guide provides a brief introduction to slip resistance; discusses the slip resistance suitability of wood with respect to location and use; describes methods for enhancing slip resistance; summarises relevant Australian obligations and recommendations; and outlines slip resistance testing methods.

The graphs and tables in Section 3 of this guide show slip resistance test results of various wood products and conditions. They are case studies and are not necessarily indicative of other instances of the products or conditions. Similarly, the table at the end of Section 3 provides a ranked summary of the case study test results, but it is for general illustration and is not necessarily indicative of individual cases.

Consideration and selection of wood products in terms of slip resistance should be based on product and application specific tests as discussed further below.

The Australian statutory and non-statutory codes are occasionally revised. Readers should asses the relevance of such revisions to this guide.

This guide does not address slip resistance for bicycles and wheelchairs or coverings such as large-area mats.

2 Complexity of slip resistance and its measurement

'Slipperiness' and, conversely, 'slip-resistance' are commonly experienced phenomena and their implications for safety are readily appreciated: if a surface is slippery, possibly injurious falls may occur. But the simplicity of this characterisation belies the complexity of the phenomena and the difficulty in validly applying and comparing their measurement.

Appreciating this complexity will facilitate realistic expectations about testing and achieving slip resistance, assessing slip and fall risks, and appraising the performance of wood products.

2.1. Slips and falls

Very small, frequently unnoticeable slips are a normal part of ambulation. It is when slips are longer or faster that falls can occur; the chance of this increases with the slipperiness of surface.

Slips can occur in forward and rearward directions: forward slips of the leading foot as it lands, and rearward slips of the trailing foot as it leaves the surface[1].

Forward slips of the leading foot typically occur between the rear of the heel (of feet or footwear) and the surface. Rearward slips of the trailing foot typically occur between the foresole and the pedestrian surface. It is a forward slip of the leading foot that is most likely to cause a fall[2].

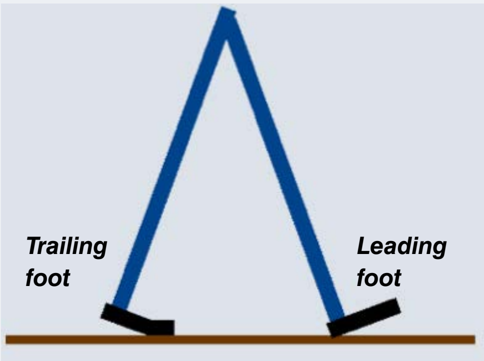


Figure 2.1: Walking gait.

At mid-stride, the heel of the leading foot contacts the ground as the trailing foot rises on its forefoot preparatory to leaving the ground.

Slips can occur at heel contact and at 'toe-off', but it is typically at heel contact that slips are most likely to occur and that can lead to imbalance and falls.

2.1.1 Reflexive downward rotation of foot for heel slip

When a heel slips, initiated by its rear edge, the foot typically reflexively and rapidly rotates downward to full-foot contact with the pedestrian surface. This helps arrest the slip and retain balance. However, if the reflex is retarded, slip arrest may not occur in time, or at all, to prevent a fall.

Consideration of the contact of the heel of the leading foot, especially at its rear edge, needs to be foremost in appraising and developing methods for slip-and-fall avoidance. A key issue is the typically very small area of contact between the rear edge of the heel and the pedestrian surface, with implications for installation of parallel slip-resistant fixtures as discussed below (see also Figure 3:29).

[1] *Rearward slips of the leading foot as it lands can also occur, but they are very small and undetectable.*

[2] *Rearward slips of the leading foot occur at the rear of the heel.*

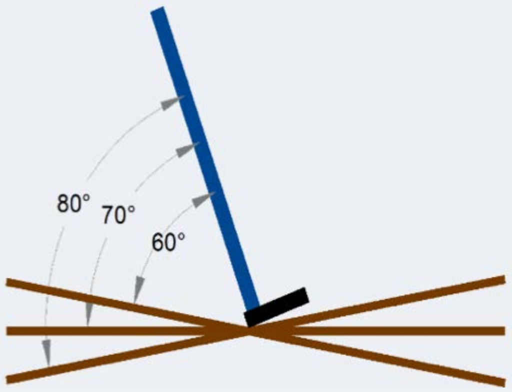


Figure 2.2: Leg angles on sloped and level surfaces.

Generally, the greater the angle of the leg to the pedestrian surface, the less is the risk of slipping. Hence, for the leading foot, there is more chance of slipping going down a slope than going on the level and even greater than going up the slope.

2.1.2 Slip length

People have been known to avoid falls for slip lengths as great as 300 mm. However, this can be considered rare. Falls can occur from slips much shorter than this. An assumption that significant fall risk is associated with slip distances of about 75-100 mm or less may be warranted – with implications for installation of intermittent attachments on pedestrian surfaces, as discussed below.

2.1.3 Slopes

Generally, a rearward slip of the trailing foot is more likely, and a forward slip of the leading foot less likely, during ascent than descent of slopes or on level surfaces. Conversely, a forward slip of the leading foot is more likely, and a rearward slip of the trailing foot less likely, during descent than during ascent of slopes or on level surfaces. In other words, slip and fall risk increases for descent travel. It also increases with increasing slope.

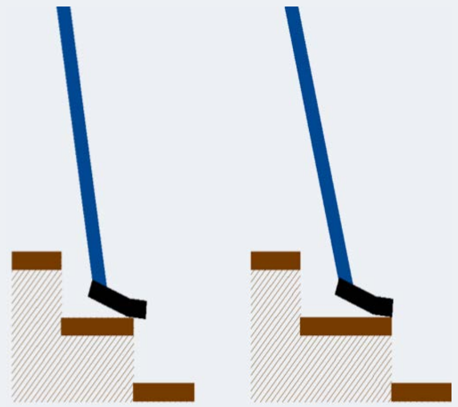


Figure 2.3: Leg and foot angles during stair descent.

The angle of the leg to the surface is considerably less on stair treads than on other pedestrian surfaces. It decreases slightly with increasing tread going, especially if the rise of the treads also decreases, which may increase slip risk although, in this circumstance, there is less likelihood of the foot projecting over the nosing and therefore less risk of an overstep slip on it.

2.1.4 Stairs

During stairway ascent and descent, contact and the possibility of a slip generally occurs between the sole of the foot and the stair treads and/or bottom and mid landings. An exception is that, if misjudgement occurs at the top landing just prior to descent, contact and the possibility of a slip can occur between the heel and the landing. At top stairway landings, the risk of a slip and fall can therefore be greater than for elsewhere on the stairway. Contact and the possibility of slips can also occur when the foot lands too far forward on a tread or top landing during descent. If this happens, the foot can slip over the nosing of the tread or landing.

Slips and falls are less likely on stairways than on slopes or on the level but, particularly during descent, if falls do occur, they are likely to be more injurious.

2.1.5 Slips and slip resistance factors

Slips and slip resistance, as discussed further below, are an outcome of three factors: the interaction of the underside of the foot (or footwear) with the ground (or floor); any matter between the two surfaces; and gait characteristics (the manner of walking or running). Consequently, for a given surface, it is difficult to predict whether a slip will be inevitable; even more difficult to predict whether falls will occur because of a slip; and impossible to do so without reference to all three factors. Moreover, the change of these factors with time also needs to be considered, whether on the scale of minutes (e.g. after rainfall or spillages) or months or years (in terms, for example, of wear and weathering).

2.2 Slip resistance measurement

Results of slip resistance testing devices and methods can be very precisely expressed, but they are specific to the tribometer and test method, to the surface sample and to any matter on it. Moreover, the heterogeneity of wood, at least in its natural state, frustrates the generalisability of test results for the same type of wood, especially with respect to surface change over time.

Additionally, the diversity of naturally occurring surface matter means that instances of it will affect slipperiness and its measurement differently, in many cases quite differently to water and oil – the surface matter used for testing under Australian slip resistance testing standards.

The variability of human gait and footwear also frustrates the formulation of broadly applicable slip resistance requirements.

Unless testing is conducted for a specific combination of factors, slip risk assessment becomes a risk management exercise and it is in this context that Standards Australia's guidelines on slip resistance is useful (see Non-statutory codes below).

The test pads of most tribometers can be regarded as poorly replicating footwear and its diversity, and their action as very poorly replicating human gait and its diversity. Nevertheless, the standardisation of tribometers and their operation (such as under Australian standards) enables a reliable and consistent comparison of surfaces, including between different wood products, and between wood and non-wood products.

Apart from the three types of tribometer addressed by the Australian slip resistance testing standards, there are other types that can be validly and valuably used in the development of new products; in the analysis of slip and fall incidents; for establishing 'alternative solutions' under the performance provisions of Australian codes; and in circumstances where the use of tribometers under the Australian standards are impracticable. A complicating factor, however, is that results with these other types of tribometer will not necessarily correlate strongly with results from the tribometers covered by the Australian standards.

Descriptions of the tribometers covered by the Australia standards are given in Section 7.

3 Slip resistance suitability of wood

3.1 Wood species

The primary relevance of different wood species to slip resistance is their response to wear and weathering and their retention of slip resistant fixtures and coatings. Generally, the slip resistance of denser hardwoods will be more sustainable indoors and outdoors than less dense hardwoods, and more sustainable than softwoods. For internal applications, differences between species becomes less important to the extent that they are coated or otherwise covered, in which case as noted below, the coatings or coverings become the slip resisting attribute.

More information about wood species can be found WoodSolutions website, in the Species and Materials section: woodsolutions.com.au.

3.2 Location and use

Slip risk and the need for slip resistance depends on where and how a surface is used, i.e. the nature of activities and of human gait on it and, particularly for outdoor settings, environmental conditions. It is also influenced by the susceptibility of the surface to wear from traffic and, for outdoor settings, weathering.

3.2.1 Activities and gait

The relevance of "activities" here is their possible contribution of surface matter such as mud, grease, food, oils and cleaning products, and spilt water such as from swimming, bathing and showering and from water fountains.

Gait refers to whether people are running or walking and, if walking, whether quickly or slowly and, in either event, whether abrupt changes in direction occur. People are more likely to slip when running, walking quickly or abruptly changing direction. Examples of where abrupt changes of direction can occur are at intersections or bends in footpaths and corridors, and anywhere that activities predispose to abrupt directional changes, such as in recreational, sports and employment settings.

Figure 3.1: Well-worn heel.

This heel was removed from a shoe worn by a person who slipped going down a wet ramp of boards laid across the ramp. It was surmised that the smoothed and rounded rear edge of the heel from long-term wear was a key factor in the slip. For the leading foot, the rear of the heel is critical in forward slips (or avoidance of them).

Slip propensity on surfaces is also influenced by whether people are bare-foot or in footwear. For footwear, key factors in slips or slip resistance are: a) the material of outersoles (heels and fore-soles) and their degree of stiffness or compressibility; b) whether outersoles have treads or are otherwise textured and, regardless, whether they have been smoothed, including rounded at the rear of heels through prolonged wear; and c) the style of shoe, particularly in terms of the size and shape of the heel (stiletto high-heeled shoes are an extreme example).

There obviously can be a prevalence of certain types of footwear in some settings, such as deeply treaded outersoles in industrial settings, firm smooth outersoles in commercial settings, and highly compressible outersoles in sports and recreational settings. The relevance of footwear is not discussed in this guide, but it is necessary to acknowledge the complicating contribution of footwear to consideration of slip resistance and its measurement, and to the risk assessment of falls.

The Standards Australia handbook *HB 198:2014 – Guide to the specification and testing of pedestrian surfaces* (see further below) provides general guidance on minimum slip resistance for various activities and settings. Importantly, it also illustrates the wide range of slip resistance ratings that may be suitable, depending on the setting. Nevertheless, published indications of products' slip resistance or of slip resistance requirements need to be regarded as approximations because they cannot fully account for the complex interaction of the key factors identified above.

3.2.2 Environmental conditions

The significance of environmental conditions is their contribution of moisture or moist matter or, at the other extreme, dry matter such as grit and very small seed pods and twigs[3]. Wet vegetative debris, moss and fungal growth on boardwalks in bushland; moisture from rainfall and mist; and sand on foreshore boardwalks are common instances[4]. The adverse effects of this matter in outdoor settings can be ameliorated if there is exposure to driving rain and drying wind that remove the matter.

Regardless of its origin, it is matter between the wood and the foot or footwear that facilitates slipping, even if the matter is so minute as to be invisible to the naked eye. Moisture or moist matter is the most common slip-inducing type. Regardless of the type, regular monitoring and, if necessary, cleaning and maintenance is necessary to mitigate slip risk[5].

Slippery matter on pedestrian surfaces can be particularly hazardous if it is not evident or anticipated, as discussed in section 3.1.6 Adjacent dissimilarity.

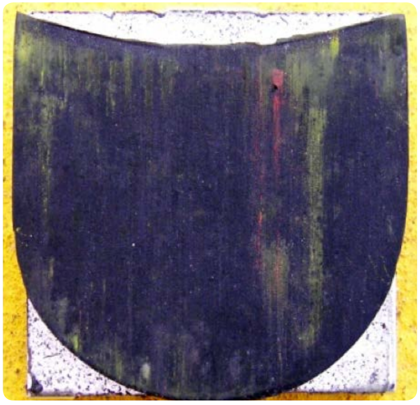


Figure 3.2: Matter between heel and pedestrian surface. Paint residue has been transferred to the test heel from the pedestrian surface.

Figure 3.3: Matter between heel and pedestrian surface. Fungal matter has been transferred to the test heel form from the pedestrian surface.

Example: slip resistance of fungal-coated wood

An example of reduced slip resistance of a wood boardwalk in a swampy area is shown at Figure 3.4. Test results indicated that, along the boardwalk (perpendicular to the boards), the slip resistance of wet fungal-coated boards was 23% less than of the clean boards when wet. Across the boardwalk (parallel with the grain), the slip resistance was reduced by 14%. Differences in slip resistance for movement perpendicular to ('across') and movement parallel with ('along') the grain is discussed below.

[3] Ice on pedestrian surfaces is not considered in this guide.

[4] The risk of dry particles is greater when they are on smooth dense surfaces such as smooth-trowelled concrete or ceramic tiles than it is for textured surfaces and surfaces of wood that are sufficiently easily indented as to restrain the particles or whose surfaces cavities can receive the particles that are rolled into them by the foot; it is also greater for un-textured, non-resilient soles.

[5] Matter can also include microscopic abraded debris from the pedestrian surface and the outersoles of footwear, although their contribution is unlikely to be significant in most situations.

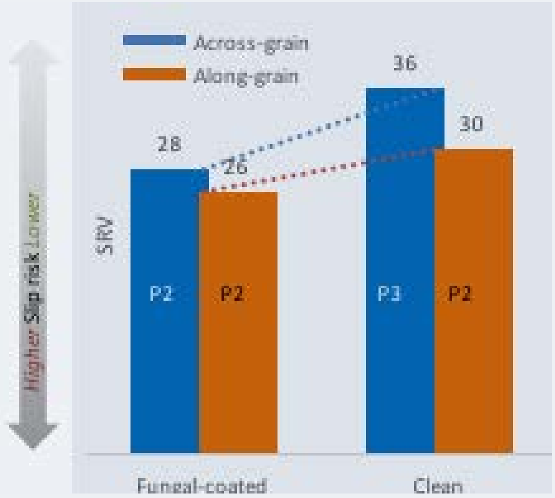


Figure 3.4: Effect of fungal growth.

In this example, the fungal-coated boards were about 23% less slip resistant across the grain and 14% less slip resistant along the grain than the clean boards. There was very little difference in slip resistance across and along the grain for the fungal-coated boards but, for the clean boards, slip resistance across the grain was 20% greater than for along the grain. In other words, fungal growth negated the greater across-grain slip resistance of the board in its clean state.

Notes:

1. *'SRV' = Pendulum slip resistance value.*
2. *'P' ratings as per AS 4586 and AS 4663.*
3. *Surfaces were tested when water-wet.*
4. *'Across-grain' = perpendicularly across the grain. 'Along-grain' = parallel with the grain.*
5. *The percentage variances indicated in the caption are for illustration only and should not be relied-on.*
6. *The sample size was very small, and results should therefore be treated cautiously.*

3.2.3 Wear and weathering

Wear by pedestrian traffic and, for outdoor settings, weathering (erosion and rot) can cause the edges and the textured surface of wood to become smoother over time and therefore diminish its slip resistance.

Figure 3.5: Naturally textured surface from weathering.

Weathered boards, when dry, are typically more slip resistant across the weathered grain than along it. However, when they are wet the opposite can apply because the wood fibres can be dislodged by heels at contact with the surface. Suitable wood preservatives will retard the process and help sustain the integrity of the surface.

The effect of weathering is complex. Apart from possible surface smoothing, checking and cracking of wood and the raising of grain from weather exposure can also occur, imparting surface texture (see Figures 3.5 and 3.6) and therefore possibly increasing slip resistance (for traffic transverse to the wood grain). However, in moist conditions, the surface fissuring can facilitate rotting, surface disintegration and deposition of matter and possibly to reduction of slip resistance.

Wind-borne grit can cause gouging of the wood and possibly contribute to slip resistance (across the gouges) but it can also smoothly scour the ridges and diminish slip resistance. The complexity of wear and weather effects over the service life of boards is indicated by the example at Figure 3.7.

Figure 3.6: Magnified view of a severely weathered hardwood board.

This photo shows surface texture caused by weathering, and the accumulation of stone particles in fissures. The particles have been imported from much further away, by pedestrians and cyclists, and have been dislodged from treaded footwear and tyres.

More-detailed accounts of weathering can be found in other Wood Solutions Technical Guides[6].

Gaps between external boards can drain water and help surface matter to be removed by wind and foot traffic. However, these benefits can be negated if the boards become concave (cup) with time[5]. The machining of boards so that their upper surface is slightly convex can assist natural drying and compensate for cupping. However, the convexity would need to be sufficiently shallow for the downward slopes on either half of each board not to contribute to slips (nor hinder comfortable wheeled movement).

Example of wear, weathering and slip resistance

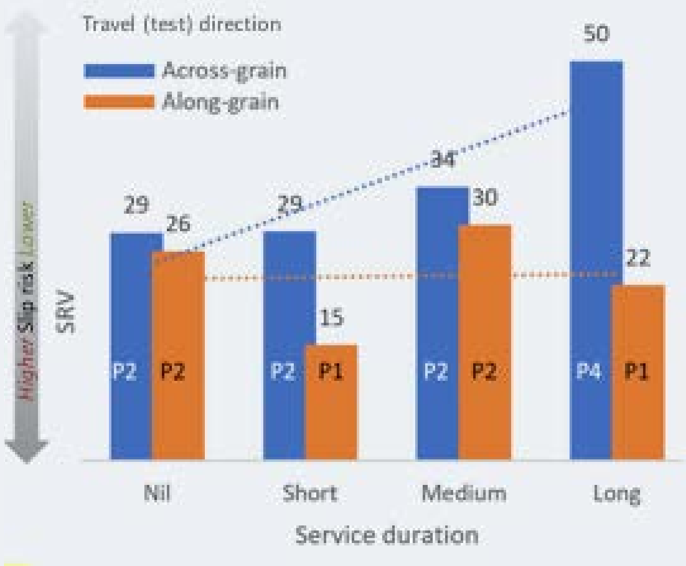


Figure 3.7: Effect of wear/weathering duration.

In this example: Slip resistance across the grain (blue bars) increased with increased service duration, but there was no clear order for along-grain slip resistance (orange bars) over time.

The greatest slip resistance was across-grain for long duration (SRV 50, P4). This is attributable to fissures in the wood caused by long-term weathering.

The least slip resistance was along the grain for Short duration (SRV 15, P1). This is attributable to the wood having been smoothed by wear and prior to the eventual opposing effect of weathering.

The greatest dissimilarity between across-grain and along-grain slip resistance was for short and long duration.

Notes:

1. *'SRV' = Pendulum slip resistance value.*
2. *'P' ratings as per AS 4586 and AS 4663.*
3. *Surfaces were tested when water-wet.*
4. *'Across-grain' = perpendicularly across the grain.*
5. *'Along-grain' = parallel with the grain.*
6. *The Sample size was very small; results should be treated cautiously.*

[6] These include Guide No. 5: Timber Service Life Design – Design Guide for Durability, Guide No.13: Finishing Timber Externally, and Guide No. 21: Domestic Timber Deck Design. These and other timber-related guides can be downloaded for free from the WoodSolutions website.

[7] Convexity and concavity of boards can prevent easy wheeled movement (by people with prams, wheelchairs and bicycles).

3.2.4 Slopes

As for other materials, wood on slopes is less slip resistant for movement down them than for movement on horizontal surfaces (for identical wood, conditions, footwear and gait[8]) because the angle of the leg at heel strike on downward-sloping surfaces (and therefore the angle of the incident force) is shallower than on a horizontal surface. This is illustrated at Figure 2.2. Additionally, in the event of a slip, there is a greater angle through which the foot must rotate downward to full foot contact, which tends to reflexively happen, and so it takes longer for full foot contact to occur, which diminishes slip-arrest. Generally, as indicated above, the steeper the slope, the less the slip resistance.

The increase in slip risk is not necessarily proportionate to the increase in slope because, if people are aware of the change in plane, they typically adapt their gait. However, there are many reasons why people might be unaware of a change in plane and therefore not adapt their gait.

3.2.5 Stairs

Three parts of stairways need to be considered for slip resistance: top landings (near the top tread), treads, and the nosings of treads and top and mid landings.

Slips during stair ascent are unlikely to cause falls down stair flights but slips during descent do predispose to falls. The risk of slips and falls during descent can be considered as low; however, there is a high risk of injury, and even fatality, if a fall does occur.

Stair descent

On landings at the top of flights, the foot, as with normal walking, usually lands heel first, although typically at a shallow angle to the landing. On treads and bottom and mid landings, the foot usually lands foresole first and at a very shallow or no angle to the tread, and the leg at little or no inclination to the vertical.

Generally, people place their feet slightly further back from the nosings of treads of larger goings than they do for smaller goings; the same also occurs for slower rather than quicker descent.

As an example, for a size 9 adult male shoe, the potential contact length of the foresole on the tread is 110 mm. Hence, if the front of the foot is placed at the tread nosing, the corresponding contact length on the tread is 110 mm. However, especially on smaller but even on larger treads, the front of the foot typically projects over the nosing, in which case the contact length on the tread is 60 mm. Foresole contact lengths much less than this can occur.

The slip resistance of treads near their nosings becomes critical if a person's foot lands too far forward (the person over-steps the nosing), in which event the centre of pressure is too close to or in front of the nosing and, if there is insufficient slip resistance, the foot can slip off the tread. However, it is possible that slip-resistant nosings merely retard slips and that loss of balance and falls down stairs occur regardless of slip resistance.

Figure 3.8: Foot and leg at touchdown.

The person's leading foot has just contacted the tread (at its nosing). The foresole is at a very shallow angle to the tread (shown in yellow), and the angles of the leg (shown white) and the force vector (shown red) with respect to the vertical is small. The ball of the foot (and centre of pressure) is very close to but not in front of the nosing.

[8] *Gait will be slightly different on inclined compared with level surfaces.*

Figure 3.9: Foot and leg just after contact.

As weight becomes borne on the leading foot, the leg angle becomes vertical, the force vector angle becomes closer to vertical, and the centre of pressure on the tread moves further back from the nosing.

Figure 3.10: Risk of overstep slip.

This person's foot has just contacted the tread (at its nosing), with the ball of the foot perilously close to over-stepping the tread. However, the compressibility of the outersole and the presence of the metal insert helped avoid an overstep slip.

3.2.6 Adjacent dissimilarity

Dissimilarity of adjacent surfaces or surface conditions (even if the conditions are momentary) is an easily overlooked slip hazard. People stepping unknowingly from a slip-resistant to a slippery surface, and so not adjusting their gait, risk slipping and possibly falling.

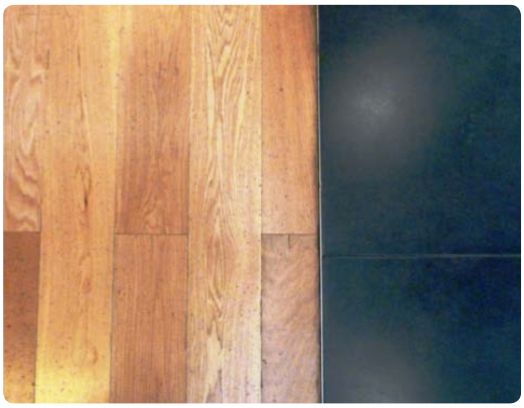


Figure 3.11: Adjacent dissimilar materials.
Highly polished ceramic tiles adjacent to sealed hardwood floorboards.

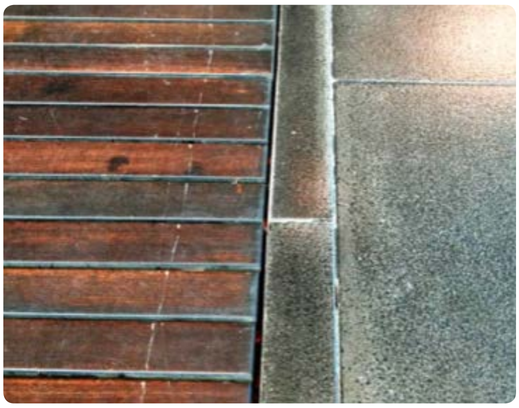


Figure 3.12: Adjacent dissimilar materials.
Stone tiles adjacent to sealed hardwood boards in an area prone to walked-in moisture.

Transitions between indoor and weather-exposed outdoor areas, and between food stalls and adjacent walkways, or dining areas and retail spaces, are where matter can be transferred. For the area onto which matter is transferred, it is good practice to provide a surface whose slip resistance is the same as that of the surface of the area from which the matter is transferred (and for each of the adjacent surfaces to be sufficiently slip resistant). Isolated momentary dissimilarity of conditions can also occur within any setting because of spillage and, while this is a facilities maintenance issue, consideration should be given to transient matter in the choice of surface products.

Figure 3.13: Walked-in moisture.
The walked-in rainwater is evident from this vantage point but not necessarily from other vantage points or to a person too busy striding or running across the wet patches or towards the doorway to notice.

Another example of adjacency effect is outdoors where part of an area is exposed to breeze and sunlight and therefore kept clean and dry, but another part is shaded and wind-protected and therefore remains moist, harbours fungal growth and accumulates small debris.

Alleviation of these problems entails a combination of good design of adjacent spaces, good selection of pedestrian surfaces and good maintenance.

Figure 3.14: Dissimilar surface conditions.
This boardwalk over a swampy area is shaded from the sun and sheltered from the wind on one side; hence, that side is moist and has fungal growth whereas the other side is comparatively clean and dry. A sudden divergence by a pedestrian from the dry side to the moist side could lead to a slip.

See also Figure 3.4.

3.3 Enhancing the slip resistance of wood

There are several options for increasing wood's slip resistance, including by: - choice of orientation with respect to traffic; surface texturing; incorporating inserts and attachments; and applying coatings. Coverings, such as mats, is another option, but are not discussed in this guide.

3.3.1 Orientation

Wood surfaces with directional characteristics because of grain, texture (from rough-sawing, roughening, profiling or weathering) or board juxtaposing are generally more slip resistant across than along those characteristics. Wood should be installed so that its predominant directionality is perpendicular to predominant directions of traffic (or to direction of most rapid traffic[9]).

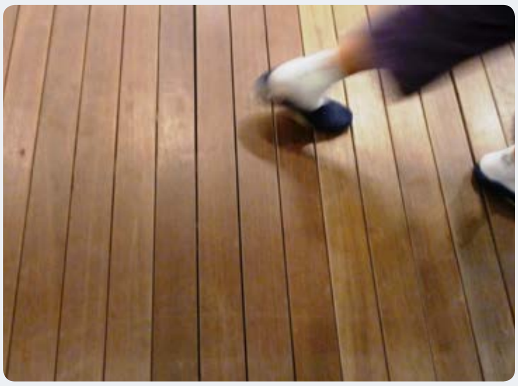


Figure 3.15: Board orientation and traffic direction.

Wood installed transversely to predominant traffic tends to be more slip resistant than when installed parallel to it. However, this depends on the saliency of the grain. Coatings will diminish the slip resistance difference, increasingly so with thicker coatings, which then become the paramount slip-resisting surface.

Examples of grain direction and slip resistance

The effect on slip resistance of the orientation of wood grain and other directional surface features is illustrated in Figures 3.4 and 3.7. A comparison of slip resistance ratings across and along wood boards is shown in Figures 3.15 to 3.18.

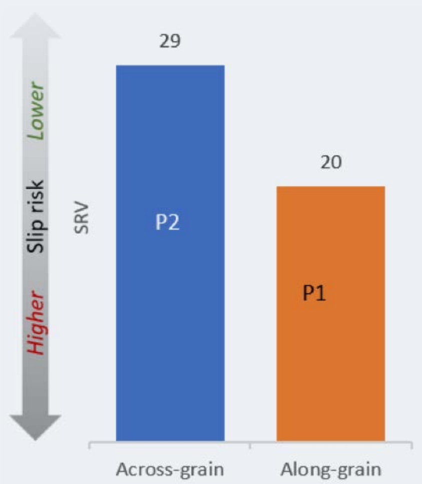


Figure 3.16: Effect of grain direction: grouped results.

For this group of dissimilar boards, slip resistance across the grain (SRV 29, P2) was 47% greater than along it (SRV 20, P1). However, the difference does not accurately indicate individual results. For example, the greatest difference between two boards was 57% and, in one instance, there was no difference. Results for the individual boards are shown in Figure 3.17.

For a different or larger group of boards, the average slip resistance values and the difference between them could be very different. This highlights the difficulty of providing generalised slip resistance guidelines for timber surfaces.

Notes:

1. *'SRV' = Pendulum slip resistance value.*
2. *'P' ratings as per AS 4586 and AS 4663.*
3. *Surfaces were tested when water-wet.*
4. *'Across-grain' = perpendicularly across the grain. 'Along-grain' = parallel with the grain.*
5. *The Sample size was very small; results should be treated cautiously.*

[9] *Rapid traffic (such as fast walking and running) and erratic traffic (that entails sudden acceleration and deceleration) increases slip risk.*

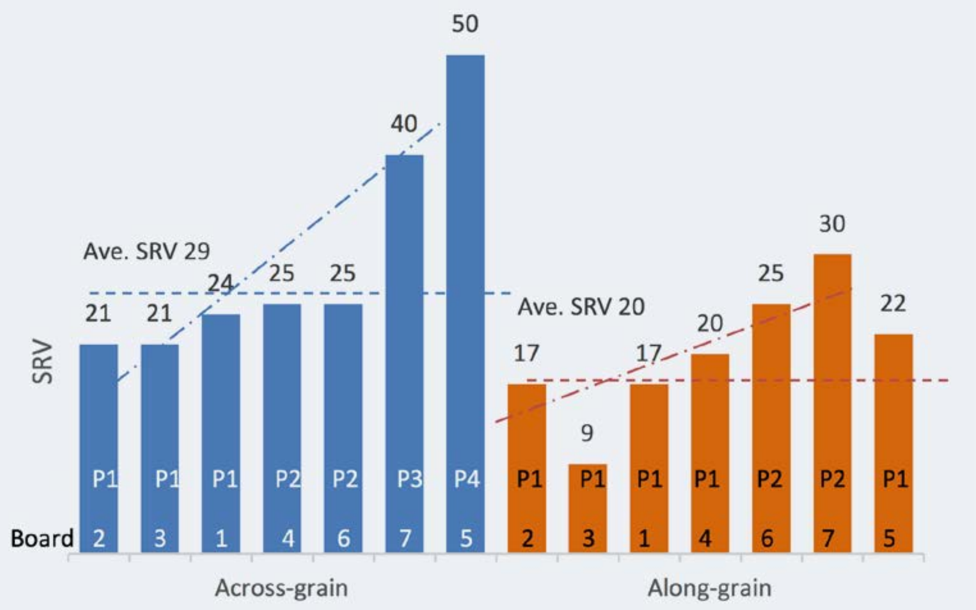


Figure 3.17: Effect of grain direction: individual results.

When arranged in order of increasing across-grain slip resistance (blue bars), the same boards also show a generally increasing order of along-grain slip resistance (orange bars), although not as consistently.

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. 'Across-grain' = perpendicularly across the grain. 'Along-grain' = parallel with the grain.
5. The Sample size was very small; results should be treated cautiously.

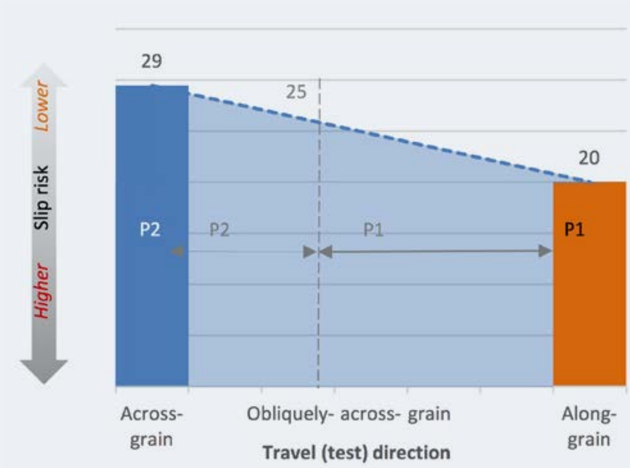


Figure 3.18: Intermediate slip resistance for travel directions obliquely across the grain.

In this example the slip resistance was found to be 29 SRV across the grain and 20 SRV along it. Test results for intermediate angles would be approximately proportionate to the difference between these two values.

The risk of slipping decreases with higher SRVs; however, categorisation of slip risk, recognised internationally for Pendulum test results and reflected in Australian standards and building regulations, is at a larger scale – as expressed by the P classifications. In the above case-study, test results at intermediate angles would be either P1 (for SRVs less than 25) or P2 (for SRVs less than 29 and equal to or greater than 25). The P classification scale is further illustrated in Figure 3.17.

Figure 3.19: Incorrect orientation of boards at landings.

Figure 3.20: Incorrect orientation of boards at landing.

The boards on the landings in Figures 3.19 and 3.20 are parallel to the travel direction and are therefore more likely to facilitate slips than if they were transverse. The wood grain is also less likely to restrain underfoot movement of sand particles and other small vegetative matter, which also facilitates slips. Installation of wide, slip resistant fixtures at the landing edge would help alleviate the slip risk, preferably rebated to avoid accumulation of surface matter at their rear edge.

In some settings, such as around swimming pools, it might be feasible to lay boards in different orientations in different areas to correspond with the different types of traffic, although assessing or predicting predominant traffic characteristics could be difficult. Moreover, there might be slip risk at the junction of the different orientations precisely because of the difference: someone moving across the directional characteristic of wood in one part might unknowingly step onto adjacent wood in a direction parallel with the adjacent wood's directional characteristics.

One method sometimes adopted is a chequerboard configuration of short boards in groups perpendicular to each other. However, unless each board is extremely short, there will be adjacent dissimilarity of board direction and possibly therefore slip risk for the reasons previously indicated. The risk might be the greater because people might perceive the chequerboard configuration as being slip resistant and so move on it with false confidence.

Figure 3.21: Chequerboard configurations can give the illusion of slip resistance.

3.3.2 Board width

Surfaces such as boardwalks and decks that have boards with gaps between them may be more slip resistant if the boards are narrow than if they are wide. This is suggested by a preliminary study that found that 30 mm wide boards were 40% more slip resistant than 20 mm wide boards 67% more slip resistant than 90 mm wide boards, as shown in Figure 3.22.

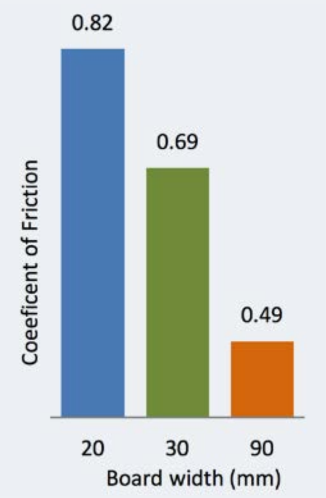


Figure 3.22: Contribution of board width to slip resistance.

In this study, 20 mm wide boards, with gaps between them, were found to be 20% more slip-resistant than 30 mm wide boards, and 67% more than 90 mm boards

Notes:

1. *Slip resistance is expressed in terms of Coefficient of Friction, not Slip Resistance Value (SRV).*
2. *Surfaces were tested when water-wet.*
3. *The sample size was very small; results should be treated cautiously.*
4. *These results were obtained with a slip resistance tester that measures slip distances up to 450 mm, compared with the 124 mm of the pendulum.*

3.3.3 Texture

Machine-roughened wood

Multi-directional roughness imparts slip resistance in any direction but has the disadvantage of retaining moisture and very small debris and being resistant to cleaning, including by wind and rain. Uni-directional roughness will impart slip resistance across the texture but diminished slip resistance along it, depending upon the sharpness and configuration of the texture (and upon the compressibility, 'treadedness' and rigidity of footwear outersoles).

Depending on pedestrian traffic loads and characteristics, the slip resistance of roughened wood will diminish with time as it is abraded and smoothed by wear. Sustained slip resistance for the setting will require board replacement or re-roughening.

Figure 3.23: Saw-roughened boards.

Because of the semi-circular configurations, boards such as these can provide slip resistance in all directions and, in this example, opportunity for water run-off. However, the texture might not be sustained over time if installed in high traffic areas or continuously moist settings.

Example of machined roughness and slip resistance

For boards with gaps, such as outdoor decking and boardwalks, narrow boards may be more slip resistant than wide boards. This is suggested by a preliminary study that found that 30 mm wide boards were 40% more slip resistant and 20 mm wide boards 67% more slip resistant than 90 mm wide boards, as shown in Figure 3.22.

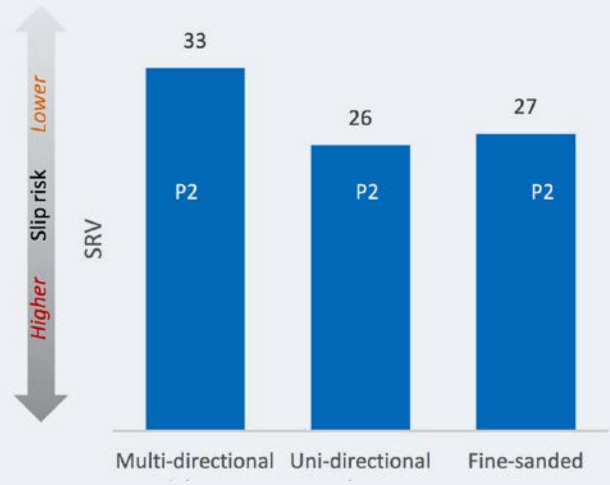


Figure 3.24: Effect of machined roughness on slip resistance.

The multi-directional rough-sawn texture was 22% more slip resistant than the fine-sanded board, and the uni-directional rough-sawn texture was 4% less slip resistant.

The multi-directional texture was 27% more slip resistant than the uni-directional texture.

The boards were each tested twice, once from each end of the sample.

The machined roughness did not increase the P classification compared with the fine-sanded board.

Notes:

1. *'SRV' = Pendulum slip resistance value.*
2. *'P' ratings as per AS 4586 and AS 4663.*
3. *Surfaces were tested when water-wet.*
4. *'Along-grain' = parallel with the grain.*
5. *Tests were conducted along the grain only*
6. *The sample size was very small; results should be treated cautiously.*

Reeding

Reeded boards can provide slip resistance across the reeding but much-less-so along them (see Figure 3.27). They are best-suited for uni-directional traffic such as on narrow boardwalks and footbridges.

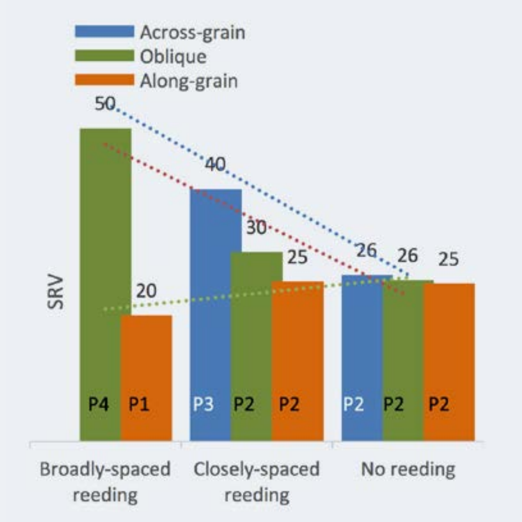
Increasing the gap size between reeds (see Figures 3.25 and 3.26) can substantially increase slip resistance across the reeding but decrease it along it, as shown in Figure 3.27.

A significant disadvantage of reeded boards is that they are not self-draining unless they are in short lengths.

Figure 3.25: Reeded board 4 mm diameter, closely spaced (1 mm gaps).

Figure 3.26: Reeded board 4 mm diameter, broadly spaced (6 mm gaps).

Example of reeded wood and slip resistance



Notes:

1. *'SRV' = Pendulum slip resistance value.*
2. *'P' ratings as per AS 4586 and AS 4663.*
3. *Surfaces were tested when water-wet.*
4. *'Across-grain' = perpendicularly across the grain.*
   'Along-grain' = parallel with the grain.
5. *'Oblique' = offset by 12° from perpendicular to grain (the pendulum is unsuitable for perpendicularly across-grain testing of broadly spaced reeding).*
6. *The sample size is very small; results should be treated cautiously.*

Figure 3.27: Effect of reeded wood on slip resistance

Across-reeding slip resistance of closely-spaced reeding was found to be 52% greater than for plain boards

Obliquely-across-grain slip resistance of closely-spaced reeding was 18% greater than for plain boards, and broadly-spaced-reeding was 94% greater. Broadly-spaced-reeding had 65% greater slip resistance than closely-spaced reeding.

Along-grain slip resistance of closely-spaced reeding was almost the same as for plain boards; however, broadly-spaced reeding was 25% less slip resistant than plain boards. Broadly-spaced reeding was 21% less slip resistant than closely-spaced reeding.

For closely-spaced reeding, the slip resistance across the grain/reeding was 58% greater than for along the grain/reeding, and 18% greater than for obliquely across the grain/reeding.

For broadly-spaced reeding, the slip resistance obliquely across the grain/reeding was 148% greater than for along the grain/reeding.

For plain boards, there was very little difference in slip resistance of different grain/reeding directions.

Of the different surfaces, broadly-spaced reeding offers the greatest slip resistance obliquely across the grain/reeding (SRV 50, P4) but the least for along the grain/reeding (SRV 20, P1).

Example of reeded wood with grit-augmented coating

Slip resistance of reeded boards can be further improved by applying grit-augmented coating. In a test of broadly-reeded boards, slip resistance was increased by a P classification for along-grain and obliquely across-grain (see Figure 3.28).

The susceptibility of the tops of reeds to traffic wear, and therefore to gradual removal of the grit coating, would diminish the slip resistance with time. Regular re-coating would be required.

Example of reeded wood and slip resistance

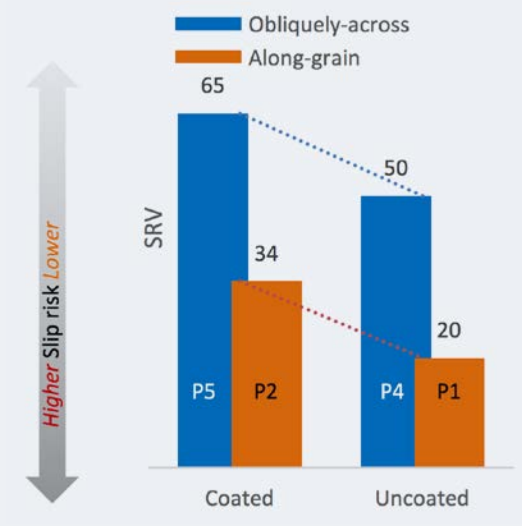


Figure 3.28: Broadly reeded board with grit-augmented coating

The coating provided 30% more slip resistance across the reeding and 70% more along it.

The coated board was 95% more slip resistant and the uncoated board 148% more slip resistant obliquely across the reeding than along it.

P classifications increased from P4 to P5 obliquely across the reeding and from P1 to P2 along it.

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. 'Along-grain' = parallel with the grain.
5. 'Obliquely across' = offset by 12° from perpendicular to grain (the pendulum is unsuited for perpendicularly across-grain testing of broadly spaced reeding).
6. The sample size is very small; results should be treated cautiously

3.3.4 Fixtures

Slip resistant 'fixtures' are the wide variety of single and multiple inserts and attachments that can be used for enhanced localised and broad-area slip resistance. 'Inserts' are thin metal strips inserted on edge into rebated wood but protruding slightly above it—commonly applied in parallel configurations on stair treads and top landings. 'Attachments' are thin flexible or rigid slip-resistant straps, or rigid metal sections incorporating one or more thin flexible or rigid slip resistant straps, each type being attached to or rebated into the wood. Single attachments are commonly used on stair treads, and multiple attachments on walkways and ramps.

Figure 3.29: Boards with inlaid grit straps.

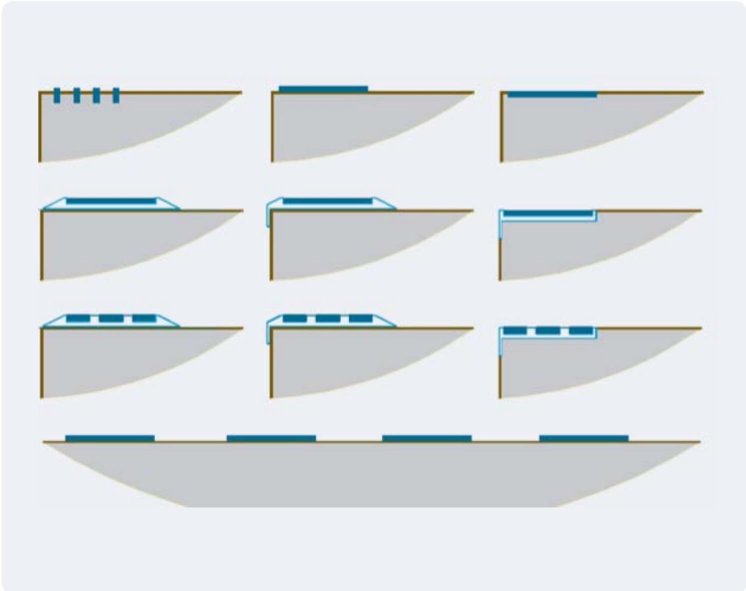


Figure 3.30: Examples of fixtures on treads and walkways.

On stair treads, the primary function of fixtures is to resist slipping of foresoles during ascent and descent. On ramps and walkways, their primary function is to resist heel slips and, secondarily, fore-sole slips.

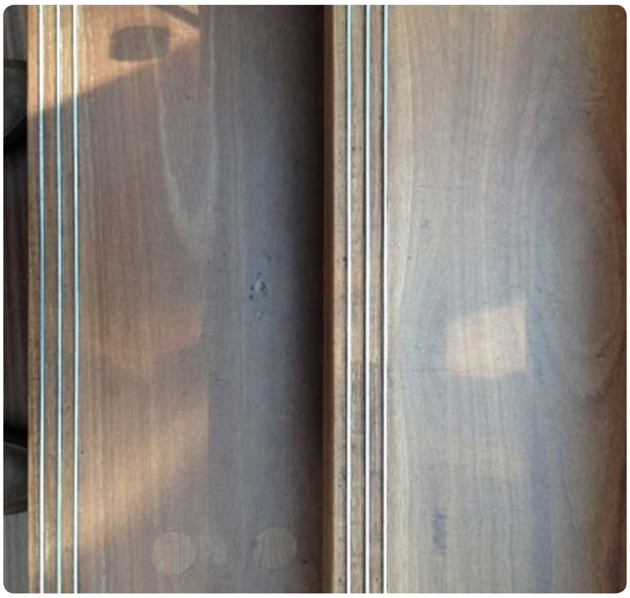


Figure 3.32: Metal ribs at stair tread nosings.

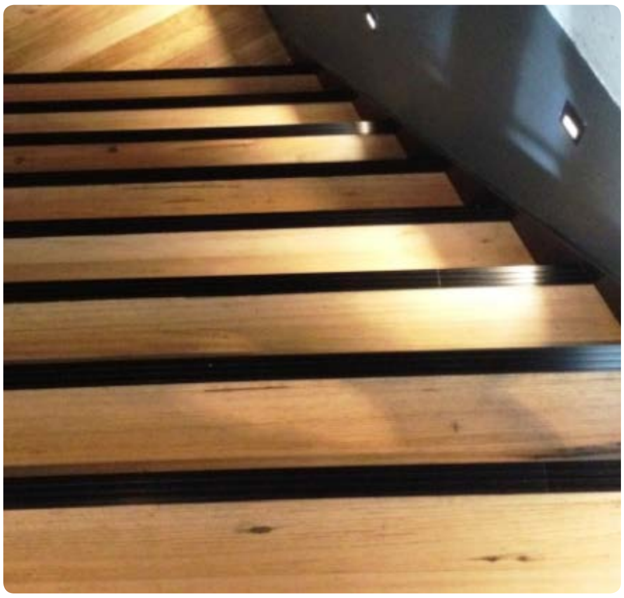


Figure 3.31: Polymer straps at stair tread nosings.

Parallel fixtures and heel slip arrest

Key factors in heel slip arrest are heel slip distance and the area of contact of the heel's rear edge at touchdown.

Parallel fixtures perpendicular to pedestrian traffic

With respect to heel slip distance, a key dimension in the placement of parallel fixtures orientated perpendicular to traffic is the distance between them.

On level surfaces, distances between fixtures of no greater than about 100 mm or, for greater safety, no greater than about 75 mm may be required. On sloped surfaces, depending on the slope, or on level surfaces on which rapid traffic is common, distances of much less than about 75 mm may be necessary.

Parallel fixtures can function in two ways: a) a heel slip on the substrate just after one of the fixtures is arrested or retarded by the next fixture; and b) if reflexive downward rotation of the foot occurs with slip initiation, the foresole will engage the next fixture (if it is close enough).

For traffic oblique to the fixtures, the fixtures need to be increasingly closely spaced with increasing angular divergence from the perpendicular.

Another key dimension is the width of each fixture. A reflexive downward rotation of the foot when a slip occurs contributes to slip arrest and fall avoidance because it brings increasingly greater foot area (heel then foresole) into contact with the pedestrian surface. Fixtures whose width wholly or substantially accommodates the heel will contribute the most to slip arrest.

Parallel fixtures parallel with pedestrian traffic

Heel rear edge contact area has the greatest relevance to parallel fixtures that are also parallel to pedestrian traffic and when foot contact occurs between the fixtures.

The contact area of the heel at touchdown (when heel slips occur) is at its rear edge and is very small. Unless fixtures are very closely spaced, they will allow a heel slip to continue and possibly cause a fall (unless the substrate and the person's reactions enable balance recovery). Even if reflexive downward rotation of the foot occurs on heel slip initiation, and notwithstanding the progressively greater heel and foresole contact area, unless the fixtures are close enough, insufficient of the increasing area of the heel and foresole will engage the adjacent fixtures to aid slip arrest.

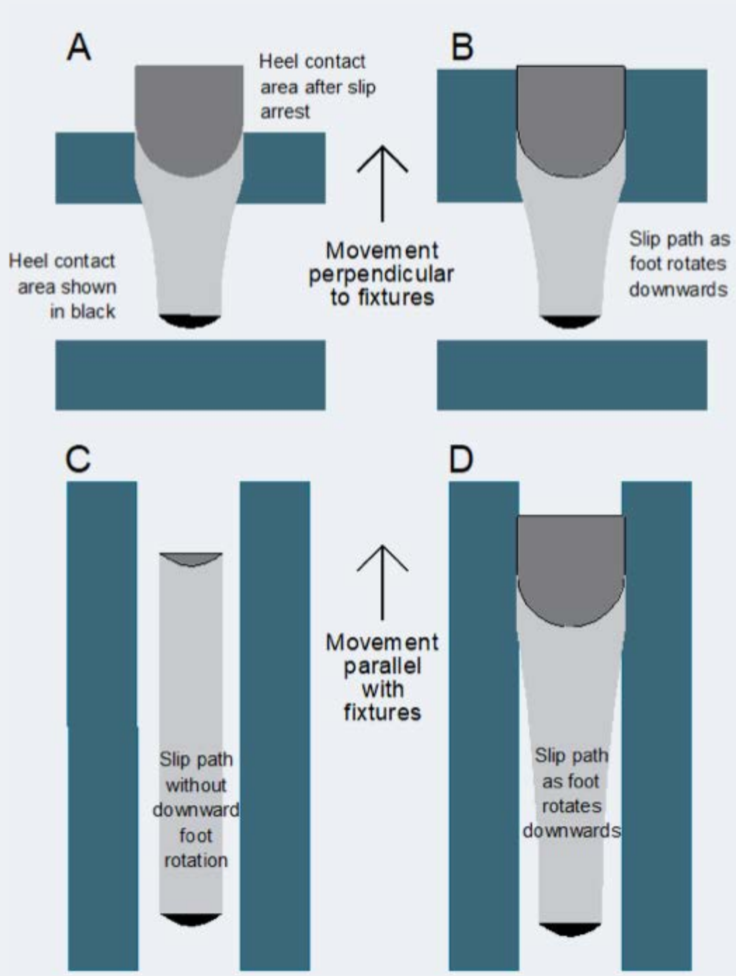


Figure 3.33: Diagrammatic representation of heel slips and slip resistant fixtures.

The heel/ground contact area at touchdown is shown in black. The shoe is that of an adult male and has been rounded from wear. New shoes or shoes with smaller heels would have a much smaller contact area. The foresole area has been omitted from the diagrams.

Diagrams A and B show the heel having been arrested by the second fixture.

Diagram B indicates the contribution to slip arrest provided by wider fixtures.

Diagram C shows heel contact between fixtures. In this example, there has been no reflexive downward rotation of the foot and it can be assumed that a fall occurs at the end of the slip. It is evident that the surface fixtures have no slip arrest role.

Diagram D shows the typical reflexive downward rotation of the heel (and foot) as a means of regaining balance and fall avoidance. In this example, the slip arrest depends on the wood substrate, not the slip-resistant fixtures at each side of the foot.

Example of grit-tape and slip resistance

Grit fillings or grit-tape products are amongst the most effective way of achieving slip resistance for wood surfaces. Their effectiveness depends on their extent of coverage. In the test in Figure 3.34, for partial coverage, 13 mm wide strips with 40 mm gaps between them were the most slip resistant, slightly more slip resistant than the 13 mm strips with 15 mm gaps and the half-width strips.

Different results would be derived for different grit type and size, different width strips, different-sized gaps and, to a much lesser extent, different starting positions for the Pendulum test foot.

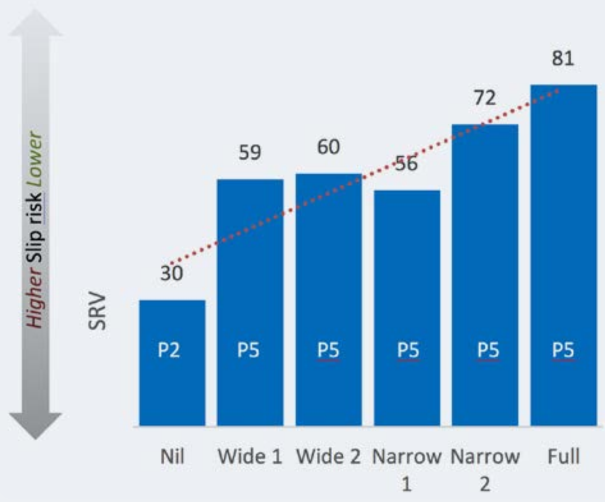


Figure 3.34: Contribution of grit-tape to slip resistance.

Compared with the Nil (uncovered) condition, grit tape increased slip resistance substantially, the most for full coverage (170% increase to SRV 81, P5) and, in descending order, Narrow 2 (by 139% increase to SRV 72, P5), Wide 2 (by 100% to SRV 60, P5), Wide 1 (by 96% to SRV 59, P5) and Narrow 1 (by 87% to SRV 56, P5). with very little difference between Wide 1 and Wide 2.

Grit-tape with 40 mm gaps was 28% more slip resistant than for 15 mm gaps.

The grit tape increased the slip resistance classification from P2 to P5, regardless of the extent of tape coverage.

See Figure 3.35 for the Narrow 2 sample.

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. Slip resistance was assessed along the grain only (as the worst case). The grit tape was perpendicular to the grain.
5. The sample size is very small; results should be treated cautiously.
6. 'Full' = whole surface covered with grit-tape
7. 'Narrow 1' =13 mm wide grit-tape with 15 mm gaps
8. 'Narrow 2' = 13 mm wide grit-tape with 40 mm gaps
9. 'Wide 1' = half-coverage (63 mm wide strip); test foot commenced on tape.
10. 'Wide 2' = half-coverage (61 mm wide strip); test foot commenced on wood.
11. 'Half coverage' = half of the 124 mm travel length of the Pendulum test foot.

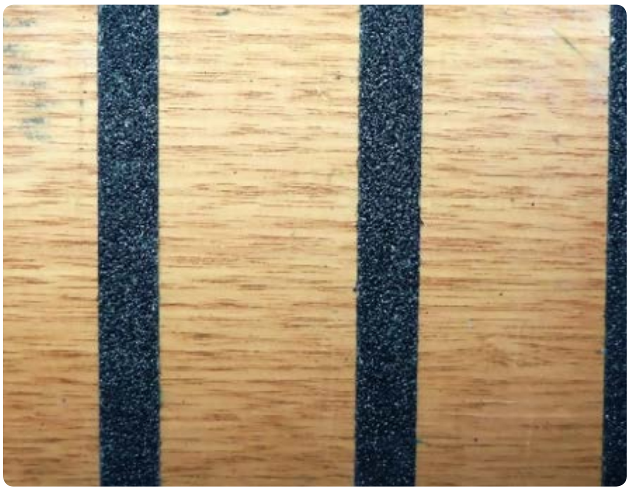


Figure 3.35: Victorian Ash hardwood with grit-tape strips.

This is the sample 'Narrow 2' referenced in Figure 3.34. It has 13 mm wide grit-tape straps with 40 mm gaps.

Example of metal rib inserts and slip resistance

As indicated above, fixtures such as metal rib inserts can be effective for uni-directional traffic. The test described in Figure 3.37 found that thin metal rib inserts at 36 mm and 18 mm centres (see Figure 3.36) substantially increased the slip resistance of wood, increasing the P classification across the ribs from P1 to P3 for ribs at 36 mm centres, and from P1 to P4 for 18 mm centres. In other words, of the two spacings, the closer-spaced ribs were more slip resistant (when measured with the Pendulum).

Figure 3.36: Victorian hardwood with thin metal rib inserts at 18 mm centres

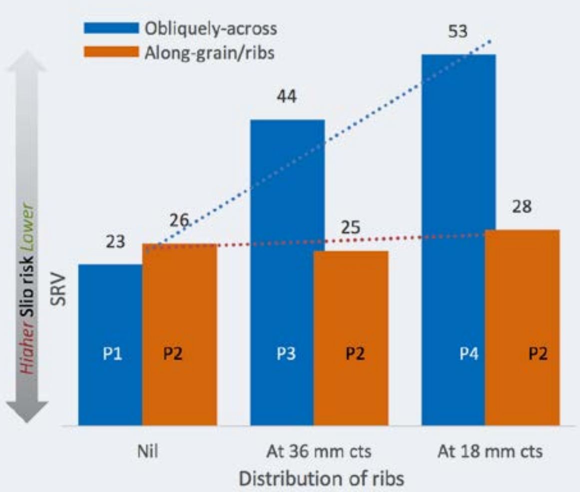


Figure 3.37: Enhancing slip resistance with thin metal rib inserts.

For travel obliquely across the grain/ribs, increased slip resistance of the plain board was increased by 90% for ribs 36 mm apart, and by 130% for ribs at 18 mm (the closer-spaced ribs were 21% more slip-resistant than the broader-spaced ribs).

The P classification increased from the P2 of the plain board to P3 for the 36 mm spaced ribs and P4 for the 18 mm spaced ribs.

For travel along the grain/ribs, there was very little difference in slip resistance between the three conditions.

Slip resistance obliquely across the ribs was 75% and 89% greater than for along them for the 36 mm and 18 mm spaced ribs respectively.

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. 'Along-grain' = parallel with the grain.
5. 'Obliquely across' = offset by 12° from perpendicular to grain (the pendulum is unsuited for perpendicularly across-grain testing of surfaces with protuberances such as these ribs).
6. The sample size is very small; results should be treated cautiously.

Fixtures for multi-directional pedestrian traffic

For slip resistance, fixtures for multi-directional traffic need to be closely spaced and in a matrix configuration.

Fixtures for fore-sole slip resistance during stair descent

Optimum widths of slip resistant fixtures during descent on stair treads depend on tread, footwear size, and descent speed.

Consistent with the foresole contact area indicated above, a slip-resistant fixture measuring about 110 mm from the nosing can be considered generally effective. Other factors influence this but are not discussed here.

Example of nosing fixtures to slip resistance

In this example, two nosing straps were tested with the Pendulum: a 60 mm wide grit-coated metal strap, and a 25 mm wide fluted metal strap (see Figures 3.38 and 3.39). As shown in Figure 3.40, both straps increased the slip resistance substantially, with the wider grit-coated strap being slightly more slip resistant than the narrower fluted metal strap. The slip resistance can be partly attributed to the fact the fixtures were surface mounted and not rebated into the wood. Results would vary considerably with less slip resistant wood, different fixture widths and fixture surfaces. As indicated above ('Parallel fixtures perpendicular to pedestrian traffic'), fixtures wider than the 25 mm and 60 mm of these two samples is recommended.

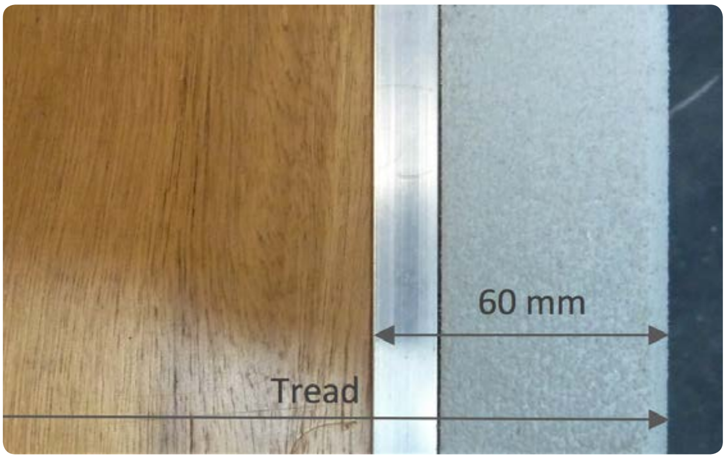


Figure 3.38. Grit-coated metal strap, 60 mm wide, at nosing of wood tread.

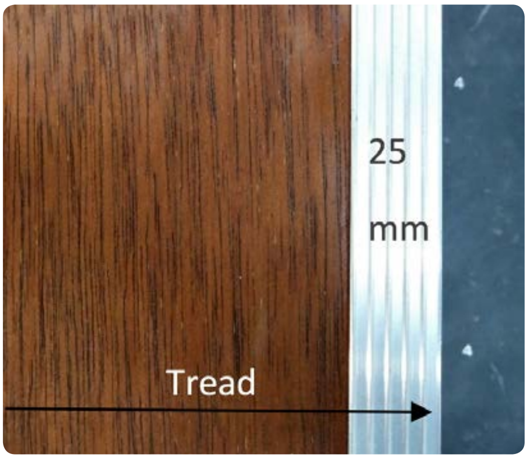


Figure 3.39: Fluted metal strap, 25 mm wide, at nosing of wood tread.

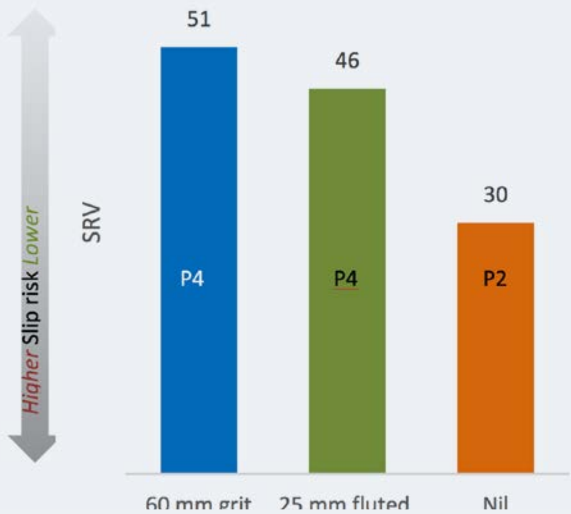


Figure 3.40: Contribution of nosing features to slip resistance.

For this small sample of treads:

The 60 mm wide grit-covered metal strap increased the slip resistance of the plain board (finely sanded, coated with non-micro-grit polyurethane) by 70%; the 25 mm wide fluted metal strap increased it by 53%.

The tread underlying the 60 mm wide grit-covered metal strap was finely sanded and coated Australian hardwood with non-micro-grit polyurethane; the tread underlying the fluted metal strap was fine-sanded uncoated merbau (both with SRV of 30).

The grit-coated strap was 11% more slip resistant than the fluted metal strap (due to the extra width, or possibly the greater effectiveness of the grit coating, or both).

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. 'Across-grain' = perpendicularly across the grain.
5. The samples were positioned so that the end of the Pendulum's test foot run ended at the front edge of each fixture.
6. Very few samples were tested; results should therefore be treated cautiously.

Failure of fixtures over time

Slip-resistant fixtures need to be fixed to resist dislodgement or delamination by foot traffic or items dragged across them so that they do not deteriorate to protrude and become even more susceptible to mechanical damage (and contributing to trip risk). The rebating of fixtures minimises their susceptibility to dislodgment including, for fixtures lipped over and rebated into tread nosings, by people ascending stairs[10].

For wood outdoors, particularly if unseasoned, weather can cause differential thermal and moisture movement between the wood and strips, bands and fixings, thus admitting moisture and promoting failure of fixings, rotting of the wood and increased vulnerability of dislodgement of the strips and bands. Figures 3.41 and 3.42 illustrate the failure of inset straps.

[10] *Lipped fixtures that are rebated into stair risers also avoids the risk of people's feet or the tips of their walking aids being snared during ascent.*

Figure 3.41: Delamination of inset grit-tape straps.

Figure 3.42: Dislodgement of inset straps.

Figure 3.43: Wire netting attachments.

Wire-netting is commonly used for increasing slip resistance. However, it is susceptible to damage and consequent contribution to trip risk.

Tactile ground surface indicators

Tactile ground surface indicators (TGSIs) should not be used as slip-resistant attachments. TGSIs must be slip resistant, but this is not their primary purpose: it is to act as navigation aids for people with low vision.

3.3.5 Coatings

Coatings (applied in liquid or viscous form) tend to supplant the wood substrate's role in slip resistance, increasingly with coating thickness and extent of coverage. Coatings might not increase wood's slip resistance, and could even decrease it, unless they contain suitable particles.

Slip-resistant coatings are available as alkyd, 'latex'[11], polyurethane, epoxy and wax types, and in transparent, semi-transparent and opaque forms.

Particles in slip-resistant coatings are of various sizes, shapes and materials. The materials include silicon dioxide (sand, ground quartz), aluminium oxide, silicon carbide (carborundum), ceramic microspheres, polymer microspheres, ground polypropylene, ground polycarbonate, demineralised potash, and crushed glass[12].

The particles are supplied already within the coatings; separately, for mixing prior to application; or broadcast over freshly-applied coating. The suppliers' stated advantages of their particles over others include lighter weight and therefore greater retention at the finished surface, less reduction of transparency, sharper asperities, and greater resistance to wear.

A review of slip resistance ratings published by suppliers of the various coatings indicate slip resistance ratings of between P3 and P5 when tested new and water-wet with the Pendulum, and between R9 and R11[13] when tested new and oil-wet with the Inclining Platform. The P ratings are similar to those found in the study in Figure 3.43.

[11] *'Latex' is an inaccurately used term to denote coatings that contain synthetic polymers such as acrylic, vinyl acrylic (PVA) and styrene acrylic.*

[12] *In North America and possibly other countries, ground rubber and ground walnut shells are also used as admixtures.*

[13] *'R' ratings are explained in 'Statutory codes'.*

Example of micro-grit-augmented coatings

In the study shown in Figure 3.43, wood with a micro-grit augmented coating was compared with uncoated wood. The coated wood had substantially greater slip resistance than the uncoated wood, achieving a P classification of P3. Different results would be obtained with a combination of coatings of different git type and size and different characteristics of the uncoated wood.

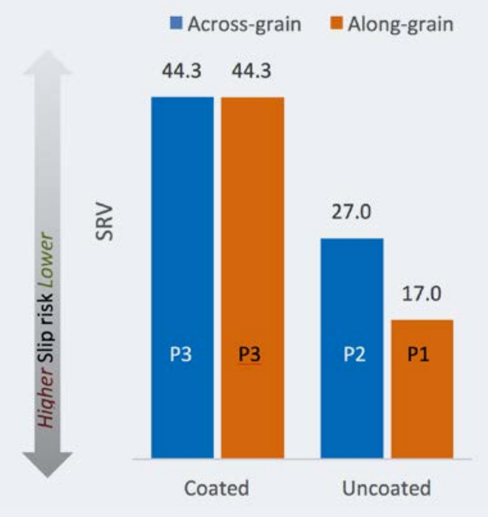


Figure 3.44: Increasing slip resistance with micro-grit augmented coatings

The micro-textured coating increased the slip resistance of the uncoated wood by 64% across the grain and 160% along it, increasing the Class to P3 (almost Class 4) in each case.

The micro-textured coating nullified any difference between across-grain and along-grain slip resistance. For the uncoated condition, across-grain slip resistance was 59% greater than the along-grain slip resistance.

Notes:

1. 'SRV' = Pendulum slip resistance value.
2. 'P' ratings as per AS 4586 and AS 4663.
3. Surfaces were tested when water-wet.
4. 'Across-grain' = perpendicularly across the grain. 'Along-grain' = parallel with the grain.
4. The Sample size was very small; results should be treated cautiously.

Example of micro-roughness of coated and uncoated wood

The micro-roughness of coated and uncoated wood was measured; results are shown in Figure 3.45. Consistent with previous indications of differences between across-grain and along-grain slip resistance, the study demonstrates a similar difference in three travel directions on the wood, with the greatest micro-roughness being for across the grain, the least for along the grain, and intermediate values for the intermediate angle.

Regardless of the travel direction, the thickly coated sample shows the substantially least micro-roughness, consistent with comments above about the paramount slip resisting role of thick coatings.

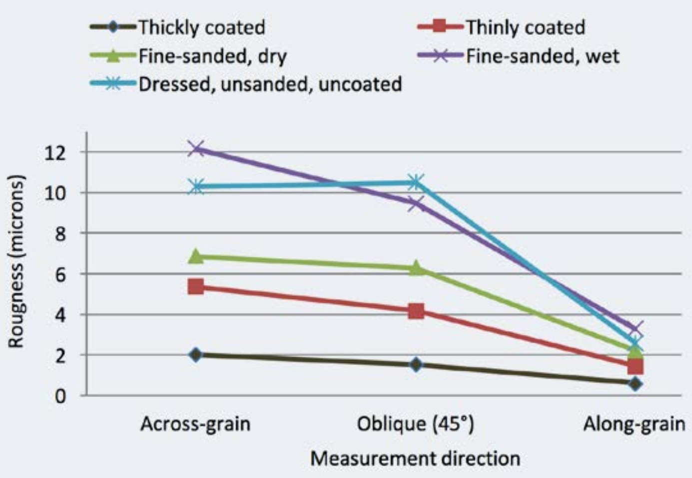


Figure 3.45: Micro-roughness of coated and uncoated wood.

Greatest micro-roughness was for water-saturated, uncoated and fine-sanded wood measured across the grain and, as can be expected, the smoothest was for the thickly coated wood measured along the grain.

Generally, micro-roughness increased with increasing measurement angle from the grain direction.

The contribution of differential surface swelling of the saturated wood is evident.

The small variance in micro-roughness for the thickly coated wood in the three directions affirms the nil difference between across-grain and along-grain slip resistance for the coated wood reported in Figure 39.

Notes:

1. The one sample, Vic. Ash HW, was used for all conditions.
2. 'Fine-sanded, wet' = measured after 10 minutes water saturation
2. 'Across-grain' = perpendicularly across the grain. 'Along-grain' = parallel with the grain.

Degradation of coatings with time

Delamination and erosion of coatings from their wood substrates because of wear, mechanical damage or failure of adhesion, including from moisture ingress outdoors, obviously compromises the efficacy of the slip resistant coatings. The coating and its intended location should therefore be carefully considered, and the coating properly applied for sustained suitability of the coating.

Figure 3.46: Coating degradation outdoors.

The textured coating is delaminating due to weathering, and abrasive wear from traffic. Regular monitoring and maintenenace is necessay to avoid the degradation.

Figure 3.47: Coating degradation due to abrasive wear.

Abrasive wear of smooth-coated timber in an indoor high traffic area has exposed the wood grain. Because traffic through the doorway is across the wood grain, it is possible that the resulting surface is more slip resistant in the entry and exit direction than the original smooth coating.

3.4 Decrease of slip resistance over time

A new surface that only just satisfies the NCC and HB 198 is unlikely to satisfy them after being subject to wear from traffic and, for outdoor settings, weathering. Even short-term use can reduce slip resistance ratings.

Testing the slip resistance of newly completed coatings in situ is recommended to ensure they have been applied effectively in terms of consistency of texture across the coated area and, for sequenced coating systems, that the top coating has not submerged the broadcast particles and diminished the intended slip resistance. Regular periodic testing of coatings should also be conducted to ensure retention of slip resistance.

One coatings supplier, to its credit, indicates the results of slip resistance tests of samples of its products after they have undergone accelerated wear tests so effects of traffic wear on slip resistance can be anticipated (see later for accelerated wear tests).

3.5 Wood slip resistance factors: effectiveness ranking

From the 14 slip resistance tests presented above, factors contributing to slip resistance can be aggregated and ranked for effectiveness, as shown in Table 3.1.

The ranking is expressed in terms of the ‘P’ classifications of AS 4586 and AS 4336, from the lowest (P1) to the highest (P5).

The table seeks to enhance a general understanding of wood slip resistance factors. However, it is based on a very small number of tests and is not based on a rigorously objective compilation. Consistent with previous discussions, it cannot be relied upon for consideration of individual wood products: each one must be considered in the context of its unique environmental conditions and occupational setting.

Table 3.1: P Classifications and Wood Conditions.

Class	Factors	Basis of assessment
P1	**Lowest rating is likely for:** • travel along reeding, grain, weather-textured grain • fungal-coating • finely sanded finish • smooth-sawn finish.	57% of results were for tests along reeding or grain, including weather-textured grain, and fungal-coated boards. 43% were for tests across the grain (obliquely or perpendicularly) of finely sanded or smooth-sawn boards, including some with fungal-coating.
P2	**Low rating is likely for:** • travel along reeding, grain • finely sanded finish • smooth-sawn finish • fungal-coating.	47% of results were for tests along reeding, grain, or metal ribs, including some with fungal coating. 43% were for tests obliquely or perpendicularly across reeding or of the grain of finely sanded or smooth-sawn boards including some with fungal-coating. Note: Fungal-coated boards had grit-augmented coating. Along-grain and across-grain test results were the same, notwithstanding minor surface degradation due to weathering (likely to yield dissimilar results between across-grain and along-grain tests with increased weathering and degradation).
P3	**Mid-rating is likely for:** • travel across grain • grit-augmented coating • metal ribs • reeding (for travel across) • long-term weather-texturing • multi-directional rough-sawn texture.	25% of results were for along-grain tests . 75% were for across-grain tests (obliquely or perpendicularly). Along-grain samples had grit-augmented coating or multi-directional rough-sawn texture. Across-grain test samples had grit-augmented coating, metal ribs, reeding, or long-term weathered texture. The multi-directional rough-sawn textured sample was not tested across the grain because the multi-directionality would probably have masked the underlying grain.
P4	**High rating is likely for:** • travel across-grain • grit-augmented coating • fluted metal nosing band • grit-coated nosing band • metal ribs • broadly spaced reeding (for travel across) • weather-texturing.	18% of results were for along-grain tests of boards with grit-augmented coating. 82% were for across-grain tests (obliquely and perpendicularly) of boards with grit-augmented coating; fluted metal or grit-coated nosing band; broadly spaced (10 mm) reeding; metal ribs; or weather-texturing.
P5	**Highest rating is likely for:** • grit tape covering • grit-augmented coating including broadly spaced reeded board tested obliquely across reeding.	Grit-tape covering or grit-augmented coating; broadly spaced reeded board tested obliquely across the grain/reeding. Except for the full grit tape coverage, other instances of grit tape were of intermittent strips laid perpendicularly across and tested along the grain (slightly higher results would probably occur if the intermittent strips were laid along and tested across the grain).

4 Statutory codes

4.1 National Construction Code

The National Construction Code (NCC)[14] mandates slip resistance for ramps, and stairway treads or near their nosing – at performance requirement DP2 in NCC Volume 1, and P2.5.1 in NCC Volume 2.

NCC Volume 1 is for 'public' buildings and structures (Classes 2 to 9, 10a and 10b). NCC Volume 2 is for 'private' buildings and structures (Classes 1 and 10).

The NCC quantifies the required degree of slip resistance in its Deemed-to-Satisfy provisions (DTSPs)[15]. The DTSPs are expressed identically in NCC Volume One (for Classes 2 to 9 buildings) and Volume Two (for Classes 1 and 10 buildings). In NCC Volume 1, the slip resistance is specified at Part D2.10(c) for ramps, D2.13(A)(v) for stair treads, and D2.14(A)(ii) for landings and summarised in Table D2.14.

In NCC Volume 2, it is specified at Part 3.9.1.4 and in its Table 3.9.1.3.

Fixed platforms, walkways, stairways and ladders are also required to satisfy NCC Parts D2.10, D2.13 and D2.14, except for the concessions given below. The slip resistance ratings are those determined in accordance with Australian Standard *AS 4586-2013: Slip resistance classification of new pedestrian surface materials.*

Table 4.1: NCC slip resistance performance requirements

Classes of building and pedestrian surfaces within them that require slip resistance				**Pedestrian surface**		
				Treads[1]	**Ramps[2]**	**Intermediate & top landings[3]**
Vol.	Class		Building type	Y	Y	Y
Two	1	1a	Detached dwellings	Y	Y	Y
		1b	Attached dwellings, e.g. row house, terrace house, town house	Y	Y	Y
	10	10a	Non-habitable structure, e.g. garage, carport, shed	Y	Y	Y
		10b	Structure such as fence, retaining or free-standing wall[4].	Y	Y	Y
		10c	Private bushfire shelter	-	Y	-
One	10b		Accessible swimming pool[5]	Y	Y	Y
	2-9		All others	Y	Y	Y

Notes

[1] Or nosing of treads; [2] But not ramp landings; [3] Or edge strip of landings;

[4] Relevant here if a ramp or stair is integrated with the structure; [5] Ref NCC Spec D3.10.2.

Table 4.2: NCC DTS slip resistance provisions

Application	**Building class**	**Surface condition**	
		Dry	**Wet**
Ramp <= 1:8	1, 10	P4 or R10	P5 or R12
Ramp > 1:14	2 – 9	P4 or R11	P5 or R12
Ramp > 1:20 <= 1:14	2 – 9, 10B	P3 or R10	P4 or R11
Stair tread	1, 2 - 9, 10		
Stairway landing	2 - 9	P3 or R10	P4 or R11
Tread nosing strip	1, 2 - 9, 10	P3	P4
Stair landing edge strip			

> 'steeper than'; < 'not steeper than'

[14] The NCC is enforced at a State and Territory level by means of States' and territories' building acts and subsidiary regulations. Volumes 1 and 2 of the NCC constitute the Building Code of Australia Volumes 1 and 2.

[15] DTS is titled 'Acceptable Construction' in Volume Two of the NCC.

Slip resistance for NCC DTSP compliance must be determined with a Pendulum or an Inclining Platform (see further below). The 'P' values nominated in the NCC DTSPs are Pendulum test results and the 'R' values are Inclining Platform test results. The P values are for water-wet surfaces and dry textile surfaces (carpets, mats etc.). The R values are for dry and oil-wet surfaces. The values are derived from the Standards Australia handbook *HB198-2014 Guide to the specification and testing of slip resistance of pedestrian surfaces* (see further below). Testing with both tribometers is not required by the NCC: testing with only one is permissible.

Slip resistance features and testing with devices other than those stipulated in the NCC DTS are allowed by the NCC under its Alternative Solutions provisions, if they can be demonstrated to have at least equivalent effectiveness as the DTSPs.

AS 4586

The slip resistance ratings required by the NCC DTSPs must be determined in accordance with Australian standard AS 4586. See Section 5.2. In some circumstances under the NCC DTSPs, the earlier version of AS4586 may be used.

Limitation of the NCC

Under the NCC, stairways are those having more than one riser; hence, neither a single step (one riser) nor two steps (one tread and two risers) needs to satisfy the NCC for slip resistance[16].

The NCC DTSPs do not address slip resistance for stepped aisles in theatres, auditoria, spectator stands and the like.

The NCC has statutory force only for buildings (and associated structures and facilities) that require a building permit. It is therefore not applicable to existing buildings, or structures and facilities that do not require a permit under the NCC, although there is no reason why the NCC requirements cannot be used as criteria in circumstances where the NCC is inapplicable.

Concessions and limitations of the NCC DTSPs

If a fixed platform, walkway, stairway or ladder 'only serves (a) machinery rooms, boiler houses, lift-machine rooms, plant-rooms, and the like; or (b) non-habitable rooms, such as attics, storerooms and the like that are not used on a frequent or daily basis in the internal parts of a sole-occupancy unit in a Class 2 building or Class 4 part of a building', the NCC DTPSs may be disregarded in favour of the non-prescriptive AS 1657[17] (see further below). This also applies to farm buildings or sheds[18]. Similarly, a stairway 'that provides access to a service platform, rigging loft, or the like' in Class 9b buildings need only comply with AS 1657[19].

Because the NCC DTSPs reference AS 4586, they are limited to new, yet-to-be-used surfaces; the NCC DTSPs do not address existing surfaces (these are addressed by AS 4663). In other words, NCC DTSPs compliance relates only to new surfaces although, outside the matter of DTSPs compliance, the DTSPs they can be validly applied to existing surfaces under the NCC performance provisions AS 4663 (and HB:198-2014).

Slip-resistance ratings for ramps into and the sloped entries of swimming pools[20] are not included in the NCC DTSPs.

Configuration of slip resistance on ramps, stair treads and landings

The NCC does not stipulate the configuration of slip resistant characteristics, nor therefore the width and spacing of intermittent slip resistant elements. It simply states that they be provided on (NCC-required) ramps and, on stairways, on their treads and landings or on their leading edges. For characteristics at leading edges, the NCC does not provide any dimensions, nor therefore the width and spacing of intermittent slip resistant elements. It simply states that they be in the form of a 'nosing strip'.

[16] *There is no definition of stairway in the NCC. However, it is inferred in D2.13 (a)(i) Goings and risers.*

[17] *Ref. NCC Part 2.18.*

[18] *Ref. NCC Part H3.5.*

[19] *Ref NCC Part H1.6.*

[20] *Ref. NCC Part D3.10*

Slip-resistant luminance-contrasting nosing strips

Volume 1 of The NCC at Part D3.3(a)(ii) requires compliance with 11.1(f) of *AS1428.1-2009: Design for access and mobility, Part 1: General requirements for access – New building work*. AS 1428.1 requires the provision of a 50 mm to 75 mm wide luminance contrasting strip at tread nosings. However. neither the NCC nor AS1428.1 directly associates slip resistance requirements with luminance contrast strips, although it is clearly subsumed under NCC's requirement for slip resistance under Parts D2.13(A)(v) and D2.14(A)(ii).

It is common practice that luminance contrasting strips have slip-resistant coatings or inserts, but which are not commonly provided for the whole width of the strip and that they are provided in a width of 50 mm. Consequently, the slip resistance might have a width of only 40 mm. Such widths represent industry practice, not specifically NCC requirements.

NCC Volume 1 does not require luminance contrasting strips at nosings.

4.2 Disability Discrimination Act Standards

The Disability Discrimination Act (DDA) requires compliance with its buildings[21] and public transport standards[22].

The slip-resistance requirements of the DDA buildings standards are in the DDA Access Code for Buildings (ACB) and are the same as those of the NCC. Compliance with the NCC constitutes compliance with the ACB and vice versa, including with respect to slip-resistance ratings.

The DDA public transport standards apply to public transport conveyances, premises and infrastructure. The only stipulation in the standards for slip resistance is for 'boarding devices' (such as manually or mechanically operated ramps and lifts); there is no stipulation for other types of pedestrian surface. However, the standards require compliance of ground and floor surfaces (including 'access paths' and ramps[23]) with Australian standard *AS 1428.2-1992 Design for access and mobility Part 2: Enhanced and additional requirements – Buildings and facilities* which, in turn, requires compliance with the then current version of *AS1428.1 Design for access and mobility Part 1 – New building works.* AS1428.1 requires that ground or floor surfaces of 'continuous accessible paths of travel[24] and circulation spaces[25] be slip resistant; however, it is silent about the need for slip resistance of steps and stairways[26,27,28] and does not stipulate minimum slip-resistance ratings.

Public pedestrian areas under the DDA

Public pedestrian areas come under the meaning of 'premises' in the DDA. There is no DDA slip-resistance requirement for public pedestrian areas, except to the extent that they are referenced within the DDA buildings and public transport standards.

4.3 Other statutory obligations

Other obligations reside in municipal bylaws and policies for public areas; in States' and Territories' health and safety acts and codes of practice; and in States' and Territories' regulations that mandate essential safety measures and building maintenance. However, the obligations are expressed in performance terms without quantification of minimum slip resistance ratings.

[21] *Disability (Access to premises – Buildings) Standards, 2010*

[22] *Disability Standards for Accessible Public Transport 2002, as amended 2010*

[23] *AS1428.2 also requires kerb and step ramps to be slip resistant.*

[24] *A continuous accessible path of travel is 'an uninterrupted path of travel to, into or within a building providing access to all accessible features'. Accessible features are, at the least, those nominated by the NCC.*

[25] *A circulation space is a 'clear unobstructed area, to enable persons using mobility aids to manoeuvre'.*

[26] *Providing that the accessible paths of travel and circulation spaces are 'traversable by people who use a wheelchair and those with an ambulant or sensor disability'*

[27] *Though not stated in AS 1428.1, continuous accessible paths of travel are those that are usable by wheelchairs and therefore excludes steps and stairs.*

[28] *However, it requires stairway and ramp landings to have tactile ground surface indicators (TGSIs) that comply with AS1428.4.1 and that therefore must be slip resistant*

5 Non-statutory codes

5.1 AS/NZS 3661.2

Australian and New Zealand standard *AS/NZS 3661.2-1994, Part 2: Guide to the reduction of slip hazards* provides useful information relevant to wood surfaces.

Key sections are: Selection of pedestrian surfaces for slip resistant characteristics; Slipping problems during installation of finishing; Care and maintenance of floors; and Reduction of slip hazards on existing floors.

Part 2 refers to Part 1 of AS/NZS 3661; however, Part 1 has been superseded by AS 4586 and AS 4663.

5.2 AS 4586

Australian standard AS 4586-2013*: Slip resistance classification of new pedestrian surface materials* describes tribometers and testing procedures but does not indicate required or recommended slip resistance ratings. The tribometers are the British Pendulum, the Inclining Platform and the Dry Floor Friction Tester (see Section 7).

AS 4586 stipulates directions in which wood surfaces must tested (as indicated in Sections 7.1.1 and 7.1.2), and describes a method for measuring the roughness of very coarse textures ('Displacement Volume Test'). It also provides a formula and table for relating slip resistance values obtained with the Pendulum and Dry Floor Friction Tester with sloped surfaces.

AS 4586 is a non-statutory code, but it has statutory authority because it is referenced by the NCC.

5.3 AS 4663

A surface when new that satisfies the NCC and HB 198:2014 is unlikely to satisfy them after it has been subject to wear from traffic and, for outdoor settings, weathering.

Australian standard AS 4463-2013: *Slip resistance classification of existing pedestrian surfaces* was developed for surfaces after they had been applied and used. AS 4663 is very similar to AS 4586 insofar as it describes tribometers and testing procedures and does not indicate required or recommended slip resistance ratings. A principal difference is that it does not include the Inclining Platform or Displacement Volume test methods.

An important difference between AS 4586 and AS 4663 is that AS4586 is addresses individual samples, whereas AS 4663 also addresses larger surface areas and includes a procedure for obtaining a representative sample of the area.

Like AS 4586, AS 4663 stipulates directions in which wood surfaces must tested, as indicated in Sections 7.1.1 and 7.1.2.

5.4 AS 1657

Australian standard AS 1657-2013: *Fixed platforms, walkways, stairways and ladders – Design, construction and installation* stipulates the need for underfoot surfaces of walkways, platforms, landings, stair treads and ladder rungs to be slip resistant, although it does not specify required slip resistance ratings.

The standard creates a quandary because it references AS 3661 and HB 197, neither of which are current standards (see further below). Given the non-prescriptive nature of the standard, it would be reasonable to apply AS 4586 or AS 4663, as appropriate, and HB 198.

5.5 Standards Australia Handbook HB:198-2014

Standards Australia Handbook HB:198-2014 (HB 198) is an important guideline because it is the source of the NCC DTSPs for slip resistance (Table 3A of HB 198), and for slip resistance ratings in circumstances not covered by the NCC (Table 3B).

The handbook addresses the methods of measuring and clarifying slip resistance using the slip testing devices and procedures referenced by AS 4586 and AS 4663. The standards relate to water-wet, oil-wet and dry surfaces as tested by the Pendulum, the Dry Floor Friction Tester and the Inclining Ramp.

The classifications in the schedule should be interpreted in the context of risk management. Slip resistance results less than the classifications will increase the probability of slips (and falls) and results greater than the classifications will decrease the probability.

A schedule of slip resistance classifications indicated by Tables 3A and 3B of HB 198 is reproduced below at Table 6.1.

HB 198 is an interim, partial, successor to Standards Australia Handbook HB:197-1999 (HB 197). HB 197 references an earlier version of AS 4586-2013. HB 197 has no statutory relevance to new building works.

6 Common Law

Apart from statutory obligations, duty of-care obligations exist under Common Law. These are not prescribed but are established by legal precedent or as new findings on a case-by-case basis.

Compliance with the NCC DTSPs can provide substantial and even sufficient protection against legal action under Common Law, although not necessarily if the compliance was with a superseded version of the DTSPs and legal action was taken for an incident occurring after the introduction of the new version. Conversely, non-compliance with the NCC DTSPs can provide the basis of action at Common Law.

The performance provisions of the NCC, and other non-prescriptive statutory obligations do not provide the same defence against or basis of action at Common Law because of their non-prescriptiveness. In these instances, it would need to be proven by a defendant that duty-of-care obligations had been discharged or, by a plaintiff, that they had not been discharged.

Satisfying Australian standards, regardless of statutory requirements can also provide defence against or the basis of legal action.

Table 6.1: Recommended minimum slip resistance classifications indicated of HB 198. (continued on next page).

Location	Slip testing device	
	Pendulum	**Inclining platform**
External Pavements and Ramps		
External ramps including sloping driveways, footpaths etc. steeper than 1 in 14	P5	R12
External ramps including sloping driveways, footpaths, etc. under 1:14, external sales areas (e.g. markets), external carpark areas, external colonnades, walkways, pedestrian crossings, balconies, verandas, carports, driveways, courtyards and roof decks	P4	R11
Undercover car parks	P3	R10
Hotels, Offices, Public Buildings, Schools and Kindergartens *Entries and access areas including hotels, offices, public buildings, schools, kindergartens, common areas of public buildings, internal lift lobbies.*		
Wet area	P3	R10
Transitional area	P2	R9
Dry area	P1	R9
Toilet facilities in offices, hotels and shopping centres	P3	R10
Hotel apartment bathrooms, en-suites and toilets	P2	A
Hotel apartment kitchens and laundries	P2	R9

**Entries and access areas including hotels, offices, public buildings, schools, kindergartens, common areas of public buildings, internal lift lobbies.*

Table 6.1: Recommended minimum slip resistance classifications indicated of HB 198. (continued).

Location	Slip testing device	
	Pendulum	Inclining platform
Supermarkets and Shopping Centres		
Fast food outlets, buffet food servery areas, food courts and fast food dining areas in shopping centres	P3	R10
Shop and supermarket fresh fruit and vegetable areas	P3	R10
Shop entry areas with external entrances	P3	R10
Supermarket aisles (except fresh food areas)	P1	R9
Other separate shops inside shopping centres – wet	P3	R10
Other separate shops inside shopping centres – dry	P1	R9
Loading Docks, Commercial Kitchens, Cold Stores, Serving Areas		
Loading docks under cover and commercial kitchens	P5	R12
Serving areas behind bars in public on hotels and clubs, cold stores	P4	R11
Swimming Pools and Sporting Facilities		
Swimming pool ramps on stairs leading to water	P5	C
Swimming pool surrounds and communal shower rooms	P4	B
Communal changing rooms	P3	A
Undercover concourse areas of sports stadiums	P3	R10
Hospitals and Aged Care Facilities		
Bathrooms and en-suites in hospitals and aged care facilities	P3	B
Wards and corridors in hospital and aged care facilities	P2	R9

Equivalence of P values and Coefficient of Friction

Using the formula given in F1 of AS 4586 and C1 of AS 4663, the Coefficient of Friction values corresponding with the Slip Resistance Values determined with the Pendulum slip resistance tester are shown in the following table.

Table 6.2: Equivalence of P values and coefficient of friction

Pendulum Slip Resistance Values converted to Coefficient of Friction				
Class	Slider 96 rubber		Slider 55 rubber	
	SRV	CoF	SRV	CoF
P1	12 – 24	0.11 – 0.24	<20	<0.19
P2	25 – 34	0.25 – 0.35	20 – 34	0.19 – 0.35
P3	35 – 44	0.36 – 0.46	35 – 39	0.36 – 0.40
P4	45 – 54	0.47 – 0.59	40 – 44	0.41 – 0.46
P5	>54	>0.59	>44	>0.46

7 Assessing Slip Resistance

7.1 Slip Resistance Measuring Devices

There is a multitude of slip resistance measuring device types throughout the world. They include pendulum, skid, trolley, ramp and impact types. Many are portable and suitable for on-site and *in situ* testing, but others, such as the inclining platform, are not. Very few are suitable for *in situ* testing of mid-flight stair treads.

Three types of devices are recognised by Standards Australia: the portable British Pendulum, portable ‘Dry Floor Friction Tester’, and the non-portable Inclining Platform. AS 4586 and AS 4663 describe the standardised testing procedures, and HB 198-2014 provides supporting information and consumer guidance.

7.1.1 British Pendulum

This portable device comprises a suspended radial rod at the end of which is a spring-tensioned rubber pad (‘slider’). When the rod is released, it swings down onto the test surface and drags the edge of the pad along it. The extent to which the rod rotates about its fulcrum indicates the resistance of the surface to the sliding of the rubber edge. The pendulum can be used for testing wet and dry surfaces and a wide variety of surface textures, although surfaces with coarse texture or protuberances, cavities or troughs can adversely affect the travel of the test pad over the surface. This can be ameliorated by adjusting the orientation of the surface being tested.

Material other than rubber can be used for the pad, and it can be used at reduced lengths, although this would place tests results outside the scope of the Australian Standards.

The Australian Standards AS 4586 and AS 4663 require that wood is tested parallel with the grain, and that stairway treads be tested in the direction of descent[29]. To avoid the adverse effects of very coarsely textured surfaces, including therefore multi-protuberant surfaces, the Standards require that they be orientated at an angle to the pendulum so that a part of the pad is always on the top of protuberances and so that the pad remains parallel with the plane of the tops of the protuberances.

Figure 7.1: The British Pendulum.

[29] *Because the pendulum is too large for treads, the standards allow for top landings, if they have surfaces identical to the treads, to be tested as substitutes for the treads.*

7.1.2 The 'Dry Floor Friction Tester'

The 'Dry Floor Friction Tester', as it is identified in AS 4586 and AS 4663, is a portable, autonomously travelling device that drags a very small rubber pad across the test surface, digitally recording the slip resistance of the surface to the pad as it moves. Its name is prefixed here with 'Dry' to indicate that it is widely considered to be only suitable for testing dry surfaces[30], and for which it is the only application considered by the Australian standards for it.

The smallness of the test pad restricts its use to surfaces that are finely textured and that do not have protuberances, cavities or troughs.

Figure 7.2: Tortus Dry Floor Friction Tester. *From http://www.mastrad.com/tortus3.pdf*

7.1.3 Inclining Platform

Sometimes referred to as a ramp[31], the inclining platform is a non-portable device. Samples of a pedestrian surface are placed on the narrow platform and a person takes several steps (forwards down, then backwards up the platform). The angle of inclination at which the person slips is recorded as the slip resistance of the samples.

Figure 7.2: Inclining platform *From https://oshwiki.eu/wiki/File:ERO-10-06-b-7b.fig1.jpg#file*

[30] *It is widely accepted internationally that results of tests with the Dry Floor Friction Tester on wet surfaces are insufficiently reliable.*

[31] *Such as in earlier versions of the Standards Australia standards; they now refer to it as 'inclining platform'.*

The inclining platform is customarily used with people in footwear of standardised synthetic outsoles or in bare feet because the SA standards provide for these conditions. However, it can be used with any type of footwear or foot covering.

The platform is customarily used with water-wet or oil-wet surfaces, although surfaces can be tested for any fluid or other matter, or when they are clean and dry. Under AS 4586, wood needs to be tested so that the grain or directional texture is aligned with the slope of the platform.

The inclining platform enables the testing of surfaces that range from very smooth to coarsely textured or multi-protuberant. There are very few inclining platforms in Australia.

7.1.4 Other slip resistance measuring devices

Another portable slip testing device commercially available and used by some practitioners in Australia is a very small trolley fitted with a very small rubber test pad like that of the Dry Floor Friction Tester.

It operates by being placed on a small ramp so that, when released, it rolls down the ramp onto the test surface. The travel distance on the test surface is used to determine the slip resistance of the surface. Its use is best suited to surfaces that are not coarsely textured nor that have protrusions, cavities or troughs.

A prototype portable tribometer has been developed by the author. It can test horizontal and inclined surfaces, stair treads and nosings, and a range of textured surfaces including highly textured ones such as Tactile Ground Surface Indicators. It can be fitted with an electronically recordable and multi-articulated foot (including with footwear), and can simulate reflexive responses to slips (see Figure 7.3). An earlier prototype was used in preliminary research of which some results are shown in this guide.

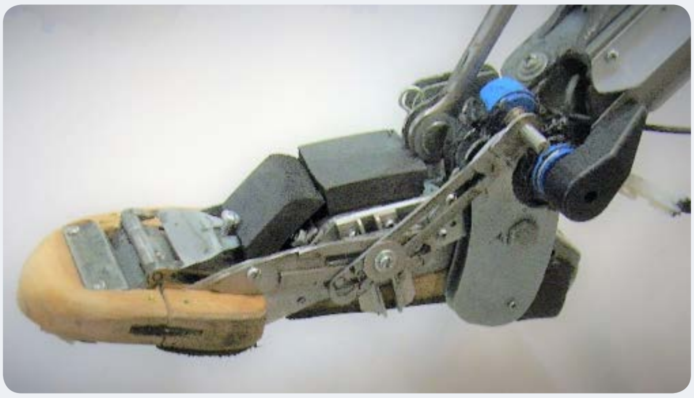


Figure 7.3 Replica of human foot

Test foot used in a prototype tribometer. It is multi-articulated and, with the leg, is electronically recordable. It can be shod with footwear; used for a large range of conditions; and simulate reflexive responses to slips.

7.2 Texture measurement

An ancillary method for assessing slip resistance, although not for directly measuring it, is surface texture measurement.

It is customary to refer to microtexture and macrotexture, with the former regarded as the primary agent in slip resistance. However, there can be several scales of texture that can simultaneously influence slip resistance. Generally, there is a correlation between surface texture and slip resistance such that an increase in texture tends to increase the slip resistance. However, there are many criteria for quantifying texture and there is no broadly accepted quantum of texture that reliably and accurately correlates with slip resistance.

There are two methods of texture measurement methods, one with profilometers for microtexture and another by concavity volume calculation for macrotexture.

7.2.1 Profilometers

There are two common types of hand-held profilometers used for testing pedestrian surfaces—stylus and optical. The stylus type has a diamond tipped needle whose vertical displacement is recorded as it is moved across the surface; the optical type relies upon reflected light which records variation of texture as the device is moved.

Although surface texture as an indicator of slip resistance is promoted in the United Kingdom, for example, the reliability of the method is not universally accepted. A difficulty is that there are many ways of defining texture, including with respect to shape, spacing and heights of asperities.

Hand-held profilometers are limited to measurement of very fine textures, such as dressed or coated wood. Moreover, granular textures such as carborundum finishes can exceed the capability of stylus profilometers.

Figure 7.4: Stylus profilometer

This is the profilometer used for the study at Figure 3.44. Stylus profilometers are only suitable for micro-textured surfaces and, typically, only those that are not comprised of individual sharp particles such as comprise grit-tape and grit-coated fixtures.

7.2.2 Concavity volume calculation

This method can be used for textures that cannot be measured by profilometers. The procedure entails filling a particular sized area of a textured surface with very fine sand or paste and identifying the volume that occupies the area. AS 4586 describes this method using paste.

7.3 Simulated wear

Because surface modification from traffic and, for exterior surfaces, weather and other environmental conditions can alter slip resistance, it can be valuable to simulate this by subjecting the surfaces to an accelerated wear procedure prior to measuring or re-measuring the surfaces for slip resistance.

As indicated above, two opposite results of modification can occur from wear and weathering. If the wood is initially polished or coated, the surface texture will tend to increase as will the slip resistance (at the cost of appearance). If the wood is initially textured or has protuberances, it will tend to become less textured or more planar, and therefore less slip resistant.

7.3.1 Abraders

Two types of abraders are commonly used for accelerated simulation of wear: a powered or manually operated horizontally reciprocating type, and a rotating (Tabor) type. The reciprocating device can test a larger area than the rotating type which can only treat a very small area, and samples are simpler to prepare for it than for the rotating type. However, the rotating device can be better suited than the reciprocating device for surfaces that are coarsely textured or have protuberances.

Neither of the reciprocating and rotating devices replicates foot impact forces and they are therefore unable (nor intended) to replicate mechanical damage to surface protuberances or the edges of surface cavities and troughs. These features can contribute to slip resistance, so their blunting or removal will tend to reduce slip resistance.

49

Long-span Timber Floor Solutions

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Metsä Wood

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-69-0

Prepared by:
Bella Basaglia, Kirsten Lewis
Dr Rijun Shrestha
Emeritus Prof. Keith Crews

First Published: July 2019
Revised: January 2020

Contents

Contents

1 Introduction

This guide covers two timber floor solutions – cassette type floors (using LVL or glulam web and LVL or CLT flanges) and panel-type floors (using CLT or combination of CLT with LVL or glulam secondary members) – that have the potential to be used for at least 9 x 9 metre mid-rise commercial building.

These floor alternatives have been arrived at, based on industry input, to address key concerns when designing long-span timber floors: constructability and floor dynamics.

Recommendations on design criteria, procedure and parameters for vibration design are based on existing knowledge from literature and supported by extensive laboratory tests.

It is well understood from previous studies that once the floor span exceeds 6 m, serviceability limit state requirements, especially vibration behaviour, rather than strength limit state requirements tend to govern the design. This guide addresses the performance requirements of the floors to meet the strength and serviceability limit state design requirements and the focus will be on design considerations for floor dynamics. The design process for the two floors is presented in two separate sections (Sections 3 and 4) but some of the steps and design criteria are common for both types.

These floors have been designed to be able to satisfy serviceability and ultimate limit state design as well as to ensure that both the systems are modular, suit prefabrication and are simple to assemble on site. The proposed panel-type floor can be built using CLT only or a combination of CLT supported on secondary LVL or glulam members while the cassette floors can be built into box-beam type sections but it may be beneficial to use the floor cavity for installing services and insulation. Access to the floor cavity in such case will require either the top or the bottom flange to be a non-structural component.

2 Floor Design Requirements

Performance requirements of a ribbed deck floor must address ultimate and serviceability limit states. Load type, load combinations and modification factors for both ultimate and serviceability limit states have been defined in accordance with the AS 1170 standards. The limit states that require checking, which have also been identified in previous studies on design requirements for long-span timber floors (WoodSolutions Technical Design Guide #31), are:

- Short-term ultimate limit state – response of the structure under maximum load.
- Long-term ultimate limit state – response of the structure to quasi-permanent loading and avoiding failure due to creep of the timber member in particular.
- Short-term serviceability limit state – instantaneous response of the structure to an imposed load.
- Long-term serviceability limit state – time-dependent variations of the material properties to identify the service life behaviour.
- Serviceability limit state – instantaneous response to an imposed load of 1.0 kN at mid-span as an indication of dynamic behaviour. This criterion alone is, however, not sufficient in satisfying vibration design. Further checks, particularly for spans greater than 6 m, are required and are detailed in the later sections.

Once the span of the floors exceeds 6 m, it is likely that the design will be governed by vibration and therefore more rigorous vibration design checks will be essential. This design guide will, therefore, focus on these additional design checks. Fire and acoustic designs are outside the scope of this design guide.

3 Design of Cassette Floor System

3.1 Overview

Ribbed-deck cassette floors consist of timber joists rigidly connected to a flange. Engineered wood products (EWPs) are used to make up the cassette where LVL or glulam can be used as the web while the flange can be made from LVL or CLT. The cassette can be manufactured from off-the-shelf products to reduce costs and streamline the fabrication process. Typical dimensions and grades for an LVL ribbed-deck cassette, shown in Table 1, are based on Nelson Pine LVL products. To achieve composite action, the flange should be connected to joists with a combination of adhesives (e.g. Purbond) and mechanical connectors (screws). Composite action is essential to increase stiffness and allow for longer spans.

From an ultimate limit state perspective, spans over 9 m are achievable, making it a suitable option for commercial buildings. However, the design is typically governed by vibration serviceability under human-induced walking excitations, which is a current area of investigation internationally and at the University of Technology, Sydney.

Several configurations of the web and the flange can be used e.g. flange connected to the top of the web only (Figure 1(a)), flange connected to the bottom of the web only (Figure 1(b)) or web sandwiched between both a top and bottom flange (Figure 1(c)). Each configuration has its own advantages and disadvantages in terms of construction and services installation and should be chosen based on the building requirements. This guide refers to a design procedure based on a ribbed-deck floor with top panel only. However, all the design criteria and considerations remain the same for other configurations.

Table 1: Typical dimensions of ribbed deck cassette floor based on Nelson Pine LVL products.

Component	Typical dimensions	Typical grade
Web	45 or 63 mm breadth x 300 or 360 or 400 mm depth	LVL13
Flange	45, 63 or 90 mm thickness x 1220 mm width	LVL11 or LVL13

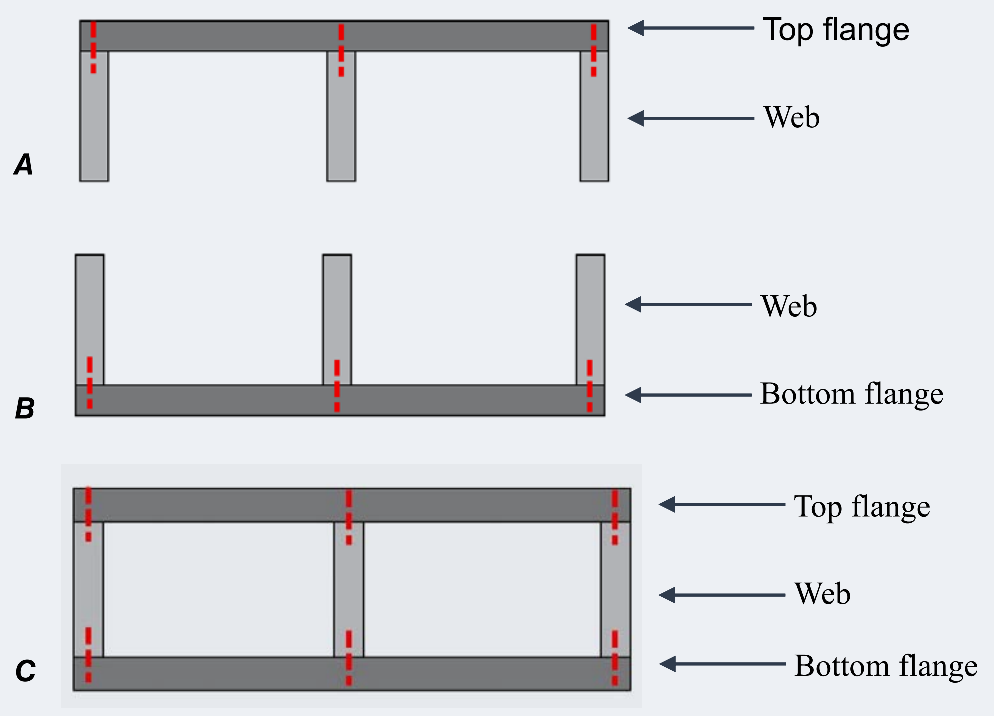


Figure 1: (a) top flange and web configuration; (b) bottom flange and web configuration; (c) web members sandwiched between top and bottom flange.

3.2 Design Considerations

3.2.1 Shear Deformation

Deflection of beams consists of bending and shear deformation. Typically, in concrete and steel, shear deformation in steel and concrete floor systems is negligible as most deflection is dominated by bending. However, timber has a low shear modulus, for example 0.66 × 10^3 MPa for LVL13 grade compared to 80 × 10^3 MPa for structural steel. As a result, the ratio of Modulus of Elasticity to shear modulus is high (i.e. 25 for LVL13), which can indicate that the shear component of total deflection becomes more significant (Skaggs and Bender 1995). Consequently, it is recommended that shear deformation is considered in both design and finite element modelling methods.

The American Institute of Steel Construction (AISC) Design Guide 11 (Murray et al. 2016) provides an equation for a reduced effective moment of inertia, I_e, for composite steel-concrete beams and trusses which accounts for shear deformation. It is recommended that this reduced effective moment of inertia be used for serviceability design:

$$I_e = \frac{I_{comp}}{1 + 0.15\, I_{comp}/I_{chords}}$$

where I_{comp} = fully composite transformed moment of inertia and I_{chords} = moment of inertia of chord or joist areas alone.

3.2.2 Shear Lag Effect

As the floor undergoes bending, there is a shear transfer between the web and flange members. However, Figure 2 shows that the stress distribution across the flange is not uniform. The shear lag effect considers this variation or 'lag' in stress via an effective flange width. This is defined as the width of panel, which effectively contributes to the stiffness of the floor system.

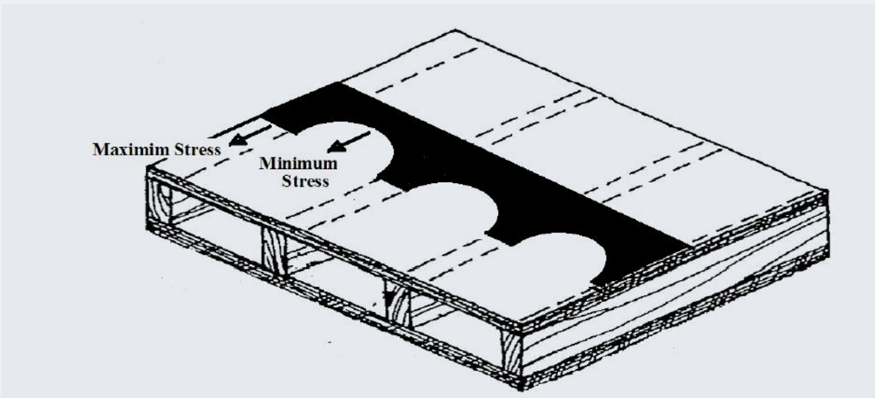


Figure 2: Shear lag effect in a ribbed deck floor system (Zabihi 2014).

Eurocode 5 (2004b) provides effective width calculations for shear lag effect in thin flanges such as plywood, oriented strand board and particleboard. The assembly is considered as a number of I-beams or U-beams (see Figure 3). Plate buckling effects are also considered for the compression flange (top flange), however, this is unlikely to occur in thick flange members used in long-span cassette floors.

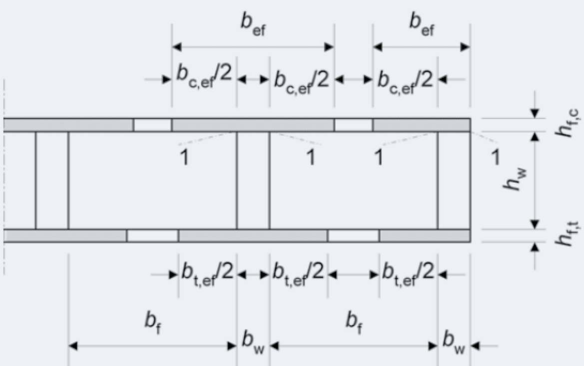


Figure 3: Thin-flanged beam (CEN 2004b).

Although there is no guidance for LVL or CLT flanges, which will be thicker than typical materials used in residential buildings such as plywood, OSB, etc, designers should ensure that the centre-to-centre web spacing is such that shear lag effects in the flange do not occur. If the effective width is greater than the web centre-to-centre spacing, shear lag effects will not occur. Based on Clause 9.1.2. Eurocode 5 (CEN 2004b), Equations 3.2 and 3.3 can be used to calculate the effective width of the top and bottom flanges. As an example, a 9000 mm spanning cassette with 90 mm top flange and 63 mm wide joist can have a maximum centre-to-centre spacing of 963 mm before shear lag effects occur in the flanges.

Bottom flange:

$$b_{f,t} = b_w + 0.1 \times span \tag{3.2}$$

Top flange:

$$b_{f,c} = \min(b_w + 0.1 \times span, b_w + 20 \times h_f) \tag{3.3}$$

where $b_{f,t}$ and $b_{f,c}$ are the effective width of the top bottom (tension) and top (compression) flange, respectively, and b_w and h_f is the web breadth and thickness of top flange, respectively. For $b_{f,c}$, the effective width considers plate buckling; however, this is unlikely to occur in thick flange members used in long-span cassette floors.

3.2.3 Blocking

Blocking or bridging provides lateral stability to the web which is particularly crucial for long-span cassette floors with no top flange. According to Ozelton and Baird (2006), lateral buckling of a beam depends on:

- depth-to-breadth ratio (or I_x/I_y ratio)
- the geometrical and physical properties of the beam section
- the nature of the applied loading with respect to the neutral axis of the section
- the degree of restraint provided at the vertical supports and at points along the span.

For ribbed deck floor systems, the top flange provides a degree of lateral restraint. Equation 3.4 from Clause 3.2.3.2 (b) in AS 1720.1 (2010) should be satisfied in order to determine whether the top flange provides continuous lateral restraint to the web member. L_{ay} refers to the distance between points of effectively rigid restraints against lateral movement i.e. distance between screws connecting the flange to web. If this equation is satisfied, blocking is not structurally required for ultimate limit strength, however, should still be used for ease of fabrication. For cassettes with only a bottom flange, blocking will be required to satisfy lateral buckling checks as per Clause 3.2.3 in AS 1720.1 (Standards Australia 2010).

$$\frac{L_{ay}}{d_w} \leq 64\left(\frac{b_w}{\rho_b d_w}\right)^2 \tag{3.4}$$

where d_w is the depth of the web and ρ_b is the material constant for a beam (Clause 3.2.4 AS 1720.1 (Standards Australia 2010).

3.2.4 Thinner Outer Joists

Outer joists for ribbed deck floor cassettes can be designed to have a thinner breadth since they only take half the load. This will create a more structurally efficient cross-section and save costs on material.

3.3 Design Procedure

The design procedure has following three fundamental stages:

1. Identifying the characteristics of the ribbed deck cross-section
2. Evaluation of the strength capacity
3. Assessment of the serviceability limit.

3.3.1 Cross-section Characteristics

Section properties of ribbed deck floor sections can be calculated assuming fully composite action between the web and the flange. Transformed section method can be used when the flange and the web have differing Modulus of Elasticity. Figure 4 shows notations for a typical cross-section of a ribbed deck floor with top and bottom flange.

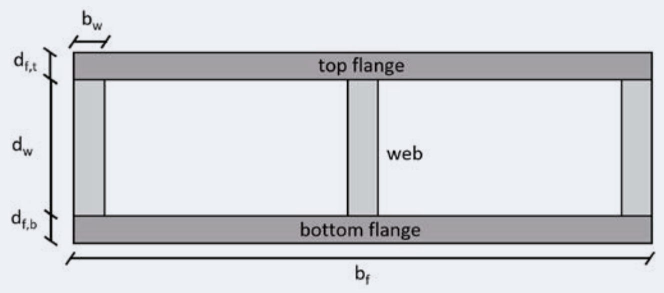


Figure 4: Notation for typical cross-section of ribbed deck floor system with top and bottom flange.

3.3.2 Evaluation of Strength Capacity

The following capacity checks need to be made:

- bending capacity of the cross-section
- axial capacity (compression) of the top flange
- axial capacity (tension) of the bottom flange (if present)
- combined bending and compression of the top flange
- combined bending and tension of the bottom flange (if present)
- shear capacity in web
- shear flow at interface between web and flanges
- shear strength at glue line
- bearing strength.

Appropriate equations of these checks are given in WoodSolutions Technical Design Guide #31: Timber Cassette Floors and can similarly be used for the design of ribbed deck cassette floors. The design procedure is based on AS 1730.1:2010 Timber structures Part 1: Design methods with load actions predicted using the AS 1170 Structural Design Action series.

3.3.3 Serviceability

Deflection
Deflection must be checked for short-term and long-term serviceability load combinations. The limits depend on the functional requirements of the building being designed. Appropriate equations for serviceability checks can be found in Wood Solutions Technical Design Guide #31: Timber Cassette Floors and can similarly be used for the design of ribbed deck cassette floors. A reduced effective moment of inertia is to be considered in deflection calculations as per Equation 1.

Vibration
Two approaches exist for the vibration design of floors: simplified design using hand calculations or a spreadsheet and finite element (FE) modelling. Choice of the method primarily depends on the complexity of the floor but is also influenced by the stage of structural design and the end-use of the floor. The orthotropic nature of timber cassette floors can result in closely spaced modes that can amplify the motion, negatively affecting the dynamic response of the floor (Khokhar, 2004). People tend to be more annoyed when there are two closely spaced frequencies (Ljunggren 2006; Ljunggren, Wang & Ågren 2007) and consideration of these higher modes during design is recommended (Brownjohn & Middleton 2008; Ljunggren 2006). Consequently, finite element modelling is recommended to obtain modal properties including frequency and mode shapes. Response analysis and assessment can then be performed, either through hand calculations using classic dynamic theory or through the FE model if the software allows. The following sections outline a vibration design procedure for long-span ribbed-deck cassette floors that stems from literature review, observations from testing at UTS on a 9 m span ribbed-deck floor (with top flange only) and current vibration design guides for other floor materials. Only dynamic loading from walking excitation is considered as it is the most common form of human induced excitation. The final project report, PNA 341-1415, for this FWPA-funded research is available on the FWPA website.
An overview of the stages of vibration design and associated questions is presented in Figure 5 and Table 2, respectively. The answers to the questions in Table 2 are primarily influenced by the floor use. Design processes for vibration are detailed in Section 4.5.

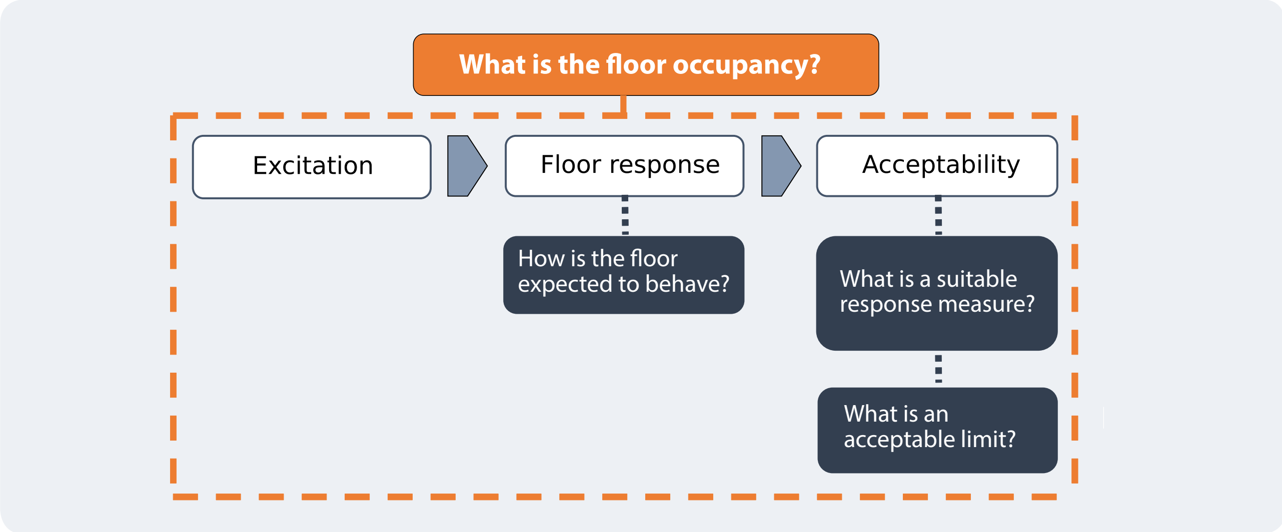


Figure 5: Flowchart of vibration assessment.

Table 2: Associated questions in reference to vibration assessment flowchart.

Question	Description
What is the floor occupancy?	Categorise the occupancy of the floor. This will indicate dead and live load requirements and the demands regarding floor vibration.
How is the floor expected to behave when subject to dynamic forces?	Determine modal properties of the floor using the simplified method or finite element modelling. Boundary conditions, loading, material properties, connection to adjacent cassettes and the main structure will influence the modal properties.
What limitation measure will you use?	Vibration characteristics include frequency, deflection, velocity or acceleration. Response factor is a commonly used measure.
What is an acceptable vibration limit for your floor?	This directly relates to the floor occupancy. High-importance buildings, such as hospitals and laboratories, will have more stringent vibration limits than a residential building.

3.4 Vibration Design Considerations for Ribbed Deck Floors

3.4.1 Modal Separation and Participation

It is apparent from experiments at UTS (final project report, PNA 341-1415) that the frequencies of the first bending and torsion modes of the floor are close together. Figure 6 shows the measured mode shapes, frequencies and damping ratios up to 40 Hz for a single cassette floor with overhanging flange bearing onto a rigid timber frame. Closely spaced modes are often caused by orthotropic floor systems in which the flexural rigidity along joists is higher than that across-joists and can amplify floor motions (Khokhar 2004). Further, people tend to be more annoyed when there are two closely spaced frequencies (Ljunggren 2006; Ljunggren, Wang & Ågren 2007). Although there is no clear definition of spacing required between modes to avoid interaction, a minimum separation of 5 Hz has been suggested (Ohlsson 1982; Weckendorf & Smith 2012).

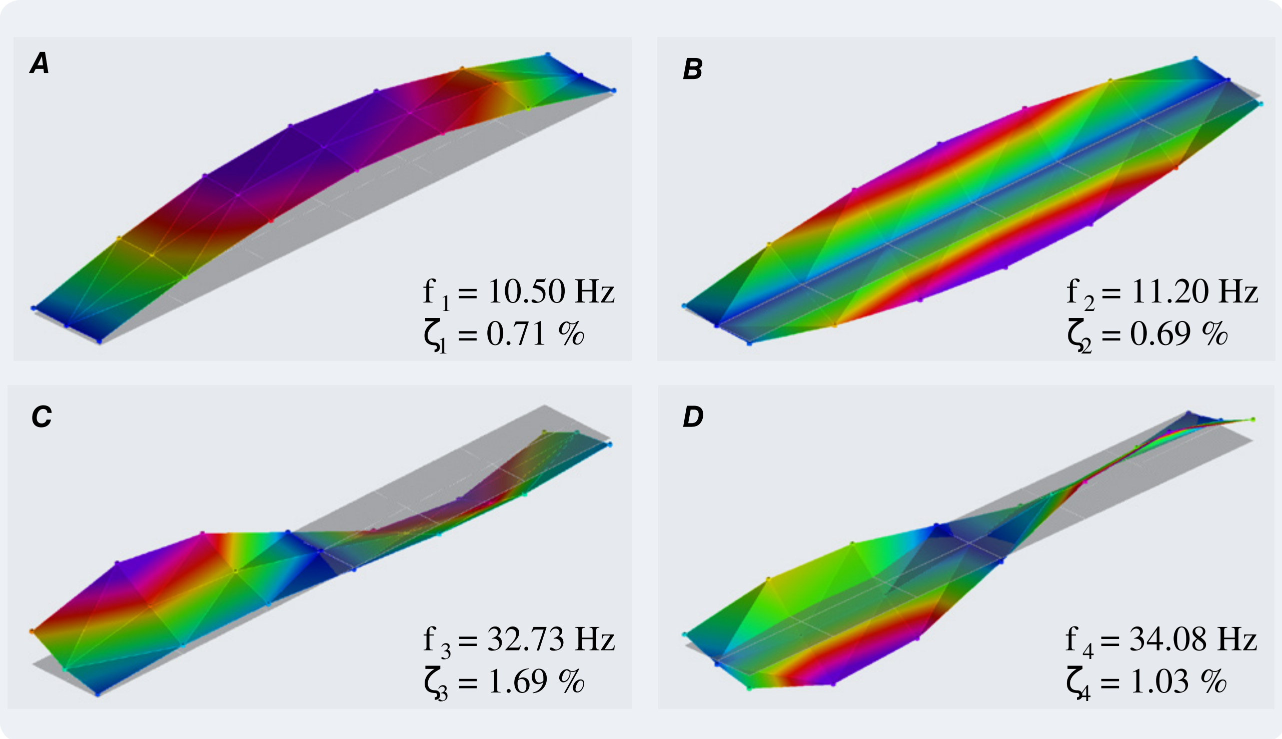


Figure 6: Extracted mode shapes for (a) Mode 1 – first bending mode (b) Mode 2 – first torsion mode (c) Mode 3 – second bending mode and (d) Mode 4 – second torsion mode.

3.4.2 Damping Ratio

Damping is the structure's ability to dissipate vibration energy through friction. It is made up of two main categories: material and structural damping. Material damping involves the internal friction within the material that is created through energy dissipation associated with microstructure defects such as grain boundaries and impurities (Labonnote 2012). Structural damping is a form of mechanical energy dissipation by friction of movement between components such as at support connections. Structural damping also depends on the occupancy, where friction between the floor and partitions or furniture can also contribute.

For timber floor systems, the main contribution to damping will be from structural damping. However, predicting damping is complex as slight differences in superimposed load and connection systems can affect the value. In addition, workmanship will also differ from site to site. The damping ratio also differs between modes and varies with load amplitude. For example, field testing of 13 different timber joist floors found that the average damping ratio for impulse loads was 5.05% as opposed to 0.95% for ambient vibration (Xiong, Kang & Lu 2011). For these reasons, standards often suggest a conservative value.

Damping values for long-span ribbed deck floors for commercial applications is currently limited. Testing on both single cassette and two adjacent connected 9 m spanning cassettes has shown that the damping ratio for the first and second bending mode is about 1%. Interestingly, under a simply-supported case, the damping ratio increased to 3–4% when a subject walked across the floor at various pace frequencies. Other studies on light-weight floor structures have also highlighted the positive influence of human-structure interaction on the damping ratio (Sachse 2002; Živanović, Diaz & Pavic 2009). This highlights the potential benefit of considering human-structure interaction in design approaches.

In current standards, Eurocode 5 suggests a damping ratio of 1% should be assumed 'unless other values are proven to be more appropriate' (CEN 2004b). This value is based on investigations undertaken by Ohlsson (1988b, 1991) on residential timber joist floors, which are typically constructed from solid timber joists nailed to thin top flanges made from oriented strand board (OSB) or plywood. More recent investigations have been carried out by Weckendorf et al. (2008) on composite LVL joists glued and screwed to an OSB decking among other configurations. Damping ratios of 2.0–3.5% were measured for the first mode while second and third modes had a mean value of about 1%. At this stage, it is recommended that a value of 1% is assumed for design. Table 3 compares the various damping ratios found in literature for more conventional timber joist floors and the influence of a screed layer. Concrete and steel floors have also been included for reference.

Table 3: Damping ratios for various floor systems.

Code/guideline	Floor type	Damping ratio
CCIP-016 (Willford & Young 2006)	Bare reinforced concrete floors	1%–2%
	Completed reinforced concrete floors with typical fit out	2.2–3.5%
	Completed steel composite, post tensioned or reinforced concrete floors with extensive fit out and full height partitions	3% 4.5%
Eurocode 5 (CEN 2004c)	Timber floors[1]	1%
UK NA to Eurocode 5 (BSI 2008b)	Timber floors	2%
ISO 10137:2007 (ISO 2007)	Wood joist floors – preliminary design value	2%
	Wood joist floors – typical range	1.5–4%
	Wood joist floors – extreme range	1.0–5.5%
HIVOSS (HIVOSS 2008)	Bare wood floor[2]	6%
Mohr (Mohr 1999)	Timber floors without any additional boardings for sound insulation	1%
	Plain glulam timber floors with additional boarding for sound insulation	2%
	Girder floors and nail laminated timber floors with additional boarding for sound insulation	3%
Hamm et al. (2010)	Timber floors without any floor finish	1%
	Plain glued laminated timber floors with floating screed	2%
	Girder floors and nail laminated timber floors with floating screed	3%

Notes:

[1] *Unless other values are proven to be more appropriate.*

[2] *For open plan office, it is suggested to add another 1% damping due to the furniture.*

3.4.3 Load Case

Overestimating the mass can be non-conservative for footfall vibrations (Willford & Young 2006). Floor loading should include the unfactored self-weight of the structure plus any superimposed dead load, such as any floor finishes and ceiling and services. Due to the requirement for commercial spaces to be flexible for different layouts and occupancies, it is recommended that about 10% of nominated live load is considered for vibration design (Smith, Hicks & Devine 2009). Mass for floor vibration can be calculated using Equation 3.5, where G, SDL and Q represent the self-weight, superimposed dead load and live load, respectively.

$$m = G + SDL + 0.1Q \tag{3.5}$$

3.4.4 Boundary Conditions

Boundary conditions significantly influence the modal properties of floor systems. For footfall analysis of steel and concrete floors using finite element models, connections between floor system and main structure are assumed to act as fixed due to the very small strains associated with footfall loading. However, there has been limited research into how accurate this assumption is for timber structures.

Ribbed deck floors will typically be connected into a timber frame system spanning between primary beams (see Figure 7(a)) or clamped between CLT walls (see Figure 7(b)). The clamping effect of the top flange will result in an increased rotational stiffness. However, the effect may be limited due to the compressibility of wood, which allows for rotational movement of the floor in the joints between walls and floor (Jarnero 2014). Impact hammer tests on a 9 m spanning ribbed deck cassette have shown that a flange-bearing support condition similar to Figure 7(a) acted very closely to a pin-support. Addition of an added load of up to 2000 N at each support location showed minimal increase in natural frequency for the first bending and torsion modes.

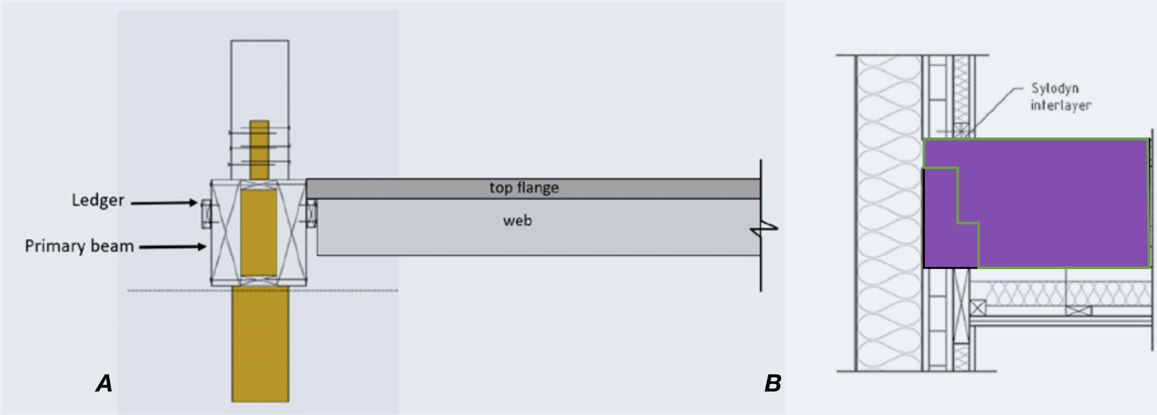


Figure 7: Example of connection of ribbed deck floor cassette into main structure (a) supported by primary beams, adapted from WoodSolutions Technical Design Guide #26 (Forsythe 2015) (b) supported on CLT load-bearing walls (Jarnerö, Brandt & Olsson 2015).

Another support condition often found in timber frame buildings in Europe involve the addition of an acoustic interlayer at boundaries between floor and supporting beam/wall, as shown as the 'Sylodyn interlayer' in Figure 7(b). Jarnerö, Brandt and Olsson (2015) investigated the effect of acoustic interlayers (Sylomer® and Sylodyn® manufactured by Getzner) on the damping ratio of a cross-laminated timber ribbed deck floor both in a laboratory environment and in-situ. Sylomer has a combination of both spring and damper properties while Sylodyn has stronger spring and smaller damping properties. Both elastomers are suitable for a wide range of applications including as a vibration isolation element in the rail industry, elastic machine bearing and to minimise footfall noise in buildings including those made from mass timber (Getzner GmbH 2016). Test results showed that the damping ratio increased to 6% when the last storey was added (floor tested was on second floor of an 8-storey building). This was significantly different to the 2.5% value obtained from the laboratory for the same elastic interlayer boundary condition.

Experiments on the influence of the Sylomer® SR 55 and Sylodyn® NB interlayer on the damping ratio of a long-span ribbed deck floor system were also undertaken at UTS. The ratio of utilisation was 70% and 96% for the Sylodyn and Sylomer interlayer, respectively; this means that the elastomers were still within the static range of operation. The elastomers were placed underneath an overhanging top flange bearing onto a timber frame support which was secured to the ground. Figure 8 shows the measured damping ratio for the first bending and torsion modes with and without the interlayer. Results show that for the Sylomer® interlayer, the damping ratio for modes 1 and 2 increased by over six- and nine-fold, respectively, compared to the case with no interlayer. Under walking tests at a pace frequency equivalent to the fifth integer of the fundamental frequency, the response factor at the centre of the floor reduced by 61% and 78% for Walker 1 and 2, respectively. It is important to note that the addition of the elastomer also introduced new modes under 50 Hz. Further tests would be needed to verify these results and whether any formal recommendations can be made on the use of an elastomer to mitigate floor vibration.

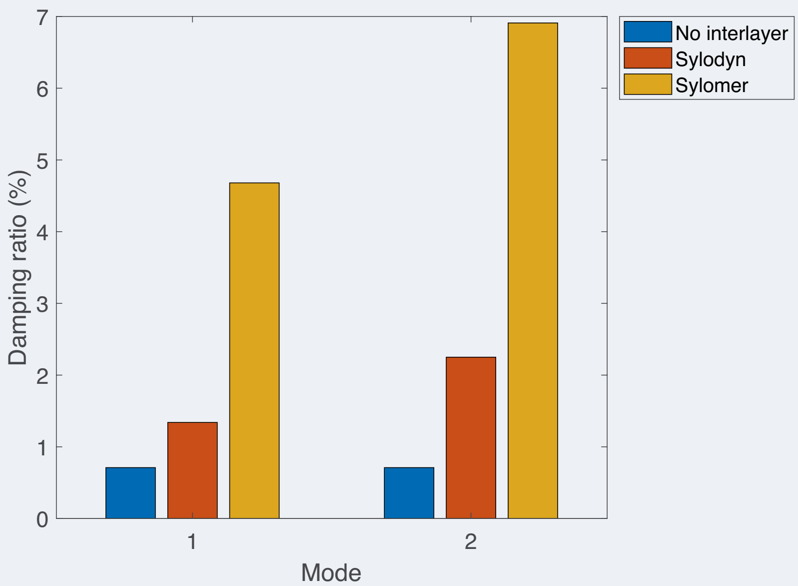


Figure 8: Effect of different interlayers on damping ratio of first bending and torsion modes.

3.4.5 Architectural Considerations

Dynamic behaviour of long-span ribbed deck floor systems will greatly benefit from cooperation between architects and structural engineers at an early design stage. If the footfall loading is close to the maximum deflection points in the mode shape, it is likely that there will be a higher floor response. Walking paths or corridors can be strategically placed closer to beams and columns as these areas will be less responsive than areas in the middle of the floor. Corridor length should also be considered as the longer the corridor, the more time is associated with walking (Smith, Hicks & Devine 2009).

3.5 Vibration Design Procedure

The following design procedure for a floor under footfall excitation is common to a number of vibration design guides, including CCIP-016(Willford & Young 2006), SCI P354 (Smith, Hicks & Devine 2009) and AISC DG 11 (Murray et al. 2016):

Step 1. Calculation of modal properties:

a. Natural frequency

b. Modal mass

c. Mode shape

Step 2. Categorisation as a low or high frequency floor.

Step 3. Evaluation of response.

Step 4. Checking response against acceptance criteria.

The following sections provide commentary from current literature and equations, where available, on each step with reference to ribbed deck floor systems. A design for a simply-supported cassette within a commercial building is shown in Appendix A.

3.5.1 Step 1: Calculation of Modal Properties

As mentioned in Section 3.4.1, timber cassette floors are prone to having closely spaced modes with the first longitudinal bending mode not always being the most critical. Step 1 is recommended to be undertaken using a finite element model. Hand calculations using closed-form solutions for the fundamental mode can be used as a guide as to the potential nature of floor response.

Several assumptions are made when using an idealised situation (Bishop & Johnson 1960):

1. The system can be isolated from its surroundings. It is supported by rigidly-fixed points and are not affected by external forces. This ideal boundary does not exist in the real world.
2. The materials used to make up the system are perfectly homogenous and dimensions are exact. Wood, in particular, is a cellular material and is non-homogenous due to its anisotropic nature. Although engineered wood provides the user with a more reliable product, uncertainties still remain.
3. Some systems are assumed to have finite freedom only, they must be constructed of rigid bodies and massless springs.

Natural frequency

Ribbed deck floors have a similar structure to steel-concrete composite floor systems in which there are secondary beams or joists which are compositely connected to a flange element. With the floor cassettes most likely being supported by primary perimeter beams, consideration should be taken of the primary beam mode shapes about the columns. SCI P354 (Smith, Hicks & Devine 2009) suggests calculating the natural frequency of both secondary beam (Figure 9(a)), and primary beam modes (Figure 9(b)), and choosing the lower value. Mode A assumes that the ribbed deck cassettes vibrate as simply-supported members about the primary beams while Mode B assumes that the primary beams vibrate about the columns as simply-supported members and the ribbed deck cassettes are taken as fixed-ended.

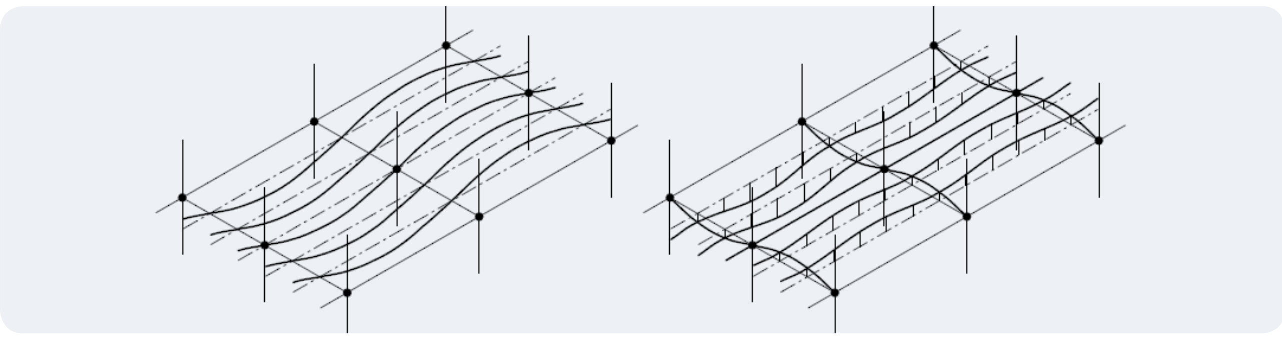


Figure 9: Mode shape governed by (a) secondary beam flexibility (b) primary beam flexibility (Smith, Hicks & Devine 2009).

The fundamental frequency can be calculated using the following equation:

$$f_0 = \frac{18}{\sqrt{\delta}} \tag{3.6}$$

Where δ is the total deflection (in millimetres) of the ribbed cassette and primary beams, depending on the mode shape being considered. For Mode A, only the deflection of the ribbed deck floor needs to be considered. For Mode B, the deflection of the primary beam needs to be added to the deflection of the ribbed deck floor. If the primary beam dimensions and span are unknown, assessment of Mode A is sufficient. Equation 3.6 is a rearrangement of the equation for free elastic vibration of a simply-supported beam of uniform cross-section:

$$f_n = \frac{\pi^2}{2\pi}\sqrt{\frac{EI_e}{mL^4}} \tag{3.7}$$

Deflection of a simply supported and fixed-fixed element subjected to a uniformly distributed load is shown in Equations 3.8 and 3.9.

Simply-supported case:

$$\delta = \frac{5mgL^4}{384EI} \tag{3.8}$$

Fixed-fixed case:

$$\delta = \frac{mgL^4}{384EI} \tag{3.9}$$

where:

E	is the static Young's modulus [N/mm²]
I_e	is the effective second moment of area of one cassette as a composite section with consideration of shear deformation [mm⁴]. For Mode A, if flange and web members are of different grades, the transformed section method can be used to calculate the neutral axis of the cassette cross-section.
m	is the mass per unit length as per Equation 3.9
L	is the length of the cassette [mm]

Modal mass

The modal mass is the amount of mass involved in the mode shape i.e. how much kinetic energy exists within the system. For the purpose of a simplified calculation, the modal mass of a simply-supported ribbed deck cassette can be as assumed to be similar to that of a simply-supported beam of uniform cross-section.

The modal mass can then be calculated as:

$$\hat{m} = \frac{mL}{2} \tag{3.10}$$

Mode shape factor

The mode shape is the structure's preferred maximum displacement pattern when excited by a sudden impact and differ for each mode. The mode shape for the *j*th mode is:

$$\mu_j = sin\left(\frac{j\pi y}{L}\right) \tag{3.11}$$

where, *y* is the distance along the beam of the excitation or response point.

The mode shape factor can be conservatively taken as 1, since the worst case for both the excitation and response point will occur at maximum deflection points in the mode shape. For first bending mode, *y/L*=0.5, and for second bending mode, *y/L*=0.25 and so on.

3.5.2 Step 2: Categorisation as a low- or high-frequency floor

Many guides, including CCIP-016, SCI P354 and DG11, categorise the floor response based on the fundamental frequency as either resonant or transient and suggest a different human loading force model for each case. Low-frequency floors are assumed to sustain resonance with a higher harmonic of the walking frequency with amplitudes building up as each footstep is taken. Guides suggest conservatively that the walking force is continuous and perfectly period and thus can be represented as a Fourier series of harmonic force contributions:

$$F(t) = Q + \sum_{h=1}^{H} \alpha_h Q \sin\left(2\pi h f_p t - \phi_h\right) \tag{3.12}$$

where $F(t)$ = vertical walking force; Q = static weight of an 'average' person (normally 76 kg × 9.81 m/s^2 = 746 N); h = harmonic number; n = total number of contributing harmonics; α_h = Fourier coefficient of the h-th harmonic generally known as the Dynamic Loading Factor (DLF); f_p = pace frequency (Hz); and Φ_h = phase lag for the h-th harmonic. CCIP-016, SCI P354 and DG 11 all suggest different DLFs based on various literature.

On the other hand, a high-frequency floor response is assumed to be characterised by an initial peak associated with each heel drop and decaying vibrations at a rate depending on the damping ratio. For this case, an impulsive footfall force model representing the heel strike is suggested. All guides have agreed on the method proposed by Willford et al. (2006) in which the initial and hence maximum velocity under an impulsive action can be calculated by dividing the magnitude by the modal mass. For unit mass, the initial velocity is numerically equal to the applied impulse, referred to as the 'effective impulse' and expressed as:

$$I_{eff} = \frac{f_p^{1.43}}{f_n^{1.3}} \frac{Q}{17.8} \tag{3.13}$$

CCIP-016 suggests a 'design' value of Equation 3.13 that has a 25% chance of being exceeded while SCI P354 incorporates requirements provided in EN 1990 Annex C (Gulvanessian 2001; Smith, Hicks & Devine 2009) that results in an 18% larger effective impulse than CCIP-016. Higher natural frequencies and low pace frequencies result in a lower effective impulse (Willford, Young & Field 2006). Guides only consider the fastest pace frequency expected in the occupancy, suggested as 2.5 Hz in CCIP-016 and 2.2 Hz for SCI P354 and DG11.

Despite this distinction, there have been many observed cases where 'high-frequency' floors have exhibited a resonant response or 'low-frequency' floors have localised high frequency modes with low modal mass which are easily excited by footstep impulses (Brownjohn, Racic & Chen 2016).

For long-span timber floors with low modal mass and first modal frequency in the 8 to 12 Hz range (depending on the loading considered), such a finding may be particularly relevant.

Walking tests have shown that although the fundamental frequency would classify the floor as a high-frequency floor, a resonant response was generated. Interestingly, contrary to the common assumption that only natural frequencies up to fourth harmonic should be considered, the resonant response was generated from a pace frequency in which the fifth harmonic was coinciding with the fundamental mode. This may indicate that the classification of long-span ribbed-deck floors as low- or high-frequency may not be appropriate. In addition, having two closely spaced modes around the cut-off frequency may create further uncertainty of this approach.

Adding to the ambiguity of the categorisation, various guidelines and standards recommend different cut-offs (see Table 4). For current designs, as per SCI P354, it is suggested that low-frequency floors are checked for both resonant and transient responses while high-frequency floors are checked for only transient response.

Table 4: Low to high frequency floor cut-off for various guidelines and standards.

Reference	Low to high frequency cut-off
SCI P354 (Smith, Hicks & Devine 2009)	10 Hz
Concrete Centre (Willford & Young 2006)	10.5 Hz[1]
Toratti and Talja (2006)	10 Hz
BS 6472-1:2008 (BSI 2008a)	7–10 Hz
Allen and Murray[2] (1993)	9 Hz
Wyatt and Dier[2] (1989)	7 Hz
Ohlsson[2] (1988a)	8 Hz

Notes:

[1] 4.2×maximum footfall rate (2.5Hz)

[2] As per Pavic et al. (2003)

3.5.3 Step 3: Evaluation of Response

Apart from the natural frequency, deflection, velocity and acceleration can also be used to quantify the response of the floor. These parameters stem from the dynamic equilibrium equation for a single-degree-of-freedom system subjected to an external dynamic force, *p(t)*:

$$m\ddot{y}(t) + c\dot{y}(t) + ky(t) = p(t) \qquad (3.14)$$

where, *m, c* and *k* are the matrices of mass, damping and stiffness, respectively and $\ddot{y}(t)$, $\dot{y}(t)$ and $y(t)$ are acceleration, velocity and deflection, respectively. Acceleration is the most commonly used evaluation parameter as easily correlated to measurements from accelerometers and also appears to be the best parameter to relate to acceptable magnitudes of human perception of motion (Irwin 1978). Depending on the guide, peak or root-mean-square (RMS) acceleration are typically suggested as an evaluation for both resonant and transient response floors. Table 5 shows the different evaluation parameters suggested by other guides and the subsequent final evaluation criterion. Note that the RMS acceleration of a sine wave is about 70% of the peak acceleration (Equation 3.15).

$$a_{rms} = \frac{a_{peak}}{\sqrt{2}} \qquad (3.15)$$

Table 5: Response evaluation for CCIP-016, SCI P354 and DG 11.

Guideline	Resonant response parameter	Transient response parameter	Final assessment
CCIP-016	Peak acceleration	RMS velocity	Response Factor
SCI P354	RMS acceleration	RMS acceleration	Response Factor and/or Vibration Dose Value
DG 11	Peak acceleration	Equivalent sinusoidal peak acceleration	%g

Resonant response

Resonant response analysis involves the calculating the total response of each harmonic of walking which is found through the square-root sum of squares of the acceleration response of each relevant vibration mode of the system. Typically, the first four harmonics are considered while all modes up to 15 Hz for CCIP-016 and 12 Hz for SCI P354 are included. The general expression for total acceleration response at a position *r* from excitation at a point *e* is shown in Equation 3.16 taken from SCI P354; note that CCIP-016 and DG 11 expressions will vary slightly to the equation.

$$a_{w,rms,e,r} = \frac{1}{\sqrt{2}}\sqrt{\sum_{h=1}^{H}\left(\sum_{n=1}^{N}\left(\mu_{e,n}\mu_{r,n}\frac{F_h}{M_n}D_{n,h}W_h\right)\right)} \qquad (3.16)$$

where H = number of harmonics; N = number of modes; h = harmonic number; n = mode number; $\mu_{e,n}$ = mode shape amplitude at the point on the floor where excitation force is applied; $\mu_{r,n}$ = mode shape amplitude at the point where response is calculated; F_h = excitation force for the h-th harmonic ($F_h = \alpha_h Q$); M_n = modal mass of mode n; $D_{n,h}$ = dynamic magnification factor for acceleration; W_h = appropriate code-defined weighting factor for human perception of vibrations for the frequency of the harmonic under consideration h_{fp}. The worst case for the mode shape amplitudes is when the excitation and response locations are at the same point. When using a finite element model for vibration design, response at each node for excitation at each node should be checked to obtain the worst case. When using hand calculation methods, mode shape amplitudes can conservatively be taken as 1.

The dynamic magnification factor for acceleration is the ratio of the peak amplitude to the static amplitude and can be calculated as follows where β_n is the frequency ratio of f_p/f_n:

$$D_{n,h} = \frac{h^2\beta_n^2}{\sqrt{(1-h^2\beta_n^2)^2+(2h\zeta\beta_n)^2}} \tag{3.17}$$

A resonance build-up factor, shown in Equation 3.18, can also be applied for each harmonic at each relevant mode; this factor, related to the damping and the number of footsteps taken to cross the span, reduces the extent of full resonant build-up. Although, since architectural layout of corridors and partitions may not be known, it is commonly taken as 1.

$$\rho_{h,n} = 1 - e^{-2\pi\zeta N} \quad \text{where} \quad N = 0.55h\frac{L}{l} \tag{3.18}$$

where ζ = damping ratio; N = number of footsteps; L = span; l = stride length (typically 0.75 m for 2 Hz walking pace).

Transient response
Transient response analysis typically involves all modes up to twice the fundamental frequency. Since faster walking speeds generally induce a higher response, only the fastest walking pace that is expected on the floor is considered; for corridors and circulation zones, this is typically 2.5 Hz (Willford & Young 2006). The acceleration of each impulse is typically expressed as the sum of responses for each relevant mode:

$$a_{w,e,r}(t) = \sum_{n=1}^{N} 2\pi f_d \mu_{e,n}\mu_{r,n}\frac{I_{eff}}{M_n}\sin(2\pi f_d t)\cdot e^{-2\pi\zeta f_n t}W_n \tag{3.19}$$

where f_d = damped natural frequency $f_d = f_n\sqrt{1-\zeta^2}$; W_n = appropriate code-defined weighting factor for human perception of vibrations for the frequency of the mode under consideration f_n. Equation 3.20 can then be used to determine the RMS acceleration where $T = 1/f_p$.

$$a_{w,e,r} = \sqrt{\frac{1}{T}\int_0^T a_{w,e,r(t)}dt} \tag{3.20}$$

Response factor
CCIP-016 and SCI P354 both recommend response factor (RF) criteria which is a multiple of the base curve shown in Figure 10 representing a minimum vibration magnitude for approximately equal human response with respect to human annoyance to continuous vibrations (BSI 2008a). The RF defines an ‘acceptable level’ of vibration for various occupancies. BS 6472 states that adverse comments of vibration are rare for vibration magnitudes below the base curves, however, this does not imply that ‘annoyance and/or complaints are necessarily to be expected at higher magnitudes (BSI 2008a). This highlights the subjectivity of human perception of floor vibrations and the importance of selecting criteria based on the expected occupation and occupant activity.

The RF based on weighted RMS acceleration is typically calculated as per Equation 3.21. The 0.005 m/s^2 in the denominator refers to the baseline perception threshold for the most sensitive frequency range of 4 to 8 Hz. Although SCI-P354 uses Equation 3.21 for evaluation of both resonant and transient responses, CCIP-016 is the only guide which acknowledges that after 8 Hz, human perception threshold is based on constant velocity.

$$RF = \frac{a_{w,rms}}{0.005} \tag{3.21}$$

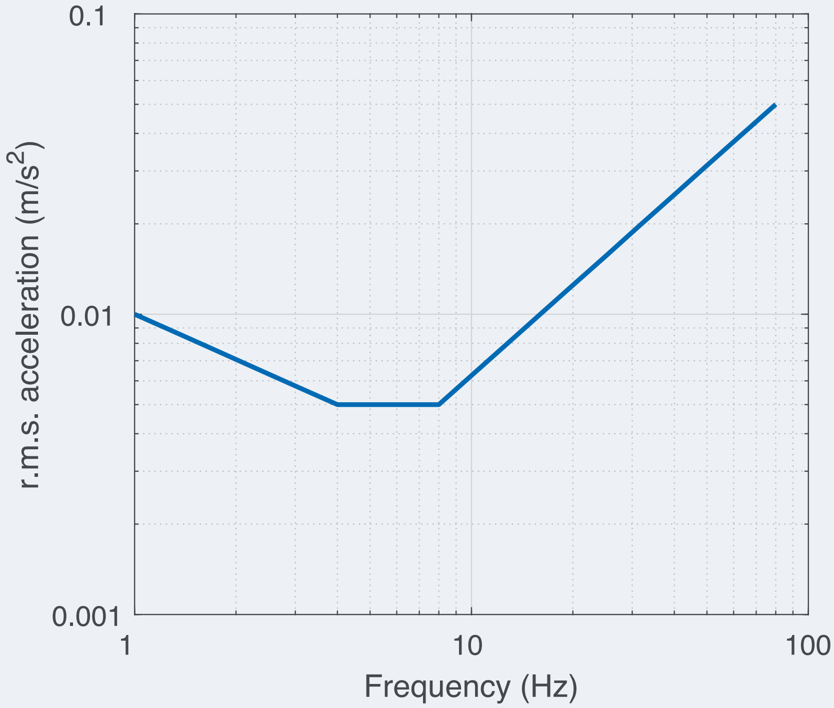


Figure 10: Building vibration z-axis base curve for acceleration (foot-to-head vibration direction) (British Standards Institution 2008).

Vibration Dose Value

A cumulative measure, such as the Vibration Dose Value (VDV), has been suggested to be more appropriate in assessing vibrations from human walking (Ellis 2001). The VDV places importance on the amplitude of vibration and effectively relaxes the response limit of those specified for continuous vibrations, but only for short periods of time when the large amplitudes occur. SCI P354 suggests that if the floor does not satisfy the conservative RF criterion, a representative VDV value as per research from Ellis (2001) can be checked:

$$rVDV = 0.68a_{w,rms}\sqrt[4]{n_a T_a} \tag{3.22}$$

where n_a = number of times the activity will take place in an exposure period; T_a = duration of an activity i.e. time taken to walk along a corridor (s). Ellis (2001) suggests that three possible scenarios thzt may be considered in calculating $n_a T_a$:

- Extremely busy scenario: a person crossing the floor every second for a 16-hour day.
- Busy scenario: as a person walking across the floor every minute for an 8-hour day.
- Quiet scenario: one person walking across the floor 4 times per hour for an 8-hour day.

Otherwise, Equation 3.22 can be rearranged based on the VDV limits provided in BS 6472-1 (2008) to identify the maximum number of times that activity will can occur in an exposure period.

3.5.4 Checking Response against Acceptance Criteria

Response factor limits

Table 6 summarises the performance criteria for various floor occupancies as suggested by CCIP-016 and SCI P354 with comparison to those suggested by BS 6472 (BSI 2008a). Human perception of vibration is not only influenced by amplitude but also by frequency and duration of the walking (Willford, Young & Field 2007). The criterion is based on a single person walking at the most critical footfall rate. Compared to CCIP-016 and BS 6472, SCI P354 has the most relaxed limit for commercial floors with a RF of 8. CCIP-016 makes note of different types of commercial spaces and has reduced the RF by a factor of 2 for many typical office scenarios. These values are in line with those suggested by BS 6472. Another difference of CCIP-016 limits is the consideration of partitions where the RF can be relaxed by a factor of 1.5 for areas which have many full-height partitions that have not been previously considered in the prediction analysis. Although DG 11 provides a response limit of 0.5% gravity for offices and residences, an equivalent RF limit is 7.

As the criteria is based on human perception of vibration magnitudes, the limiting value indicates a level of vibration at which probability of adverse comment is low (but not zero probability) (Willford & Young 2006). If these limits were doubled, adverse comment may result. This is where vibration limits can lead to some uncertainty in determining satisfactory behaviour and it is the responsibility of the engineer and client to balance risk and cost to agree on a reasonable limitation. Note that a floor having RF of 3.8 would be perceived similarly to one with RF of 4.1, i.e. these limits should not be used as a pass/fail but rather as an indication.

Table 6: RF criterion for various floor occupancies from CCIP-016, SCI P354 and BS 6472.

Environment		**Response Factor**		
		SCI P354	**CCIP-016**	**BS 6472**
Critical working areas		1	1	1
Residential	Day	-	4 – 8	2 – 4
	Night	-	2.8	1.4
Commercial	Premium quality open-plan offices and when precision tasks are to be undertaken[1]	8	4	4
	Open-plan offices with busy corridor zones near mid-span[1]	8	4	4
	Heavily trafficked public areas with seating[1]	8	4	4
	Other commercial buildings not covered by the above categories*	8	8	4
Retail	Shopping mall	4	-	-
	Dealing floor	4	-	-
Stairs	Light use (e.g. offices)	32	-	-
	Heavy use (e.g. public buildings, stadia)	24	-	-

Notes:

[1] Target can be relaxed by a factor of up to 1.5 if there are many full-height partitions that were not explicitly included in the prediction analysis.

* intermittent vibrations

- not provided

Vibration Dose Value limits

VDV limits are more holistic in that the response is classified based on probability of adverse comment. These limits reproduced from BS 6472-1 (2008) in Table 8 can be adjusted with the multiplying factors in the final column of Table 7 for various occupancies.

Table 7: RF criterion for various floor occupancies from CCIP-016, SCI P354 and BS 6472.

Place	**Low probability of adverse comment**	**Adverse comment possible**	**Adverse comment probably**
Residential buildings 16 h day	**0.2–0.4**	**0.4–0.8**	**0.8–1.6**
Residential buildings 8 h night	**0.13**	**0.26**	**0.51**

Hu and Chui's limit

Hu and Chui (2004) undertook a field test program involving 130 timber floors in order to develop an improved design method using 'designer-useable formulas' to control vibrations for wood-based floor constructions. Occupants' perception to vibrations were correlated to the measured parameters including static deflection under 1kN, natural frequency, initial velocity and acceleration and RMS acceleration. From the correlation, it was decided that a 1 kN static deflection (*d*) and natural frequency (*f*) were the most suitable design parameters, simply due to the ease of use from the designer and ease of measurement with acceptable accuracy. Through regression analysis, the following formula was proposed:

$$\frac{f}{d^{0.44}} > 18.7 \qquad (3.23)$$

The formulas for deflection under a point load at mid-span and frequency were derived from the ribbed-plate theory and considered semi-rigid connections between web and flange, torsional rigidity of joists and sheathing stiffness in the span and across-joist directions (Hu & Chui 2004). Construction details shown to enhance performance, such as between-joist bridging, strong-back and strapping, are also accounted for in the formulas. More information can be found in Hu & Chui.

The method was validated through data obtained for 106 timber floors built with wood I-joists and solid sawn timber joists with spans of 3–13 m. Depths of floor joists ranged from 140 to 450 mm. The subjective ratings of these floors were compared to the proposed acceptance criteria, (see Figure 11). It was observed that the method is quite effective in differentiating between unacceptable and acceptable floors and shows 'great potential to properly address issues which are deemed to be problematic with the design approaches' including long-span floors (Hu & Chui 2004).

Main differences to note from Hu and Chui's assumptions compared to response of ribbed deck cassette floors are, firstly, the criterion only accounts for the first fundamental vibration mode. This neglects the fact that the second mode which, as mentioned earlier, may be close to the first mode and may have more contribution to the floor response. Secondly, ribbed deck cassettes act in a composite manner rather than the semi-composite behaviour assumed. Although more investigation needs to be undertaken as to how this criterion applies to ribbed deck cassette construction, it should be kept in mind as a possible limitation measure.

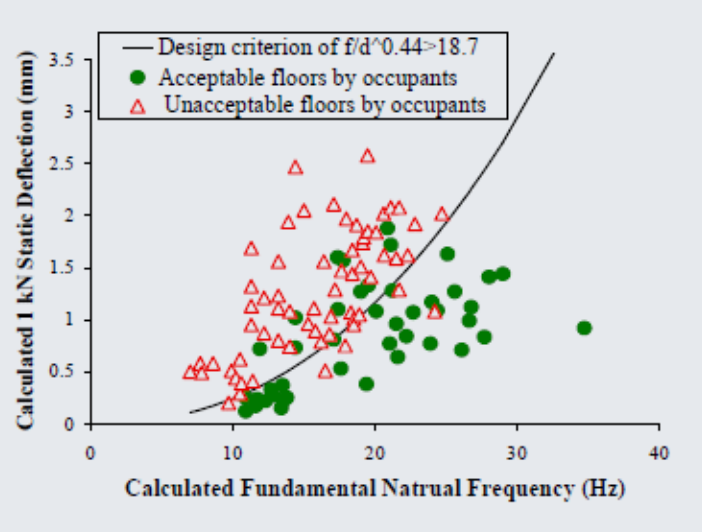


Figure 11: Comparison between subjective evaluation and proposed criterion (Hu & Chui 2004).

3.6 Modelling the floor using finite element

Finite element modelling is a useful tool to accurately analyse the modal properties of more complex or irregular structures. Typically, a complete floor plan is modelled including columns and shear walls, which represents a more realistic situation than a simplified analysis. A transient analysis can also be undertaken to identify locations of high response due to certain nodes being excited. The following modelling details should be considered for dynamic assessment of a ribbed deck cassette floor using FEM.

- **Element type:** Shell elements based are recommended to be used for the flange. The web can be modelled as an isotropic beam or shell element, depending on the desired complexity of boundary connection. Note that a Timoshenko beam element or Mindlin shell element should be used to take into account the shear deformation. Using shell elements for the web means that there are a number of nodes through the depth as opposed to one node if it was a beam element. This provides more flexibility when connecting back into the main structure. If modelling as a beam element, the flange element should be offset from the flange centreline.
- **Flange to web connection:** Web elements should be rigidly tied into the flange elements, i.e. nodes can be coincident between web and flange. Under service loads, the section can be considered to act as a fully composite section.
- **End boundary conditions**: Support conditions where the overhanging flange is secured to the primary beams with screws can be assumed to act as pins. The position of screws (i.e. edge distance) should be followed in the model to ensure the effective length is accurate. Further investigation is required for other support conditions.
- **Cassette-to-cassette connections**: Adjacent cassettes are connected through both web and flange members. Screws connecting web members can be assumed to act as translationally coupled nodes. For flange-to-flange connections such as a splice or diagonal screws, nodes at screw locations can be assumed to provide translational restraint but should be rotationally free. Further investigation is required to confirm whether rotational restraint can be considered in a numerical model.
- **Additional non-structural mass:** Parametric studies will be undertaken to determine whether strategic positioning of mass will reduce floor response.
- **Material property**: All material properties should be input from the manufacturer's technical data sheet.

- **Continuity over primary beam:** From a review of literature on continuous composite floor systems, such as steel-concrete (Ellis et al. 2010; Pavic et al. 2007; Zheng et al. 2010) and timber-concrete (Ghafar et al. 2010), it is expected that there will be little difference to modal properties when compared to a simply supported cassette. When concrete is poured continuously over the primary beams, small cracks occur in the tensile zone above the primary beams. This substantially decreases the stiffness of the section compared to analytical models of a 'continuous' floor. If small cracks in concrete can result in a not-perfect continuous system, then it is more than likely that the panel-to-panel connection between ribbed deck cassettes will create a similar situation.
- **Modelling of elastomer material:** Elastomers at support locations may be modelled as a spring-damper system.
- **Other structural considerations:** Column, shear walls, and façade restraints should be included in the model to ensure additional stiffness is accounted for. Voids in the floor plan must be modelled as floors around openings are often more susceptible to floor vibration.

3.7 Construction Considerations

Services and insulation can be incorporated into the floor structure by utilising the space or void between joists. For cassettes with a top flange, the floor is immediately accessible for fit-out works and suspended ceiling installation. For bottom flange cassettes, workers have direct access to the voids which can be used for a raised floor or other required services. However, care should be taken as the joists pose a trip hazard to workers. Adjacent panels can be connected through diagonal screws (Figure 12a) or splice plate (Figure 12b). The splice plate will typically be recessed into the panel so as to not affect finishes and is less time consuming on site.

Testing on two adjacent ribbed deck cassettes with varying connection systems with screw spacing of 300 mm and 150 mm led to the following observations:

- **Web-to-web connection:** Although the natural frequency of the first bending mode remained the same from a single cassette, the torsion mode frequency reduced by about 3 Hz to become the first mode. Damping ratio increased for the first torsion mode by about 1%. Some reduction in Response Factor from 300 mm to 150 mm spacing.
- **Splice connection + web-to-web connection @ 150 mm c/c:** Negligible change in Response Factor from 300 mm to 150 mm spacing.
- **Diagonal screw connection + web-to-web connection @ 150 mm c/c:** Negligible change in Response Factor from 300 mm to 150 mm spacing. Negligible difference in response between splice and diagonal screw connections.

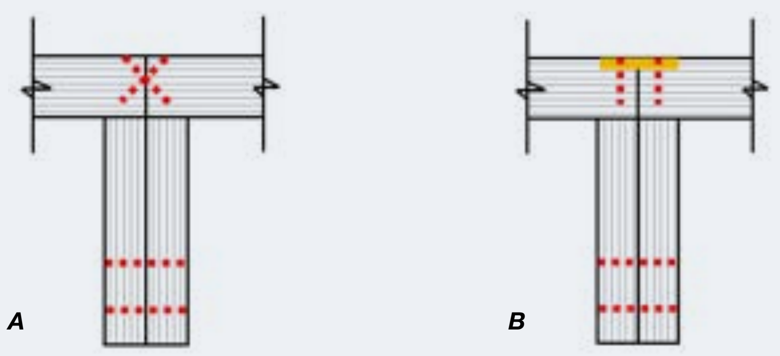


Figure 12: Panel-to-panel connection options for ribbed deck floor.

3.8 Discussion on Response Factor Results for Ribbed Deck Floor

Appendix A Section A.1.7 shows that CCIP-016, SCI P354 and DG 11 procedures significantly overestimated the response of the floor. CCIP-016 had the smallest margin of error for a 2.0 Hz walking pace at 221%; for 2.14 Hz walking pace, DG 11 produced the smallest margin of error at 115%. SCI P354 had the largest margin of error in both cases at 282% and 158% for 2.0 Hz and 2.14 Hz pace frequency, respectively. These errors are from a case where the measured modal properties have been used. This highlights that the predicted effective impulse equation determined by Willford et al. (2006) from experiments of single footfall time histories by Kerr (1998) may not accurately represent the forces occurring on a timber floor. Reasons for the inaccuracy may be a result of:

- **Human-structure interaction:** Some research has shown that the ground reaction forces from a single pedestrian on a more flexible floor surface, such as a footbridge, are less than on a rigid surface (Caprani et al. 2015; Zivanovic, Pavic & Reynolds 2005).

- **Inter-subject variability**: The predicted footfall loading equations do not consider intra-subject variability (differences in response of an individual from one moment to the next). Studies have shown that the slight differences in stride length and frequency between left and right legs have shown to generate subharmonics where excitation energy leaks between the bands of main harmonics (Sahnaci & Kasperski 2005) and becomes more pronounced for higher harmonics (Brownjohn, Pavic & Omenzetter 2004).
- **Time dependency of Response Factor:** There is some conservatism in assuming that the maximum RF will occur for a continuous period. In reality, the RF may only occur for a few seconds when a subject walks across the floor. Through a proposed stochastic model of the footfall impulse, a statistical approach of the RF methodology was investigated (Živanovié & Pavié 2009) indicating a probability of exceedance of a certain RF. Response measures in Japan such as the VLT is another example of consideration of time duration of response above a certain threshold (for measured data only) (Matsushita et al. 2015).

3.9 Recommendations

The following points should be considered when designing ribbed deck cassette floors.

- Shear deformation should be considered in design.
- Timber cassette floors may be susceptible to closely spaced modes which may interact to produce higher responses. The second mode may have a higher modal participation depending on the location of excitation. Floor assessment procedures considering only the fundamental mode may not be appropriate for long-span timber cassette floor systems.
- A damping ratio of 1% should be used until further investigations are undertaken on ribbed deck floor systems.
- The human loading model proposed in design guides (CCIP-016, SCI P354 and AISC DG 11) do not consider important aspects such as human-structure interaction, inter-subject variability and the time varying nature of the RF.
- Negligible difference in modal properties and no clear trend of floor response between the two flange-to-flange connection types (splice and diagonal screws) as well as for reduction of screw spacing from 300 mm to 150 mm.
- A finite element model is recommended to determine modal properties in order to capture torsional modes.
- Support conditions to primary beam should be modelled as a pin-support until further investigation confirms otherwise.

4 Plate type floor using cross-laminated timbers

4.1 Overview

Cross-laminated timber (CLT) is solid timber engineered wood product (EWP) capable of spanning whole walls and floors. The component elements of CLT are boards of timber that are laid side by side to form a plate layer. Each layer is laid 90 degrees to the adjacent layer forming a solid wood panel that improves the structural properties of the timber. Figure 13 shows the general arrangement of a CLT plate.

This document presents the current design procedures available for CLT. It is written in conjunction with experimental work on the vibration characteristics of CLT being undertaken at UTS (final project report, PNA 341-1415, available on the FWPA website).

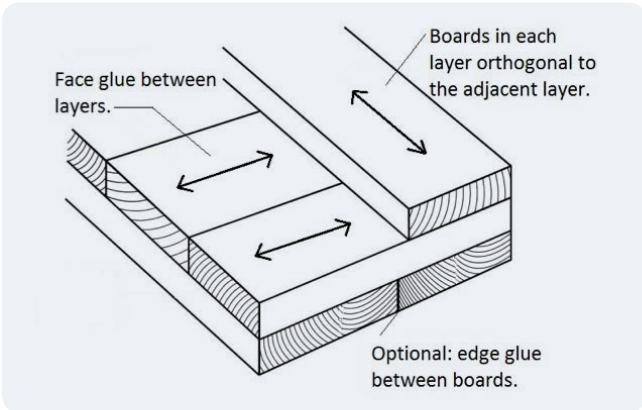


Figure 13: CLT panel arrangement: stacked panels of 5-layered CLT (left). A schematic of a 3-layered panel (right).

4.2 Design Considerations and Scope

Timber is a highly workable material with many options and possible configurations. Therefore, this design guideline is limited to the following design parameters that are within the practical limits for spanning a CLT floor up to 9 m span.

- **Spans up to 9 m**. While CLT is structurally efficient for spans up to around 6 m it can span further. Steel and concrete are capable of structurally satisfying a building with a 9 x 9 m column grid. This column grid is desirable in commercial buildings for efficient desk spacing in the office space and car parks in the basement level. This design guide includes strategies to allow CLT to satisfy a 9 x 9 m column grid.
- **Single span**. Manufacturing and transport capabilities for CLT limit the panel length to 12 m. Therefore, double spans of 6 m are economically efficient. However, there are cases where longer spans are required. This document suggests methods and design guidelines for a single non-continuous span of 9 m.
- **Framing**. While timber framing is desirable, both steel and concrete framing could provide the structural support. Framing options for providing one-way and two-way support are considered. Layouts for these framing options are shown in Figure 14.

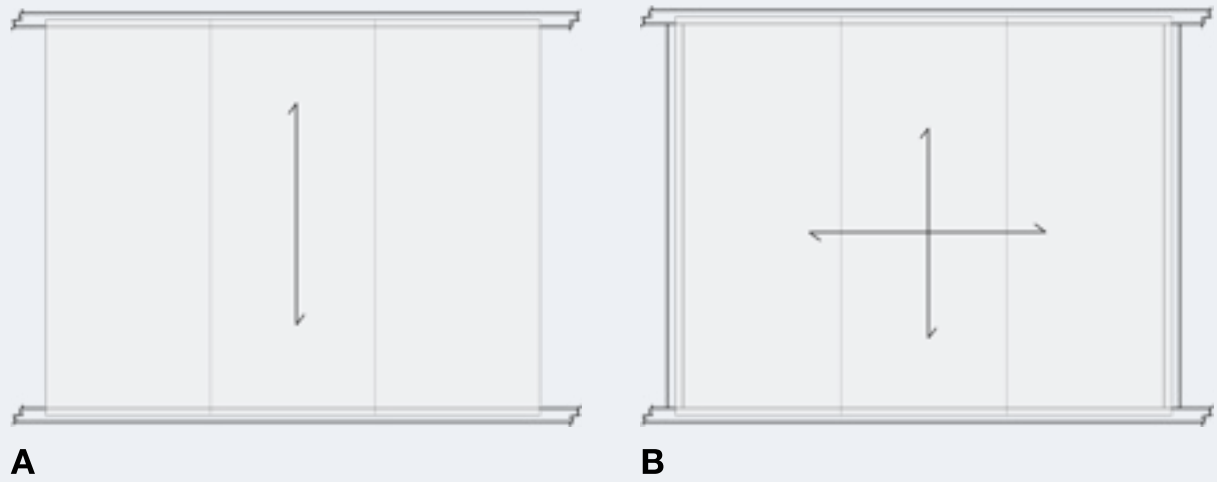


Figure 14: Framing options for CLT floors (a) CLT panels spanning one-way between primary beams and (b) CLT panels spanning two-way between primary beams and secondary beams.

- **Panel-to-panel connections.** Panel-to-panel connections are necessary to facilitate a 9 x 9 m grid. For a 9 m column grid there will be three or possibly four panels making up the cross-section of the floor plate. A half-lap connection is one of the most common connections (Figure 15a). Another common connection type is a single surface spline. This connection involves an added length of timber that can be recessed into the panel (shown not recessed) and screwed to each panel (Figure 15b).

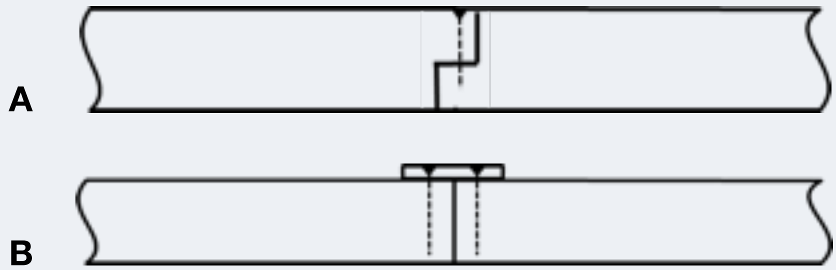
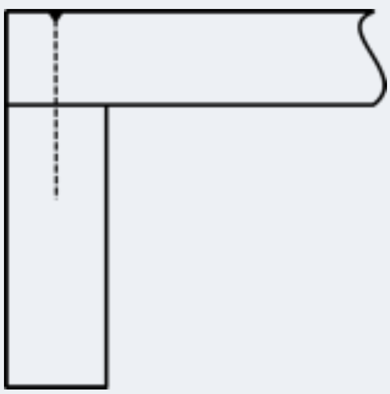


Figure 15: Panel-to-panel connection types: (a) a half-lap connection (top) and (b) a single surface spline (bottom).

- **Floor-to-supporting element connection.** Simple screws connecting the CLT floor to the supporting beam or wall in a single row were investigated. A self-tapping partially threaded screw is drilled into the timber from above at a set spacing. An end-span connection is illustrated in Figure 16. This connection can be extended to span multiple floors by supporting the floors on a wide support beam or wall (Figure 17a) and adding bending moment continuity to the floor by an additional top plate (Figure 17b).



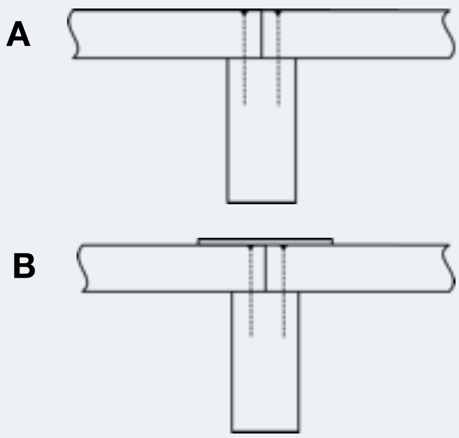


Figure 16: Floor-to-wall support connection with a single partially threaded self-tapping screw.

Figure 17: Floor-to-wall support connection for multiple span floors (a) without top plate and (b) with top plate.

4.3 Design Requirements

The design requirements for CLT floors can be divided into two stages: evaluation of the strength capacity and assessment of the serviceability limit. The design criteria can be summarised as:

Strength Design:

- bending, shear and bearing strength for vertical loads
- design for in-plane strength if diaphragm action present
- fire and earthquake design.

Serviceability Design:

- short-term deflection
- long-term deflection
- vibration.

Due to the high strength-to-weight ratio of timber, serviceability generally governs the design of CLT floors. For this reason most of the research to-date has focused on the serviceability design of CLT (Gagnon & Pirvu, 2011). This document presents current research and methods on the strength and serviceability design for CLT floors.

4.4 Design Procedure

4.4.1 Material Property Considerations

Characteristic strength values for engineered wood products (EWPs) such as glued laminated timber (Glulam) and laminated veneer lumber (LVL) are determined from experimental testing. However, the problem with this approach for CLT is that there are numerous possible layups, material types and configurations. A standard released by the American National Standard ((ANSI 2012), has categorised CLT into grades and provided the respective strength values for a limited number of cross-section sizes. This approach has also been adopted by some manufacturers for the products they regularly produce. The design methods presented in this document, however, use the characteristic values of the lumber that makes up a CLT panel. Therefore, design calculation of any configuration of CLT, material type, thickness, and number of layers and be used. Characteristic values and test configurations for CLT are presented by Unterwieser and Schickhofer (2014).

An important characteristic of CLT is that it cannot be viewed as a homogenous material due to a phenomenon known as rolling shear. This occurs in CLT due to the low shear capacity in the radial and tangential directions of timber and is an important consideration for CLT design of (Figure 18). Rolling shear can contribute significantly to a panel's deflection under bending due to the shear deformation of the transverse layers.

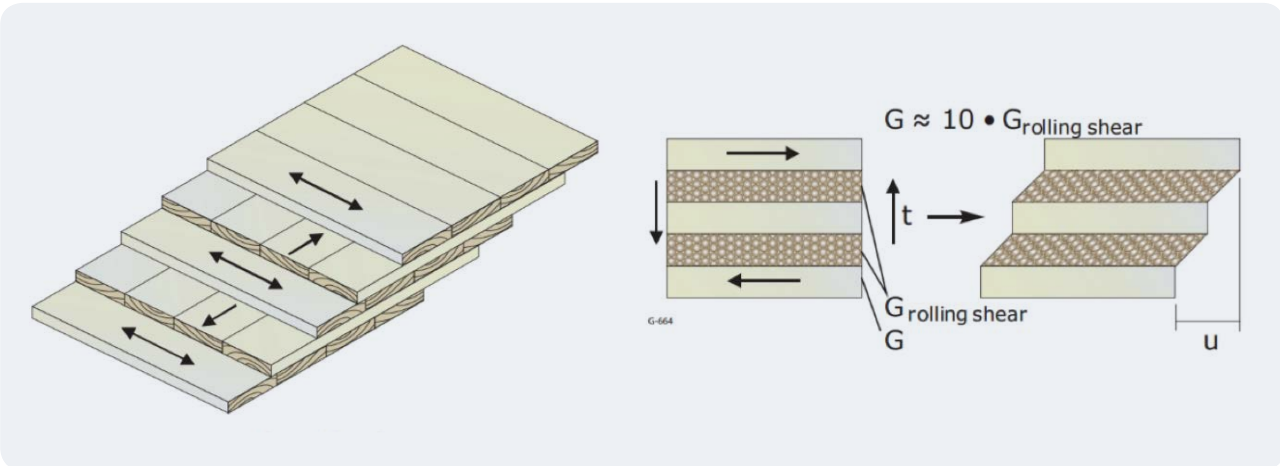


Figure 18: The effect of the low shear strength of the transverse layers in a CLT panel (Gagnon & Pirvu, 2011).

While more research is needed to provide values for rolling shear modulus of various timber species, experimentation to date indicates the shear modulus (G0) to be between 1/12 and 1/20 of the true modulus of elasticity and the rolling shear modulus (GR) to be 1/10 of the shear modulus (Gagnon & Pirvu 2011).

4.4.2 Current Design Guidelines

Several design guidelines have been published that present the holistic design of CLT. European research groups have been the leaders in design and manufacturing of CLT. Design software for CLT has been released by the University of Graz, called CLTDesigner, the calculation methods for this software are presented in this document. More recently the German-Czech company Dlubal has integrated a CLT module into its RFEM software that provides the structural analysis for CLT.

Another important document is the CLT Handbook released by FPInnovations in Canada (Karacabeyli & Douglas 2013). It provides comprehensive documentation of the manufacturing, design and construction of CLT.

Four methods for calculating the strength and serviceability properties for timber are considered in this document; CLTdesigner, Gamma method, composite k method and the shear analogy method. These methods calculate the design capacity for timber structures using a modified version of the Euler-Bernoulli hypothesis of plane sections remain plane. For CLT floors where the ratios of the length/thickness ≥ 15 all these methods converge (Thiel, 2014). Therefore, in such situations, the designer has the option to choose among these methods to calculate the properties of the CLT section depending upon relevant code and design requirements.

These theories are limited as the analytical models are based on beam theory, whereas CLT is a plate element. A more advanced examination is recommended in cases of large point loads, for accounting two-way spanning effects and for length/thickness ratio less than 15. Advanced laminated plate theories requiring higher computational input have been developed for such cases (Thiel, 2014).

Table 8 summarises these methods as well as the material and capacity factors from AS 1720.1 (2010).

Table 8: Summary of available methods for determining the design of CLT.

Method	Design Process	Properties calculated
AS 1720.1 (Standard 2010)	Factors for bending Factors for shear Factors for bearing	AS 1720.1 is used to determine the capacity and modification factors and the characteristic strengths.
CLTDesigner (Thiel 2013)	Bending strength Shear strength Bearing strength Bending stiffness Shear stiffness Vibration	Section modulus, Z Effective area, A_{eff} Effective stiffness K_{CLT} Shear stiffness S_{CLT} Frequency, acceleration
Gamma Method (Eurocode 2003)	Bending strength Shear strength Bending stiffness	Section modulus, Z Effective area, A_{eff} Effective stiffness, EI_{eff}
Composite K Method (Gagnon & Popovski 2011)	Bending strength Bending stiffness	Section modulus, Z Composite factor, k_1
Shear Analogy Method (Gagnon & Popovski 2011)	Bending strength Bending stiffness Shear stiffness	Section modulus, Z Effective bending stiffness, EI_{eff} Effective shear stiffness, GA_{eff}

4.4.3 Strength

The timber for the FWPA-funded project PNA 341-1415 was sourced from New Zealand, so the material and safety factors in AS 1720.1 (2010) are applicable for calculating the strength design capacity. However, AS 1720.1 (2010) does not present methods to determine the relevant moment of inertia, section modulus and effective area calculations for CLT. Due to the cross lamination of CLT, the reduction in these values due to the shear slip is accounted for by a number of methods presented in this section. This section is organised to first present the relevant 'k' values from AS 1720.1 and then presents the methods from international guidelines and research to calculate the section properties.

Both the bending and shear strength are required to be assessed under ultimate limit state loads for strength, earthquake, and fire. Additionally, it is important to check the bearing strength of CLT. This is due to the low compression strength of timber when loads are applied perpendicular to the grain. It is particularly important for the design of CLT buildings where the floors extend between the walls (Figure 19).

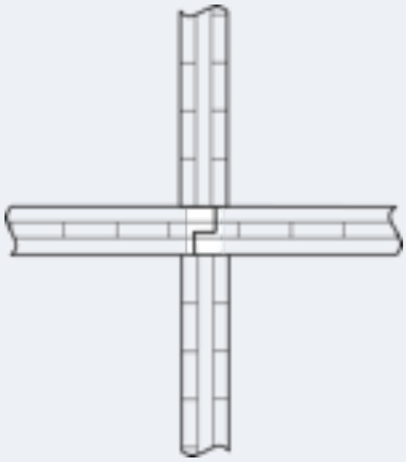


Figure 19: CLT building construction; floors sandwiched between wall plates.

Factors for bending strength AS 1720.1

The Australian standards calculate the bending moment capacity for timber structures using the Euler-Bernoulli hypothesis of plane sections remain plane. Where Z is the section modulus of the timber cross-section with for a rectangular joist can be simply calculated as the moment of inertia divided by the centroid. The section modulus for CLT, Z_{CLT} is calculated using one of the methods presented in this document as AS 1720.1 does not provide guidance for CLT.

$$M_d = \varphi k_1 k_4 k_6 k_9 k_{12} f_b' Z_{CLT} \tag{4.1}$$

φ safety factor equal to 0.95 for secondary members in structures other than houses

k_1 accounts for load effects, equal to 0.57 for permanent loading

k_4 accounts for moisture content, generally equal to 1.0 unless there is significant moist environment or where partial seasoning occurs.

k_6 accounts for temperature effects, 1.0 for covered timber under ambient conditions

k_9 strength sharing factor for Glulam is taken as unity; could be as high as 1.33 for CLT

k_{12} stability factor to be taken as unity for CLT due to the low thickness-to-width ratio

f'_b the characteristic bending strength of timber, for CLT, $f'_b = f_{m,CLT,k}$, see section 4.4.3 CLTDesigner.

Factors for shear strength AS 1720.1

The shear strength of a beam is generally more complicated to calculate, as unlike the bending stress distribution, the shear stress is not linear. For a rectangular and homogenous cross-section of a beam the shear strength is simple to calculate as the shear area is equal to 2/3 the gross area. Due to the non-homogenous cross-section of CLT the method provided in section 4.4.3 is recommended to calculate the shear plane area.

$$V_d = \varphi k_1 k_4 k_6 f_s' A_s \tag{4.2}$$

A_s is the shear plane area which is for a non-composite rectangular section 2/3 of the gross area. For CLT $A_s = A_{eff}$ discussed in section on CLT.

f'_s is the characteristic shear strength, for CLT the shear strength at mid-section, $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are 3.0 N/mm^2 and 0.7 N/mm^2 respectively (Unterwieser & Schickhofer, 2014). If edge bonding has occurred in the manufacturing the rolling shear strength can be increased to 1.25 N/mm^2.

The k modification factors are the same as those for bending strength.

Factors for bearing strength AS 1720.1

To calculate the bearing strength the area of applied load, Ap, and the characteristic bearing strength of the timber, f'_p is required. The strength of bearing calculated as:

$$N_{d,p} = \varphi k_1 k_4 k_6 k_7 f'_p A_p \quad (4.3)$$

k_7 accounts for the location of the bearing position. Location at edge of timber piece is given as unity. A location factor specific for CLT, $k_{c,90,CLT}$ is included in the CLTdesigner section below.

f'_p For CLT is given as 2.85 N/mm² (Thiel, 2014).

The other modification factors are the same as those for bending strength.

CLTdesigner

Bending strength

The software program CLTdesigner developed by the Centre of Competence (holz.bau.forschungs.gmbh) in Graz, Austria, uses the Bernoulli-hypothesis of plane sections remaining plane to calculate the bending strength. The program assumes there is negligible bending stress in the cross layers. This is due to the cross layers being orientated so that the weak grain of the timber contributes to the cross-section stiffness. Further, the transfer of normal stresses in the cross layers is likely not possible due to lack of edge gluing (Thiel 2013). The bending stress distribution for the longitudinal and transverse layers is shown in Figure 20, and the bending stiffness, K_{CLT} is calculated using Equation 4.4.

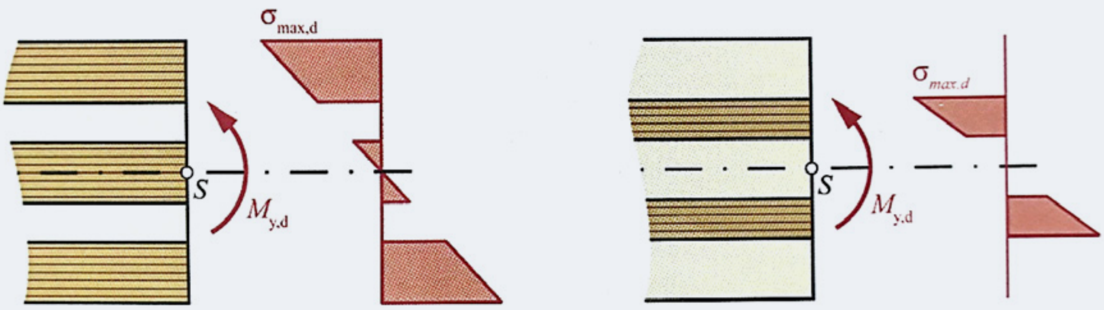


Figure 20: The normal stress distribution of a CLT panel for the bending moment of a floor panel for longitudinal bending (left) and transverse bending (right) (Thiel 2013).

$$K_{CLT} = \sum (E_i I_i) + \sum E_i A_i z_i^2 \quad (4.4)$$

E_i the elastic modulus of the *i*th layer

I_i the moment of inertia of the *i*th layer

A_i the area of the *i*th layer

z_i the distance from the centroid of the *i*th layer to the centroid of the entire cross-section.

The maximum stress, $\sigma_{max,d}$, of the cross-section is calculated using Equation 4.5.

$$\sigma_{max,d} = \frac{M_{y,d}}{K_{CLT}} \frac{t_{tot}}{2} E_1 \quad (4.5)$$

$_{tot}$ the total thickness of the CLT panel

E_1 the elastic modulus of the outermost layer

To calculate the bending design stress of CLT, $f_{m,CLT,d}$, two methods are suggested, this first is based on the tensile strength of the timber and the second on the Glulam product with an equivalent strength grade. The tensile strength value is presented here as it's more easily translated to the base timber material properties. The characteristic values that are used for CLT are presented in more detail by Unterwieser and Schickhofer (2014).

$$f_{m,CLT,k} = k_{m,CLT} f_{t,0,1,k}^{0.8} \tag{4.6}$$

$k_{m,CLT}$ is equal to 3 for timber with a tensile strength CV of 25% and 3.5 for a CV of 35%

$f_{t,0,1,k}$ is the characteristic tensile strength of the timber

Rearranging Equations (4.5) and (4.6), we find an expression for the design bending moment My,d:

$$M_{y,d} = \frac{2K_{CLT}}{t_{tot}E_1} f_{m,CLT,k} \tag{4.7}$$

Equation 4.7 is in the form of Euler Bernoulli's beam theory, $M_{y,d} = fZ_{CLT}$, where Z_{CLT} is the section modulus for cross-laminated timber calculated using Equation 4.8. This section modulus and the value for design bending stress $f_{m,CLT,k}$ can be used to calculate the bending moment capacity in accordance with AS 1720.1.

$$Z_{CLT} = \frac{2K_{CLT}}{t_{tot}E_1} \tag{4.8}$$

Shear Strength

CLTdesigner uses the classical procedure for unidirectional layered cross-sections to calculate the shear stress distribution given by Equation 4.9. The assumption that $E_{90} = 0$ means that there is no shear stress increase in the cross layers (Figure 21).

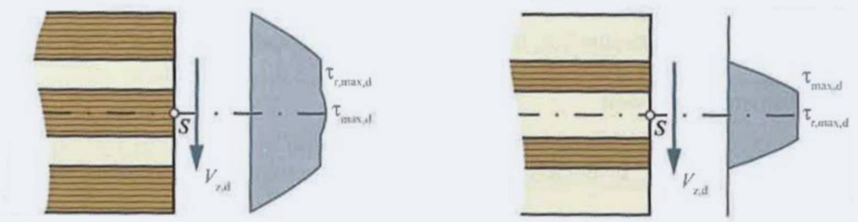


Figure 21: The shear stress distribution in a CLT cross-section for shear caused by longitudinal bending (left) and transverse bending (right), (Thiel 2013).

$$\tau = \frac{V_z \int_A E(z).z.dA}{K_{CLT}.b(z_0)} \tag{4.9}$$

The shear stresses need to be assessed for both the rolling shear stress, $\tau_{r,max,d}$ (at the inter layers) and the maximum shear stress $\tau_{max,d}$ at the centre of the CLT panel and therefore satisfy Equation 4.10.

$$\frac{\tau_{max,d}}{f_{v,CLT,d}} \leq 1.0 \quad \text{and} \quad \frac{\tau_{r,max,d}}{f_{r,CLT,d}} \leq 1.0 \tag{4.10}$$

Solving Equation 4.9 for a 5-layered CLT plate and combining with Equation 4.10 an expression for shear force at the mid cross-section (4.11) and at the rolling shear layers (4.12) is given. These equations need to be multiplied by appropriate material and safety factors.

$$V_{mid} = f_{v,CLT,d} \frac{K_{CLT}}{\left(E_1 t_1 z_1 + \frac{E_3 t_3^2}{8}\right)} \tag{4.11}$$

$$V_{rolling} = f_{r,CLT,d} \frac{K_{CLT}}{(E_1 t_1 z_1)} \tag{4.12}$$

Unless experimental testing has occurred, the values for the shear strength $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are currently 3.0 N/mm^2 and 0.7 N/mm^2 respectively (Unterwieser & Schickhofer, 2014). If edge bonding has occurred in the manufacturing, the rolling shear strength can be increased to 1.25 N/mm^2.

Rearranging Equations 4.11 and 4.12, the effective area for shear strength at the mid-point of the section and shear at the transfer layers are given by:

$$A_{eff,mid} = \frac{K_{CLT}}{\left(E_1 t_1 z_1 + \frac{E_3 t_3^2}{8}\right)} \quad (4.13)$$

$$A_{eff,rolling} = \frac{K_{CLT}}{(E_1 t_1 z_1)} \quad (4.14)$$

K_{CLT} is the bending stiffness in Equation 4.4

$E_{1,t1}$ are the elastic modulus and thickness of the outer layer of a 5-layer CLT panel

z_1 is the distance between the centroid of layer 1 and the centroid of the entire cross-section

$E_{3,t3}$ are the elastic modulus and thickness of the middle layer of a 5 layer CLT panel

Bearing Strength

The bearing strength is calculated by multiplying the contact area with the characteristic compressive strength of CLT perpendicular to the plane of the CLT floor. CLTdesigner uses a characteristic value of $f_{c,90,CLT,k}$ = 2.85 N/mm^2 which has been determined from testing. This value must be multiplied by the appropriate modification factors (see Factors for bearing strength AS 1720.1). Table 9 gives the appropriate multiplying factors to account for the location of bearing, k_7.

Table 9: Factor to account for location of bearing (Thiel, 2014).

Load Type	Load Location	k_7
Point	Central (away from edge)	1.8
Point	Edge of panel (not a corner)	1.5
Point	Corner	1.3
Line	Central and parallel to span	1.3
Line	Central and perpendicular to span	1.8
Line	Edge and parallel to span	1.0
Line	Edge and perpendicular to span	1.5

Gamma method

Bending strength

The gamma method has been developed from mechanically jointed beam theory and is detailed in Eurocode 5 and in the CLT Handbook by FPInnovations. Therefore, only limited equations are presented in this document.

To calculate the effective bending stiffness using the gamma method the reduction in stiffness due to shear slip is accounted for by a stiffness component (γi) which is calculated using Equation 4.16. The effective stiffness can then be determined using Equation 4.15.

$$EI_{eff} = \sum_{i=1}^{n} \left(E_i I_i + \gamma_i E_i A_i z_i^2\right) \quad (4.15)$$

$$\gamma_i = \frac{1}{1 + \pi^2 \frac{E_i A_i}{l^2} \cdot \frac{t_1'}{G_R b}} \quad (4.16)$$

G_R, is the rolling shear modulus

b is the width of the cross-section

A is the area of layer i

E is the elastic modulus of layer i

l is the length of the floor

t'_1 is the thickness of the slip layer

The section modulus of bending moment is calculated from Equation 4.17 and can be used to determine the bending moment capacity of CLT.

$$Z_\gamma = Z_{CLT} = \frac{(EI)_{eff}}{E_1(\gamma_1 z_1 + 0.5t_1)} \tag{4.17}$$

z_1 is the distance from the centroid of the cross-section to the centroid of the outer layer.

Shear Strength

Shear stresses are calculated using mechanically jointed beam theory. The difference between the CLTDesigner method and the Gamma method is that the latter includes the shear strength contribution of the cross layers. The resulting effective shear area for the mid-point of the cross-section and at the transverse layers are given by:

$$A_{eff,mid} = \frac{\left(EI_{eff}\right)b}{\left(\gamma_1 E_1 A_1 z_1 + E_1' A_1' z_1' + \gamma_2 E_2 \frac{A_2}{2}\frac{t_2}{4}\right)} \tag{4.18}$$

$$A_{eff,rolling} = \frac{\left(EI_{eff}\right)b}{\left(\gamma_1 E_1 A_1 \left(z_1 - \frac{t_2}{2}\right) + E_1' A_1' \left(z_1' - \frac{t_2}{2}\right)\right)} \tag{4.19}$$

Composite method

Composite theory was developed for calculating the bending strength of plywood. From composite theory, the design bending moment is calculated using Equation 4.20. The composite factor, k, is a value that accounts for the reduced stiffness of the entire cross-section due to the transverse layer's flexibility. For a CLT floor with the outer layers running longitudinal to the span the value for k is given by Equation 4.21. Any shear deformation is not considered using the k-method.

$$M_{y,d} = \varphi k_1 f_{b,eff} S_{gross} \tag{4.20}$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right)\left(\frac{a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3}{a_m^3}\right)\right] \tag{4.21}$$

The value for a_m is shown in Figure 22. E0 is the elastic modulus of the longitudinal layers and E90 is the elastic modulus of the transverse layers. The elastic modulus relationship is given as E90 = E0/30.

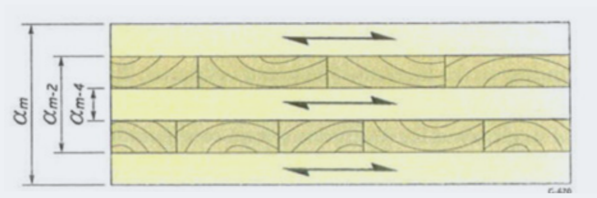


Figure 22: Cross section values for calculation of value k using composite theory.

The section modulus for bending strength for composite theory is given by:

$$Z_k = Z_{CLT} = k_1 S_{gross} \tag{4.22}$$

S_{gross} is the section modulus for the complete rectangular cross-section of the CLT panel without considering the reduced section due to the transverse layers.

Shear analogy method

The final method presented is considered the most precise method as it does not neglect the effects of shear deformation (Blass & Fellmoser 2004). The shear analogy method splits the CLT panel into two virtual beams, A and B. Beam A is treated as the sum of flexural strength of the individual plies along their local neutral axis, while beam B accounts for the flexible shear strength of the panel and the flexibility of the connectors. Equation 4.23 calculates the true bending stiffness, where the values for BA and BB for the two virtual beams, are given in Equations 4.24 and 4.25, respectively.

$$EI_{eff} = B_A + B_B \tag{4.23}$$

$$B_A = \sum_{i=1}^{n} E_i I_i \tag{4.24}$$

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2 \tag{4.25}$$

The shear stiffness of the beam is considered for this method and is calculated using:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{t_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i b_i}\right) + \left(\frac{t_n}{2G_n b}\right)\right]} \tag{4.26}$$

Where:

$$a = t_{total} - \frac{t_1}{2} - \frac{t_n}{2} \tag{4.27}$$

t_i is the thickness of layer i

G_i is the shear modulus of layer i

b_i is the width of layer i

The method in the CLT Handbook by FPInnovations presents a simplified method to calculate the bending moment capacity where the section modulus is given by the following:

$$Z_{simp} = Z_{CLT} = \frac{(EI)_{eff}}{0.5 E_1 t_{tot}} \tag{4.28}$$

Design for in-plane loads

At the Graz University, the representative volume element, RVE, is proposed to calculate in-plane loads (Bogensperger, Moosbrugger & Silly 2010). The size of one RVE is dictated by the thickness of the CLT panel and the width of a single board plus half of the gap width on each side. The RVE is subjected to only in plane stresses (normal and shear) and therefore the stresses and strains are constant over the entire cross-section. If the thickness is equal for all layers the RVE can be further divided into a representative volume sub-element, RVSE (see Figure 23). An RVSE has the same square surface but with a thickness composed of half the board thickness on both sides of an adhesive layer acting as a plane of symmetry.

In manufacturing, it is common not to edge glue (narrow face) all CLT boards. Even with edge gluing, cracks can form due to swelling and shrinkage. This means that shear forces will be acting in different directions on adjacent planes and cause a torsional stress at the glued interface. Therefore, the RVSE is used to calculate both shear and torsional stresses. This method is only valid for constant layer thickness, therefore for layouts with various thicknesses and strength grades it is recommended to adopt load bearing and design models that are available for glued laminated timber.

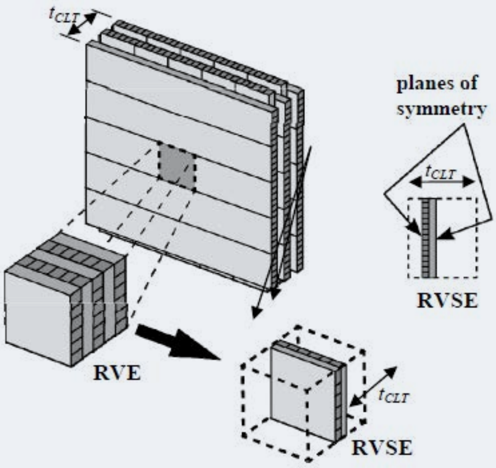


Figure 23: Definition of RVE and RVSE on a CLT element (Bogensperger, Moosbrugger & Silly 2010).

Earthquake design

Timber structures are lightweight and therefore the seismic actions are lower. However, the key to seismic design is structural ductility as it allows energy dissipation. Compared to the steel connections used to connect CLT panels, the panels themselves have infinite stiffness. The ductility of the structure therefore needs to be designed into the connections to ensure good seismic behaviour. Capacity-based design is proposed for seismic design as it aims to prevent brittle failure. By oversizing the CLT panels there is a global ductile failure mechanism at the connections (Gavric, Fragiacomo & Ceccotti 2015). Formulas provided by Eurocode 5 for connections with metal fasteners are used to ensure the connections are dissipative. In a CLT building these connections are the vertical screwed connections between wall panels, connections of wall to floor using angle brackets to resist shear and hold down connections at each end of a wall element to resist uplift.

Fire design

To check the strength capacity of the CLT during a fire, design is based on the reduced cross-sections per EN 1995-1-2 (2004). The charring depth depends on the adhesive applied, the gap size between boards and the availability of fire protection. The charring rate for softwoods and beech with density greater than 290 kg/m^3 is β = 0.65 mm/min for gaps up to 2 mm and β = 0.80 mm/min for gaps up to 6 mm. If the adhesive is not temperature proofed it has been observed that the charred layers of CLT elements loaded out of plane can detach.

Fire tests were conducted on CLT panels composed of various pine species in accordance with AS /NZS 3837:1998. The charring rate of pine with no gaps calculated from AS 1720.4-2006 is 0.75 mm/min for Hoop and Radiata and 0.64 mm/min for Slash pine. The experimental results on CLT panels with gaps displayed larger charring rates, close to the value of β = 0.80 mm/min provided by EN1995-1-2.

4.4.4 Serviceability Design

Short-term deflection

It is critical to calculate the deflection of CLT elements out-of-plane. Due to the cross-layers in CLT, the deformation due to the shear slip in the transverse layers is considered. The gamma method and the composite method incorporate this by using a reduction factor of the effective stiffness E_{Ieff}. The shear analogy method calculates the deflection due to shear slip.

Gamma method

The gamma method is straight forward to implement after the EI_{Ieff} has been calculated using Equation 4.15. The effective bending stiffness can then be used to calculate the deflection at any point. For the mid-span deflection under a uniformly distributed load it is calculated using:

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} \quad (4.29)$$

Composite method

The composite method is straight forward to implement with the composition factor, k_1 calculated using Equation 4.21. The effective bending stiffness can then be used to calculate the deflection at any point. For the mid-span deflection under a uniformly distributed load it is given by:

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1E_0I_{gross}} \quad (4.30)$$

Shear analogy method

The maximum deflection in the middle of a uniformly loaded CLT slab using the shear analogy method is given by Equation 4.31. The first term is the amount of deflection due to bending deformation, while the second term is the amount of deflection due to shear deformation.

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} \tag{4.31}$$

k is a shear coefficient factor equal to 1.2 according to Timoshenko.

Long-term deflection

The factors for creep have been given by prEN 16351 (2011) and are dependent on the amount of moisture and relative humidity the structure is exposed to. The values proposed to be used are k_{def} = 0.85 and k_{def} = 1.1 for service class 1 and 2 respectively.

There is currently little information on how these values compare with CLT panels composed of Australian and New Zealand pine species.

4.5 Vibration

The design of CLT floors for vibration performance depends on three aspects of design: floor loads that cause the vibration response; response of the structure defined by the modal properties; and vibration perception/experience by the user measured using acceptability criteria (see Figure 24).

In regard to loading, the worst-case scenario – where the most problematic floors will have a resonant response due to a cyclic load, commonly walking – is considered. These floors will generally have a lower fundamental frequency. Annoyance in floor vibration can also occur due to an impact load. However, floors susceptible to impact load may not necessarily have a low fundamental frequency and the transient floor response needs to be computed for such floors. Once the loads are determined in step 1 of Figure 24, the modal properties in step 2 can be calculated. It is important to understand the loading as this can change the values of the natural frequencies and the damping. The modal properties can be calculated either by closed form solutions of beam or plate formulas for vibration or alternatively a finite element analysis can be used.

The acceptability of the floor can be determined by either a simplified prescriptive based method in step 3a or a more complex response factor analysis in step 3b. Generally, the prescriptive-based methods are for a limited floor types while the response factor analysis covers any floor type and loading case.

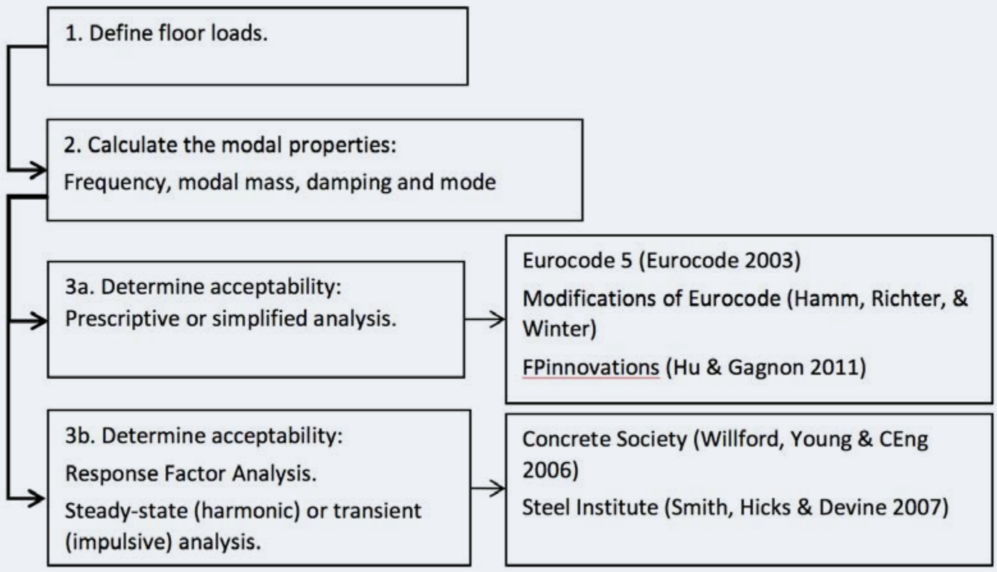


Figure 24: Summary of procedure to determine vibration performance of a floor.

4.5.1 Step 1 – Load Type

Walking loads are considered a cyclic load that can cause resonant frequency with a floor if the walking frequency is close to or equal to one of the natural frequencies of the floor. Generally, people walk with frequencies between 1.5–2.5 Hz. However, it is not as simple as avoiding floor designs with these low frequencies as up to the 4th harmonic of the walking load can excite a natural frequency – if a natural frequency is a multiple of 1-4 of the walking load, a resonant response can occur. Therefore, floors with the first natural frequency below 10 Hz (2.5Hz x 4) are considered resonant response floors while floors above 10Hz are considered to have a transient response.

4.5.2 Step 2 – Modal Properties

The modal properties can be calculated by simply using closed form solutions. However, the limitations of using these equations are that the boundary conditions are limited to the derived formula and the solutions are commonly based on beam theory. Cross-laminated timber is a plate-like element that is capable of spanning both one-way and two-way. While its flexural modes as a one-way spanning structure can be predicted using beam formulas, more advanced plate formulas are required for torsional modes and two-way spanning behaviour. This section contains formulas, where available, for predicting the modal properties of CLT. It also contains advice on predicting these properties using finite element analysis (FEA).

Frequency

The closed form solution for the natural frequency of a simply supported beam is given by Equation 4.32. The natural frequency is proportional to the ratio of the stiffness of the structure to the modal mass.

$$f_j = \frac{j^2\pi^2}{2\pi l^2}\sqrt{\frac{EI}{m}} \tag{4.32}$$

Where:

j is the mode number

l is the length

EI is the stiffness of the cross-section

m is the modal mass

This equation is limited to the Euler-Bernoulli theory of slender beams where the effects of shear deformation of the cross-section are assumed negligible. Due to the cross lamination of CLT, it is particularly susceptible to shear deformation. The slender beam formula can still, however, be adopted by finding an effective EI value that accounts for the loss of stiffness due to shear.

The formula can also be modified by multiplying it by a value of, K, to account for fixity type to become Equation 4.33. Values for K are given in Table 10.

$$f_j = \frac{K}{2\pi l^2}\sqrt{\frac{EI}{m}} \tag{4.33}$$

Table 10: Values for K, for beams with different end conditions (Willford, Young & CEng 2006).

End Condition	1st mode, K_1	2nd mode, K_2	3rd mode K_3
Pin – Pin	9.87	39.5	88.8
Fix – Free (Cantilever)	3.52	22.0	61.7
Fix – Pin	15.4	50.0	104
Fix – Fix	22.4	61.7	121

If the end fixity cannot be idealised as the examples in Table 10 and has some sort of partial restraint, the beam can be represented as a symmetrically elastically supported beam shown in Figure 25. Advanced computations are required to calculate the K values for this type of beam. Generalised solutions for this type of beam are given in (Karnovsky, Lebed & Karnovskii 2004).

Figure 25: Beam with symmetrically elastically restrained ends, (Wang & Wang 2013).

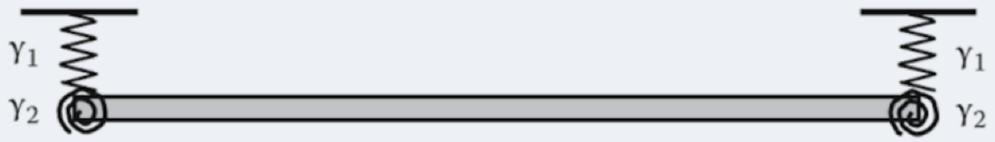


Figure 25: Beam with symmetrically elastically restrained ends (Wang & Wang 2013).

Beam theory, however, will only account for the flexural modes of vibration and will ignore the torsional and transverse modes present in the CLT plate structure. Experiments have indicated that the 2nd mode of a single simply supported CLT plate, which is a torsional mode, also has a large modal contribution factor and should also be considered for the design of CLT (see final project report, PNA 341-1415, available on the FWPA website). Table 12 shows the first five modes of vibration for a single panel of CLT, simply supported on each side,. Therefore, formulae for plate theory are required to capture these vibration modes. Unfortunately, this is not a trivial calculation and there are several textbooks, including Timoshenko Theory of Plates and Shells that are dedicated to solving closed form solutions for plate and shell structures (Timoshenko & Woinowsky-Krieger 1959).

The two-way spanning vibration modes can also be calculated using plate theory. Frequency for a simply supported plate with isotropic material properties can be calculated using Equation 4.34. In Equation 4.34, D is the flexural rigidity defined in Equation 4.35 and the dimensions of the plate are shown in Figure 26.

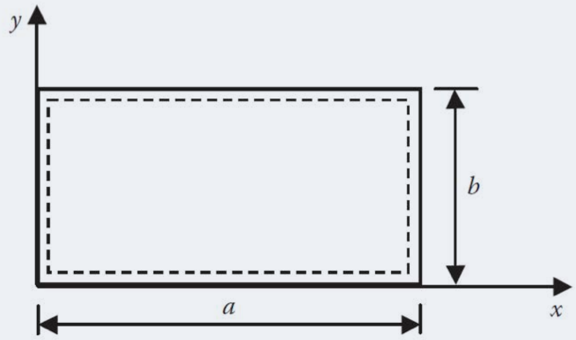


Figure 26: Coordinates and dimensions of two-way spanning plate.

$$f_{m,n,iso} = \frac{\pi}{2}\sqrt{\frac{D}{\rho t}}\left[\left(\frac{m}{a}\right)^2 + \left(\frac{n}{b}\right)^2\right] \tag{4.34}$$

Where:

ρ	is the density of the material
t	is the thickness of the plate
m	is the number of half sine waves in the x direction
n	is the number of half sine waves in the y direction
a,b	are the dimensions of the plate

$$D = \frac{Eh^3}{12(1-\upsilon^2)} \tag{4.35}$$

The isotropic equation can be modified to include the effects of the orthotropic nature of CLT. In each spanning direction, x and y, CLT has a strong and weak direction depending on the cross lamination. Equation 4.36 includes the orthotropic effect of CLT. Equation 4.37 gives the flexural and torsional rigidity.

$$f_{n,n,ortho} = \frac{\pi}{2a^2}\sqrt{\frac{m^4 D_x + 2Hm^2n^2\left(\frac{a^2}{b^2}\right) + n^4 D_y\left(\frac{a^4}{b^4}\right)}{m}} \tag{4.36}$$

Where:

m is the mass per unit area

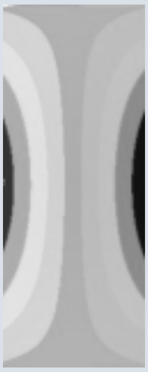
D_x, D_y are the flexural rigidity in the x and y direction

H is the torsional rigidity

$$D_x = D_y = H = \frac{Et^3}{12(1-\upsilon^2)} \tag{4.37}$$

However, these exact solutions are complex to derive and require significant computation. Finite element programs are becoming significantly easier to use and readily available. Therefore, in some cases it is more straightforward to perform FEA analysis to determine the frequency. The benefit of FEA is that mode shapes and modal masses are also easily extracted. Closed form equations discussed in this section can be used to check the validity of the FEA model.

Table 11: First five modes of vibration for a simply supported single 5.9 m long CLT panel.

Mode	1	2	3	4	5
Shape					
Frequency (Hz)	8.4	13.8	29.7	35.5	57.2

The theories discussed so far are for slender plates and beams. For some cases, using an effective EI value to capture the deformation is acceptable, but for span-to-depth ratios less than 15 and for concentrated loads these methods fail (Thiel 2013). Studies by Stürzenbecher et al. (2010) have compared various composite laminated advanced plate theories to produce a two-dimensional plate calculation. The advanced plate theories examined using Equivalent Single Layer Methods (ESLM), which means the number of independent variables is not dependent on the number of layers. ELSM is derived from 3D elasticity theory by making suitable assumptions concerning the stress state through the thickness of a laminate (Reddy 2006). These assumptions allow the reduction of the 3D problem to a 2D problem. The plate theory that was found to most accurately represent the plate behaviour of CLT was developed by (Ren 1986). The Ren theory contains a zigzag term that allows for discontinuous shear strains to represent the laminate specific characteristics. The theory by Ren (1986) was compared with a more accurate and time consuming exact solution by (Pagano & Hatfield 1972). It was found that the Ren plate theory closely matched the results of the exact solution without its computational difficulty.

Stürzenbecher et al. (2010) conducted further research to simplify this model by developing a 6-solution independent variable model (one less than Ren) that considers the relationship between the plane stress reduced stiffness components and the transverse shear stresses. It was found that compared to the Ren (1986) plate theory, the new theory delivers at least the same accuracy, and for transverse shear stresses even better, and at a lower computational cost. The extension of the presented plate theory to angle ply laminates and its implementation into finite element software is planned to make it applicable to structural simulations of plates with arbitrary lay-up, shape, and boundary conditions.

Modal Mass and Mode Shape

If performing finite element analysis, the modal mass can be extracted from a modal analysis. Care must be taken on whether a unity normalised or a mass normalised analysis is conducted. Most FEA packages allow you to choose. Either is okay to use if it is understood how each analysis affects the mode shape. If the structure is unity normalised, then the maximum displacement of the structure is set to 1 for every mode. The modal mass will then vary for each mode and should be used with the mode shape values for the unity normalised shape.

For a mass normalised analysis, the mode shape displacements are calculated from a modal mass of 1 kg for each mode. Therefore, a unity normalised measurement should occur if the modal masses are explicitly required. For example, in ANSYS, these can be extracted from the eigenvalue solutions or by converting the maximum kinetic energy for each mode into modal mass using Equation 4.38.

$$M_n = \frac{KE_n}{2\pi^2 f_n^2} \tag{4.38}$$

This equation can be checked by assessing that the first flexural mode is about half the weight of the floor.

Damping

In the CLT Handbook by FPInnovations values of damping ratios as low as 1% are used (Hu & Gagnon 2011). Tests found one-way spanning, simply supported CLT has a damping ratio for the first mode between 0.5% and 1.5%. Subsequent modes did not vary significantly. (See final project report, PNA 341-1415, available on the FWPA website.)

The damping is affected by the configuration, material, support conditions and the loading type. The occupant of a floor can change the damping characteristics. Onsite damping has been reported to be higher than in laboratory experiments. A study conducted in Sweden found that the damping ratio could be four times the value found in laboratory studies (Jarnerö, Brandt & Olsson 2015). Studies at UTS found laboratory CLT floors have damping ratios as low as 1%, while CLT floors tested in situ had damping ratios of 2.5% and 5% with no topping and with a 50 mm screed, respectively.

4.5.3 Prescriptive or Simplified Analysis

Prescriptive-based methods provide a simplified assessment of the vibration performance of a floor for a set of scenarios defined by the standard or design guide. These methods provide assessment of one or more of the following properties; stiffness, natural frequency, velocity, and acceleration of the floor. The methods generally contain equations to calculate the modal properties and give limits based on the type of floor. The methods compared here are from Eurocode 5 (2008), modifications of Hamm et al. (2010), modification by Mohr (1999) and the CLT Handbook criteria (Hu & Gagnon 2011). These methods and the criteria they use to assess the floor, including limit values are summarised in Table 12.

Table 12: Comparison of available analytical models for determining vibration performance.

	Stiffness (Unit Displacement)		Floor Natural Frequency		Floor Velocity		Acceleration (floors under 8 Hz only)	
Vibration Performance Method	Load kN	Limit mm	Load Case	Limit Hz	Velocity	Limit	Frequency Range Hz	Limit m/s2
Eurocode 5	1	≤1	G_{TOT}	≥8	Eq.(4.43)	Eq.(4.42)		
Hamm et al	2	≤0.5	G_{TOT}	≥8			4.5 – 8	≤0.05
Mohr	1	≤1	G_{TOT}+0.3Q	≥8	Eq.(4.47)	Eq.(4.47)	3.4 – 8	≤0.1
CLT Handbook	1	*	G_{TOT}	*				

* According to CLT Handbook criteria the floor frequency is dependent on the floor stiffness and vice versus.

The methods from European research and standards (Eurocode 5, Hamm et al. and Mohr) requires the vibration requirements of the floor to be defined first – either normal or high. High requirements are considered for commercial buildings and multi-storey residential blocks, whereas normal requirements are considered for single unit dwellings. Since this guide is concerned with long-span floors, primarily found in commercial buildings, high requirements for vibration are considered.

Eurocode 5

Eurocode 5 provides guidelines for providing acceptable vibration design of residential timber floors. Longitudinal stiffness (EI_l) and stiffness transverse to the span (EI_t), for a 1 m wide cross-section of CLT are used to calculate the natural frequency, deflection limit and floor velocity.

The natural frequency of the timber floor calculated using Equation 4.39, is limited to a minimum of 8 Hz, to avoid vibrations caused by resonance. Eurocode states that frequencies of 8 Hz can be acceptable with a 'special investigation' required; however, it does not provide guidelines for this investigation. The factor for support stiffness (k_m) in Eurocode 5 is equal to π^2 which represents a single span simply supported floor. For other end conditions the factors in Table 11 can be used.

$$f_1 = \frac{k_m}{2\pi l^2}\sqrt{\frac{(EI)_l}{m}} \geq 8\ Hz \tag{4.39}$$

The mass, m, is treated as a static mass; equal to the self-weight of the floor plus any extra imposed loads depending upon the use of the floor. Further to checking natural frequency, the deflection due to a unit force (Equation 4.40) is limited to a maximum value 'a', which is dependent on the required vibration performance level of the floor. A graph is provided in the code that displays the relationship between the limit value for deflection (a) and the limit value for velocity (b) (see Figure 27). The calculations are based on a rectangular floor supported on all four sides. Therefore, an equivalent beam width, b_{eff}, is calculated to determine the panel's equivalent beam effective stiffness (EI_b) taking into account the transverse stiffness using Equation 4.41 (Mohr, 1999).

$$w_{EC5} = \frac{1}{48}\frac{Fl^3}{EI_b} \leq a\, mm/k\, N$$

$$b_{eff} = \frac{l}{1.1}\sqrt[4]{\frac{EI_t}{EI_l}}$$

The velocity (v) due to an impulse of 1 Ns is then calculated using Equation 4.43 and limited by Equation 4.42. Only the number of first order modes with natural frequencies up to 40 Hz is considered and calculated using Equation 4.44. A value for damping, ζ = 1%, is provided by the code.

$$v \leq b^{(f_1\zeta-1)}\ m/(Ns^2)$$

$$v = \frac{4(0.4 + 0.6n_{40})}{mbl + 200}$$

$$n_{40} = \left\{\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{b}{l}\right)^4 \frac{(EI)_l}{(EI)_b}\right\}^{0.25}$$

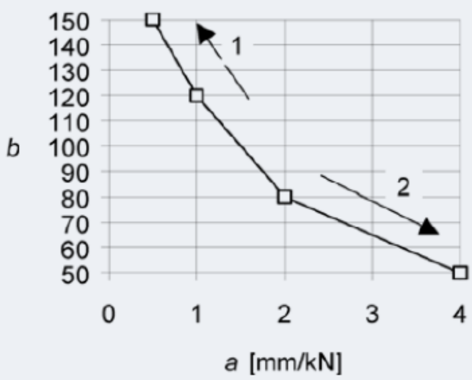


Figure 27: Interaction between the limit values of a, and b; directions 1 and 2 correspond to better and worse behaviour respectively (CEN 208).

Modifications of Hamm et al. (2010)

Modifications of the Eurocode 5 method were developed by Hamm et al. (2010) in Germany to account for the stricter requirements on vibration performance and for floors with natural frequencies less than 8 Hz. The research, which was based on the assessment of 50 buildings and 100 floors, found timber floors with natural frequencies less than 8 Hz, particularly heavy floors, could have acceptable vibration performance. A light floor on the other hand could perform poorly when subjected to frequencies over 8 Hz. Figure 28 is a flow chart that outlines the design procedure.

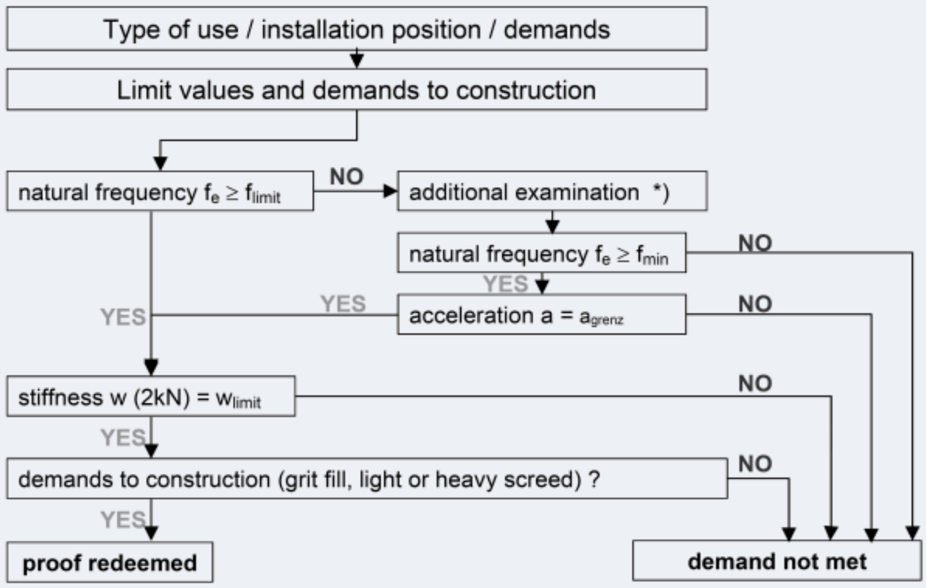


Figure 28: Flow chart for the design and construction of timber floors, the additional examination only applies for heavy floors with wide spans, or timber concrete composite systems (Hamm et al. 2010).

The frequency is calculated using the same method as Eurocode 5 considering only the static mass of the floor. The stiffness criterion is also calculated using a similar method as the Eurocode, however, it is given a more stringent limit value of 0.5 mm and a concentrated load value of 2 kN as opposed to 1 kN. The more stringent criteria were determined by studying the behaviour of several floors (Hamm et al. 2010).

If the frequency of the floor is less than 8 Hz, the floor is not necessarily deemed unacceptable, unlike the Eurocode. An additional examination of the acceleration is provided along with the original criteria also being met. The acceleration is calculated using Equation 4.45 and is limited to 0.05 m/s^2. In Equation 4.45, P_0 is the force of one person (taken as 700 N) and the values for the Fourier coefficient α_i and the forcing frequency FF are given in Table 13. The generalised mass, M_{gen}, is equivalent to half the effective area contributing to vibration performance (Equation 4.46) where the mass, m, is the self-weight of the floor plus any super-imposed dead load. Values for damping were taken as 1% as outlined by Eurocode 5.

$$a \approx 0.4 \frac{P_0 \alpha_i (f_1)}{M_{gen}} \frac{1}{\sqrt{\left[\left(\frac{f_1}{f_F}\right)^2 - 1\right]^2 + \left(2D\frac{f_1}{F_F}\right)^2}} \leq 0.05 \ m/s^2 \qquad (4.45)$$

$$M_{gen} = m \frac{l}{2} b_{eff} \qquad (4.46)$$

Table 13: Fourier coefficient, dependent on the fundamental frequency of the floor (Mohr, 1999).

Fundamental Frequency Hz	Fourier coefficient	Forcing frequency F_F Hz
3.4 < f1 ≤ 4.6	0.2	f_1
4.6 < f1 ≤ 5.1	0.2	f_1
5.1 < f1 ≤ 6.9	0.06	f_1
f_1 > 6.9	0.06	6.9

Mohr Criteria

The International Council for Building Research Studies and Documentation provides an alternate modification to the Hamm et al. (2010) method for frequencies below 8 Hz and was developed at the Technical University of Munich (Mohr 1999). This method considers a quasi-static floor mass that includes a portion of the live load in the total floor mass (G + 0.3Q) for calculating the natural frequency. Apart from the floor mass being quasi-static, both the frequency and the floor stiffness are calculated by the same method as Eurocode 5. A floor velocity check is included that was derived from the action of a 'heel drop' and is given by Equation 4.47. A damping value of 1% is assigned to floors without any additional boarding's for sound isolation as outlined by Mohr (1999).

$$v_{MOHR} = \frac{0.6}{m_f^{0.5} EI_l^{0.25} EI_t^{0.25}} < v_{lim,MOHR} = 6 \times 100^{(f\zeta - 1)} \quad (4.47)$$

For floors with frequency below 8 Hz the acceleration is calculated using the same methods as outlined by Hamm et al. (2010). However, the acceleration limit is less stringent at 0.1 m/s^2.

CLT Handbook

A Canadian research team, FPInnovations, developed a simplified method to specifically assess the vibration performance for CLT floors, which was published in the CLT Handbook (Hu & Gagnon 2011). The criterion given by Equation 4.48 provides an inequality based on the fundamental frequency and the effective stiffness of the floor under a unit load.

$$\frac{f}{\Delta^{0.7}} \geq 13 \quad (4.48)$$

The deflection is calculated considering a 1 m-wide CLT panel and the frequency is calculated considering static mass only.

4.5.4 Response Factor Analysis

The prescriptive-based methods (discussed in the previous section) allow calculations of only certain floor types. Using response factor analysis, any floor type can be considered. Two design guides are considered here. The first was released by the Steel Construction Institute (Smith et al., 2009) and its scope is limited to steel framed floor and building types. The scope of the second guide, which was released by The Concrete Society Willford et al., 2006), is not limited to concrete structures and includes any other form of construction material that people walk on, including floors and bridges.

Both documents split the analysis up into resonant and transient response structures. Resonant response structures are defined by having any natural frequencies less than 10 Hz while impulsive or transient response structures have the first natural frequency above 10.5 Hz. This categorisation is based on the possible harmonics of walking frequencies that can influence a resonant response. If the structure's first natural frequency is around 10 Hz, both transient and resonant responses are advised to be analysed.

Concrete Society design guide

This section presents an example of a vibration response function procedure in accordance with the Concrete Society design guideline (Willford et al., 2006). The guideline provides an in-depth step-by-step procedure for determining the response function and should be consulted for analysis. A summary and example calculation of a resonant response structure is only discussed in this section as long-span floors have low fundamental frequencies and are likely to fall into the resonant zone.

The floor plate considered has been tested (final report for PNA 341-1415 is available on the FWPA website). The floor properties including material values are outlined in Appendix B.1. The floor consists of three CLT panels connected by half lap connections spanning between two timber support beams. Figure 29 shows the floor plan and the walking path used to activate the vibration response of the floor.

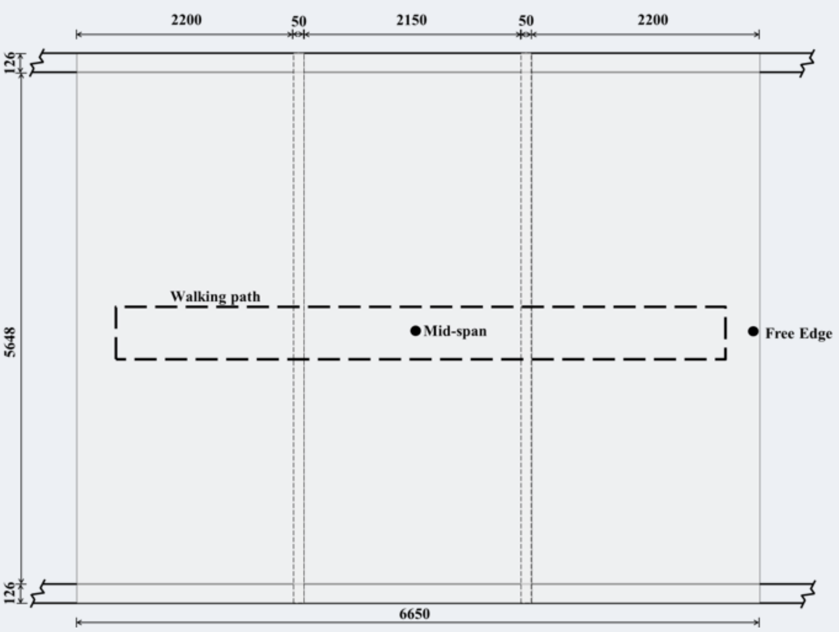
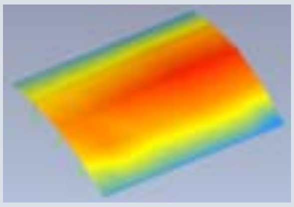
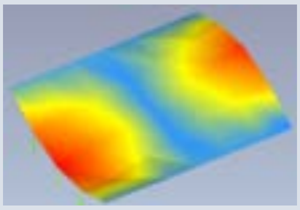
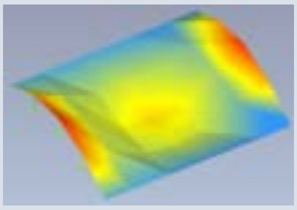
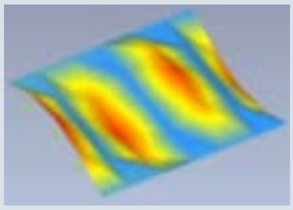


Figure 29: Floor dimensions of CLT floor plate and walking path.

The modal properties of the floor are first determined. This can be done by any appropriate method; closed form solutions, FEA and by experimentation. Table 14 shows the values of the first four natural frequencies, damping ratios and mode shapes, from walking tests conducted on the CLT floor plate.

Table 14: Results from ARTeMIS operational modal analysis software for the first four modes of vibration of the CLT floor plate.

Mode 1	Mode 2	Mode 3	Mode 4
f_1 = 7.72	f_2 = 9.74	f_3 = 13.86	f_4 = 19.15
ζ_1 = 1.2%	ζ_2 = 1.2%	ζ_3 = 1.1%	ζ_4 = 1.0%
			

The maximum floor response is going to occur due to walking loads that have a harmonic that can cause resonance with a natural frequency of a floor. The following process should occur not only for harmonics with the fundamental frequency but with any natural frequencies less than 10 Hz.

Step 1:

The first step is to calculate the harmonic forcing frequency, f_h. This is done by multiplying the walking frequency, f_w, by the harmonic number. For the floor tested the walking frequency was selected to be 2.57 Hz, which is listed as the 1st harmonic in Table 15. This walking frequency was selected as its 3rd harmonic corresponds with the fundamental frequency of the floor.

Step 2:

The forcing frequency, F_h, for each mode under 15 Hz is calculated. The guide contains the table to conduct this calculation.

Step 3:

The real and imaginary acceleration is then calculated and summed to find the magnitude of the response using the following equations:

$$a_{real,h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{m_m} \frac{A_m}{A_m^2 + B_m^2} \qquad (4.49)$$

$$a_{imag,h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{m_m} \frac{B_m}{A_m^2 + B_m^2} \qquad (4.50)$$

Where:

$A_m = 1 - \left(\frac{f_h}{f_m}\right)^2$ and $B_m = 2\zeta_m \frac{f_h}{f_m}$

Where:

$\mu_{r,m}$	is the mode amplitude at the response location
$\mu_{e,m}$	is the mode amplitude at the excitation location
f_m	is the mode frequency
ζ_m	is the mode damping ratio
m_m	is the modal mass
$\rho_{h,m}$	is a correction factor to account for the likelihood of resonant response being reached by the number of footfalls possible and the length of the floor

The real and imaginary accelerations are summed for each mode the magnitude of the acceleration response is found using Equations 4.53 and 4.54.

$$a_{real,h,} = \sum_{m} a_{real,h,m};\ a_{imag,h} = \sum_{m} a_{imag,h,m} \tag{4.51}$$

$$|a_h| = \sqrt{a_{real,h}^2 + a_{imag,h}^2} \tag{4.52}$$

This process was conducted for two locations of the floor (see Figure 29), at the mid-span of the floor and at its edge.

Step 4:

The response factors are calculated by dividing the magnitude of the acceleration response by the baseline peak acceleration for a response factor of 1. Table 15 gives the results from the guideline at the free edge and at the mid-span.

For a walking speed of 120 bpm or 2Hz we find a response factor of 48 at the mid-span and 56 at the free edge. When the pace is increased to 2.57 Hz which has a harmonic that coincides with resonance the RF increases to 250 at mid-span and 300 at the free edge.

Table 15: Response factor results from the Concrete Society design guideline.

Walking Frequency	RF mid-span	RF free-edge
2 Hz (120bpm)	48	56
2.57 Hz (resonance)	250	300

The CLT floor produced very high response factors both at mid-span and at the edge of the floor plate. This is a bare CLT floor under laboratory conditions and finishes and partitions in an actual floor would add additional damping to the system. An observation from this method is that the response factors are highly sensitive to changes in frequency, mass and damping values. This leads to the tendency to want to increase mass and damping to lower the response factor value. For example, doubling the damping from 1.2% to 2.4% reduces the mid-span response factor from 250 down to 120. Changing the mass of a system requires re-analysis of the floor to assess the resulting natural frequencies. Care should be taken with both approaches as an increased mass leads to lower natural frequencies and known damping values are currently poorly defined.

Steel construction institute design guide

The document provided by the Steel Construction Institute (Smith et al., 2009) provides a comprehensive vibration design assessment for steel-framed floors. While the document states that its scope is limited to steel structures, it provides a framework that can be used for any material type. The design process is dependent on the availability of finite element (FE) modelling. A simplified method is provided if FE analysis is not available.

The response analysis is based on the conservative assumption that the vibrating force will be applied to the most responsive part of the floor. This is a logical assumption given the most responsive location is generally the centre of a floor where movement will likely occur.

For low frequency floors (defined by Table 16) both a steady state response and a transient response are conducted. This is because the higher frequencies of a low frequency floor can cause the transient response to be greater than the steady-state response. For floors defined as high frequency, only a transient response is considered.

Table 16: Definition of low frequency floors provided by Smith, Hicks & Devine (2007).

Floor Type	Low to high frequency cut-off
General floors, open plan offices, etc	10 Hz
Enclosed spaces, e.g. operating theatre, residential	8 Hz
Staircases	12 Hz
Floors subject to rhythmic activities	24 Hz

Steady state response of floors (resonant response)

The natural frequencies up to 2 Hz higher than the cut-off frequency in Table 17 for a low frequency floor are examined. The weighted root mean square (RMS) acceleration of the floor for a single mode of acceleration is calculated using Equation 4.53.

$$a_{w,rms,e,r,n,h} = \mu_{e,n}\mu_{r,n}\frac{F_h}{M_n\sqrt{2}}D_{n,h}W_h \tag{4.53}$$

Where:

e	is the point of excitation
r	is the response location
n	is the mode number
h	is the harmonic number
$\mu_{e,n}$	is the mode shape amplitude at the excitation point
$\mu_{r,n}$	is the mode shape amplitude at the response location
F_h	is the excitation force for the hth harmonic
M_n	is the modal mass
$D_{n,h}$	is the dynamic magnification factor for acceleration
W_h	is the weighting factor for human perception of vibrations

A discussion of how to calculate modal mass and the relationship of modal mass to the mode shape amplitude are included in section 3.5.1. The dynamic magnification factor $D_{n,h}$, which is the ratio of the peak amplitude to the static amplitude is calculated using Equation 4.54.

$$D_{n,h} = \frac{h^2\beta_n^2}{\sqrt{(1-h^2\beta_n^2)^2+(2h\zeta\beta_n)^2}} \tag{4.54}$$

Where:

β_n	is the frequency ratio of the walking frequency f_p to the mode frequency f_n (f_p/f_n)
ζ	is the damping ratio

A weighting factor, Wh, is included to account for the perception of vibration that will cause discomfort in different building uses. For example, perceivable floor vibration in a hospital will cause more discomfort to an occupant than in an office or residential building. The weighting factors for vertical floor vibrations are dependent on the natural frequency of the floor and included in the design guide.

The total response of the system is then calculated by adding the accelerations of each mode contributing to the vibration response. The document discusses three methods to perform the summation of the response acceleration:

- **Full time history**: The most accurate methods which yields both peak and RMS accelerations but is computationally intensive. Since RMS accelerations are only required some simpler methods is presented.
- **Sum of peaks (SoP)**: Provides a conservative calculation as it assumes all the components of the response peak at the same time and continue to peak at the same time, i.e. it will calculate the RMS acceleration as proportional to 1/√2 of the peak acceleration where in reality the RMS will actually be somewhat lower. Figure 30 shows the relationship of SoP acceleration to the actual accelerations.
- **Square-root sum of squares (SRSS)**: The recommended method to determine the RMS acceleration response that will produce the same RMS acceleration as a full time history summation is calculated using the following equation:

$$a_{w,rms,e,r} = \frac{1}{\sqrt{2}}\sqrt{\sum_{h=1}^{H}\left(\sum_{n=1}^{N}\left(\mu_{e,n}\mu_{r,n}\frac{F_h}{M_n}D_{n,h}W_h\right)\right)^2} \tag{4.55}$$

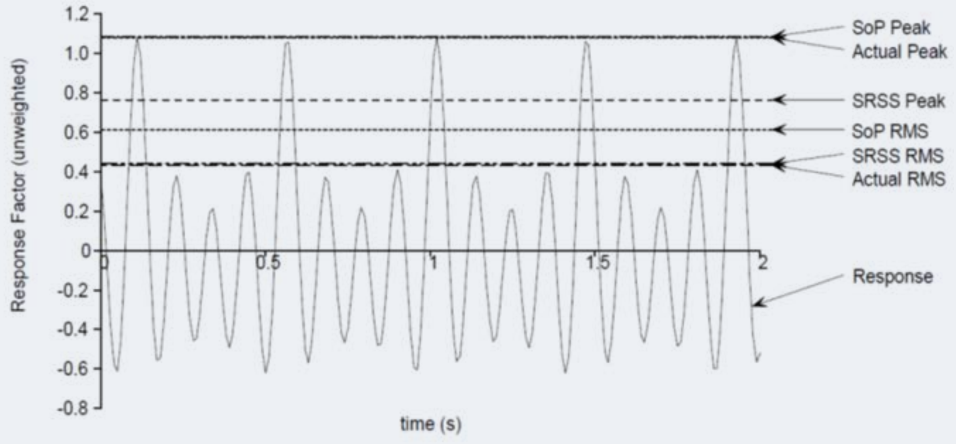


Figure 30: The relationship of peak and RMS accelerations for the time history vibration response of a floor.

The response factor is calculated by dividing the RMS acceleration with a base value acceleration of 0.005 m/s^2. The response factors for the CLT floor example are included in Table 18. At a walking speed of 120 bpm the mid-span response factor was found to be 51 while the free-edge was found to be 74. These values increased significantly to 270 and 390 when the walking speed corresponds to a floor harmonic.

Table 18: Response factor results from the SCI design guideline.

Walking Frequency	RF mid-span	RF free-edge
2 Hz (120 bpm)	51	74
2.57 Hz (resonance)	270	390

4.6 Experimental Observations

The CLT floor in Figure 29 was tested using two walking subjects. The walkers were instructed to walk at speeds of 60, 90 and 120 bpm and at a speed that corresponds to a harmonic of the fundamental frequency of the floor. The walkers took a number of different paths on the slab and would walk each path three times to ensure resonance build up was possible and that all areas of the floor were activated. Walker 1 had a weight of 52 kg while walker 2 was 65 kg. The accelerations were recorded, then weighted, filtered and the response factors calculated to compare with the predicted results.

4.6.1 The Effect of Using Extra Self-Tapping Screws at the Support

The size and number of screws at the support connection were varied and the response factors were recorded. The screws are partially threaded self-tapping screws with diameters of 6 mm and 8 mm and spacing from 125 mm up to 1000 mm. The cross-section of the span with the location of the self-tapping screw is shown in Figure 31.

There was a general trend for response to increase with increasing number of screws. The initial response from 0 screws to 6 screws at 1000 mm spacing was the most pronounced and then the change tapers off (see Figure 32). The maximum response factor was 14 for walker 1 and 18 for walker 2, this is an increase from 9.5 and 13 for having no screws at the support. Therefore, increasing the number of screws and hence the stiffness at the support has shown to have a negative influence on the vibration response of the floor.

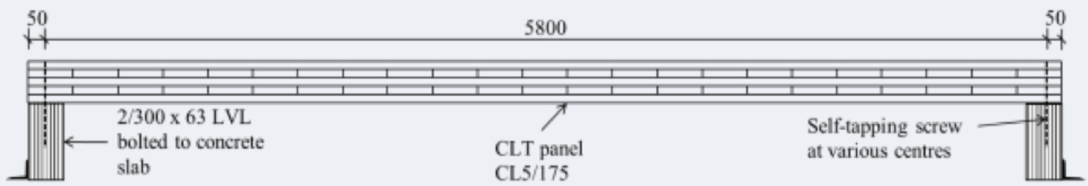


Figure 31: Cross section of single CLT panel connected to frame with a single screw connection.

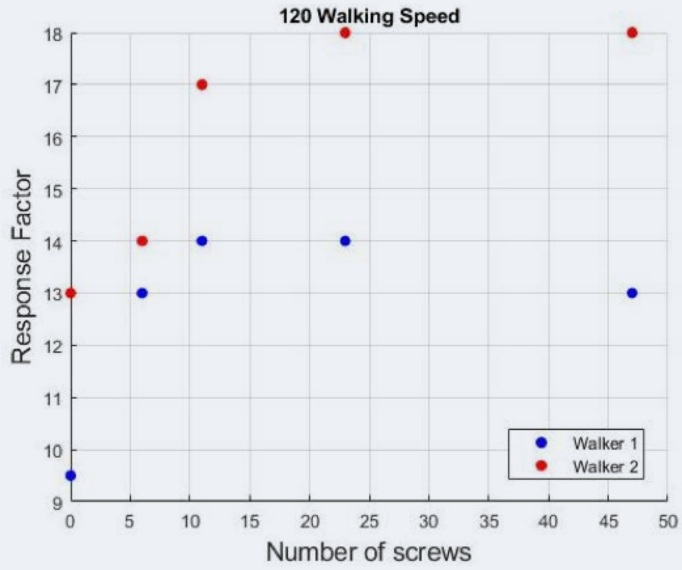


Figure 32: Response factors from walking speeds of 120 bpm on a single panel with increasing number of self-tapping screws at the support.

4.6.2 The Response of the Floor with 1, 2 and 3 Panels Adjacent to Each Other

Experiments were conducted on CLT floors spanning between two LVL beams. Three floor panels were tested with configurations of a single, double and triple panel floor (see Figure 33). The response factors were calculated over the entire floor of each configuration. However, for the purpose of this report the response factors are discussed at the centre of the span at both its mid-point and at the free edge (see Figure 33).

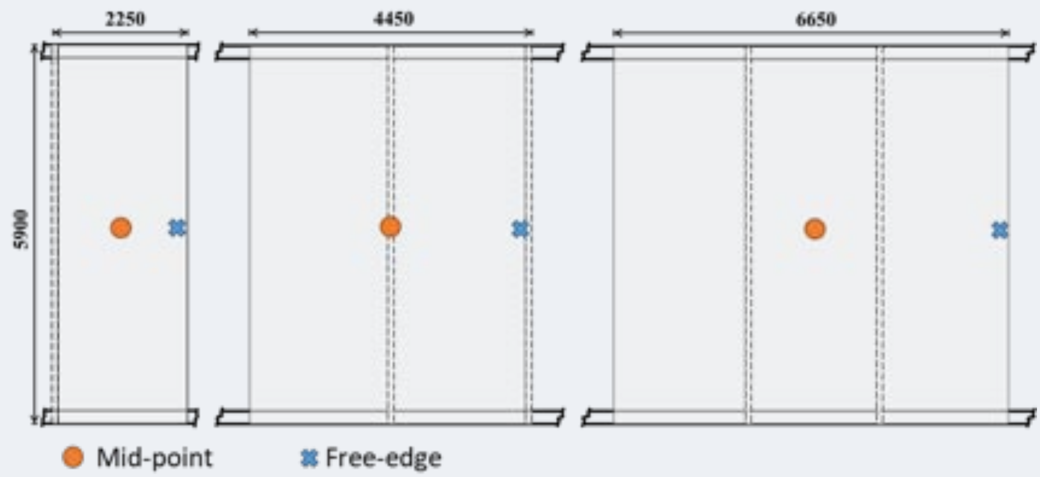


Figure 33: Layout of testing for CLT panels with locations of response factor calculations.

The response factor (RF) results from the walkers are recorded as a range in Table 18 at speeds of 60, 90 and 120 bpm. The response factor is dependent on the walker maintaining a steady and consistent walking pace. Because this is difficult to monitor, the range of response factors are recorded to indicate the variation in the data. It was found at the free edge, the response factors were relatively the same for each floor configuration. However, the response factors significantly decreased between the single panel and triple panel configuration at the mid-point of the floor. This is due to the extra mass of the three-panel floor compared to a single panel.

Table 18: Response factors of single, double and triple panel CLT floors.

	At free edge			At mid-point of floor		
Walking Speed	Single	Double	Triple	Single	Double	Triple
60	3.2–5.2	3.5–4.3	2.9–4	2.9–4.9	1.8–2.1	1.7–2
90	7.1–9.8	6–8.7	9.1–11	6.6–9	3.4–4.8	4.3–4.9
120	7.2–20	14–17	15–18	6.2–19	5.4–6.1	7.9–9.3

4.6.3 Added Support to the Free Edges of the CLT Floor

Generally, the largest vibration response occurs at the free edges of the CLT floor. Therefore, LVL support beams were added at the free edges to assess any improvement in the vibration response. By adding in the additional support, the CLT floor should theoretically act like a two-way plate (see Figure 34) as opposed to one-way spanning when there is no additional support.

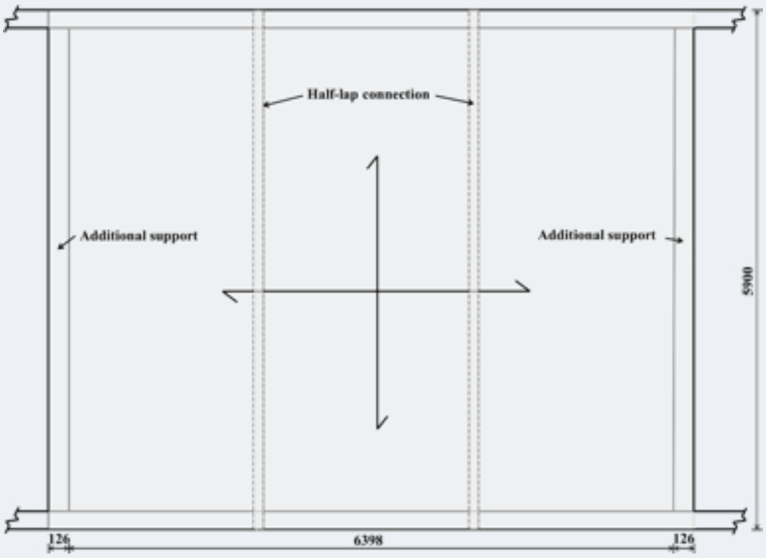


Figure 34: CLT floor with additional support at the free edges.

Table 19 gives the maximum response factors from the walkers and with the results from the one-way spanning floor. The maximum response factors have more than halved, from values of 15–18 for the one-way span down to 3.9–7.5 for the two-way floor at a walking speed of 120. Due to the extra support the maximum response factor of the two-way spanning floor is at the centre of the floor. The RFs at the centre of the two-way floor are smaller but not dissimilar to the RFs at the centre of the one-way spanning floor.

Table 19: Comparison of response factors of the one-way and two-way floor configurations.

	At free edge		**At mid-point of floor**	
Walking Speed	Max RF	Mid-point RF	Max RF	Mid-point RF
60	2.9–4	1.7–2	0.7–1.8	0.7–1.8
90	9.1–11	4.3–4.9	1.5–2.6	1.5–2.6
120	15–18	7.9–9.3	3.9–7.5	3.9–7.5

5 Discussion on Response Factor

Acceptable levels of vibration magnitudes are typically evaluated in industry as response factors (RF). The value indicates a level for which the probability of adverse comment is low. For commercial buildings, floors are mostly designed with a RF of less than 4, although this can be relaxed through consultations with the client.

The RF is proportional to the mass and damping of a structure. The higher the mass and damping ratio, the lower the response factor. Timber floors are light in weight and, therefore, produce high response factors that may not represent how they 'feel' in reality.

The floors tested at UTS (final project report, PNA 341-1415) were not designed to satisfy perception. These floors were built to examine their modal properties under various support and boundary conditions. However, it was noted by the staff and students working on each of the floors that the floors are significantly stiff and an examination of the RFs was then undertaken. The one-way spanning CLT system was found to have RFs up to 18 while the two-way spanning system values were up to 7.5. These are higher than the general accepted response factor of 4 for most systems. The ribbed deck cassette floor was also found to have higher RFs compared to accepted response factor (13 and 23 for pace frequencies of 2 and 2.14 Hz, respectively). However, procedure in CCIP-06, SCI P354 and DG 11 significantly overestimated the RF values for this type of floor, which was as high as 221% of the measured RF value.

On-site testing found that bare CLT floors produced RFs of about 30 while CLT floors with a concrete screed had RFs of about 10.

6 References

Australian Standard™, 2010, Timber Structures, Part 1: Design Methods, vol. AS 1720.1–2010, Standards Australia, Homebush (NSW), Australia.

Australian/New Zealand Standard™ 2002a, Structural Design Actions, Part 0: General Principles, vol. AS /NZS 1170.0:2002, Standards Australia, Homebush (NSW), Australia.

Australian/New Zealand Standard™ 2002b, Structural Design Actions, Part 1: Permanent, Imposed and Other Actions, vol. AS /NZS 1170.1:2002, Standards Australia, Homebush (NSW), Australia.

Allen, D. E., & Murray, T. M. (1993). Design criterion for vibrations due to walking. Engineering Journal, 4th quarter, 117–129.

ANSI, A. P. A. (2012). ANSI/APA PRG 320–2012 Standards for performance-rated cross-laminated timber. ANSI/APA, Tacoma, US.

AS . (2010). AS 1720.1 – 2010 Timber Structures Part 1: Design Methods. Australia: SAI Global.

Bernard, E. S. (2009). Improving Dynamic Behaviour in Lightweight Engineered Timber Floors. Australia.

Bishop, R. E. D., & Johnson, D. C. (1960). The analysis of real systems; approximate methods. In The Mechanics of Vibration (pp. 212–244). Cambridge, Great Britain: Cambridge University Press.

Blass, H. J., & Fellmoser, P. (2004). Design of solid wood panels with cross layers. In 8th world conference on timber engineering (Vol. 14, p. 2004).

Bogensperger, T., Moosbrugger, T., & Silly, G. (2010). Verification of CLT-plates under loads in plane. In 11th World Conference on Timber Engineering (WCTE2010), Riva del Garda.

British Standards Institute. BS 6472-1:2008 Guide to evaluation of human exposure to vibration in buildings, BS 6472-1:2008 (2008).

British Standards Institute. UK National Annex to Eurocode 5: Design of timber structures. General. Common rules and rules for buildings (2008).

Brownjohn, J., Racic, V., & Chen, J. (2016). Universal response spectrum procedure for predicting walking-induced floor vibration. Mechanical Systems and Signal Processing, 70–71, 741–755. https://doi.org/10.1016/j.ymssp.2015.09.010

CEN. EN 1995-1-1:2004 Eurocode 5: Design of timber structures – Part 1-1:General Common rules and rules for buildings (2004). The European Committee for Standardization. https://doi.org/10.1680/cien.2001.144.6.39

CEN. (2004b). Eurocoe 5: Design of Timber Structures.

Eurocode. (2003). Eurocode 5–Design of timber structures–Part 1-1: General rules and rules for buildings. prEN 1995-1-1. Bruxelles, Belgium.

Gagnon, S., & Pirvu, C. (2011). CLT handbook. Cross-Laminated Timber. FP Innovations Special Publication SP-528E.

Gagnon, S., & Popovski, M. (2011). Structural design of cross-laminated timber elements. Québec, FPInnovations.

Gavric, I., Fragiacomo, M., & Ceccotti, A. (2015). Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures. Materials and Structures, 48(6), 1841–1857.

Hamm, P., Richter, A., & Winter, S. (n.d.). Floor vibrations – new results.

HIVOSS. (2008). Vibration Design of Floors Guideline.

Hu, L., & Chui, Y.-H. (2004). Development of a design method to control vibrations induced by normal walking action in wood-based floors. In 8th World Conference on Timber Engineering. Lahti, Finland.

Hu, L. J., Chui, Y. H., & Onysko, D. M. (2001). Vibration serviceability of timber floors in residential construction. Progress in Structural Engineering and Materials, 3, 228–237. https://doi.org/10.1002/pse.69

Hu, L. J., & Gagnon, S. (2011). Vibration performance of cross-laminated timber floors. CLT Handbook. FPInnovations, Canada.

Jarnero, K. (2014). Vibrations in timber floors – dynamic properties and human perception. Linneaus University Dissertations.

Jarnerö, K., Brandt, A., & Olsson, A. (2015). Vibration properties of a timber floor assessed in laboratory and during construction. Engineering Structures, 82, 44–54. https://doi.org/10.1016/j.engstruct.2014.10.019

Karacabeyli, E., & Douglas, B. (2013). CLT handbook: cross-laminated timber. FPInnovations.

Karnovsky, I. A., Lebed, O., & Karnovskii, I. A. (2004). Free vibrations of beams and frames. McGraw-Hill Professional Publishing.

Labonnote, N. (2012). Damping in Timber Structures. Norwegian University of Science and Technology.

Mohr, B. (1999). Floor Vibrations. International Council for Building Research Studies and Documentation. Graz, Austria.

Ohlsson, S. V. (1988a). Springiness and human-induced floor vibrations: a design guide. Stockholm, Sweden: Swedish Council for Building Research.

Ohlsson, S. V. (1988b). Ten years of floor vibration research – a review of aspects and some results. In Symposium/Workshop on Serviceability of Buildings (pp. 435–450). Ottawa.

Ohlsson, S. V. (1991). Serviceability criteria – especially floor vibration criteria. In International Timber Engineering Conference (p. 1.58-1.65). London.

Pagano, N. J., & Hatfield, H. J. (1972). Elastic behavior of multilayered bidirectional composites. AIAA Journal, 10(7), 931–933.

Pavic, A., Reynolds, P., Prichard, S., & Lovell, M. (2003). Evaluation of Mathematical Models for Predicting Walking-Induced Vibrations of High-Frequency Floors. International Journal of Structural Stability and Dynamics, 3(1), 107–130. https://doi.org/10.1142/S0219455403000756

Reddy, J. N. (2006). Theory and analysis of elastic plates and shells. CRC press.

Ren, J. G. (1986). A new theory of laminated plate. Composites Science and Technology, 26(3), 225–239.

Smith, A. L., Hicks, S. J., & Devine, P. J. (2007). Design of floors for vibration: A new approach. Steel Construction Institute Ascot, Berkshire, UK.

Smith, A. L., Hicks, S. J., & Devine, P. J. (2009). Design of Floors for Vibration: A new approach. UK: The Steel Construction Institute.

Standard, A. (2010). AS 1720. 1—timber structures—part 1: design methods. Standards Australia.

Stürzenbecher, R., Hofstetter, K., & Eberhardsteiner, J. (2010). Structural design of Cross-laminated Timber (CLT) by advanced plate theories. Composites Science and Technology, 70(9), 1368–1379.

Taylor, S., & Hua, G. (2000). Dynamic performance of wood framed floor systems with poured toppings. In Sixth World Conference on Timber Engineering (p. 4.6.3.1-4.6.3.6.). Whistler.

TDA. (2015). Rethinking office construction – consider timber. Sydney, Australia: Timber Development Association NSW Ltd.

Thiel, A. (2013). ULS and SLS design of CLT and its implementation in the CLTdesigner. In Focus solid timber solutions—European conference on cross-laminated timber (CLT). The University of Bath, Bath.

Timoshenko, S. P., & Woinowsky-Krieger, S. (1959). Theory of plates and shells. McGraw-hill.

Toratti, T., & Talja, A. (2006). Classification of Human Induced Floor Vibrations. Building Acoustics, 13(3), 211–221. https://doi.org/10.1260/135101006778605370

Wang, C. Y., & Wang, C. M. (2013). Structural vibration: exact solutions for strings, membranes, beams, and plates. CRC Press.

Weckendorf, J., Zhang, B., Kermani, A., Reid, D., & Andersen, P. (2008). Damping Characteristics of Timber Flooring Systems with Respect to Low-Frequency Vibration Modes. In 10th World Conference on Timber Engineering. Miyazaki, Japan: Elsevier Ltd.

Willford, M. R., & Young, P. (2006). A design guide for footfall induced vibration of structures (CCIP-016). Surrey, UK.

Willford, M. R., Young, P., & CEng, Mim. (2006). A design guide for footfall induced vibration of structures. Concrete Society London, UK.

Willford, M., Young, P., & Field, C. (2007). Predicting footfall-induced vibration : Part 1. Proceedings of the Institution of Civil Engineers – Structures and Buildings, 160(SB2), 65–72.

Wyatt, T. A., & Dier, A. F. (1989). Floor Serviceability under Dynamic Loading. In International Symposium Building in Steel – The Way Ahead (p. 20.1-20.22). Stratford on Avon, UK: ECCS Publication No. 57.

Xiong, H., Kang, J., & Lu, X. (2011). Field Testing and Investigation of the Dynamic Performance and Comfort of Timber Floors. Journal of Asian Architecture and Building Engineering, 10(2), 407–412. https://doi.org/10.3130/jaabe.10.407

Zabihi, Z. (2014). Investigation of a Proposed Long Span Timber Floor for Non-Residential Applications. University of Technology Sydney.

A Appendix A. Worked example for a ribbed deck floor

A.1 Design of ribbed deck floor

This example demonstrates the ultimate and serviceability design of a ribbed deck cassette with top flange only. The flange and web are made from LVL 11 and LVL 13, respectively. The floor is located in a typical open plan commercial building. Vibration design as per procedures in CCIP-016, SCI P354 and DG 11 have also been undertaken including suggestions from this design guide.

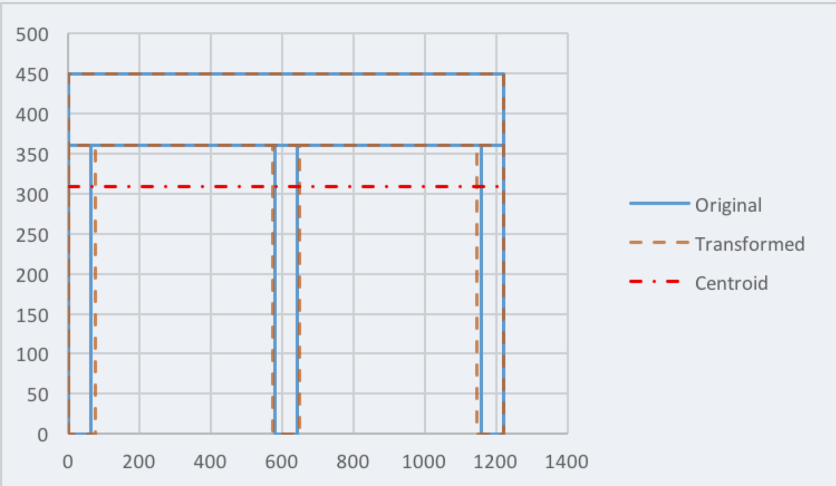


Figure A-1: Cross section of ribbed deck floor.

A.1.1 Floor structure

The floor cassette has a span of 9000 mm and consists of three 360 x 63 joists spaced equidistant apart. The cassette is 1220 mm wide. The cross-section can be seen in Figure A-1.

Cross section			
	depth	breadth	length
	mm	mm	mm
web	360	63	9000
top flange	90	1220	9000
bottom flange	0	0	0
total depth	450		
no. of joists/cassette	3		
no. of top panels	1		
no. of bottom panels	0		
Flange overhang	100	mm	

A.1.2 Material properties

The web and flange can be assumed to act compositely due to the glue and screw connection. The transformed section method has been used to convert the web to an equivalent width for the flange properties. The transformed cross-section can be seen in the dashed orange outline. The material properties have been taken from the manufacturer's technical data sheet. A density of 570 kg/m³ has been assumed.

Material properties								
	E	density	mass	f'b	f'c	f't	f's	$f'p_{edge}$
	MPa	kg/m3	kg/m	MPa	MPa	MPa	MPa	MPa
web	13200	569	12.9	48	38	33	5.3	10
top flange	11000	584	64.1	38	38	26	N/A	10
bottom flange	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

A.1.3 Section properties

This example demonstrates the ultimate and serviceability design of a ribbed deck cassette with top flange only. The flange and web are made from LVL 11 and LVL 13, respectively. The floor is located in a typical open plan commercial building. Vibration design as per procedures in CCIP-016, SCI P354 and DG 11 have also been undertaken including suggestions from this design guide.

Section properties

Element		A	y	Ay	Ixx	d	Ad2
		mm2	*mm*	*mm3*	*mm4*	*mm*	*mm4*
FLANGE	Top	109800	405	44469000	74115000	95.96	1E+09
FLANGE	Bottom	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	1	27216	180	4898880	293932800	129.0	5E+08
JOIST	2	27216	180	4898880	293932800	129.0	5E+08
JOIST	3	27216	180	4898880	293932800	129.0	5E+08
JOIST	4	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	5	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	6	N/A	N/A	N/A	N/A	N/A	N/A
SUM		191448		59165640	955913400		2E+09

$\bar{y}$ =	309.0	mm
I=	3.327E+09	mm4
EI=	3.659E+13	Nmm2
Z_bot=	1.076E+07	*mm3*
Z_top=	2.360E+07	*mm3*
hc	141	*mm*
ht	309	mm

The web centre to centre spacing was checked for shear lag effects.

Cross section characteristics

Section 3.1 Wood Solutions Design Guide #30 Timber Cassette Floor System

Web c/c spacing	610	mm
If bottom flange exists c/c spacing - minimum of below		
bf,t	N/A	mm
bf,t	N/A	mm
If top flange exists, spacing to be less than number below		
bf,c ≤	963	mm
Check	*Satisfactory for shear lag effects*	

A.1.4 Load combinations and modification factors

Loading

G

	kg/m	kg	kN/m	
web	12.9	348.3	0.38	
top flange	64.1	703.8	0.77	
bottom flange	0	0	0	
		TOTAL G=	1.15	
		G=	116.9	kg/m
		G=	95.8	kg/m2

SDL

	kPa	kN/m	
services	0.25	0.305	
lighting	0.25	0.305	
floor finishes	1.5	1.83	
	TOTAL SDL=	2.44	
	G+SDL=	365.6	kg/m
	G+SDL=	299.7	kg/m2

LL

	kPa	kN/m	
commercial	3	3.66	
	TOTAL LL=	3.66	
	(G+SDL)+0.1LL=	402.9	kg/m
	(G+SDL)+0.1LL=	330.3	kg/m2

Load combinations

ψ_s= 0.7

ψ_l= 0.4

		w*	M*	V*	N*c	N*t
		kN/m	*kNm*	*kN*	*kN*	*kN*
ULS	**1.35G**	4.8	49.0	21.8	159.9	N/A
	1.2G+1.5ψ_lQ	6.5	65.8	29.3	214.7	N/A
	1.2G+1.5Q	9.8	99.2	44.1	323.4	N/A

			w*	
SLS	**Short term**	**G+ ψ_sQ**	6.15	kN/m
		1Kn deflection	1	kN/m
	Long term	**G+ψ_lQ**	5.1	kN/m
		G only	3.6	kN/m

Modification factors

k4, k6, k9, k12 factors

Category	2
Material	Structural LVL
Lay	355 mm

Force	Φ	k_4	k_6	k_9	k_{12}	k_7
Tension	0.9	1	1	-	-	-
Compression	0.9	1	1	-	1	-
Shear	0.9	1	1	-	-	-
Bending	0.9	1	1	1	1	-
Bearing	-	-	-	-	-	1

k12 calculation - stability factor

Cl 8.4.7

r	0.25
pb	1.08
S1	0

Check of continuous lateral restraint

LHS	0.99
RHS	1.68

Continuous lateral restraint system

k1 modification factors

Table G1

Load type	Combination	k_1	Comment
Permanent	1.35G	0.57	50+ years
Long-term	1.2G+1.5Ψ_lQ	0.8	5 months
Short-term	1.2G+1.5Q	0.94	5 days

A.1.5 Flexural design capacities

Flexural design capacities for the various loading conditions were checked using the *WoodSolutions Technical Design Guide 31: Timber Cassette Floors*. As shown in the utilisation table, the design is well below capacity.

Flexural design capacities

Combination	Bending		Axial		Shear	Bearing	
	Above centroid	*Below centroid*	*Top flange*	*Bottom flange*	*web*	*top flange*	*Bottom flange*
	Mdtop	Mdbot	Ndtop	Ndbot	Vd	Nptop	Nptop
	kNm	kNm	kN	kN	kN	kN	kN
1.35G	460.1	265.1	2140.4	N/A	347.0	625.9	N/A
1.2G+1.5Ψ_lQ	645.7	372.0	3004.1	N/A	487.0	878.4	N/A
1.2G+1.5Q	758.7	437.1	3529.9	N/A	572.3	1032.1	N/A

Utilisation

Combination	Bending		Axial		Bending and Compression		Bending and tension		Shear	Bearing
	Above centroid	*Below centroid*	*Top flange*	*Bottom flange*	*About xx*	*About yy*	*Eq 3.5(3)*	*Eq 3.5(4)*	*web*	*flange*
1.35G	0.11	0.18	0.07	N/A	0.18	N/A	N/A	N/A	0.06	0.03
1.2G+1.5Ψ_lQ	0.10	0.18	0.07	N/A	0.17	N/A	N/A	N/A	0.06	0.03
1.2G+1.5Q	0.13	0.23	0.09	N/A	0.22	N/A	N/A	N/A	0.08	0.04

A.1.6 Serviceability – deflection

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

Minimum required EI_{ef} for short-term and long-term deflection was less than the EI_{ef} of the section (3.659×10^{13} *Nmm*^{2}) and therefore satisfied the requirement. Deflection limits for short-term and long-term deflection are span/300 and span/400, respectively. This has been determined in accordance with Guidelines presented in Appendix B of AS 1720.1.

Serviceability - Deflection

**Short term and long term EI should be less than EI effective*

		Short term	Long term	1kN point load
EI_{ef}	*Nmm2*	1.75E+13	2.88E+13	
Δ mm		14.4	11.8	0.4
Δlimit mm		30	22.5	2
CHECK		OK	OK	OK

j2 factor - creep

Loading	j2
Instantaneous live load	1
Long-term loads in a controlled environment	2
Long-term loads in a variable environment	3

A.1.7 Serviceability – vibration design

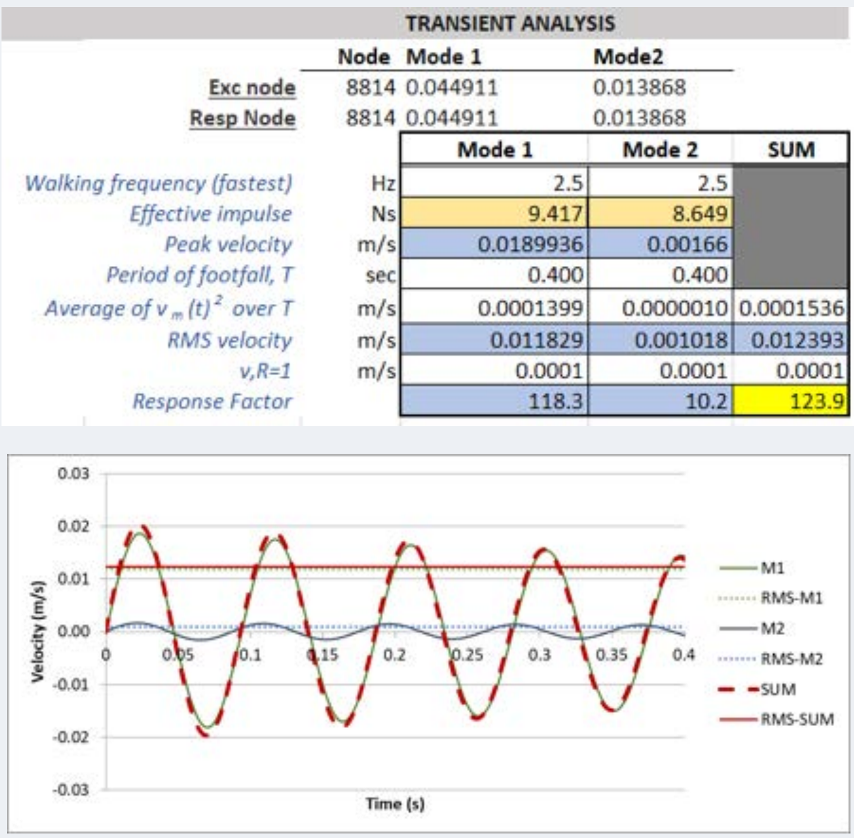
Vibration performance was calculated using procedures in CCIP-016, SCI P354 and DG 11 to highlight differences between guides and assessed based on the Response Factor. Comparison of predicted to measured results of a ribbed deck cassette with dimensions as shown in Figure A-1 have been presented. The support condition is assumed to be the overhanging portion of the flange secured to the primary beam at 40 mm from the edge and is numerically represented as a pin-pin condition. The damping ratio is taken as 1%. Although in a design situation, the loading would include 10% live load as well as any other superimposed dead load, the loading condition for this assessment only considered the self-weight of the floor. With the aforementioned details, a finite element model was created with a mesh size of 20 mm. The modal properties of the first two modes obtained from a modal analysis are shown in Table 20 (other modes were greater than 30 Hz). Mode shape amplitudes for both modes were also extracted. Excitation and response locations were taken at mid-span along the centre joist. From the mode shapes shown in Figure 6(a) and (b), this means that there should be minimal contribution from mode 2. The fundamental frequency is 10.67 Hz which is above the cut-off frequency of all guides and thus a transient analysis assuming a 76 kg person has been undertaken.

Table 20: Measured modal properties from impact hammer testing

	Mode 1 (Bending 1)	**Mode 2 (Torsion 1)**
f_n Hz	10.67	11.48
m kg	464.3	87.6
ζ,%	1	1

A.1.7.1 CCIP-016

For CCIP-016, the cut-off frequency between low and high frequency floors is determined using the formula 4.2 ×maximum walking frequency. The maximum walking frequency has been taken as 2.5 Hz (as recommended in CCIP-016) which results in a cut-off frequency is 10.5Hz. As such, this section follows the transient analysis procedure. The guide states that it is only necessary to check the response based on the maximum expected walking frequency since faster walking speeds induce greater responses. The modal superposition method is used to calculate the total response from both modes. As shown below, the predicted response factor considering the contribution of both modes is 124. The predicted velocity response for the period of one footstep (0.4 s) is shown in Figure A-2 for both modes where ‘Mx’ refers to Mode ‘x’. The root-mean-square (RMS) values of each mode and total response from both modes (SUM) are also shown.



TRANSIENT ANALYSIS

	Node	Mode 1	Mode2
Exc node	8814	0.044911	0.013868
Resp Node	8814	0.044911	0.013868

		Mode 1	Mode 2	SUM
Walking frequency (fastest)	Hz	2.5	2.5	
Effective impulse	Ns	9.417	8.649	
Peak velocity	m/s	0.0189936	0.00166	
Period of footfall, T	sec	0.400	0.400	
Average of $v_m(t)^2$ over T	m/s	0.0001399	0.0000010	0.0001536
RMS velocity	m/s	0.011829	0.001018	0.012393
v,R=1	m/s	0.0001	0.0001	0.0001
Response Factor		118.3	10.2	123.9

Figure A-2: Predicted velocity response for one footstep based on CCIP-016 procedure.

A.1.7.2 SCI P354

The cut-off frequency recommended for general floors and open plan offices is 10 Hz and subsequently a transient analysis according to the 'general assessment' procedure (Section 6 of SCI P354) is followed. The effective impulse equation provided in SCI P354 is expressed in conjunction with requirements provided in EN 1990 Annex C (Gulvanessian 2001; Smith, Hicks & Devine 2009). As a result for a 76 kg person the impulse applied is approx 18% higher than the design effective impulse in CCIP-016 and therefore a higher predicted response is expected. Despite SCI P354 recommending a maximum design pace frequency of 2.2 Hz, a pace frequency of 2.5 Hz is selected to compare with CCIP-016 results. As shown below, the predicted response factors for both modes is 199. The predicted acceleration response for one footstep as per SCI P354 approach is shown in Figure A-3.

TRANSIENT ANALYSIS

	Node	Mode 1	Mode2	
Exc node	8814	0.044911	0.013868	
Resp Node	8814	0.044911	0.013868	
		Mode 1	**Mode 2**	**SUM**
Wn (Wb weighting)		1	1	
Walking frequency	Hz	2.5	2.5	
F_i	Ns	11.437	10.505	
Peak Acceleration	m/s2	1.522	0.142	
Period of footfall, T	sec	0.400	0.400	
Average of a,w(t)^2	m/s	0.898	0.008	0.992
RMS accel	m/s2	0.948	0.087	0.996
accel,R=1	m/s2	0.005	0.005	0.005
Response Factor		189.6	17.4	199.2

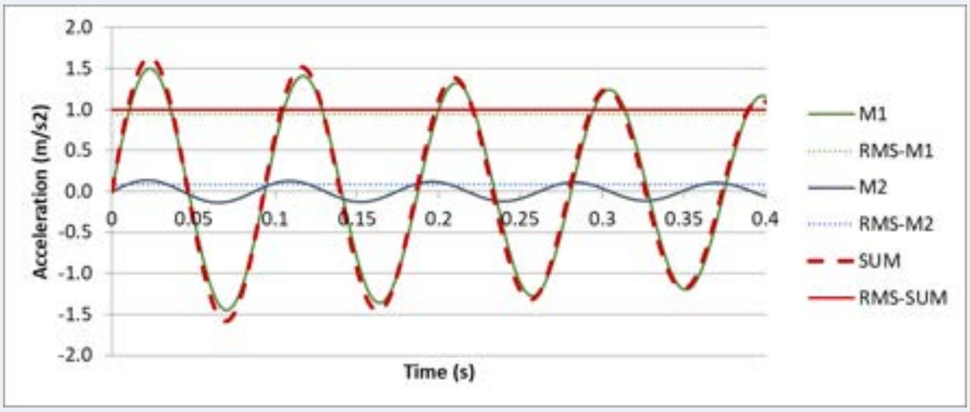


Figure A-3: Predicted acceleration response for one footstep based on SCI P354 procedure.

A.1.7.3 AISC DG 11

In AISC DG11 (2016), the cut-off frequency is taken as 9 Hz which means that, similarly to CCIP-016 and SCI P354, the response prediction follows a transient analysis using the finite element method (Section 7 in the guide). In a typical design scenario, the effective impulse would be calculated based on a 5th to 9th integer division of the dominant mode. However, in this case to allow direct comparison to predictions from other guides, a pace frequency of 2.5 Hz was used. As shown below, the total predicted response factor is 130. The predicted acceleration response for one footstep is shown in Figure A-4.

		TRANSIENT ANALYSIS (from Section 7.4)		
	Node	Mode 1	Mode2	
Exc node	8814	0.044911	0.013868	
Resp Node	8814	0.044911	0.013868	
		Mode 1	Mode 2	SUM
Walking frequency	Hz	2.5	2.5	
I_eff	Ns	7.338	6.675	
Peak Acceleration (mass normalised)	m/s2	0.99263	0.09260	
Period of footfall, T	sec	0.4	0.4	
Average of a,w(t)²	m/s	0.3821861	0.0032127	0.4221163
RMS accel	m/s2	0.618212	0.056680	0.649705
Accel_ESPA	m/s2	0.874283828	0.080158	0.9188213
	%g	8.91%	0.82%	9.37%
accel,R=1	m/s2	0.005	0.005	0.005
Response Factor		123.6	11.3	129.9

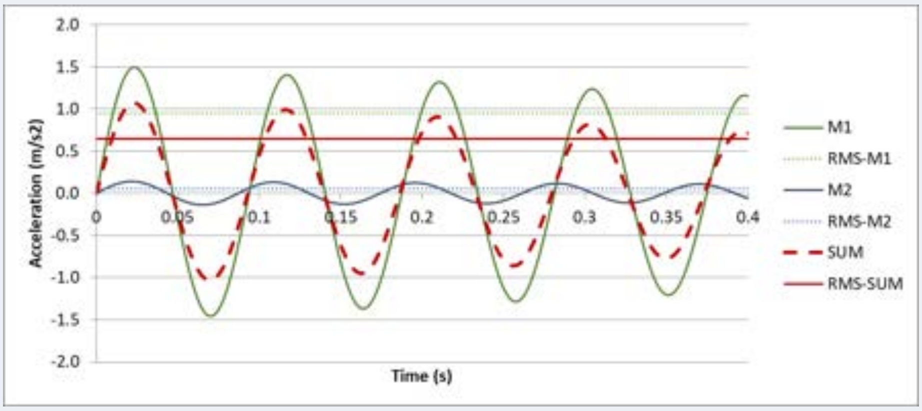


Figure A-4: Predicted acceleration response for one footstep based on AISC DG11 procedure.

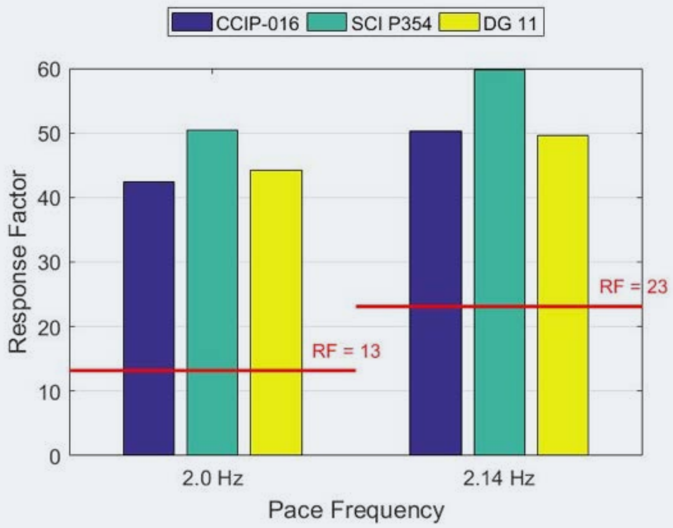


Figure A-5: Comparison of predicted Response Factors based on CCIP-016, SCI P354 and DG 11.

B Appendix B. Worked example for a Cross-laminated Timber Floor

B.1 Floor Properties

This section gives the geometry and material properties of a CLT floor that is used to demonstrate the design procedures required for a CLT floor.

The floor consists of three 2.2 m wide panels spanning 5.8 m, making a 5.8 x 6.6 m^2 floor plate. The example calculations will discuss the design for the panel supported by primary beams in a one-way span arrangement and also supported on all four sides, thereby spanning two-ways. The floor geometry is summarised as:

Floor span, L = 5.8 m
Panel width, w = 2.2 m
Panel thicknesst_{tot} = 175 mm
Number of panels, n = 3

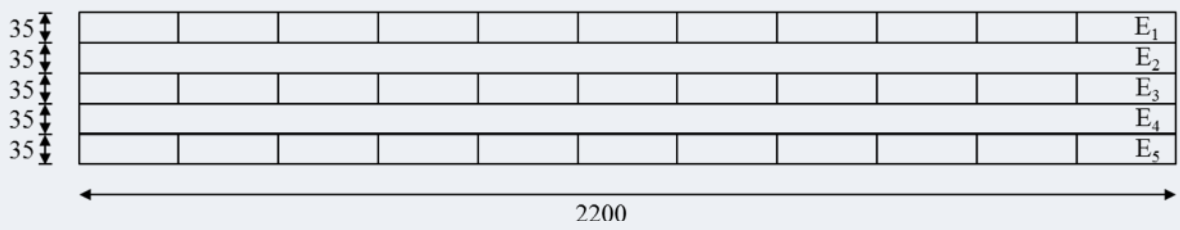


Figure B-1: Cross section geometry of 5 layered CLT panel.

The material used to compose the panel is New Zealand Radiata Pine. The cross-section shown in Figure B-1 is made up of five equal thickness layers of timber. The characteristic strength properties required to complete the design process in this section is included in Table 21.

Table 21: Material properties of CLT panel for example calculation.

Layer number	Thickness (mm)	Elastic Modulus (MPa)	Bending Strength (MPa)	Tension Strength (MPa)
1	35	8000	14	6

The density is 460 kg/m^3.

B.2 Strength Design

This design guideline covers several methods and numerous calculations are involved for each method. This section contains the results from the example floor and demonstrates the abilities of each method.

Table 22: Material properties of CLT panel for example calculation.

Property	CLT Designer	Gamma	Composite	Shear Analogy
$Z_{eff,1m}$ mm^3	4.04 x 10^6	3.99 x 10^6	4.06 x 10^6	4.09 x 10^6
Bending Moment, kNm	36.6	36.2	36.8	37.1
$A_{eff,1m}$ mm^2	136 x 10^3 (mid) 144 x 10^3 (rolling)	133 x 10^3 (mid) 189 x 10^3 (rolling)		
Shear strength, kN	55	72	-	-
Bearing Strength, kN	637	-	-	-

B.2.1 CLT Designer

Using Equation 4.4 to calculate the floors bending stiffness, we find:

$$K_{CLT} = (2\times3.573\times10^6\times8000 + 3.573\times10^6\times8000) + (2\times8000\times35000\times70^2)$$
$$= 2.83\times10^{12} Nmm^2$$

And therefore, the section modulus:

$$Z_{CLT} = \frac{2\times2.83\times10^{12}}{175\times8000} = 4.04\times10^6\ mm^3$$

The bending strength is given by:

$$Z_{CLT} = \frac{2\times2.83\times10^{12}}{175\times8000} = 4.04\times10^6\ mm^3$$

Using the equation for moment capacity in accordance with AS 1720.1 the ultimate moment capacity for the CLT is calculated under permanent loads:

$$M_d = 0.95\times0.57\times1.33\times12.6\times4.04\times10^6 = 36.6\ kNm$$

The shear stress requires calculation of the effective area using (4.13) for the mid-span and (4.14) for the rolling shear layers:

$$A_{eff,mid} = \frac{2.83\times10^{12}}{\left(8000\times35\times70 + \frac{8000\times35^2}{8}\right)} = 136\times10^3\ mm^2$$

$$A_{eff,rolling} = \frac{2.83\times10^{12}}{8000\times35\times70} = 144\times10^3\ mm^2$$

The shear strength at mid-section, $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are 3.0 N/mm^2 and 0.7 N/mm respectively (Unterwieser & Schickhofer, 2014).

The shear strength is then the minimum of the shear at mid-span or the shear at the transverse layers:

$$V_{mid} = 0.95\times0.57\times3\times136\times10^3 = 221\ kN$$

$$V_{rolling} = 0.95\times0.57\times0.7\times144\times10^3 = 55\ kN$$

The bearing area is a line load along the support of the CLT panel with a support area of 125 mm x 2200 mm. Therefore, the bearing capacity is:

$N_{d,p}$ =0.95×0.57× 1.5 ×2.85 ×125×2200= 637 kN

B.2.2 Gamma Method

Using Equation 4.16 we find the gamma reduction value:

$$\gamma_1 = \frac{1}{1+\pi^2\frac{8000\times35000}{5800^2}\cdot\frac{35}{50\times1000}} = 0.95$$

Where the rolling shear modulus is taken as 50 MPa.

$\gamma_1 = \gamma_3$ = 0.95 and γ_2 = 1

$z_1 = z_3$ = 70 mm and z_2 = 0

And the effective stiffness is found to be:

$$EI_{eff} = 2.68\times10^{12}\ Nmm^2$$

$$Z_\gamma = \frac{(EI)_{eff}}{E_1(\gamma_1 z_1 + 0.5t_1)} = \frac{2.68\times10^{12}}{8000(0.95\times70 + 0.5\times35)} = 3.99\times10^6\ \text{mm}^3$$

Finally the bending moment capacity is calculated in accordance with AS 1720.1:

$$M_d = 0.95\times0.57\times1.33\times12.6\times3.99\times10^6 = 36.2\ \text{kNm}$$

The effective shear area is calculated using:

$$A_{eff,mid} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1 z_1 + E_1' A_1' z_1' + \gamma_2 E_2 \frac{A_2}{2}\frac{t_2}{4}\right)} = \frac{2.68\times10^{12}\times1000}{\left(0.95\times8000\times35000\times70+267\times35000\times35+1\times8000\times\frac{35000\times35}{2\quad 4}\right)} = 133\times10^3\ \text{mm}^2$$

E_1' is taken as $E_1/30$ = 267 MPa

z_1' = 35 is the distance from centroid of rolling shear layer and the centroid of the cross-section.

$$A_{eff,rolling} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1\left(z_1 - \frac{t_2}{2}\right) + E_1' A_1'\left(z_1' - \frac{t_2}{2}\right)\right)} = \frac{2.68\times10^{12}\times1000}{\left(0.95\times8000\times35000\left(70-\frac{35}{2}\right)+267\times35000\left(35-\frac{35}{2}\right)\right)} = 189\times10^3\ \text{mm}^2$$

The shear strength is then the minimum of the shear at mid-span or the shear at the transverse layers:

$$V_{mid} = 0.95\times0.57\times3\times133\times10^3 = 216\ \text{kN}$$

$$V_{rolling} = 0.95\times0.57\times0.7\times189\times10^3 = 72\ \text{kN}$$

B.2.3 Composite K-Method

The composite factor is calculated:

$$k_1 = 1 - \left[\left(1 - \frac{200}{8000}\right)\left(\frac{105^3 - 35^3}{175^3}\right)\right] = 0.797$$

$$EI_{eff} = 0.797\times8000\times\frac{1000\times175^3}{12} = 2.85\times10^{12} mm^4$$

$$Z_k = 0.797\times\frac{1000\times175^2}{6} = 4.06\times10^6\ \text{mm}^3$$

$$M_d = 0.95\times0.57\times1.33\times12.6\times4.06\times10^6 = 36.8\ \text{kNm}$$

B.2.4 Shear Analogy Method

The elastic modulus, shear modulus and rolling shear modulus for this calculation are taken as:

$E_{90} = E_0/30 = 8000/30 = 267$ MPa
$G_0 = E_0/16 = 8000/16 = 500$ MPa
$G_{90} = G_0/10 = 500/10 = 50$ MPa

$$B_A = 2{\times}8000{\times}3.573{\times}10^6 + 8000{\times}3.573{\times}10^6 + 2{\times}267{\times}3.573{\times}10^6 = 87.66{\times}10^9 \text{ Nmm}^2$$

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2 = 2.767{\times}10^{12} \text{ Nmm}^2$$

$$EI_{eff} = B_A + B_B = (0.08 + 2.767){\times}10^{12} = 2.86{\times}10^{12} \text{ Nmm}^2$$

To calculate the distance a:

$$a = t_{total} - \frac{t_1}{2} - \frac{t_n}{2} = 175 - \frac{35}{2} - \frac{35}{2} = 140 \text{ mm}$$

$$GA_{eff} = \frac{140^2}{\left[\left(\frac{35}{2{\times}500{\times}1000}\right) + \left(\frac{35}{50{\times}1000} + \frac{35}{500{\times}1000} + \frac{35}{50{\times}1000}\right) + \left(\frac{35}{2{\times}500{\times}1000}\right)\right]} = 12.7{\times}10^6 N$$

A simplified method that is used in the CLT Handbook USA is then used to calculate the section modulus:

$$Z_{SAM} = \frac{2{\times}2.86{\times}10^{12}}{8000{\times}175} = 4.09{\times}10^6 \text{ mm}^3$$

$$M_d = 0.95{\times}0.57{\times}1.33{\times}12.6{\times}4.09{\times}10^6 = 37.1 \text{ kNm}$$

B.3 Serviceability

B.3.1 Short-term deflection

The deflection is straight forward to calculate once the effective stiffness has been calculated. The effective stiffness for the Gamma, Composite and Shear analogy method are detailed in B.2. This calculation does not consider load combinations for design. When calculating the deflections for a design scenario the appropriate load case factors in accordance with AS 1170 and AS 1720 should be considered. Only the self-weight of the floor is considered in this example. A comparison of the deflection results for short-term and long-term deflection are included in Table 23.

The density is 460 kg/m^3. Therefore:

w = 460 x 5.8 x 1 =

$$w = 460{\times}0.175{\times}1{\times}\frac{9.81}{1000} = 0.8 \; kN/m$$

Gamma method

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} = \frac{5{\times}0.8{\times}5800^4}{384{\times}2.68{\times}10^{12}} = 4.4 \text{ mm}$$

Composite method

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1E_0I_{gross}} = \frac{5{\times}0.8{\times}5800^4}{384{\times}2.85{\times}10^{12}} = 4.14 \text{ mm}$$

Shear analogy method

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} = \frac{5{\times}0.8{\times}5800^4}{384{\times}2.86{\times}10^{12}} + \frac{0.8{\times}5800^2{\times}1.2}{8{\times}12.7{\times}10^6} = (4.12 + 0.32) = 4.44 \text{ mm}$$

Table 23: Deflection results comparison of a 2 m wide, 5.8 m long CLT panel.

Property	Gamma	Composite	Shear Analogy
EI_{eff} Nmm^2/m	2.68×10^{12}	2.85×10^{12}	2.86×10^{12}
GA_{eff} N	-	-	12.7×10^{6}
Δ_s mm	4.4	4.14	4.44
Δ_l mm	9.24	8.7	9.32

B.3.2 Long-term deflection

A service class 2 structure is considered and therefore a k_{def} = 1.1 is used to calculate the long-term deflections.

Gamma method

$$\Delta_{mid,\gamma}=\frac{5wl^4}{384(EI)_{eff}}=\frac{5\times0.8\times5800^4}{384\times2.68\times10^{12}/(1+1.1)}= 9.24 \text{ mm}$$

Composite method

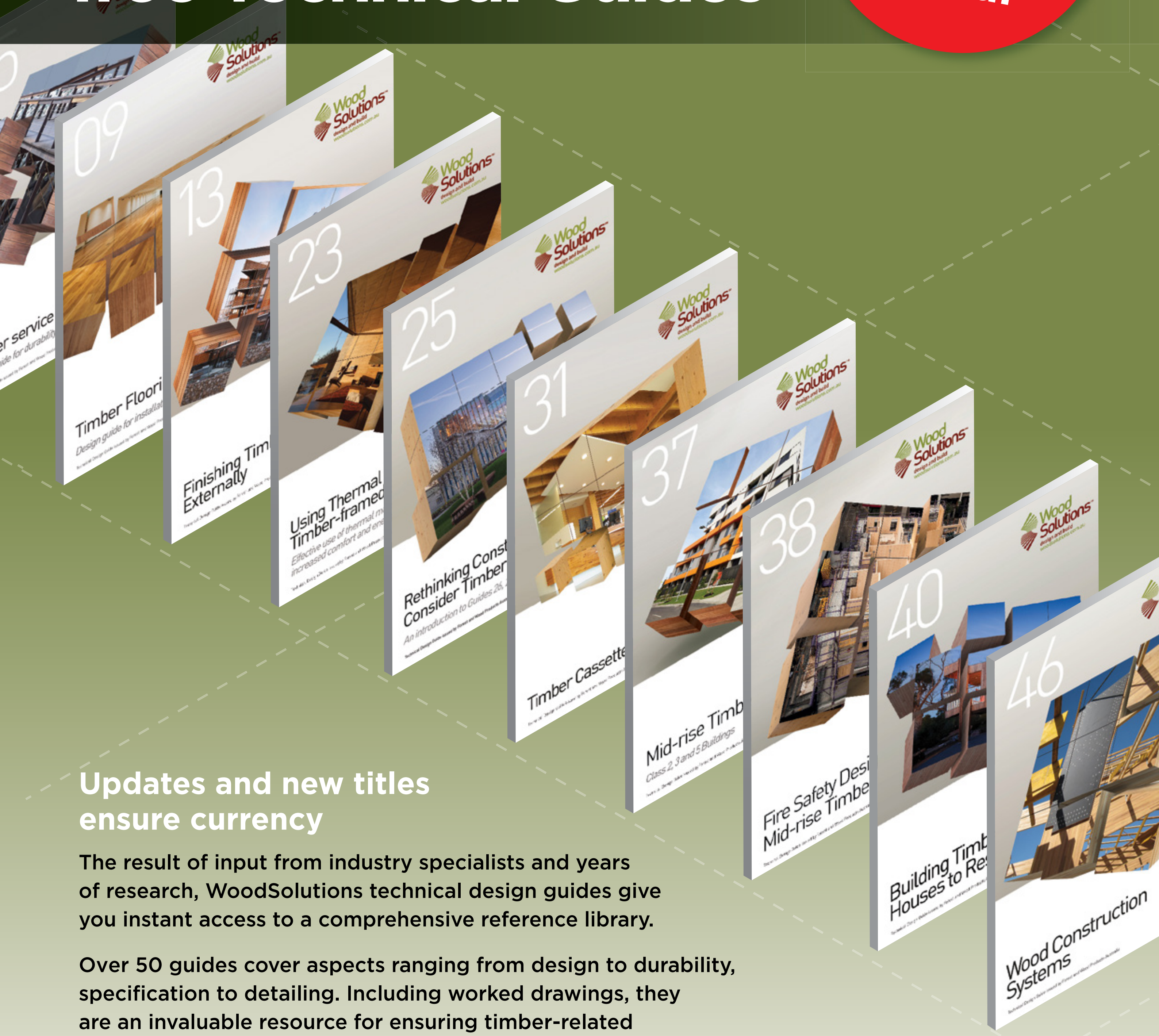
$$\Delta_{mid,k}=\frac{5wl^4}{384k_1E_0I_{gross}}=\frac{5\times0.8\times5800^4}{384\times2.85\times10^{12}/(1+1.1)}= 8.70 \text{ mm}$$

Shear analogy method

$$\Delta_{mid,SAM}=\frac{5wl^4}{384(EI)_{eff}}+\frac{wl^2k}{8(GA)_{eff}}=\frac{5\times0.8\times5800^4}{384\times2.86\times10^{12}/(1+1.1)}+\frac{0.8\times5800^2\times1.2}{8\times12.7\times10^6/(1+1.1)}=(8.65+0.67)=9.32 \text{ mm}$$

B.3.3 Vibration design

Worked solution for the floor is covered in section 4.5.



50 A1

WoodSolutions™
design and build
woodsolutions.com.au

This guide was published in Dec 2019
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Mid-rise Timber Building Structural Engineering

Appendix 1: Worked Example for a Timber-framed Apartment Building

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Williams Landing
Figurehead Constructions

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-74-4

Researchers:
Geoff Boughton & Jon Shanks
TimberEd Services Pty Ltd

First Published: December 2019

Contents

Introduction: Worked Example Scope - Lightweight Timber Framed Mid-Rise Structure

The building utilised for this worked example is the WoodSolutions mid-rise model apartment building. It was designed to provide a basis for defining, comparing and presenting different mid-rise timber-based construction solutions and it is utilised in a range of WoodSolutions mid-rise resources, where it provides a prototypical situation for modelling spatial, loading, fire and noise resistance conditions. WoodSolutions has referred to this building in several design guides. More information on the mid-rise model apartment building can be found in *Design Guide #27 Rethinking Apartment Building Construction*. In this Appendix it is used to illustrate some of the structural analysis methods and approaches that may be adopted in the design of a lightweight timber-framed mid-rise structure.

The worked examples:

- Provide guidance to the structural engineer to assist in understanding some of the key steps required in the design of timber mid-rise buildings but are not a full set of structural computations that will be required in a real project.
- Discuss key decision points and assumptions. The calculation steps are provided with a narrative to highlight the key steps. Simple calculation steps are not presented, nor are detailed steps copied from design standards.
- Are based on one set of engineering assumptions that may or may not be valid for a particular design.
- Present one possible approach to the engineering design. It is not intended that all the calculations necessary in design are presented here, or that the methods are the only valid methods.
- Make simplifying assumptions on architecture. The engineering design has not been completed as part of an iterative design process, and so broad assumptions on what is feasible have been made.

Had this worked example been completed as a real project, an iterative design process would have been utilised, and feedback to the client and design team would have been provided in order to inform discussion, e.g. on construction time, a particular facade finish, deflection criteria from the lift manufacturer, acoustic or thermal efficiency requirements beyond the code compliance.

As far as reasonably practical, hand-based methods of analysis have been used. Reference is made to computer analysis where appropriate. It is the intention of the worked examples to present the thought behind the process, that is best highlighted through hand calculations.

The worked examples in Appendices 1 & 2 present element design for both lightweight timber-framed construction and CLT construction respectively. The calculations of the different element types follow a similar format and so some cross-referencing is possible. But, the systems are designed to different loads and assumptions and so cannot be easily swapped like-for-like.

Typical timber-framing

Overview of the Design Process - Lightweight Timber Framed Mid-Rise Structure

As discussed in Section 1.2.2, while design of a building in practice is an iterative process, it does generally follow three phases and design steps. This worked example illustrates computations for the steps identified below.

1

Phase 1: Preliminary Design

Step 1: Building layout and performance considerations – Early Contractor Involvement (ECI):

Project team to discuss and determine:

- overall building design and layout
- performance requirements for: structure, fire, acoustics, robustness, etc

Step 2: Preliminary structural design – Early Supplier Involvement (ESI)

Engineer to provide preliminary advice on:

- the structural approach used
- structural element layout (roof trusses, floors, walls, cores, bracing walls, etc)
- initial member sizing information for 'preliminary costings' to be developed
- non-timber elements: concrete basement, transfer structures, etc

Also consider as part of ESI process: the construction sequence; limits of transportation, lifting and manual handling; site restrictions; temporary bracing and propping required; building services and voids required.

2

Phase 2: Detailed Design

Based on decisions agreed in the preliminary design phase, the actual design loadings can be determined. The major structural design steps are:

Step 3: Vertical load – Roof and floor cassette design

Step 4: Vertical load – Wall design

Step 5: Vertical movement design

Step 6: Lateral load – Stability design

Step 7: Check robustness

Step 8: Other details for consideration

Step 9: Engineering drawings and documentation for certification

3

Phase 3: Fabrication & Assembly

Step 10: Engineered timber systems fabrication (shop drawing review)

Step 11: On-site construction assembly supervision - certification

WoodSolutions Mid-rise Model Apartment - Building Overview

The WoodSolutions mid-rise model apartment building was designed to meet high-end consumer needs, including large and open room layouts. An emphasis was placed on characterising a building that could apply to many suburban/urban apartment situations across Australia. The model apartment typical floor plan is shown Figure A1.1, the building has a breadth of 34m and a depth of 22.5m, providing a 765 m² floor plate per level. Visualisations of the model apartment are provided in Figure A1.2. A section through the model apartment are provided in Figure A1.3. A summary of the design approaches utilised including relevance and reasons is provided in Table A1.1.

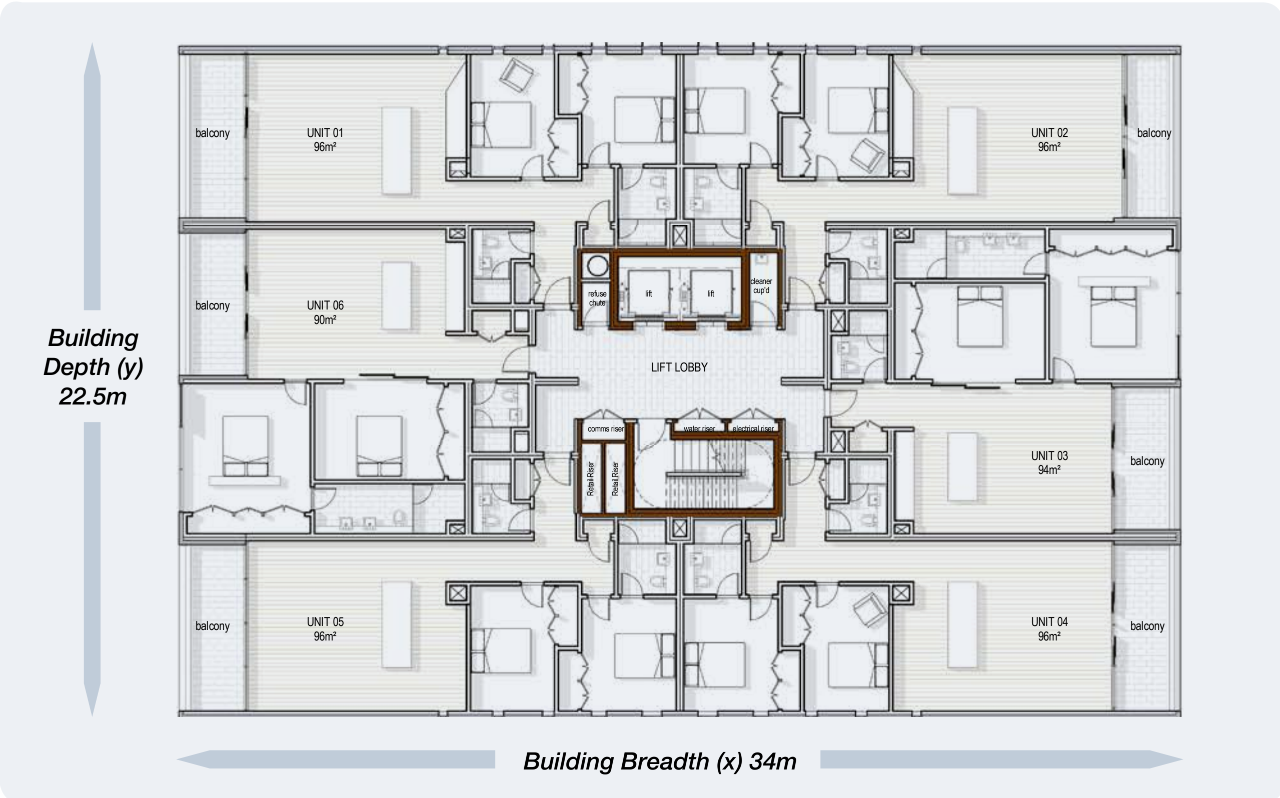


Figure A1.1 – WS model apartment, typical floor plan.

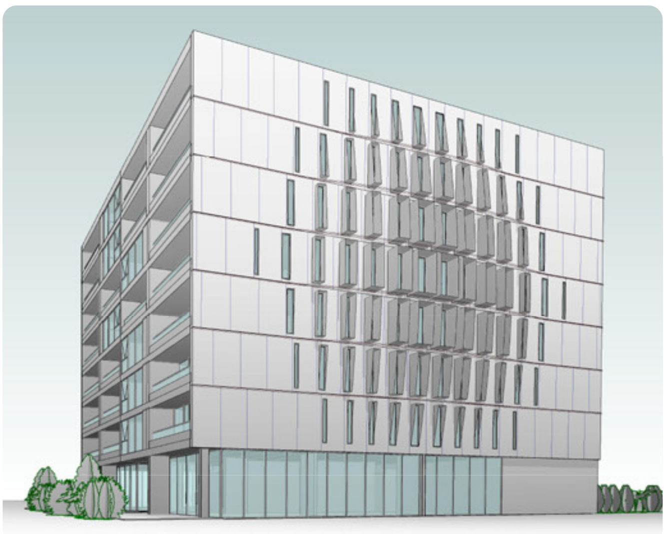


Figure A1.2: WS model apartment visualisations.

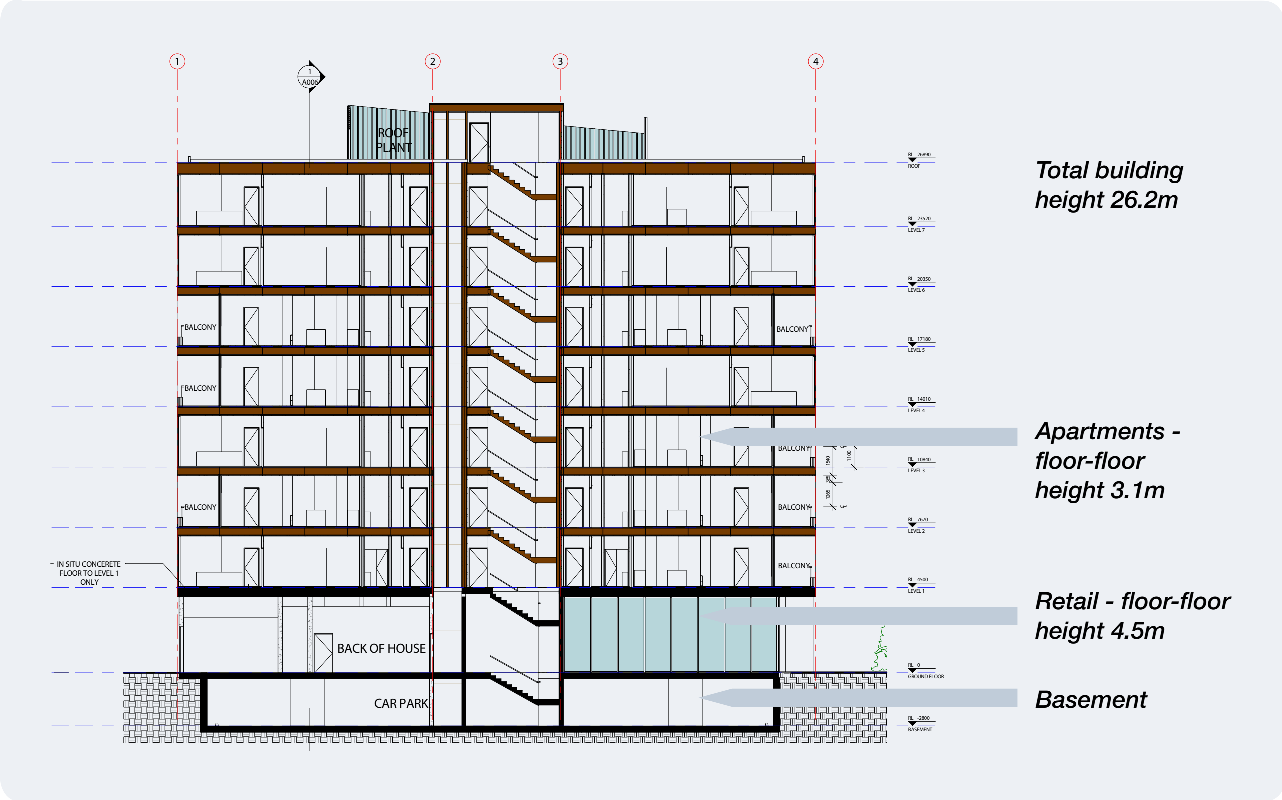


Figure A1.3: WS model apartment section

Table A1.0: Summary of design approaches utilised including relevance and reasons.

Item	Design Approach	Relevance and Reasons
Height	• 8-storey design height above ground level, including 7 apartment levels and 1 retail level. • A 26.2 m overall building height with an NCC effective height of 23.1 m. • A 3.1 m floor to floor height for the apartment levels and 4.5 m floor to floor height for the retail level.	• The apartment levels provide a habitable height plus depth for the structure and services within minimum NCC limits. • The retail level provides for a maximum depth 500 mm thick transfer slab above, i.e. used to transition loads from the timber to concrete parts of the building.
Area	• A floor plate area of 765 m^2. The apartment levels include 42 SOUs (94–96 m^2 each). • The retail level assumes three shops varying in area from 77–150 m^2. It also includes a foyer area, an entrance to basement car parking, utility meter rooms, an electrical substation and a waste area.	• Feedback and analysis indicate that many suburban mid-rise apartment buildings fit this scenario.
Key set out criteria	• Plant room, lift shaft or machinery room, public corridor, public lobby.	• The width of the building accommodates the size and set-out of the large, high-end apartments. • The grid layout accommodates car parking in the basement.
Building ownership and fire separation	• The building is considered to be strata titled, including the retail area on the ground floor.	• The building is considered to be strata titled, including the retail area on the ground floor.
Setbacks	• External wall distances are (at minimum) less than 1.5 m from the property boundary.	• External wall distances are (at minimum) less than 1.5 m from the property boundary.

Phase 1 Preliminary Design

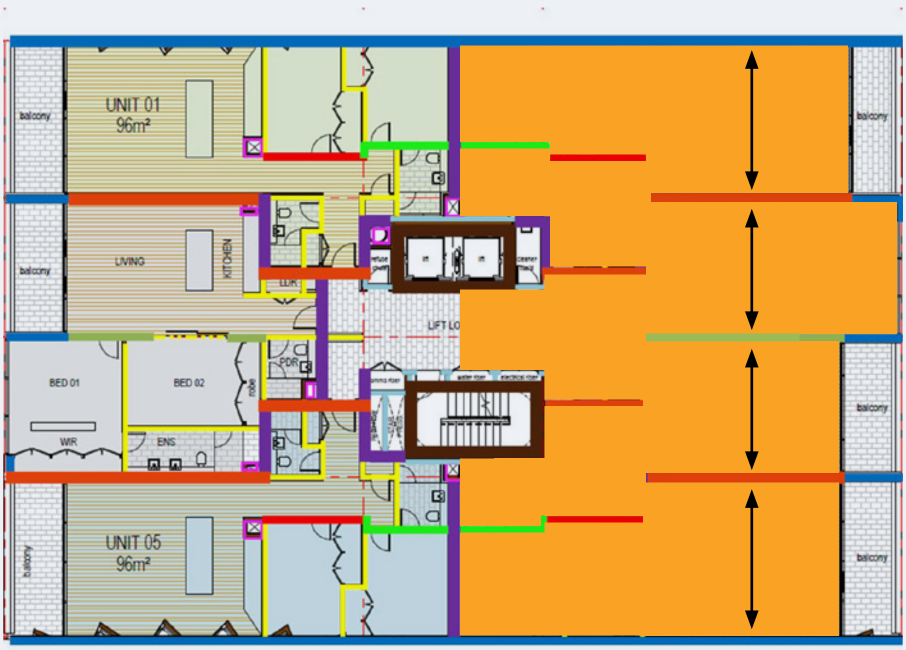
Step 1: Building layout and performance considerations – Early Contractor Involvement (ECI)

The architect determines the massing, orientation, floor plan, façade design and building area functions, see Figures A1.1 – A1.3 and Table A1.0 for general specifications and details.

The architect and design team (including acoustic and fire consultants) have determined the following:

- FRL: Class 2 building, 26.2 m, 8-storeys ➔ Type A construction, therefore, requires an FRL of 90/90/90; therefore
  - walls to be lined with 2 layers of 13 mm fire-rated plasterboard and
  - ceilings to be lined with 2 layers of 16 mm fire-rated plasterboard
- Acoustic Performance:
  - Walls: Airborne, $R_w+C_{tr} \geq 50$, between apartments use double leaf discontinuous walls (see Figure A1.4)

Step 2: Preliminary structural design – Early Supplier Involvement (ESI)



Floor elements to span between apartment walls, assume max span 6.0m

Loadbearing walls as indicated on plan and in table

Wall Location	Wall Number & Colour Code	Description
External	1	Single stud load-bearing fire-rated wall
Apartment bounding Walls	2	Double stud load-bearing fire and acoustic rated wall
	3	Double stud non load-bearing fire and acoustic rated wall
	6	Single stud load-bearing fire-rated wall
	7	Single stud non load-bearing fire-rated wall
	8	Single stud non load-bearing wall
Shafts	9	Lift and stair shaft
	10	Service shaft

Figure A1.4 – Floor plan showing proposed timber framed wall types and floor panel layout.

Acoustic Performance

Floors: Airborne, $R_w + C_{tr} \geq 50$ (apartments), Impact $L_{n,w} \leq 55$ (AAAC 3 Star rating)

Proposed floor/ceiling system build up:

- 12 mm Hardwood overlay flooring
- Mass overlay, 2x18mm Promat 40kg/m^2 (see Table 3.2)
- 10 mm acoustic mat
- 22 mm plywood flooring
- 400 mm deep lightweight floor trusses (approx. 6.0 m span)
- 75 mm Glasswool insulation
- Resilient mounts (40 mm)
- 2x16 mm fire-rated plasterboard
- 135 mm suspended ceiling
- 13 mm standard plasterboard

Total depth of floor = 700 mm (502 mm no suspended ceiling)

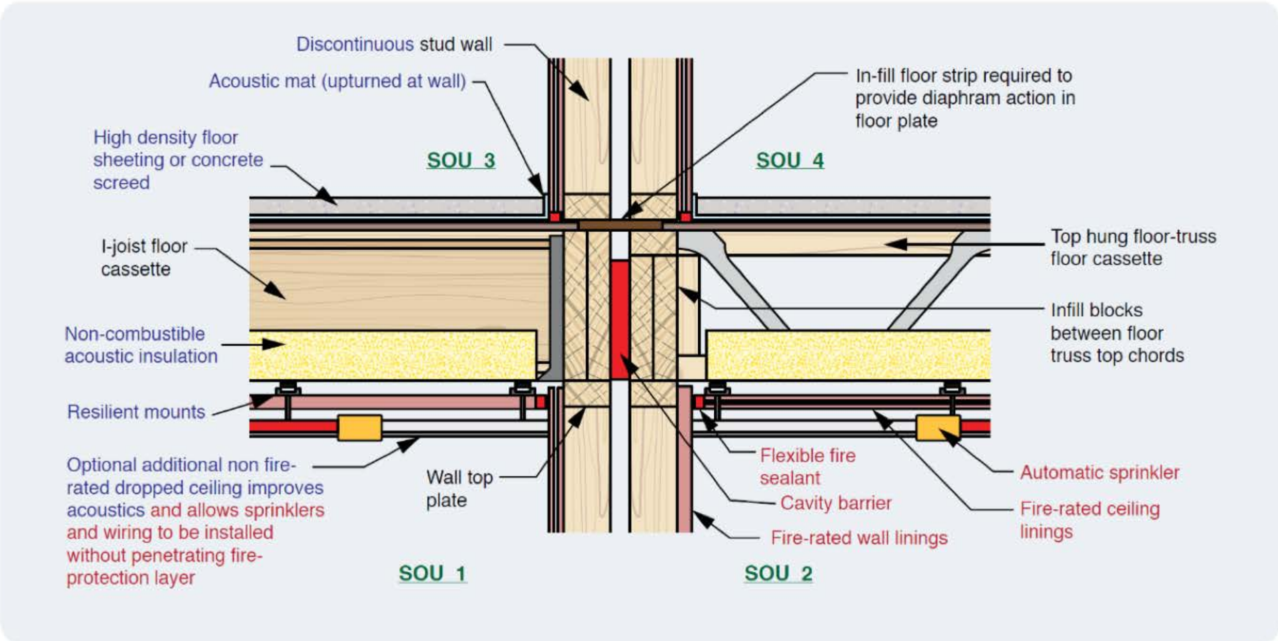


Figure A1.5: Floor build-up.

Preliminary floor member sizing

The walls between apartments are required to be discontinuous for acoustics and will be used as loadbearing walls. The floor elements will span between the apartment walls, assume a max span 6.0 m, prefabricated floor cassette systems will be utilised.

Preliminary sizing of floor cassettes can be obtained from:

- frame and truss manufacturers
- an approximate span to depth ratio estimate for structure (see section 3.4.2) for a floor truss: span: depth = 16-18, therefore 6,000/18 = 400 mm; or
- tables in *Design Guide 46, Guide to Wood Construction Systems* (see Figure A1.6). for a floor live load of 2.0kPa, 400 mm deep nail plated floor trusses at 450mm crs will span just over 6.0m simply supported.

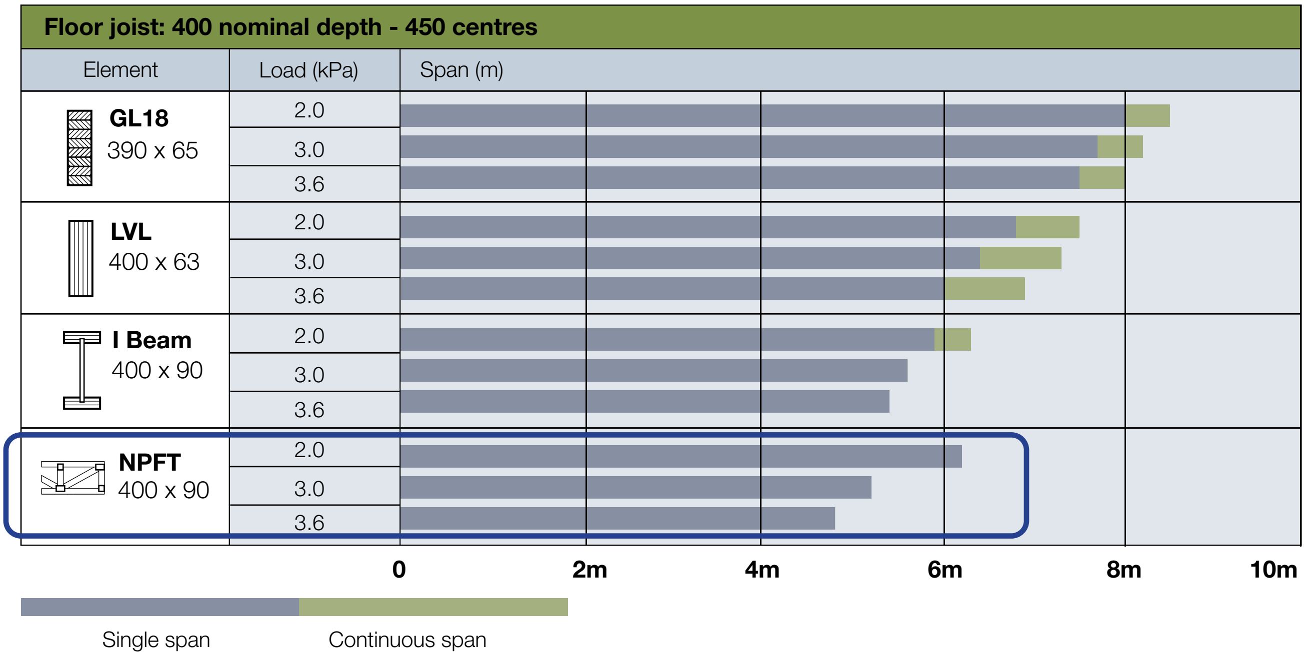


Figure A1.6: Floor joist size estimate using WoodSolutions Design Guide #46 (Table 10).

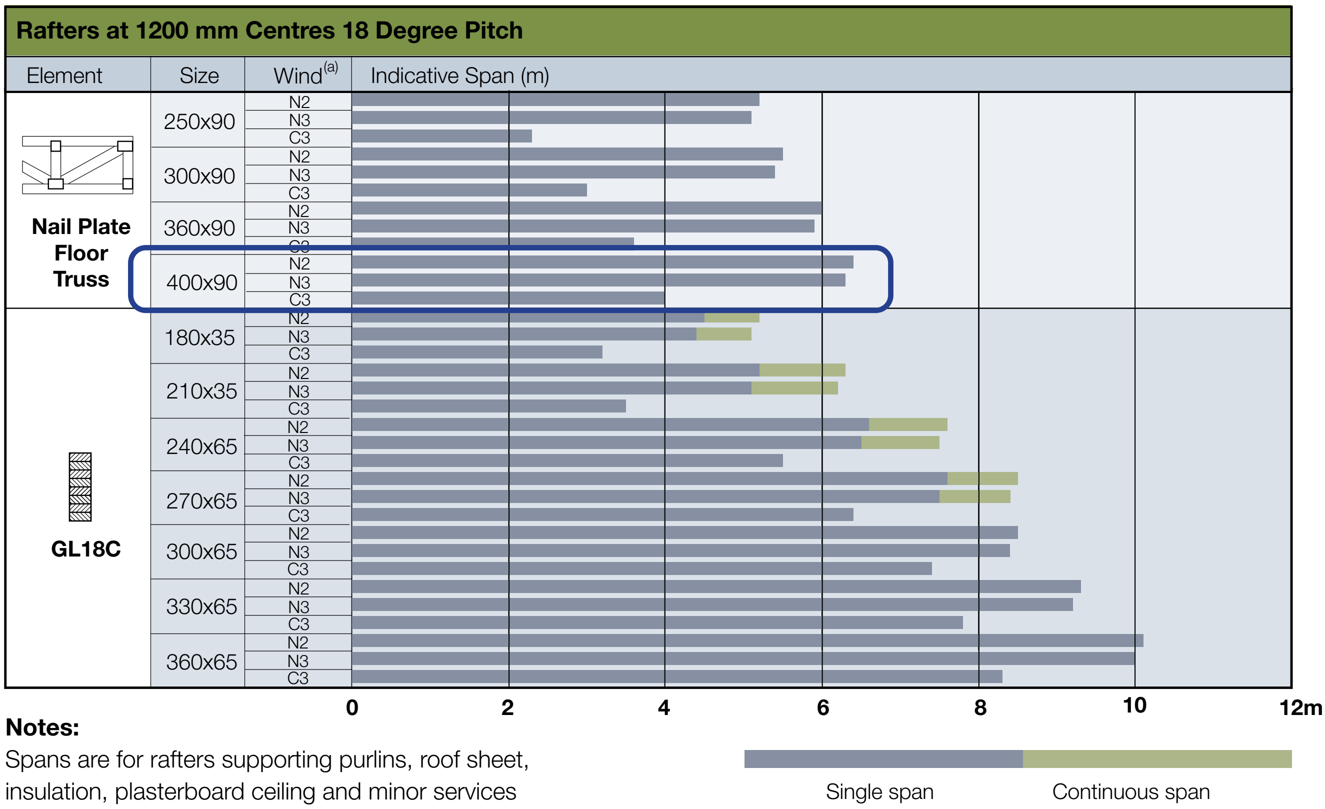
Preliminary wall sizing

Assume walls will be constructed using 90mm deep studs at 450 mm crs – may be single, double, triple or quadruple studs depending on level in building and framing material stress grade utilised.

Preliminary roof sizing

Assume roof will be constructed using nail plated trusses at 1,200 mm crs supported on the same loadbearing walls as the floor systems, so assume a max span 6.0m. Preliminary sizing of roof rafters can be obtained from

- frame & truss manufacturers; or
- tables in *Design Guide 46, Guide to Wood Construction Systems* (see Figure A1.7). for an N3 wind zone a 400 mm deep nail plated roof trusses at 1,200 mm crs will span over 6.0m simply supported for a Class 1 building for example.



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

Figure A1.7: Indicative span – Rafters at 1200 crs WoodSolutions Design Guide #46 (Table 15).

Determine Loadings for Different Structural Elements

Walls Loads 3.1 m high

- 4 x 13mm Fire rated plasterboard		0.52 kPa
- Insulation		0.02 kPa
- 2 x 90x45 studs staggered 450 cts		0.1 kPa
- Wall plates and noggins		0.05 kPa
	Total	**0.7 kPa**
		2.14 kN/m

Floor Loads

- Hardwood flooring		0.16 kPa
- 2x9mm fibre cement, 10mm acoustic mat		0.40 kPa
- 22mm particleboard		0.17 kPa
- 190mm joists @ 450mm crs inc blocking etc		0.20 kPa
- False ceiling structure and services*		0.40 kPa
- 75mm glasswool batts (14 kg/m^3)		0.02 kPa
- 2x16mm fire rated plasterboard		0.35 kPa

* assuming separated ceiling and services

Total	**1.70 kPa**

Wind Loading

Note that wind loading calculations are simplified for the purpose of the example calculations.

1) Calculate site wind speed — ***AS1170.2, CL2.2***

$$V_{sit,B} = V_R M_d (M_{z,cat} M_s M_t)$$

V_r	45	Region A3
M_d	1.0	
$M_{z,cat}$	1.0	
M_s	1.0	
M_t	1.0	
V_{sit}	45	m/s

2) Determine the site wind pressure — ***AS1170.2, CL2.4***

$$p = 0.5 p_{air} V_{des}^2 C_{fig} C_{dyn}$$

p_{air}	1.2	kg/m^3
C_{dyn}	1.0	
p	1215 $\times C_{fig}$	

3) Determine aerodynamic shape factor — ***AS1170.2, CL5.2***

$$C_{fig,ext} = C_{p,e} K_a K_c K_l K_p$$

$$C_{fig,int} = C_{pi} K_{c,i}$$

Windward Wall	
$C_{p,e}$	0.7
K_a	1
K_c	0.9
K_l	1
K_p	1

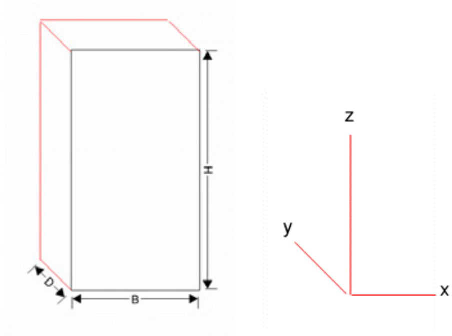
Leeward	
$C_{p,e}$	-0.5
K_a	1
K_c	0.9
K_l	1
K_p	1

Internal	
$C_{p,e}$	-0.3
K_{ci}	1

$C_{fig,\,ext}$	0.63		$C_{fig,\,ext}$	-0.45		$C_{fig\ int}$	0	or -0.3
p	0.77	kPa		-0.55	kPa		0	kPa
p net	**1.31**	**kPa**				***p wall***	**1.13**	**kPa**
						for wall out of plane bending		

1) Area of load, per floor

Tributary breadth	34	m
Floor to floor height, hi	3.1	m
Building height (timber)	25	m
Breadth (b)	34	m
Depth (d)	22.5	m
Trib Area (x) $= hi \times B$		
Area / floor (wind x)	69.75	m^2
Area / floor (wind y)	105.4	m^2



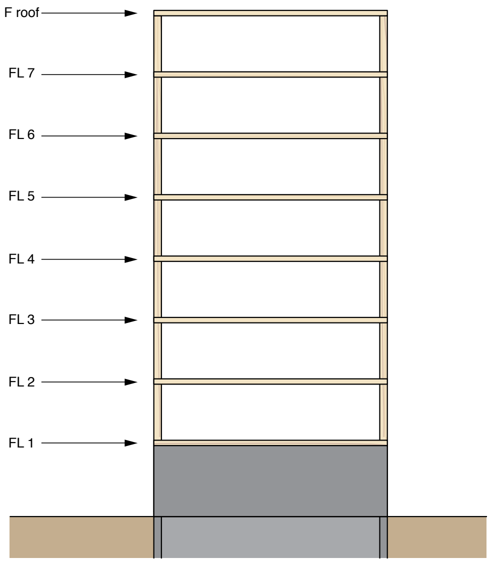
2) Calculate Wind loads for Each Floor

FL	A m^2 (x)	A m^2 (y)	F_{wi} x (kN)	F_{wi} y (kN)	Sum F_{wi} y (kN)
R	34.9	52.7	45.8	69.2	69.2
7	69.8	105.4	91.5	138.3	207.5
6	69.8	105.4	91.5	138.3	345.8
5	69.8	105.4	91.5	138.3	484.1
4	69.8	105.4	91.5	138.3	622.4
3	69.8	105.4	91.5	138.3	760.7
2	69.8	105.4	91.5	138.3	899.0
1	34.9	52.7	45.8	69.2	968.1
Total			**640.7**	**968.1**	**968.1**

**Assuming rigid diaphragm*

at interface between timber and concrete

R	69 kN
7	138 kN
6	138 kN
5	138 kN
4	138 kN
3	138 kN
2	138 kN
1	69 kN



Global overturning moment

W x width x height x height/2

13,942 kNm

Earthquake loads determined to AS 1170.4.
Importance level 2 structure, between 12 and 50m high requires Earthquake Design Category of II. ***AS1170.4, Table 2.1***

1) Calculate fundamental period of building ***AS1170.4, CL6.2.3***

$$T_1 = 1.25\, k_t h_n^{0.75}$$

k_t	0.05	All other structures
h_n	21.7	m for timber-framed design this is assumed to be from the top of the concrete. i.e. the concrete is significantly stiffer than the timber-framed portion.
T_1	0.63	s

2) Calculate horizontal static shear force ***AS1170.4, CL6.2***

$$V = \left(\frac{K_p Z C_h(T_1) S_p}{\mu}\right) W_t$$

k_p	1	1/500 years
Z	0.08	Melbourne
$C_h(T_1)$	1.81	
S_p	0.77	Braced frames (with ductile connections) ***AS1170.4, Table 6.5(A)***
μ	2	
V	0.056	xW_t

3) Calculate seismic weight for each floor ***AS1170.4, CL6.2.2***

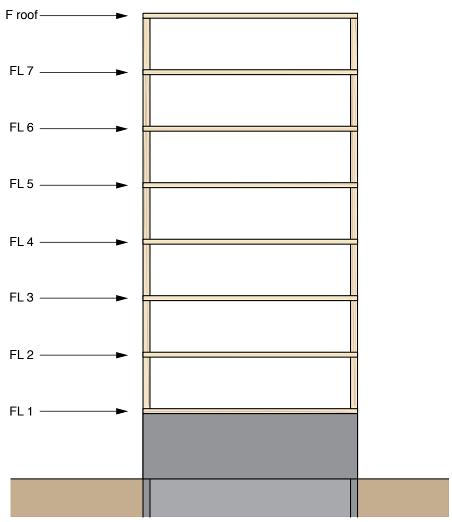
$$w_i = \sum G_i + \sum \omega_c Q$$

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n}(W_j h_j^k)}\left[k_p Z C_h(T_1)\frac{S_p}{\mu}\right] \times W_t$$

w_c	0.3	
h_i	3.1	m
A	756	m^2

	w_i (kN)	h_i (m)	w_i x h_i	w_ih_i/w_jh_j	F_i (kN)	sum F_i (kN)
R	1,399	22	30,350	22%	196	196
7	1,625	19	30,232	22%	196	392
6	1,625	16	25,194	19%	163	555
5	1,625	12	20,155	15%	130	686
4	1,625	9	15,116	11%	98	783
3	1,625	6	10,077	7%	65	849
2	1,625	3	5,039	4%	33	881
L1 -Tran	4,631	0	0	0%	0	
Total	**15,782**		**136,163**		**881**	

R	69 kN
7	138 kN
6	138 kN
5	138 kN
4	138 kN
3	138 kN
2	138 kN
1	69 kN



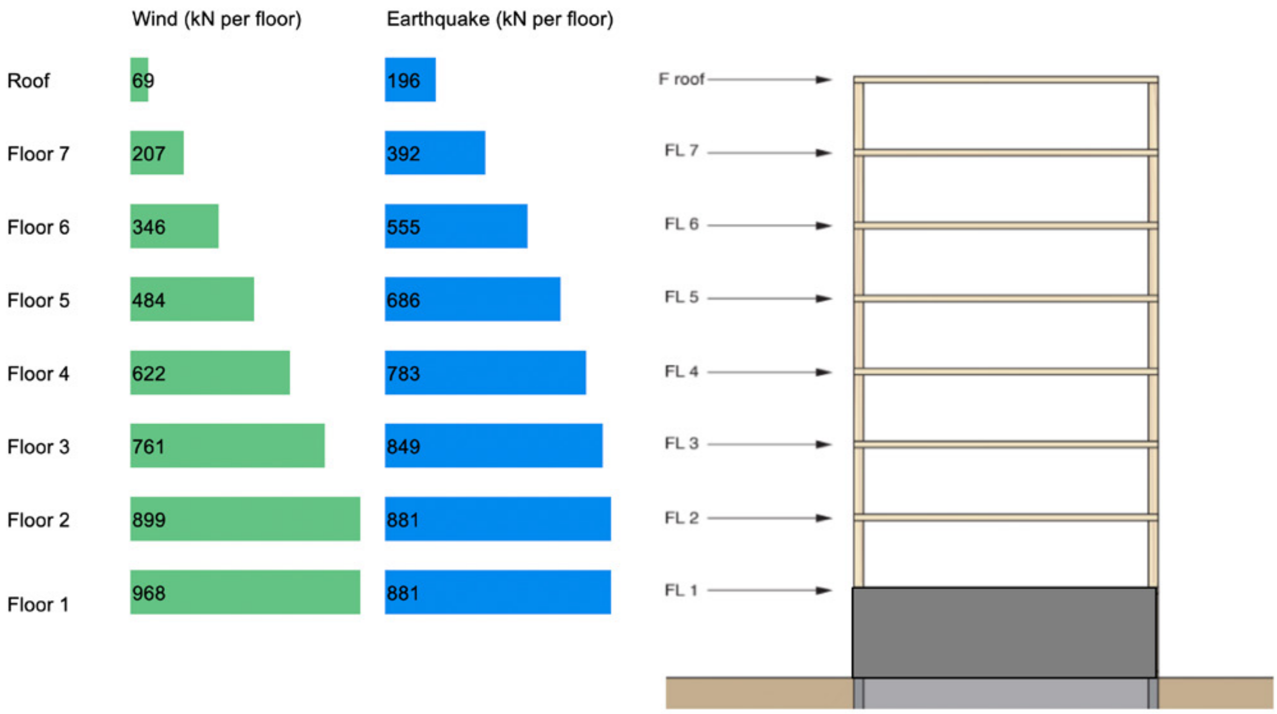
GTF
1.70 kPa

Qroof
0.5 kPa

Qapt
1.5 kPa

Gconc
14.75 kPa

Load distribution through the building height



Step 3: Vertical load – Roof and floor design

- **Detailed roof design - *not included in Worked Example*** - straight forward for all engineers.
- **Detailed floor design - *not included in Worked Example*** - final detailed design of floor cassette members will be undertaken by the frame and truss manufacturer supplying to the project in consultation with the project engineer.

Step 4: Vertical load – Wall design

- This example illustrates the design of a wall carrying a 6 m long cassette (as shown highlighted in Figure A1.8).
  - Internal wall is double stud wall with a cavity so each stud is only loaded by a single floor cassette (but diaphragm action is assumed to be maintained).
  - The Floor Load Width (FLW) is half of the cassette span. i.e. FLW = 3.0 m.

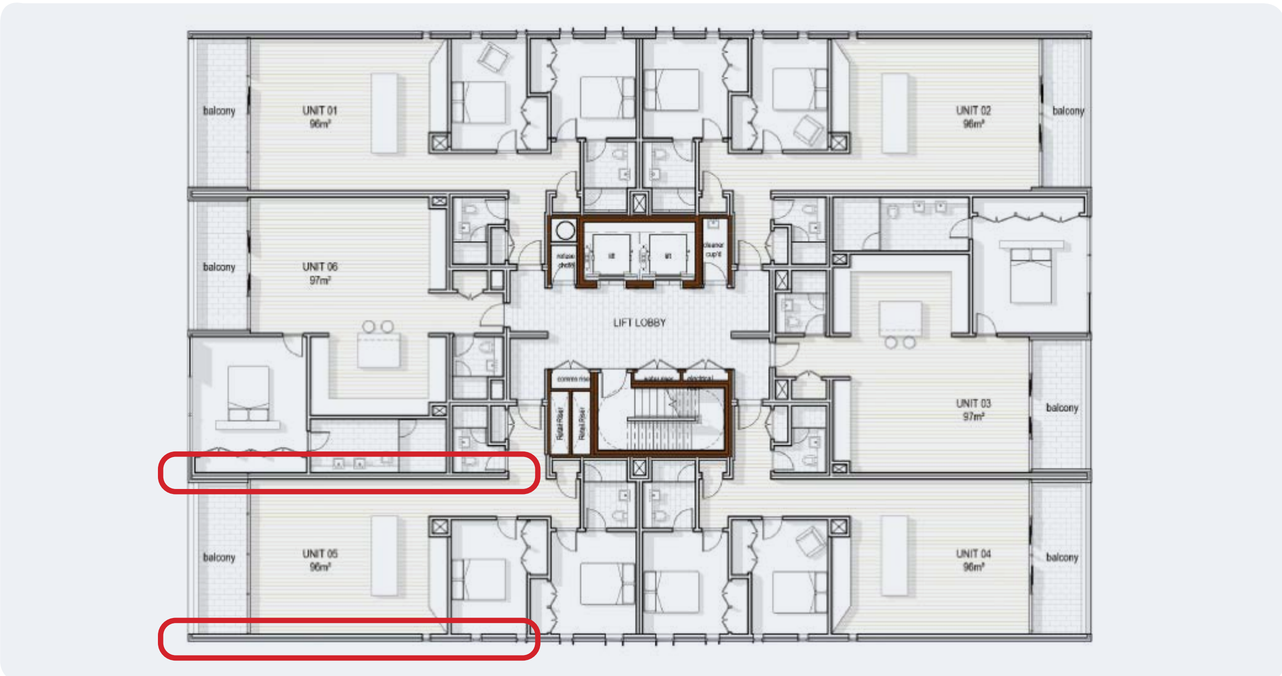


Figure A1.8: Floor plan with design walls highlighted.

The structural (load bearing) walls will all be framed with 90x45 studs and 90x35 top and bottom plates. On each level, studs and plates will be sourced from the same grade of timber to simplify construction, but grade may vary from level to level. Engineers may determine that a more efficient design can be achieved by varying grade between studs and plates.

To keep the prefabrication of the frames as simple as possible, all similar load-bearing walls at a level will have the same specification, and to enable studs to align with studs in both the wall above and the wall below, all walls will have the same maximum stud spacing, 450 mm. This will also be the joist spacing for the floor cassettes, so the studs can also align with the floor joists in order to minimise bending in the top and bottom plates of the walls.

The floor cassettes will have a structural depth of 420 mm (assumed from reasonable span/depth):

Stud length = 3100 – 420 – (2 x 35) = 2610 mm.

Stud loads

Stud gravity loads

The total dead load at each level is the dead load from the floor plus the dead load from all the walls that are supported by the floor including the weight of non-load bearing walls. As the non-load bearing walls are not continuous to the floor above, the weight of the non-load bearing walls is transferred to the floor below them and then to the load-bearing walls that support that floor.

The flat roof will be constructed like a floor and bear directly on the load bearing walls in Level FL7. It has been treated as a floor but with a heavier dead load and a lighter live load.

Tributary area of a stud = stud spacing x FLW = 0.45 m x 3.0 m = 1.35 m^2 per storey.

Maximum tributary area for a stud = 6 x 1.35 = 8.1 m^2 (the lowest studs in the building support six floors).

The accumulated tributary area on a storey FL1 stud is not enough to enable the ψa factor in AS/NZS 1170.1 Cl 3.4.2, to be applied to the live loads on the floor (less than 14 m^2).

The dead (permanent) loads include:

- the roof system weight – estimated at 2.1 kPa
- the weight of each floor system – estimated at 1.7 kPa allowing for topping and tiles as floor covering
- the total weight of all walls that are supported by the floor – estimated at 0.93 kPa distributed over the floor area (including load bearing and non-load bearing walls).

Figure A1.2 shows that the dead load on the level FL7 load bearing walls is the weight of the roof. Each lower storey wall receives load from an additional floor and the walls that bear on it.

The live (imposed) loads include:

- floor live loads within apartments of 1.5 kPa (AS/NZS 1170.1 Table 3.1 ref A1)
- balconies greater than 1 m above ground and accessed only through an apartment 2.0 kPa (AS/NZS 1170.1 Table 3.1 ref A1)
- floor live loads in public corridors and lobbies of 4 kPa (AS/NZS 1170.1 Table 3.1 ref A2)
- none of the floor cassettes with a 6 m span will carry live loads from public areas, so the live load on floors used in the stud design will be 1.5 kPa inside and 2.0 kPa outside.

There is no provision for the roof to have a floor function, so only the construction and maintenance live load will be applied.

The roof live load per stud is given by the roof tributary area per stud on Level FL7:

A = 1.35 m^2

Roof live load per stud = 1.8/1.35 + 0.12 = 1.45 kPa (AS/NZS 1170.1 Table 3.2 ref R2)
Hence roof live load per stud (Q) = 1.45 x 1.35 = 1.96 kN

As the roof does not have a floor function $\psi_l = 0$ (AS/NZS 1170.0 Table 4.1)
For all floors $\psi_l = 0.4$ (AS/NZS 1170.0 Table 4.1)

Table A1.1: Design gravity loads on studs.

			Normal service 1.2G+1.5Q, $k_1 = 0.8$		**Long-term loads 1.2G+1.5ψ_lQ $k_1 = 0.57$**		**Fire limit states G+ψ_lQ $k_1 = 0.97$**	
Wall	G	Q	Per level	***Nc****	Per level	***Nc****	Per level	***Nc****
	kN/stud		kN/stud		kN/stud		kN/stud	
Storey FL7	2.84	1.96	6.35	**6.3**	3.40	**3.4**	2.84	**2.8**
Storey FL6	3.55	2.03	7.30	**13.6**	5.48	**8.9**	4.36	**7.2**
Storey FL5	3.55	2.03	7.30	**21.0**	5.48	**14.4**	4.36	**11.6**
Storey FL4	3.55	2.03	7.30	**28.3**	5.48	**19.8**	4.36	**15.9**
Storey FL3	3.55	2.03	7.30	**35.6**	5.48	**25.3**	4.36	**20.3**
Storey FL2	3.55	2.03	7.30	**42.9**	5.48	**30.8**	4.36	**24.7**
Storey FL1	3.55	2.03	7.30	**50.2**	5.48	**36.3**	4.36	**29.0**

Stud wind loads for external walls

The studs on the outside of the building can also be subjected to out-of-plane wind loads from the combination of external and internal pressures. Wind load determined is 1.13 kPa and is the same for all levels in the building.

The load per m of stud = 0.45 x 1.13 = 0.51 kN/m

The bending moment per stud is therefore:

All levels with stud height of 2.61 m, $M_6 = \frac{0.51 \times 2.61^2}{8} =$ 0.43 kNm

(As the construction is platform framing, the studs are concentrically loaded, and the gravity loads from floors do not introduce any eccentricity and hence bending moment.

Had the construction system been a semi-balloon system, each floor load would have introduced a bending moment for that level caused by the eccentricity of the floor load bearing on a ledger attached to the side of the studs, which would need to be considered in the stud design.

Stud design

The studs could be from the following commonly available grades: MGP10, MGP12 or LVL.

The design properties for these grades are given in Table A1.2. These values were taken from AS 1720.1 for MGP grades and from a manufacturer for LVL. Capacity factor for secondary elements = ϕ = 0.9 sawn timber; and ϕ = 0.95 for LVL (AS 1720.1 Table 2.1).

Table A1.2: Design properties for studs.

Grade	*E* (MPa)	f'_c (MPa)	ρ_c	f'_b (MPa)	ρ_b	ϕ
MGP10	10000	18	0.96	**17**	0.75	**0.9**
MGP12	12700	24	0.98	**28**	0.85	**0.9**
LVL	13200	47	1.27	**37**	0.95	**0.95**

Note: The ρ_c values were taken from either AS 1720.1 Table 3.3 or calculated using AS1720.1 equation E2(3) with *r* = 0.25 (rather than the incorrect equation 8(3) in AS 1720.1–2010), and ρ_b values were taken from either AS 1720.1 Table 3.1 or calculated using AS1720.1 equation E2(1) with *r* = 0.25.

The design of studs is usually limited by major axis buckling, so calculate the design capacity for major axis buckling and then check design capacity for minor axis buckling.

Stud capacity for major axis buckling $N_{d,cx}$

Calculate Design capacity $N_{d,cx}$ (for buckling only about the major axis) using:

$$N_{d,cx} = \phi\, k_1 k_4 k_6 k_{12,x} f'_c A_c \qquad \text{AS 1720.1 equation 3.3(2)}$$

with

ϕ from Table A1.2

k_1 = 0.8 for design loads from column 5 of Table A1.1 and k_1 = 0.57 or 1 for design loads from column 7 of Table A1.1 (AS 1720.1 Table 2.3)

k_4 = 1 for seasoned timber used indoors (AS 1720.1 Cl. 2.4.2.3)

k_6 = 1 for studs within the building envelope in the cooler part of Australia (AS 1720.1 Cl. 2.4.3)

k_{12x} calculated from S_3 (AS 1720.1 equation 3.3(11))

S_3 calculated using AS 1720.1 equation 3.3(6) with g_{13} = 0.85

L = length of stud = 2.61 m

f'_c from Table A1.2

A_c = number of studs in group *n x d x b* with *d* = 90 mm and *b* = 45 mm

These calculations are summarised in Table A1.3 with the table laid out roughly in material cost order (i.e. cheapest studs at the top and most expensive at the bottom.) The first column 'n' refers to the number of studs nail laminated together.

Table A1.3: $N_{d,cx}$ for 90 x 45 studs with k_1 = 0.8 and L = 2.61 m.

n	Grade	ϕ	ρ_c	S_3	ρ_c S_3	k_{12}	f'_c (MPa)	A_c (mm^2)	$N_{d,cx}$ (kN) (k_1 = 0.8)
1	MGP10	0.9	0.96	24.7	23.6	0.36	18	4050	18.8
1	MGP12	0.9	0.98	24.7	24.1	0.34	24	4050	24.1
1	LVL	0.95	1.27	24.7	31.2	0.21	47	4050	29.8
2	MGP10	0.9	0.96	24.7	23.6	0.36	18	8100	37.7
2	MGP12	0.9	0.98	24.7	24.1	0.34	24	8100	48.3
2	LVL	0.95	1.27	24.7	31.2	0.21	47	8100	59.5
3	MGP10	0.9	0.96	24.7	23.6	0.36	18	12150	56.5
3	MGP12	0.9	0.98	24.7	24.1	0.34	24	12150	72.4
3	LVL	0.95	1.27	24.7	31.2	0.21	47	12150	89.3

These calculations are repeated to give design capacities for k_1 = 0.57 and k_1 = 1.0 and the values are presented in Table A1.4. Presenting the table in this way helps with design checks later in the calculation.

Table A1.4: $N_{d,cx}$ for 90 x 45 studs with k_1 = 0.8, k_1 = 0.57, k_1 = 1.0 and L = 2.61 m.

					k_1 = 0.8		k_1 = 0.57		k_1 = 1.0	
n	Grade	k_{12}	f'_c	A_c (mm^2)	k_1	$N_{d,cx}$ (kN)	k_1	$N_{d,cx}$ (kN)	k_1	$N_{d,cx}$ (kN)
1	MGP10	0.36	18	4050	0.8	18.8	0.57	13.4	1.00	23.5
1	MGP12	0.34	24	4050	0.8	24.1	0.57	17.2	1.00	30.2
1	LVL	0.21	47	4050	0.8	29.8	0.57	21.2	1.00	37.2
2	MGP10	0.36	18	8100	0.8	37.7	0.57	26.8	1.00	47.1
2	MGP12	0.34	24	8100	0.8	48.3	0.57	34.4	1.00	60.3
2	LVL	0.21	47	8100	0.8	59.5	0.57	42.4	1.00	74.4
3	MGP10	0.36	18	12150	0.8	56.5	0.57	40.3	1.00	70.6
3	MGP12	0.34	24	12150	0.8	72.4	0.57	51.6	1.00	90.5
3	LVL	0.21	47	12150	0.8	89.3	0.57	63.6	1.00	111.6

The gravity loads on the studs presented in Table A1.1 can be compared with the capacities in Table A1.4 to select the studs for the design as shown in Table A1.5.

Table A1.5: Selection of 90 x 45 studs to match $N_{d,cx}$ to design loads for L = 2.61 m.

	k_1 = 0.8			k_1 = 0.57		
Wall	N_c* (kN)	Design stud	$N_{d,cx}$ (kN)	N_c* (kN)	Design stud	$N_{d,cx}$ (kN)
Storey FL7	6.3	1 MGP10	18.8	3.4	1 MGP10	13.4
Storey FL6	13.6	1 MGP12	24.1	8.9	1 MGP12	17.2
Storey FL5	21.0	1 MGP12	24.1	8.9	1 MGP12	17.2
Storey FL4	28.3	2 MGP10	37.7	19.8	2 MGP10	26.8
Storey FL3	35.6	2 MGP12	48.3	25.3	2 MGP12	34.4
Storey FL2	42.9	2 MGP12	48.3	30.8	2 MGP12	34.4
Storey FL1	50.2	2 LVL	59.5	36.3	2 LVL	42.4

Stud capacity for minor axis buckling $N_{d,cy}$

The capacity of studs in minor axis buckling will be compared against the normal service loads and long-term loads with the fire-rated plasterboard on – fixed at 600 mm centres, and against fire loads and construction loads for fire-rated plasterboard missing, but with one nogging per stud at the mid-height.

With the stud restrained by the fire-rated plasterboard:

$$S_4 = \frac{L_{ay}}{b} = \frac{600}{45} = 13.3$$

AS 1720.1 equation 3.3(8)

This value is considerably less than all of the S_3 values in Table A1.3 so the stud behaviour under normal service conditions is therefore limited by major axis buckling. For this condition, the governing capacity is listed in Table A1.5.

However, $N_{d,cy}$ is required for calculation of combined action effects under wind actions with k_1 = 1, so these capacities are given in Table A1.6.

Table A1.6: $N_{d,cy}$ for 90 x 45 studs with k_1 = 1.0 and L_{ay} = 600 mm

n	*Grade*	ρ_c	S_4	$\rho_c S_4$	k_{12}	f'_c *(MPa)*	A_c *(mm²)*	k_1	$N_{d,cy}$ *(kN)*
1	MGP10	0.96	13.3	12.8	0.86	18	4050	1.0	56.5
1	MGP12	0.98	13.3	13.0	0.85	24	4050	1.0	74.2
1	LVL	1.27	13.3	16.9	0.66	47	4050	1.0	118.7
2	MGP10	0.96	13.3	12.8	0.86	18	8100	1.0	113.1
2	MGP12	0.98	13.3	13.0	0.85	24	8100	1.0	148.5
2	LVL	1.27	13.3	16.9	0.66	47	8100	1.0	237.5
3	MGP10	0.96	13.3	12.8	0.86	18	12150	1.0	169.6
3	MGP12	0.98	13.3	13.0	0.85	24	12150	1.0	222.7
3	LVL	1.27	13.3	16.9	0.66	47	12150	1.0	356.2

These calculations are repeated to give design capacities for k_1 = 0.57 and k_1 = 1.0 and the values are presented in Table A1.4. Presenting the table in this way helps with design checks later in the calculation.

With the stud restrained only by a central nogging

The design capacity of the studs can be evaluated with minor axis restraint offered by a single nogging at mid-height. This scenario simulates the loss of wall cladding:

- under fire loads when the fire-rated plasterboard has fallen away near the end of the fire.
- under construction loads before the plasterboard is fitted.

$$S_4 = \frac{L_{ay}}{b} = \frac{\left({}^{2610}/_{2}\right)}{45} = 29.0$$

AS 1720.1 equation 3.3(8)

with L_{ay} = 2610/2 (half the height of the stud)

This value is a little higher than the values for S_3 shown in Table A1.3, and will therefore limit stud behaviour under the fire limit state and for construction loads before the cladding is fitted. The design capacity with restraint by central noggings is shown in Table A1.7 for:

- 5 hour loads with k_1 = 0.97, which is appropriate for fire loads (AS 1720.4)
- 5 day loads with k_1 = 0.94, which is appropriate for construction loads.

Table A1.7: $N_{d,cy}$ for 90 x 45 studs for fire and construction with L_{ay} = 2610/2 = 1305 mm.

								k_1 = 0.97		k_1 = 0.94	
n	***Grade***	ρ_c	S_4	$\rho_c S_4$	k_{12}	f'_c (MPa)	A_c (mm²)	k_1	$N_{d,cy}$(kN)	k_1	$N_{d,cy}$ (kN)
1	MGP10	0.96	29.0	27.8	0.26	18	4050	0.97	**16.5**	0.94	**16.0**
1	MGP12	0.98	29.0	28.3	0.25	24	4050	0.97	**21.1**	0.94	**20.5**
1	LVL	1.27	29.0	36.7	0.15	47	4050	0.97	**26.1**	0.94	**25.3**
2	MGP10	0.96	29.0	27.8	0.26	18	8100	0.97	**33.0**	0.94	**32.0**
2	MGP12	0.98	29.0	28.3	0.25	24	8100	0.97	**42.3**	0.94	**41.0**
2	LVL	1.27	29.0	36.7	0.15	47	8100	0.97	**52.1**	0.94	**50.5**
3	MGP10	0.96	29.0	27.8	0.26	18	12150	0.97	**49.5**	0.94	**48.0**
3	MGP12	0.98	29.0	28.3	0.25	24	12150	0.97	**63.4**	0.94	**61.5**
3	LVL	1.27	29.0	36.7	0.15	47	12150	0.97	**78.2**	0.94	**75.8**

These stud design capacities can be compared with:

- the fire limit states load on walls shown in Table A1.1
- the long-term loads on the studs – a conservative approximation for construction loads i.e. full dead load but partial live load. This would be the case if the frame was completed and external cladding was applied to walls and roof prior to fitting internal linings, and the internal linings are stacked inside the building at each floor level.

Table A1.8: Comparison of $N_{d,cy}$ for 90 x 45 studs with design loads.

		Fire limit states $G+\psi_l Q$ k_1 = 0.97			**Construction loads** $1.2G+1.5\psi_l Q$ k_1 = 0.94		
Wall	Design stud	Per level	N_c*	$N_{d,cy}$	Per level	N_c*	$N_{d,cy}$
		kN/stud			kN/stud		
Storey FL7	1 MGP10	2.84	2.8	16.5	3.40	3.4	16.0
Storey FL6	1 MGP12	4.36	7.2	21.1	5.48	8.9	20.5
Storey FL5	1 MGP12	4.36	11.6	21.1	5.48	14.4	20.5
Storey FL4	2 MGP10	4.36	15.9	33.0	5.48	19.8	32.0
Storey FL3	2 MGP12	4.36	20.3	42.3	5.48	25.3	41.0
Storey FL2	2 MGP12	4.36	24.7	42.3	5.48	30.8	41.0
Storey FL1	2 LVL	4.36	29.0	52.1	5.48	36.3	50.5

Table A1.8 shows that the design capacity of the studs limited by minor axis buckling and restrained only by the noggings exceeds the stud loads for the fire limit state and the construction loads. (It can also nearly meet the short-term design load presented in Table A1.1)

Combined actions with wind pressure

The combination of ultimate wind bending actions and gravity axial forces on the studs meets the combination $1.2G + \phi_2 Q + W_u$ in AS/NZS 1170.0 Section 4.2.2(d).

The combination must satisfy the requirements of AS 1720.1 Section 3.5.1:

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{N_c^*}{N_{d,cy}} \leq 1.0$$ AS 1720.1 equation 3.5(1)

$$\frac{M_x^*}{M_{d,x}} + \frac{N_c^*}{N_{d,cx}} \leq 1.0$$ AS 1720.1 equation 3.5(2)

Evaluate $N_{d,cx}$, $N_{d,cy}$ and $M_{d,x}$ for the studs at each level. As the combination involves wind actions, k_1 =1.0.

$N_{d,cx}$, $N_{d,cy}$ have already been evaluated for k_1 =1.0. The bending capacity is evaluated as shown in Table A1.9 using:

$$M_{d,x} = \phi k_1 k_4 k_6 k_9 k_{12} f'_b Z$$ AS 1720.1 equation 3.2(2).

With

ϕ as presented in Table A1.3 and Table A1.9

k_1 = 1 for the wind load combination

k_4 = 1 for seasoned timber used indoors

k_6 = 1 for studs within the building envelope in the cooler part of Australia

k_{12} calculated from S_1 and ρ_b (AS 1720.1 equation 3.2(10), (11) or (12))

S_1 calculated using AS 1720.1 equation 3.2(4) or (5) – Refer section 4.3.9.

L = length of stud on storeys = 2.61 m

f'_b from Table A1.2

Z = major axis section capacity = number of studs in group n $\frac{d^2 b}{6}$ with d = 90 mm and b = 45 mm

The slenderness for flexural buckling depends on the direction of the load:

- A differential pressure from the inside of the building to the outside means that the plasterboard face (which offers lateral restraint to the stud) is on the compression face of the stud and equation 3.2(4) in AS 1720.1 applies.
- A differential pressure from the outside of the building to the inside means that the plasterboard face (which offers lateral restraint to the stud) is on the tension face of the stud and equation 3.2(5) in AS 1720.1 applies.
- Evaluate both and use the larger slenderness to determine k_{12} and the bending capacity. This will mean that the worst direction is always used to check the capacity and the other direction of loading will also satisfy the combination requirements.

Note: for a stud to be loaded in bending by the wind, linings must be installed to give the differential pressure, so it is appropriate to calculate bending capacity with the linings installed.

Table A1.9: Calculation of $M_{d,x}$ for 90 x 45 studs with linings installed.

Wall	Design stud	ϕ	S1 3.2(4)	S1 3.2(5)	ρ_b	$\rho_b S_1$	k_{12}	f'_b (MPa)	Z (mm³)	k_1	M_d (kNm)
Storey FL7	1 MGP10	0.9	6.45	4.10	0.75	4.84	1	17	60750	1	0.93
Storey FL6	1 MGP12	0.9	6.45	4.10	0.85	5.48	1	28	60750	1	1.53
Storey FL5	1 MGP12	0.9	6.45	4.10	0.85	5.48	1	28	60750	1	1.53
Storey FL4	2 MGP10	0.9	6.45	4.10	0.75	4.84	1	17	121500	1	1.86
Storey FL3	2 MGP12	0.9	6.45	4.10	0.85	5.48	1	28	121500	1	3.06
Storey FL2	2 MGP12	0.9	6.45	4.10	0.85	5.48	1	28	121500	1	3.06
Storey FL1	2 LVL	0.95	6.45	4.10	0.95	6.15	1	37	121500	1	4.27

Table A1.10: Calculation of combined action effects for k_1 =1.0.

		Bending effects kNm		Axial loads 1.2G + ψl Q		Axial capacity kN		Combined	
Wall	Design stud	M_d	M^*/M_d	Per level	N* (kN)	$N_{d,cx}$	$N_{d,cy}$	Eq 3.5(1)	Eq 3.5(2)
Storey FL7	1 MGP10	0.93	0.54	4.27	4.3	23.5	56.5	0.29	0.65
Storey FL6	1 MGP12	1.53	0.32	5.08	9.3	30.2	74.2	0.21	0.59
Storey FL5	1 MGP12	1.53	0.32	5.08	14.4	30.2	74.2	0.27	0.76
Storey FL4	2 MGP10	1.86	0.27	5.08	19.5	47.1	113.1	0.23	0.65
Storey FL3	2 MGP12	3.06	0.16	5.08	24.6	60.3	148.5	0.19	0.55
Storey FL2	2 MGP12	3.06	0.16	5.08	29.6	60.3	148.5	0.22	0.63
Storey FL1	2 LVL	4.27	0.12	5.08	34.7	74.4	237.5	0.16	0.57

The values in the two right hand columns of Table A1.10 are all less than 1.0, hence the inequality has been satisfied and the studs have adequate capacity to resist combined wind and axial loads.

Jamb studs

Where windows and doors create openings that mean some studs are bridged by lintels, extra studs must be placed either side of the opening to transmit the higher axial forces. In short, if studs are removed by the opening, then they should be placed at the side of the opening as jamb studs (extra studs that are the same grade and cross-section as normal studs). In some cases, the reserve in axial capacity makes it possible to save on jamb studs.

For a 1.8 m opening three normal wall studs are removed to create the opening. The load is concentrated at each side of the opening and the extra load is transmitted vertically down the building as all of the openings align.

The stud at the end of the lintel carries its own load plus 1.5 times the standard stud load. The total capacity at the edge of the opening must exceed 2.5 times the load on a single stud. Table A1.5 has been reworked to indicate the number of studs required to carry 2.5 times the single stud load. The results are shown in Table A1.11.

Table A1.11: Selection of 90 x 45 jamb studs for a 1.8 m opening.

		k_1 = 0.8			k_1 = 0.57		
Wall	**Normal stud**	**2.5 times design load**	**$N_{d,c}$ (kN)**	**No. studs**	**2.5 times design load**	**$N_{d,c}$ (kN)**	**Total No. studs**
Storey FL7	1 MGP10	15.9	18.8	1	8.5	13.4	1
Storey FL6	1 MGP12	34.1	24.1	2	22.2	17.2	2
Storey FL5	1 MGP12	52.4	24.1	3	35.9	17.2	3
Storey FL4	2 MGP10	70.6	37.7	4	49.6	26.8	4
Storey FL3	2 MGP12	88.9	48.3	4	63.3	34.4	4
Storey FL2	2 MGP12	107.1	48.3	6	77.0	34.4	6
Storey FL1	2 LVL	125.4	59.5	6	90.7	42.4	6

Therefore, for the opening at:

- Storey FL7, only one jamb stud is required under the lintel
- Storey FL6, two jamb studs each side of the opening
- Storey FL5, three jamb studs each side of the opening
- Storey FL4 and FL3, four jamb studs each side of the opening
- Storey FL2 and FL1, six jamb studs each side of the opening.

Similar calculations can be performed for other opening sizes, though six jamb studs is a practical limit, so larger openings may require a different solution for lintel support on the lower two timber storeys.

Summary of Example Stud Design for 3 m FLW

Non-load bearing stud walls are clear of floors and detailed not to transmit vertical loads. Load-bearing stud walls are detailed to transmit vertical loads from floors above.

- Studs at 450 mm centres, detailed to align throughout the building and align with either floor joists or rim beams in floors above.
- All top and bottom plates to be 90 x 35 timber of the same grade as the studs.
- Storey FL7 studs – single 90 x 45 MGP10
- Storey FL6 and FL5 studs – single 90 x 45 MGP12
- Storey FL4 studs – 2 x 90 x 45 MGP10, nailed together at 600 mm centres
- Storey FL3 and FL2 studs – 2x 90 x 45 MGP12, nailed together at 600 mm centres
- Storey FL1 studs – 2 x 90 x 45 LVL (E = 13,200 MPa), nailed together at 600 mm centres.

Comments

A set of options for studs based on a typical series of assumptions has been presented. For each given building design these assumptions and the process should be interrogated to ensure they are valid. The design of wall frames which form part of the vertical load path and external envelope has been discussed. Many wall frames in the building will also form part of the stability system either in association with the braced sheathed wall frames, or through the global load path for tie down. In such cases additional combined checks are needed.

The design presented here assumed stud spacing consistent through the building height and stud grade varying in order to achieve an efficient design. Such an approach comes with efficiency in wall plates and direct load paths for floor cassettes. An alternative approach is to limit the variation in timber grade and instead to vary stud spacing – as is preferred by some prefabricators. In such a case the wall plates must be sufficiently strong and stiff to transfer loads between storeys and between floors and walls.

Step 5: Vertical movement design – building shortening

The stud walls designed in Worked Example A1 will be evaluated for shortening under long-term loads.
A high-level overview of building shortening can be found in the design guide.

The ***structural (load bearing) walls*** will all be framed with ***90x45 studs and 90x35 top and bottom plates***.
On each level, studs and plates will be sourced from the same grade of timber and may vary from level to level;
Stud length = 3100 – 420 – (2 x 35) = 2610 mm.

The ***floor cassettes*** will have a depth of ***420 mm*** and fabricated from parallel chord trusses. It is assumed that ***top chords, bottom chords and webs*** at the end will be ***63 x 90 mm LVL*** with the 90 mm dimension as the thickness of the floor truss;
Web length = 420 – (2 x 63) = 294 mm.

Stud and floor joist spacing = 450 mm with studs positioned above floor trusses in platform frame construction.

The stud design covered above gives the data in Table A2.1

Table A2.1: Design gravity loads on studs.

Level	***Stud design***	***MoE (MPa)***	***k_7 plates***	***A_p Top plate (mm²)***	***A_p Bottom plate (mm²)***	***A_l Stud (mm²)***
FL7	1 MGP10	10000	1.24	5625	5625	4050
FL6	1 MGP12	12700	1.24	5625	5625	4050
FL5	1 MGP12	12700	1.24	5625	5625	4050
FL4	2 MGP10	10000	1.135	9675	9675	8100
FL3	2 MGP12	12700	1.135	9675	9675	8100
FL2	2 MGP12	12700	1.135	9675	9675	8100
FL1	2 LVL	13200	1.135	9675	9675	8100

Bearing areas

Bearing area A_p top and bottom plates

Top and bottom plates have the same thickness, so bearing area will be the same for both.

The bearing area for storey *i* is given in the following equation.

$$A_{p,i} = max\left(b_1 + \frac{d_2}{2}, k_7 b_1\right) \times min(d_1, b_2)$$

with:

b_1 = breadth of stud

d_1 = depth (wide dimension) of stud

b_2 = width of plate (wide dimension)

d_2 = thickness of plate (narrow dimension)

k_7 = bearing modification factor for the plate based on the breadth of the stud (Cl. 2.4.4 in AS 1720.1)

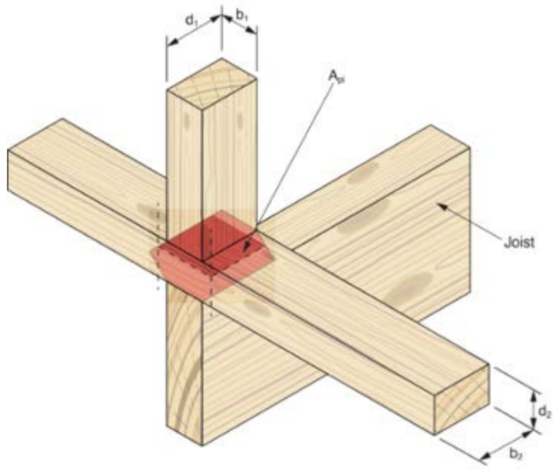
For single studs:

$$A_{p,i} = max\left(45 + \frac{35}{2}, 1.24 \times 45\right) \times 90 = 5625\ mm^2$$

For two nail-laminated studs:

$$A_{p,i} = max\left(2 \times 45 + \frac{35}{2}, 1.135 \times 2 \times 45\right) \times 90 = 9675\ mm^2$$

These values are shown in Table A2.1.

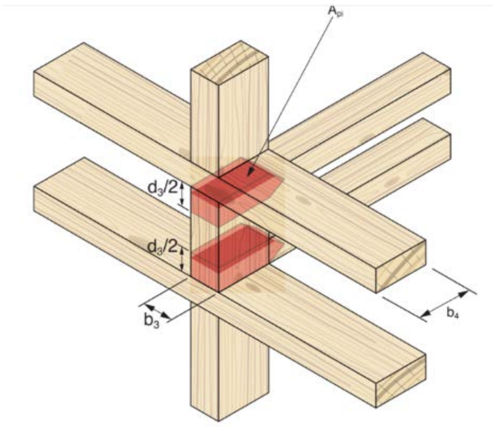


Bearing area A_p top and bottom chords in floor trusses

Top and bottom chords in the floor trusses have the same thickness (63 mm), so bearing area will be the same for both.

The bearing area for storey *i* is given in the following equation. It is the same on each level.

$$A_{p,i} = max\left(d_4 + \frac{d_3}{4}, k_7 d_4\right) \times b_3$$



Note: Flooring omitted for clarity

with:

b_3 = width of truss chord = 90 mm = width of truss web

d_3 = total depth of both truss chords = 126 mm

d_4 = depth of truss web = 63 mm

k_7 = bearing modification factor for the chord based on the depth of the web

Because the end web of the floor truss is at the end of the chord, k_7 for the chord = 1. (Cl. 2.4.4 in AS 1720.1)

$$A_{p,i} = max\left(63 + \frac{126}{4}, 1 \times 63\right) \times 90 = 8505\ mm^2$$

Bearing area A_l webs

The bearing area of the web in the floor truss is the cross-sectional area of the web.

The bearing area for storey *i* is :

$A_{l,i} = b_4 \times d_4 = 90 \times 63 = 5670\ mm^2$

with:

b_4 = width of truss web = 90 mm

d_4 = depth of truss web = 63 mm

Deformation under long-term serviceability loads

The dead load for wall and floor elements are the same as the loads on the studs given above and are applied at each top plate.

The serviceability live load is taken as $\psi_l Q$ with:

ψ_l for roof loads = 0

ψ_l for apartment floor loads = 0.4. (AS/NZS 1170.0 Table 4.1)

The loads on each element are summarised in Table A2.2. The loads accumulate down the building to give the total long-term serviceability load on each element in the right-most column.

Table A2.2: Long-term loads on wall and floor elements (kN per stud or floor element).

Level	Element	G introduced	ψ_l Q introduced	G total	ψ_l Q total	Long-term serviceability
FL7	top plate	2.84	0.00	2.8	0.0	2.8
	stud			2.8	0.0	2.8
	bottom plate			2.8	0.0	2.8
	top chord		0.81	2.8	0.8	3.6
	web			2.8	0.8	3.6
	bottom chord			2.8	0.8	3.6
FL6	top plate	3.55		6.4	0.8	7.2
	stud			6.4	0.8	7.2
	bottom plate			6.4	0.8	7.2
	top chord		0.81	6.4	1.6	8.0
	web			6.4	1.6	8.0
	bottom chord			6.4	1.6	8.0
FL5	top plate	3.55		9.9	1.6	11.6
	stud			9.9	1.6	11.6
	bottom plate			9.9	1.6	11.6
	top chord		0.81	9.9	2.4	12.4
	web			9.9	2.4	12.4
	bottom chord			9.9	2.4	12.4
FL4	top plate	3.55		13.5	2.4	15.9
	stud			13.5	2.4	15.9
	bottom plate			13.5	2.4	15.9
	top chord		0.81	13.5	3.2	16.7
	web			13.5	3.2	16.7
	bottom chord			13.5	3.2	16.7
FL3	top plate	3.55		17.1	3.2	20.3
	stud			17.1	3.2	20.3
	bottom plate			17.1	3.2	20.3
	top chord		0.81	17.1	4.1	21.1
	web			17.1	4.1	21.1
	bottom chord			17.1	4.1	21.1
FL2	top plate	3.55		20.6	4.1	24.7
	stud			20.6	4.1	24.7
	bottom plate			20.6	4.1	24.7
	top chord		0.81	20.6	4.9	25.5
	web			20.6	4.9	25.5
	bottom chord			20.6	4.9	25.5
FL1	top plate	3.55		24.2	4.9	29.0
	stud			24.2	4.9	29.0
	bottom plate			24.2	4.9	29.0

To calculate the deformation under load for each element:

Load applied parallel to grain (studs in walls and webs in floor trusses) MoE = design 1.05 MoE.
where 1.05 accounts for typical production MoE 5 - 10% higher.

Load applied perpendicular to grain (plates in walls and chords in floor trusses) MoE = design 1.05 MoE/20, where MoE perpendicular to grain is approximaely 1/20 MoE parallel to grain.

Elastic deformation was calculated without any creep factor. As the j_2 factor for seasoned timber in compression is 2, the inelastic deformation = the elastic deformation. (The sum of inelastic and elastic deformation = 2 x elastic deformation.)

Table A2.3: Deformation under load.

Level	Element	MoE (MPa)	Elastic deformation (mm)		Inelastic deformation (mm)	
			Element	Cumulative	Element	Cumulative
FL7	top plate	525	0.03	7.0	0.03	7.0
	stud	10500	0.17	6.9	0.17	6.9
	bottom plate	525	0.03	6.7	0.03	6.7
	top chord	693	0.04	6.7	0.04	6.7
	web	13860	0.01	6.7	0.01	6.7
	bottom chord	693	0.04	6.7	0.04	6.7
FL6	top plate	667	0.07	6.6	0.07	6.6
	stud	13335	0.35	6.6	0.35	6.6
	bottom plate	667	0.07	6.2	0.07	6.2
	top chord	693	0.09	6.1	0.09	6.1
	web	13860	0.03	6.1	0.03	6.1
	bottom chord	693	0.09	6.0	0.09	6.0
FL5	top plate	667	0.11	5.9	0.11	5.9
	stud	13335	0.56	5.8	0.56	5.8
	bottom plate	667	0.11	5.3	0.11	5.3
	top chord	693	0.13	5.2	0.13	5.2
	web	13860	0.05	5.0	0.05	5.0
	bottom chord	693	0.13	5.0	0.13	5.0
FL4	top plate	525	0.11	4.9	0.11	4.9
	stud	10500	0.49	4.7	0.49	4.7
	bottom plate	525	0.11	4.3	0.11	4.3
	top chord	693	0.18	4.1	0.18	4.1
	web	13860	0.06	4.0	0.06	4.0
	bottom chord	693	0.18	3.9	0.18	3.9
FL3	top plate	667	0.11	3.7	0.11	3.7
	stud	13335	0.49	3.6	0.49	3.6
	bottom plate	667	0.11	3.1	0.11	3.1
	top chord	693	0.23	3.0	0.23	3.0
	web	13860	0.08	2.8	0.08	2.8
	bottom chord	693	0.23	2.7	0.23	2.7
FL2	top plate	667	0.13	2.5	0.13	2.5
	stud	13335	0.60	2.3	0.60	2.3
	bottom plate	667	0.13	1.8	0.13	1.8
	top chord	693	0.27	1.6	0.27	1.6
	web	13860	0.10	1.3	0.10	1.3
	bottom chord	693	0.27	1.2	0.27	1.2
FL1	top plate	693	0.15	1.0	0.15	1.0
	stud	13860	0.67	0.8	0.67	0.8
	bottom plate	693	0.15	0.2	0.15	0.2

Shrinkage

To calculate the shrinkage for each element:

Height of the element parallel to grain (studs in walls and webs in floor trusses) unit movement = 0.0027/40 mm/mm/% Δmc, where shrinkage parallel to grain is approximately 1/14 shrinkage perpendicular to grain.

Height of the element perpendicular to grain (plates in walls and chords in floor trusses) – use *Equation 2.6*, unit movement = 0.0027 mm/mm/% Δmc.

Assumed timber moisture content:

Initial mc = 12% – appropriate for seasoned timber;
Final mc = 8% – appropriate for timber after some time in an air-conditioned environment;

Estimated joint closure is 0.5 mm if the joint under the element is constructed on site (bottom plates and bottom chords) otherwise, 0.2 mm per joint for elements in prefabricated assemblies (within wall frames and floor trusses).

Table A2.3: Deformation under load.

Level	Element	Unit shrinkage	Elastic deformation (mm)		Inelastic deformation (mm)	
		(mm/mm/% Δmc)	Element	Cumulative	Element	Cumulative
FL7	top plate	0.0027	0.38	12.5	0.2	11.7
	stud	0.0027	0.70	12.1	0.2	11.5
	bottom plate	0.0027	0.38	11.4	0.5	11.3
	top chord	0.0010	0.25	11.0	0.2	10.8
	web	0.0010	0.03	10.8	0.2	10.6
	bottom chord	0.0010	0.25	10.8	0.5	10.4
FL6	top plate	0.0027	0.38	10.5	0.2	9.9
	stud	0.0027	0.70	10.1	0.2	9.7
	bottom plate	0.0027	0.38	9.4	0.5	9.5
	top chord	0.0010	0.25	9.1	0.2	9.0
	web	0.0010	0.03	8.8	0.2	8.8
	bottom chord	0.0010	0.25	8.8	0.5	8.6
FL5	top plate	0.0027	0.38	8.5	0.2	8.1
	stud	0.0027	0.70	8.1	0.2	7.9
	bottom plate	0.0027	0.38	7.4	0.5	7.7
	top chord	0.0010	0.25	7.1	0.2	7.2
	web	0.0010	0.03	6.8	0.2	7.0
	bottom chord	0.0010	0.25	6.8	0.5	6.8
FL4	top plate	0.0027	0.38	6.5	0.2	6.3
	stud	0.0027	0.70	6.1	0.2	6.1
	bottom plate	0.0027	0.38	5.4	0.5	5.9
	top chord	0.0010	0.25	5.1	0.2	5.4
	web	0.0010	0.03	4.8	0.2	5.2
	bottom chord	0.0010	0.25	4.8	0.5	5.0
FL3	top plate	0.0027	0.38	4.5	0.2	4.5
	stud	0.0027	0.70	4.2	0.2	4.3
	bottom plate	0.0027	0.38	3.4	0.5	4.1
	top chord	0.0010	0.25	3.1	0.2	3.6
	web	0.0010	0.03	2.8	0.2	3.4
	bottom chord	0.0010	0.25	2.8	0.5	3.2
FL2	top plate	0.0027	0.38	2.5	0.2	2.7
	stud	0.0027	0.70	2.2	0.2	2.5
	bottom plate	0.0027	0.38	1.5	0.5	2.3
	top chord	0.0010	0.25	1.1	0.2	1.8
	web	0.0010	0.03	0.8	0.2	1.6
	bottom chord	0.0010	0.25	0.8	0.5	1.4
FL1	top plate	0.0010	0.14	0.5	0.2	0.9
	stud	0.0010	0.26	0.4	0.2	0.7
	bottom plate	0.0010	0.14	0.1	0.5	0.5

Total building shortening

Estimated total building shortening = elastic deformation + inelastic deformation + shrinkage + joint closure

Total building shortening over seven timber storeys is estimated as 38 mm (about 5.5 mm per storey).

Table A2.5: Estimated total building shortening (mm)

Level	Element	Elastic deformation cumulative	Inelastic deformation cumulative	Shrinkage cumulative	Joint cumulative	Total (mm)
FL7	top plate	7.0	7.0	12.5	11.7	38
	stud	6.9	6.9	12.1	11.5	37
	bottom plate	6.7	6.7	11.4	11.3	36
	top chord	6.7	6.7	11.0	10.8	35
	web	6.7	6.7	10.8	10.6	35
	bottom chord	6.7	6.7	10.8	10.4	34
FL6	top plate	6.6	6.6	10.5	9.9	34
	stud	6.6	6.6	10.1	9.7	33
	bottom plate	6.2	6.2	9.4	9.5	31
	top chord	6.1	6.1	9.1	9.0	30
	web	6.1	6.1	8.8	8.8	30
	bottom chord	6.0	6.0	8.8	8.6	29
FL5	top plate	5.9	5.9	8.5	8.1	28
	stud	5.8	5.8	8.1	7.9	28
	bottom plate	5.3	5.3	7.4	7.7	26
	top chord	5.2	5.2	7.1	7.2	25
	web	5.0	5.0	6.8	7.0	24
	bottom chord	5.0	5.0	6.8	6.8	24
FL4	top plate	4.9	4.9	6.5	6.3	23
	stud	4.7	4.7	6.1	6.1	22
	bottom plate	4.3	4.3	5.4	5.9	20
	top chord	4.1	4.1	5.1	5.4	19
	web	4.0	4.0	4.8	5.2	18
	bottom chord	3.9	3.9	4.8	5.0	18
FL3	top plate	3.7	3.7	4.5	4.5	16
	stud	3.6	3.6	4.2	4.3	16
	bottom plate	3.1	3.1	3.4	4.1	14
	top chord	3.0	3.0	3.1	3.6	13
	web	2.8	2.8	2.8	3.4	12
	bottom chord	2.7	2.7	2.8	3.2	11
FL2	top plate	2.5	2.5	2.5	2.7	10
	stud	2.3	2.3	2.2	2.5	9
	bottom plate	1.8	1.8	1.5	2.3	7
	top chord	1.6	1.6	1.1	1.8	6
	web	1.3	1.3	0.8	1.6	5
	bottom chord	1.2	1.2	0.8	1.4	5
FL1	top plate	1.0	1.0	0.5	0.9	3
	stud	0.8	0.8	0.4	0.7	3
	bottom plate	0.2	0.2	0.1	0.5	1

Comments

With an estimation of the building shortening due to the various contributing factors and over different load durations it is possible to verify or refine the design of the elements sensitive to movement:

- Envelope elements
- Threshold elements between different systems (e.g. concrete core).
- Suitable tie-down take-up mechanisms
- Non-loading bearing walls

The sensitivity of the shortening calculation can be tested.

Timber-framed - lateral distribution (hand calculations)

The diagram above presents a rationalised approach to the distribution of sheathed bracing walls. The approach groups them based on length. These lengths should ideally be linked to sheathing panel sizes to minimise cutting and waste, but this also needs to align with the building layout. In this case wall lengths are selected as 3.6, 4.2 and 5.4m. Double, separated stud walls are assumed where the stud walls form separation between single occupancy units or separation between single occupancy units and public spaces. In such cases the sheathed bracing is effectively two individual leaves. There are many walls in the plan above not taken into account for the stability calculations. This is to keep the process as transparent as possible with hand-calculations, and becasue short walls are less effective. If in the course of detailed design it becamce clear that more bracing was needed then the layout about could be revised.

In order to determine the lateral load distribution for each wall, the relative stiffness of each sheathed bracing (shear wall) must be calculated. The stiffness (K) is a function of unit force (kN) per unit deformation (mm), typically expressed per linear metre for wall. There are several methods for determining stiffness of sheathed bracing (shear) walls. These are discussed in Chapter 7 of the guide. The consitutient panels of sheathing mean that even for long runs of shear wall it can be appropriate to consider the stiffness as a linear function of length.

Combining clauses from international design standards should be avoided. However, where an international design standard includes a relatively simple model for determining the performance of a system then this may be considered valid for use with Australian Standards where the inputs are taken from the Australian Standards. In the case of shear walls a simple model exists in Eurocode 5 for strength design checks based on fastener performance. Stiffness design checks are generally less critical (not life-safety) than strength checks, and it can be more appropriate therefore to look for approaches internationally. The Canadian Standard CSA O86 Engineering Design in Wood provides a comprehensive model for estimating sheathed bracing wall sitffness and is used in this worked example.

Hand calculations based on simple analysis models provides a solid basis for engineering understanding and design decision-making. Computer-based analysis and design methods can provide a useful cross check against the hand calculations. As confidence in a particular piece of software grows, it can be used at various stages of design with greater confidence.

Timber-framed - shear wall capacity

Eurocode 5 9.2.4.2 provides a simplified method of analysis for the capacity of panel braced walls. The method is deemed applicable provided the individual wall panels are considered connected with a tie-down to the vertical members of the wall panels to the construction below. Construction practices of this type are common in Australia.

As a starting point for specification of plywood and nailing consider a practice as would be considered common in Australia by referencing AS 1684 table 8.18. The Eurocode method (and notation) here is used with the lateral design capacity of an individual fastener as determine by Table 4.1(B) in AS 1720.1.

EC5 9.2.4.2 Method A - SImplified analysis of wall diaphragms

$$F_{v,Rd} = \frac{N_{d,j} b_i C_i}{s}$$

Lateral fastener design capacity		$N_{d,j}$	545 N	See table below
Length of wall		b_i	4.2 m	For example. Also for 3.6 and 5.4m.
Fastener spacing		s	0.05 m	Nail spacing based on typical practices. See AS1684.2 Table 8.18
wall height		h	3.1 m	
nail diameter		d	2.8 mm	Nail diameter based on typical practices. See AS1684.2 Table 8.18
		C_i	1	See EC5 9.2.4.1. Factor based on proportion of wall panel.
Joint group		*JD5*		
9.2.4.2 (5)	factor		1.2	See EC5 9.2.4.1. Factor for increasing fastener performance around panel edges.
	$F_{iv}R_d$		46845 N	
	check per m		11.2 kN/m wall	

where lateral fastener design capacity is from AS 1720.1

AS1720 nails

Table 4.1(B): Characteristic in single shear and seasoned timber (N).

joint group	Nail diameter (mm)		
	2.5	**2.8**	**3.1**
JD4	545	665	810
JD5	445	545	680

Capacity factor		ϕ	0.8	Table 2.2
		k_1	1.14	Table 2.3
		k_{13}	1	
		k_{14}	1	
		k_{16}	1.1	
		k_{17}	0.85	Table 4.3(A)
	TOTAL		0.85	

Wall design capacity

Wall length (m)	Wall design capacity (EC5 model) (kN)	Capacity (kN) per m
3.6	40.1	11.1
4.2	46.9	11.2
5.4	60.2	11.1

This capacity does not consider stiffness limits of deflection which may govern load. See stiffness checks below for limits based on wind serviceability considerations.

See element design for associated checks such as hold-down performance.

Timber-framed - panel contribution 1

The detailed check presented below follows the method set out in Canadian Standard CSA O86 Engineering Design with Wood section 11.7.1 (refer to standard for detail and speicific clauses). The process considers contribution of stiffness (flexilibity) to the overall system of the fasteners, the sheathing, the framing and the hold downs. AS 1720.1 and CSA O86 both calculate fastener stiffnesses as reducing with increasing loads applied. Whereas Eurocode 5 fastener stiffness models are independent of applied load. Refer to section 7 of the guide for an indication as to how the different models of wall sitffness compare. In this process it is appropriate to back-calulcate the shear wall capacity that results in inter-storey drift under serviceability wind loads - h/400 say, or a different appropriate project limit.

The interstorey drift at the *i-th* level, mm, may be taken as follows:

$$\Delta_i^{storey} = \Delta_{b,i}^{storey} + \Delta_{s,i}^{storey} + \Delta_{n,i}^{storey} + \Delta_{a,i}^{storey}$$

And the total deflection at the *i-th* level, mm, may be taken as follows:

$$\Delta_i^{total} = \sum_{j=1}^{i} \Delta_j^{storey}$$

where

Δ_i^{storey} = interstorey drift at i-th level

Δ_i^{total} = total deflection at i-th level

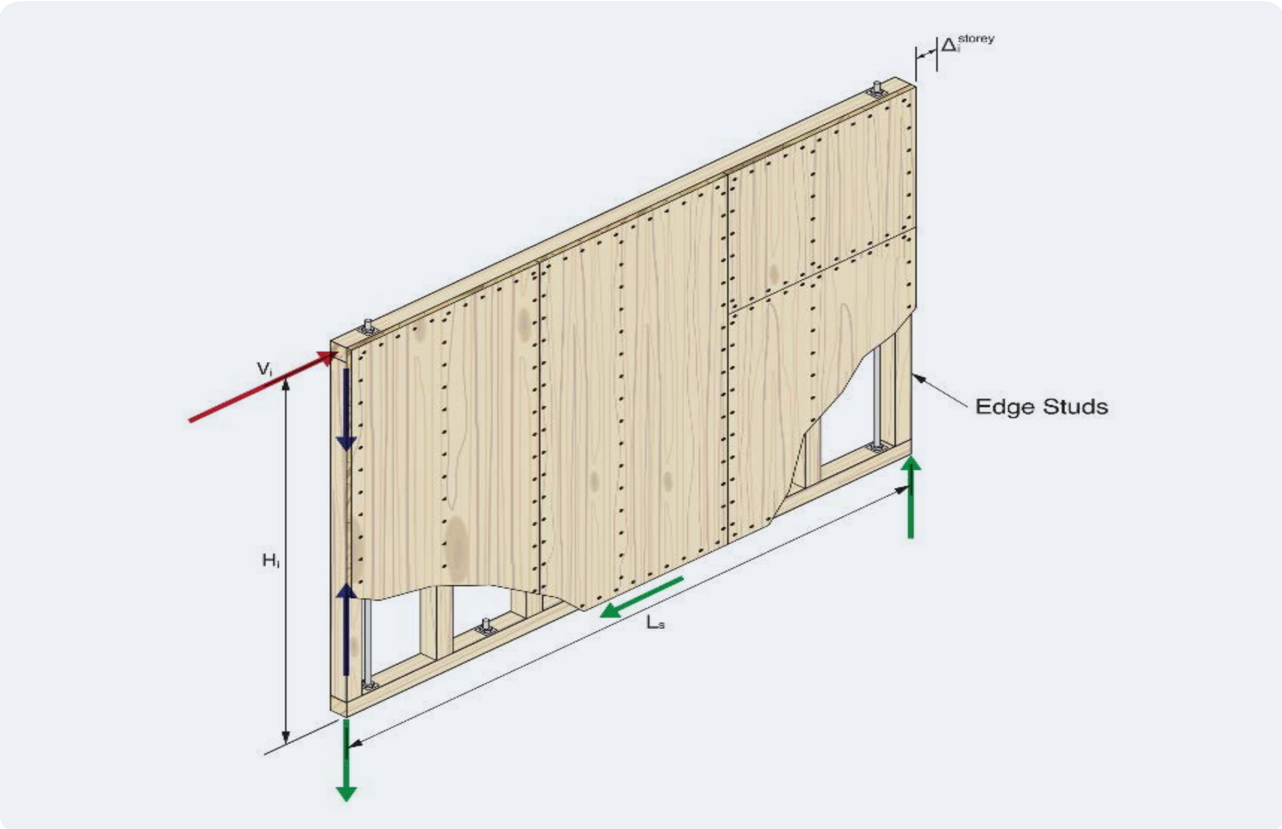
$\Delta_{b,i}^{storey}$ = interstorey drift at i-th level due to bending

$\Delta_{s,i}^{storey}$ = interstorey drift at i-th level due to panel shear

$\Delta_{n,i}^{storey}$ = interstorey drift at i-th level due to nail slip

$\Delta_{a,i}^{storey}$ = interstorey drift at i-th level due to elongation of wall tie-downs

For detailed equations see below and refer to CSA O86 A. 11.7.1.



Timber-framed - panel contribution 2

Bending Deformation

$$\Delta_{b,i} = \frac{M_i H_i^2}{2(EI)_i} + \frac{V_i H_i^3}{3(EI)_i}$$

Where

$M_i = \sum_{j=i+1}^{n} V_j H_j$

$$(EI)_i = (EI)_i \left(\frac{L_s}{2}\right)^2 \cdot 2 = \frac{(EA)_i L_s^2}{2}$$

E = Edge stud elastic modulus

A = Edge studs cross section

Panel Shear Deformation

$$\Delta_{s,i}^{storey} = \Delta_{s,i}$$

where

$$\Delta_{s,i} = \frac{V_{f,i} H_i}{B_{v,i}}$$

$V_{f,i} = \dfrac{V_i}{L_s}$ = maximum shear flow, N/mm

$B_{v,i}$ = shear-through-thickness rigidity of the sheathing at the i-th storey, N/mm

Nail Slip Deformation

$$\Delta_{n,i} = 0.0025 H_i e_{n,i}$$

where

$e_{n,i}$ = nail deformation at the i-th storey, mm

Where:

$$\boldsymbol{e_n} = \left(\frac{0.013\boldsymbol{vs}}{\boldsymbol{d_f}^2}\right)^2$$

v = shear force per unit length of wall along length of sheeting, N/mm

s = nail spacing at panel edges, mm

d_f = nail diameter, mm

Tie Down Deformation

$$\Delta_{a,i}^{storey} = \frac{H_i(d_a)}{L_s}$$

d_a = elongation of the tie-down, mm

$$d_a = \frac{V_i H_i}{L_s} \frac{l_{td}}{A_{td} E_{td}}$$

Timber-framed - panel contribution 3

Assumptions:

- a simplified version of the steps from CSA O86 are presented here for reference. Engineers are encouraged to obtain CSA O86 should they wish to follow the steps in detail or to expand the simple steps to an overall building.
- the fundamental model of shear wall deflection as followed by CSA O86 is valid for typical methods of construction. If there is significant deviation from accepted practices then the validity of the model should be tested.
- the calculation of stiffness and subsequent distrubtion of loads through the system considers only the shear walls as described here. In this case plasterboard is not considered to contribute. In some juridictions/countries the contribution of plasterboard to racking resistance is considered as significant.

The key components of the shear wall deflection model are:

- 'Bending deformation' associated with strain of the edge studs whcih form the effective flanges of the system. This can be calculated to AS1720.1
- 'Panel shear deformation' associated with shear strain of the sheathing material. This can be calculated to AS1720.1
- 'Nail slip deformation' of the nails along the top of the shear wall. This can be estimated to AS1720.1
- 'Tie down deformation' due to extension of the tie downs. In cases where this is a threaded rod this can be estimated from mechanics, or can be obtained from the manufacturer for proprietary products.

Shear deformation of the fasteners at the base of the shear wall is not considered in this model. For any given project the engineer may also wish to study the signfiicance of the this.

As noted throughout this document, the load distribution through the structure should be tested with a range of stiffnesses. Stiffness estimation is approximate.

For detailed equations refer to CSA O86 A. 11.7.1.

Key inputs are as follows:

				Reference
Studs	E	13200 MPa		LVL manufacturer for design to AS 1720.1
Stud size	Studs	90	90	Pair 90x45 studs. See stud worked example
Stud area	A	8100 mm^2		
Wall height	H	3100 mm		
Panel Shear	G	525 MPa		F11 Plywood panel performance AS 1720.1 Table 5.1
Panel Thickness	t	7 mm		Typical plywood thickness. Refer to AS 1684 Table 8.18
Nail spacing	s	50 mm		Typical nail spacing. Refer to AS 1684 Table 8.18
Nail diameter	d_f	2.8 mm		Typical nail diameter. Refer to AS 1684 Table 8.18
Tie-Down**				**Conservatively assume no load relief.
Tie-Down Dia.	d_{td}	20 mm		M12 threaded rod common
Tie-Down Rod Length	l_{td}	500 mm		Bolted through head-plate to cill-plate between floors
Tie-Down Area	A_{td}	314 mm^2		
	E_{td}	205000 N/mm^2		

Timber-framed - panel contribution 4

Wall length (mm)	Servicability Limit State (N)	Stiffness (N/mm)	Bending $\Delta_{b,j}$ (mm)	Panel shear $\Delta_{s,j}$ (mm)	Nail slip $\Delta_{n,j}$ (mm)	Tie-down $\Delta_{a,j}$ (mm)	Total Δ_j (mm)	Drift h/xxxx	Strength Limit State Load	
									Wall capacity* (kN)	Stiffness (N/mm)
3600	19088	5.3	1.9	4.5	1.5	0.3	8.2	378	28.1	7.8
4200	22301	5.3	1.6	4.5	1.5	0.25	7.85	395	32.8	7.8
5400	28655	5.3	1.3	4.5	1.5	0.2	7.5	413	42.1	7.8

* Limited by deflection

In this case the Strength Limit State wall capacity is determined by:

1. Determine the maximum Serviceablity Limit State wall loads which satisfy the inter-storey drift defleciton limits for each wall length. The narrow elevation of the building is fully glazed. Therefore target height/400 deflection.

2. Back calculate the Strength Limit State load based on the ratio between wind loads for serviceability and strength design; i.e. wind loads based on 25 year return period and 500 year return period wind speeds, squared and ratioed.

3. The Strength Limit State wall capacity will be used in the building design verifications.

Wall length (mm)	Servicability Limit State		Total Δ_j (mm)	Wall stiffness (N/mm)
	(kN)	(N/mm)		
3600	19088	5.3	8.2	2328
4200	22301	5.3	7.85	2841
5400	28655	5.3	7.5	3821

Stiffness of the wall at calculated Strength Limit State also needs to be determined in order to determine inter-storey drift under earthquake loads. The calculation below follows the CSA O86 model under wall capacity loads as determined using the EC5 model.

Wall length (m)	Wall design capacity (EC5 model) (kN)	Capacity Bending $\Delta_{b,j}$		Panel shear $\Delta_{s,j}$ (mm)	Nail slip $\Delta_{n,j}$ (mm)	Tie-down $\Delta_{a,j}$ (mm)	Total Δ_j (mm)	Drift h/xxxx
		(N/mm)	(mm)					
40.1	11.1	4.0	9.4	6.6	0.7	20.0	150	8.2
46.9	11.2	3.5	9.4	6.6	0.7	20.0	155	7.85
60.2	11.1	2.7	9.4	6.6	0.7	19.2	162	7.5

Therefore, provided the wall design capacity as determined with teh EC5 model is not exceeded then the required inter-storey drift limit of say, height/150 will be met.

Next steps:

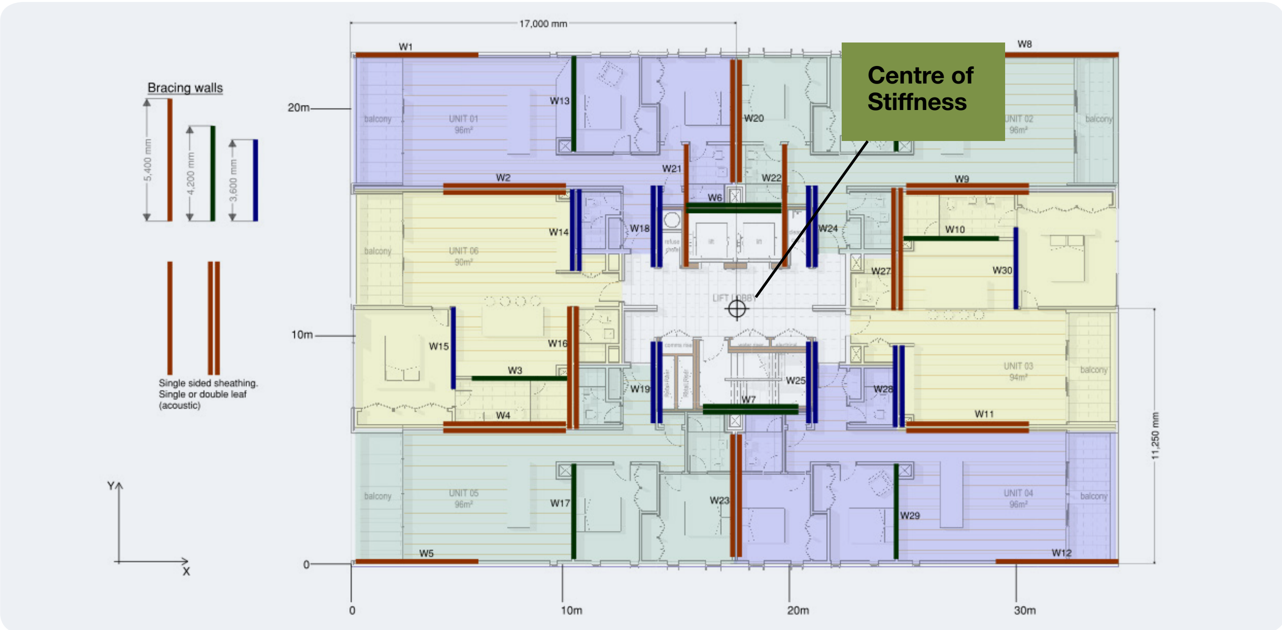
The calculations here present one pass. It would be possible to refine the bracing strategy by working through variations of plywood thickness, grade, and nail diameter, and spacing. The validitity of the stiffness model is based on 'standard practice' and so caution is needed in deviating far from those standard practices. If in doubt search for precedents (such as AS1684) or talk to a fabricator.

Determine the cumulative inter-storey drift up the full height of the building acknowledging global deformations of the walls due to overall global actions. The check above assumes a single storey and does not account for rotational deformation of the stacked shear walls over the building height.

Timber-framed wall stiffness & centre of stiffness

The following section calculates the load distribution, which is generic and not unique to timber-framed construction. The lateral load distribution for this worked example is calculated via a hand model, with rigid diaphragms assumed.

For stiffness calculations, the walls have been assumed to be broken up into a combination of 3.6m, 4.2m and 5.4m panels. These sizes are based on assumptions on sheathing sizes for braced walls. Note this is a simplification; the purpose of this worked example is to show the process only. Verification would typically be by computer software. The bracing arrangement is symmetrical in both X and Y. Therefore the centre of stiffness is located at the geometric centre of the building.



Wall ref	Length (m)	Leaves		Stiffness (N/mm)	Orientation	yi (m)	xi (m)	Kx (N/mm)	Ky (N/mm)
W1	5.4	1	3821	3821	x	22.4	0	3820.7	0.0
W2	5.4	2	3821	7641	x	16.5	0	7641.4	0.0
W3	4.2	1	2841	2841	x	8.2	0	2840.8	0.0
W4	5.4	2	3821	7641	x	5.8	0	7641.4	0.0
W5	5.4	1	3821	3821	x	0	0	3820.7	0.0
W6	4.2	2	2841	5682	x	15.5	0	5681.7	0.0
W7	4.2	2	2841	5682	x	6.8	0	5681.7	0.0
W8	5.4	1	3821	3821	x	22.4	0	3820.7	0.0
W9	5.4	2	3821	7641	x	16.5	0	7641.4	0.0
W10	4.2	1	2841	2841	x	14.1	0	2840.8	0.0
W11	5.4	2	3821	7641	x	12.9	0	7641.4	0.0
W12	5.4	1	3821	3821	x	0	0	3820.7	0.0
W13	4.2	1	2841	2841	y	0	9.7	0.0	2840.8
W14	3.6	2	2328	4656	y	0	9.7	0.0	4655.5
W15	4.2	1	2841	2841	y	0	9.7	0.0	2840.8
W16	5.4	2	3821	7641	y	0	9.7	0.0	7641.4
W17	4.2	1	2841	2841	y	0	9.7	0.0	2840.8
W18	3.6	2	2328	4656	y	0	13.4	0.0	4655.5
W19	3.6	2	2328	4656	y	0	13.4	0.0	4655.5
W20	5.4	2	3821	7641	y	0	17	0.0	7641.4
W21	5.4	1	3821	3821	y	0	14.5	0.0	3820.7
W22	5.4	1	3821	3821	y	0	18.9	0.0	3820.7
W23	5.4	2	3821	7641	y	0	17	0.0	7641.4
W24	3.6	2	2328	4656	y	0	20.3	0.0	4655.5
W25	3.6	2	2328	4656	y	0	20.3	0.0	4655.5
W26	4.2	1	2841	2841	y	0	24.1	0.0	2840.8
W27	5.4	2	3821	7641	y	0	24.1	0.0	7641.4
W28	3.6	2	2328	4656	y	0	24.1	0.0	4655.5
W29	4.2	1	2841	2841	y	0	24.1	0.0	2840.8
W30	3.6	1	2328	2328	y	0	29.2	0.0	2327.8
							Sum	**62893**	**82672**

$\sum Kxi$ 62893 N/mm $\sum Kyi$ 82672 N/mm

$\sum Yi\ Kxi$ 7.6E+05 $\sum Yi\ Kyi$ 1.4E+06

Y cos 12.0 m **X cos 17.0 m**

Timber-framed wall panel - seismic load distribution

In this section load distribution through the bracing walls is determined based on wall stiffness. This analysis also considers the torsional effect associated with eccentric loading required in seismic design.

1) Calculate torsional constant, J

Wall No	d_y	d^x	k_y x d^{x2}	k^x x d_y^2
W1	10.37521799	0	411279.1005	0
W2	4.475217988	0	153038.4525	0
W3	-3.824782012	0	41558.53863	0
W4	-6.224782012	0	296087.7712	0
W5	-12.02478201	0	552454.6137	0
W6	3.475217988	0	68618.46363	0
W7	-5.224782012	0	155100.5143	0
W8	10.37521799	0	411279.1005	0
W9	4.475217988	0	153038.4525	0
W10	2.075217988	0	12234.16512	0
W11	0.875217988	0	5853.352061	0
W12	-12.02478201	0	552454.6137	0
W13	0	-7.287650559	0	150876.6288
W14	0	-7.287650559	0	247253.5575
W15	0	-7.287650559	0	150876.6288
W16	0	-7.287650559	0	405832.9048
W17	0	-7.287650559	0	150876.6288
W18	0	-3.587650559	0	59922.19864
W19	0	-3.587650559	0	59922.19864
W20	0	0.012349441	0	1.165377923
W21	0	-2.487650559	0	23643.99888
W22	0	1.912349441	0	13972.58265
W23	0	0.012349441	0	1.165377923
W24	0	3.312349441	0	51078.69144
W25	0	3.312349441	0	51078.69144
W26	0	7.112349441	0	143705.3919
W27	0	7.112349441	0	386543.4766
W28	0	7.112349441	0	235501.48
W29	0	7.112349441	0	143705.3919
W30	0	12.21234944	0	347164.9868

For each direction, the Earthquake actions are applied at + or i 0,1b (as per AS1170.4), from the nominal centre of mass. The eccentricity will be applied to calculate the most adverse torsional moment.

A 1kN load will be applied as the nominal load to calculate the distribution. Once the distribution is determined, the real Earthquake loads (previously calculated) can be applied to the building.

$J = \ (dx_i^2 K_{iy} + dy_i^2 K_{ix})$=

J $\quad$ *5.43E+06* m^6

Applied Loads

Fx Load 1 kN

Fy Load 1 kN

e (x) 3.38 m

e (y) 2.24 m

2) Calculate the lateral load distribution based on a 1kN load

Wall No	1kN in X Direction			1kN in Y Direction		
	Fsx	Ftx	Fty	Fsy	Ftx	Fty
W1	6.1%	1.6%	0.0%	0.0%	2.5%	0.0%
W2	12.1%	1.4%	0.0%	0.0%	2.1%	0.0%
W3	4.5%	-0.4%	0.0%	0.0%	-0.7%	0.0%
W4	12.1%	-2.0%	0.0%	0.0%	-3.0%	0.0%
W5	6.1%	-1.9%	0.0%	0.0%	-2.9%	0.0%
W6	9.0%	0.8%	0.0%	0.0%	1.2%	0.0%
W7	9.0%	-1.2%	0.0%	0.0%	-1.8%	0.0%
W8	6.1%	1.6%	0.0%	0.0%	2.5%	0.0%
W9	12.1%	1.4%	0.0%	0.0%	2.1%	0.0%
W10	4.5%	0.2%	0.0%	0.0%	0.4%	0.0%
W11	12.1%	0.3%	0.0%	0.0%	0.4%	0.0%
W12	6.1%	-1.9%	0.0%	0.0%	-2.9%	0.0%
W13	0.0%	0.0%	-0.9%	3.4%	0.0%	-1.3%
W14	0.0%	0.0%	-1.4%	5.6%	0.0%	-2.1%
W15	0.0%	0.0%	-0.9%	3.4%	0.0%	-1.3%
W16	0.0%	0.0%	-2.3%	9.2%	0.0%	-3.5%
W17	0.0%	0.0%	-0.9%	3.4%	0.0%	-1.3%
W18	0.0%	0.0%	-0.7%	5.6%	0.0%	-1.0%
W19	0.0%	0.0%	-0.7%	5.6%	0.0%	-1.0%
W20	0.0%	0.0%	0.0%	9.2%	0.0%	0.0%
W21	0.0%	0.0%	-0.4%	4.6%	0.0%	-0.6%
W22	0.0%	0.0%	0.3%	4.6%	0.0%	0.5%
W23	0.0%	0.0%	0.0%	9.2%	0.0%	0.0%
W24	0.0%	0.0%	0.6%	5.6%	0.0%	1.0%
W25	0.0%	0.0%	0.6%	5.6%	0.0%	1.0%
W26	0.0%	0.0%	0.8%	3.4%	0.0%	1.3%
W27	0.0%	0.0%	2.2%	9.2%	0.0%	3.4%
W28	0.0%	0.0%	1.4%	5.6%	0.0%	2.1%
W29	0.0%	0.0%	0.8%	3.4%	0.0%	1.3%
W30	0.0%	0.0%	1.2%	2.8%	0.0%	1.8%
	100.0%	0.0%	0.0%	100.0%	0.0%	0.0%

$$F_{shear,x} = \frac{F_x K_{ix}}{\sum K_{iy}}$$

$$F_{torsion,x} = \frac{(F_x e_y) y_i K_{iy}}{(dx_i^2 K_{iy} + dy_i^2 K_{ix})}$$

$$F_{torsion,y} = \frac{(F_y e_x) x_i K_{ix}}{(dx_i^2 K_{iy} + dy_i^2 K_{ix})}$$

$$F_{shear,y} = \frac{F_y K_{ix}}{\sum K_{iy}}$$

Timber framed wall panel - seismic load distribution

The calcualted earthquake action is applied at ht position +/- 0.1b, from the nominal centre of mass. The eccentricity shall be applied in the same direction at all levels, and oriented to produce the maximum 100% and 30% loads. The load cases are simplified by taking the absolute number for each wall. For this scenario, the absolute values are used to reduce the overall number of load cases.

The timber-framed portion of the building is considered as sesmically distinct from the concrete lower levels. In such a case the analysis here considers the timber portion to be supported on an element significantly stiffer with a different frequency. As such seismic design consdiers the timber-framed structure founded at level 1, top of concrete. The assumptions for seismic design of the CLT structure consider it working with the concrete as systems are more similar.

3) Calculate load on each wall for load cases specified in AS1170.4 (based on 1kN)

LC1 100% Shear + ABS(100% Tx)+ ABS(30% Ty) Calculate for 1kN only

LC2 100% Shear + ABS(100% Ty)+ ABS(30% Tx) Calculate for 1kN only

Wall No	LC1 - x	LC1 - y	LC2 - x	LC2 - y
W1	0.08	0.00	0.09	0.00
W2	0.14	0.00	0.15	0.00
W3	0.05	0.00	0.05	0.00
W4	0.15	0.00	0.16	0.00
W5	0.09	0.00	0.10	0.00
W6	0.10	0.00	0.11	0.00
W7	0.11	0.00	0.11	0.00
W8	0.08	0.00	0.09	0.00
W9	0.14	0.00	0.15	0.00
W10	0.05	0.00	0.05	0.00
W11	0.13	0.00	0.13	0.00
W12	0.09	0.00	0.10	0.00
W13	0.00	0.01	0.00	0.05
W14	0.00	0.02	0.00	0.07
W15	0.00	0.01	0.00	0.05
W16	0.00	0.04	0.00	0.12
W17	0.00	0.01	0.00	0.05
W18	0.00	0.02	0.00	0.07
W19	0.00	0.02	0.00	0.07
W20	0.00	0.03	0.00	0.09
W21	0.00	0.02	0.00	0.05
W22	0.00	0.02	0.00	0.05
W23	0.00	0.03	0.00	0.09
W24	0.00	0.03	0.00	0.06
W25	0.00	0.03	0.00	0.06
W26	0.00	0.03	0.00	0.05
W27	0.00	0.08	0.00	0.12
W28	0.00	0.05	0.00	0.07
W29	0.00	0.03	0.00	0.05
W30	0.00	0.04	0.00	0.04

Critical Load (kN)
0.09
0.15
0.05
0.16
0.10
0.11
0.11
0.09
0.15
0.05
0.13
0.10
0.05
0.07
0.05
0.12
0.05
0.07
0.07
0.09
0.05
0.05
0.09
0.06
0.06
0.05
0.12
0.07
0.05
0.04

Timber-Framed - inter-storey drift and wall loads

Note, each wall has been checked to determine load attracted for a 1kN force. Next, the real loads are applied to the building to determine the force acting on each wall. This method is conservative with respect to the overall building shears. But is a suitable method of determining the max shear load in each wall based on the seismic load cases.

4) Calculate seismic loads for each wall at each floor

F (kN) Eq	R	7	6	5	4	3	2	Cumulative*
	196.4	**195.7**	**163.1**	**130.5**	**97.8**	**65.2**	**32.6**	**881.3**
W1	17.7	17.7	14.7	11.8	8.8	5.9	2.9	79.6
W2	28.9	28.8	24.0	19.2	14.4	9.6	4.8	129.6
W3	10.5	10.4	8.7	6.9	5.2	3.5	1.7	46.9
W4	30.8	30.7	25.6	20.5	15.4	10.2	5.1	138.3
W5	18.7	18.6	15.5	12.4	9.3	6.2	3.1	83.7
W6	20.6	20.6	17.1	13.7	10.3	6.9	3.4	92.6
W7	22.1	22.0	18.3	14.7	11.0	7.3	3.7	99.1
W8	17.7	17.7	14.7	11.8	8.8	5.9	2.9	79.6
W9	28.9	28.8	24.0	19.2	14.4	9.6	4.8	129.6
W10	9.7	9.7	8.1	6.5	4.8	3.2	1.6	43.7
W11	24.8	24.8	20.6	16.5	12.4	8.3	4.1	111.5
W12	18.7	18.6	15.5	12.4	9.3	6.2	3.1	83.7
W13	8.9	8.9	7.4	5.9	4.4	3.0	1.5	40.1
W14	14.6	14.6	12.1	9.7	7.3	4.9	2.4	65.7
W15	8.9	8.9	7.4	5.9	4.4	3.0	1.5	40.1
W16	24.0	23.9	19.9	16.0	12.0	8.0	4.0	107.8
W17	8.9	8.9	7.4	5.9	4.4	3.0	1.5	40.1
W18	12.8	12.8	10.6	8.5	6.4	4.3	2.1	57.5
W19	12.8	12.8	10.6	8.5	6.4	4.3	2.1	57.5
W20	18.2	18.1	15.1	12.1	9.0	6.0	3.0	81.5
W21	10.1	10.0	8.4	6.7	5.0	3.3	1.7	45.2
W22	9.8	9.8	8.2	6.5	4.9	3.3	1.6	44.2
W23	18.2	18.1	15.1	12.1	9.0	6.0	3.0	81.5
W24	12.7	12.6	10.5	8.4	6.3	4.2	2.1	56.9
W25	12.7	12.6	10.5	8.4	6.3	4.2	2.1	56.9
W26	8.9	8.8	7.4	5.9	4.4	2.9	1.5	39.8
W27	23.9	23.8	19.8	15.9	11.9	7.9	4.0	107.1
W28	14.5	14.5	12.1	9.7	7.2	4.8	2.4	65.3
W29	8.9	8.8	7.4	5.9	4.4	2.9	1.5	39.8
W30	8.5	8.5	7.1	5.7	4.2	2.8	1.4	38.2

AS1170.0, Table C1

Timber-framed wall panel - wind load distribution

Wall No	kix	1kN x%	kiy	1kN y%
W1	3821	6.1%	0	0.0%
W2	7641	12.1%	0	0.0%
W3	2841	4.5%	0	0.0%
W4	7641	12.1%	0	0.0%
W5	3821	6.1%	0	0.0%
W6	5682	9.0%	0	0.0%
W7	5682	9.0%	0	0.0%
W8	3821	6.1%	0	0.0%
W9	7641	12.1%	0	0.0%
W10	2841	4.5%	0	0.0%
W11	7641	12.1%	0	0.0%
W12	3821	6.1%	0	0.0%
W13	0	0.0%	2841	3.4%
W14	0	0.0%	4656	5.6%
W15	0	0.0%	2841	3.4%
W16	0	0.0%	7641	9.2%
W17	0	0.0%	2841	3.4%
W18	0	0.0%	4656	5.6%
W19	0	0.0%	4656	5.6%
W20	0	0.0%	7641	9.2%
W21	0	0.0%	3821	4.6%
W22	0	0.0%	3821	4.6%
W23	0	0.0%	7641	9.2%
W24	0	0.0%	4656	5.6%
W25	0	0.0%	4656	5.6%
W26	0	0.0%	2841	3.4%
W27	0	0.0%	7641	9.2%
W28	0	0.0%	4656	5.6%
W29	0	0.0%	2841	3.4%
W30	0	0.0%	2328	2.8%
Total	**62893**	**1**	**82672**	**1**

The lateral load attracted for each wall, is proportional to the stiffness of the individual wall to the building stiffness in the same direction.

$$F_{shear,x} = \frac{F_x K_{ix}}{\sum K_{ix}}$$

$$F_{shear,y} = \frac{F_y K_{ix}}{\sum K_{iy}}$$

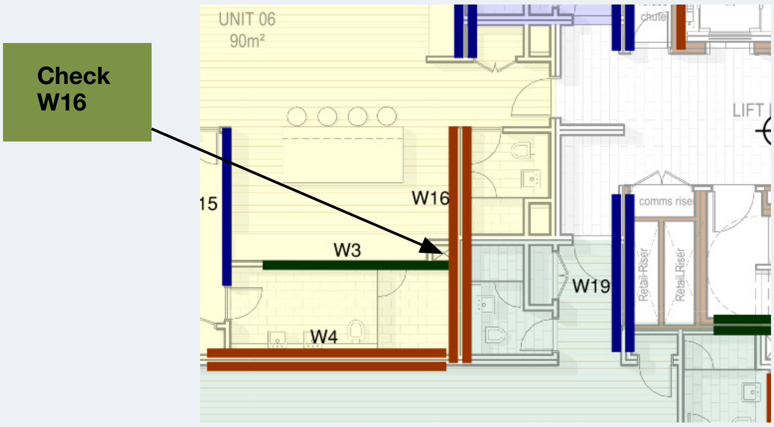
4) Calculate individual wall loads (wind)

	R	7	6	5	4	3	2	1	Cumulative*
F (kN) Wx	**45.8**	**91.5**	**91.5**	**91.5**	**91.5**	**91.5**	**91.5**	**45.8**	**594.9**
F (kN) Wy	**69.2**	**138.3**	**138.3**	**138.3**	**138.3**	**138.3**	**138.3**	**69.2**	**899.0**
W1 - x	2.8	5.6	5.6	5.6	5.6	5.6	5.6	2.8	36.1
W2 - x	5.6	11.1	11.1	11.1	11.1	11.1	11.1	5.6	72.3
W3 - x	2.1	4.1	4.1	4.1	4.1	4.1	4.1	2.1	26.9
W4 - x	5.6	11.1	11.1	11.1	11.1	11.1	11.1	5.6	72.3
W5 - x	2.8	5.6	5.6	5.6	5.6	5.6	5.6	2.8	36.1
W6 - x	4.1	8.3	8.3	8.3	8.3	8.3	8.3	4.1	53.7
W7 - x	4.1	8.3	8.3	8.3	8.3	8.3	8.3	4.1	53.7
W8 - x	2.8	5.6	5.6	5.6	5.6	5.6	5.6	2.8	36.1
W9 - x	5.6	11.1	11.1	11.1	11.1	11.1	11.1	5.6	72.3
W10 - x	2.1	4.1	4.1	4.1	4.1	4.1	4.1	2.1	26.9
W11 - x	5.6	11.1	11.1	11.1	11.1	11.1	11.1	5.6	72.3
W12 - x	2.8	5.6	5.6	5.6	5.6	5.6	5.6	2.8	36.1
W13 - y	2.4	4.8	4.8	4.8	4.8	4.8	4.8	2.4	30.9
W14 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W15 - y	2.4	4.8	4.8	4.8	4.8	4.8	4.8	2.4	30.9
W16 - y	6.4	12.8	12.8	12.8	12.8	12.8	12.8	6.4	83.1
W17 - y	2.4	4.8	4.8	4.8	4.8	4.8	4.8	2.4	30.9
W18 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W19 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W20 - y	6.4	12.8	12.8	12.8	12.8	12.8	12.8	6.4	83.1
W21 - y	3.2	6.4	6.4	6.4	6.4	6.4	6.4	3.2	41.5
W22 - y	3.2	6.4	6.4	6.4	6.4	6.4	6.4	3.2	41.5
W23 - y	6.4	12.8	12.8	12.8	12.8	12.8	12.8	6.4	83.1
W24 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W25 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W26 - y	2.4	4.8	4.8	4.8	4.8	4.8	4.8	2.4	30.9
W27 - y	6.4	12.8	12.8	12.8	12.8	12.8	12.8	6.4	83.1
W28 - y	3.9	7.8	7.8	7.8	7.8	7.8	7.8	3.9	50.6
W29 - y	2.4	4.8	4.8	4.8	4.8	4.8	4.8	2.4	30.9
W30 - y	1.9	3.9	3.9	3.9	3.9	3.9	3.9	1.9	25.3

* NB First floor loads only considered for timber-concrete shear interface at 1st floor, not in the cumulative column

Timber framed wall actions - earthquake

Consider the detiled design of a single sheathed bracing (shear) wall. In this case select a non-load bearing wall such that tie down will be most onerous.



1) Determine resultant actions at wall

H_i 3.1m Floor to floor height
L_s 5.4m W16 Panel Breadth

Earthquake actions

Shear face *V**

	F (kN)	Cumulative (kN)
R	24.0	24.0
7	23.9	47.9
6	19.9	67.9
5	16.0	83.8
4	12.0	95.8
3	8.0	103.8
2	4.0	107.8

Tie-down/bearing *T*/c**

T=C (kN)	Cumulative (kN)
13.8	13.8
13.7	27.5
11.4	39.0
9.2	48.1
6.9	55.0
4.6	59.6
2.3	61.9

G	1.7	kPa
A	0.0	m^2

2) Check for tension The wall is a stability wall but is non-loadbearing

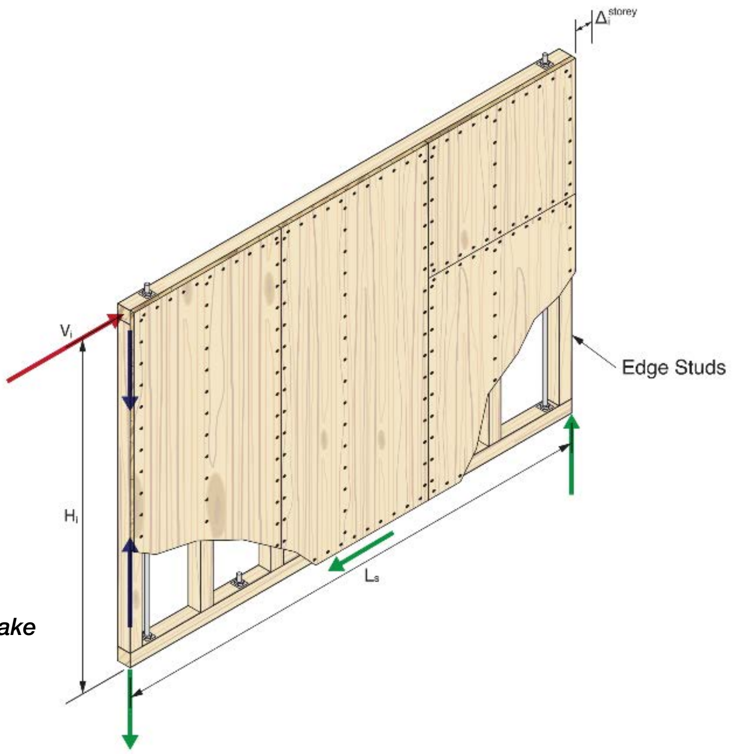
	G (kN)	0.9G - Fw
R	0	-13.8
7	0	-27.5
6	0	-39.0
5	0	-48.1
4	0	-55.0
3	0	-59.6
2	0	-61.9

3) Critical load cases

L2 - Timber/Timber Critical Connections
*V** 103.8 kN
*T** -61.9 kN

Transfer - Timber/Concrete Critical Connections
*V** 107.8 kN
*T** -61.9 kN

Timber-framed shear wall



Sheathed bracing wall dimensions

L_s	5400 mm
H_i	3100 mm

1) Calculate actions

*V** (F)	83.1 kN	Wind cumulative (lowest storey)
*V** (F)	108 kN	Earthquake cumulative (lowest storey)

2) Wall design capacity

Siesmic (strength limit)

Overall wall design checks

Determined from EC5 model	60.2 kN
Therefore two leaves	120.4 kN

Design capacity > applied loads	120.4>108

Capacity is sufficient. This takes no consideration of Servicability Limit States as the design is for earthquake loads, and therefore to different movement criteria.

Consider plywood sheathing shear capacity

Plywood thickness		7 mm	
Grade F11	f'_s	5 MPa	AS1720.1 Table 5.1
	V_i*	20.0 N/mm	
	ϕ	0.85	AS1720.1 Clause 2.3
	k_1	1	AS1720.1 Table 2.3
	k_{19}	1	AS1720.1 Table 5.2(A)
	g_{19}	1	AS1720.1 Table 5.4
Shear area 2dt/3	As	25200 mm^2	
	k_{12}	See below	AS1720.1 I2.2.2
	g_{60}	0.31	AS1720.1 Table I1, 3 ply, and theta = 90
Stud spacing	d_w	600 mm^2	
	S	26.6	
	k_{12}	0.75	
Therefore	V_{di}	80.2 kN Per leaf, i.e. 160.5 kN total therefore sufficient	

Wind (stiffness limit on strength limit state)

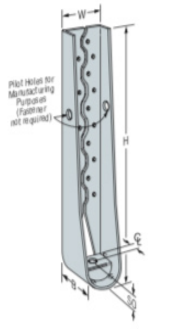
Determined from CSA O86 model	42.1 kN
Therefore two leaves	84.3 kN

Design capacity > applied loads	84.3>83.1 kN

Capacity is sufficient. This takes consideration of Servicability Limit States through the limit of Strength Limit State design capacity of the wall.

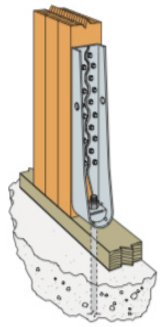
Timber-framed shear wall

Consider tie-down capacity into 90x90 mm studs



HDU8-SDS2.5
U.S. Patents
6,112,495
5,979,130

For example			
Simpson Strong Tie HDU8-SDS2.5		36.2 kN Design capacity	
NB bolt dia min 20 mm			
OK for timber-timber or timber-concrete			
Net tension for non-loadbearing (earthquake)		-61.9 kN for both leaves	
		-30.9 kN per leaf	
Design capacity > applied load		36.2 > -30.9 kN	therefore ok
Check threaded rod capacity	dia.	20 mm	
	Area	314 mm^2	
Tensile stress		98.5 N/mm^2	ok by inspection



Vertical HDU8-SDS2.5 installation

Consider panel base shear

Base shear (earthquake)		107.8 kN for both leaves	
		53.9 kN per leaf	
at timber-concrete interface		10.0 kN/m	

Say M16/16mm diameter Ramset concrete screws

Bottom plate thickness		35 mm	
Bottom plate grade		MGP10	
Joint group		JD5	
Characteristic capacity		7800 N	
	ϕ	0.65	AS1720.1 Table 2.2
	k_1	1.14	AS1720.1 Table 2.3
	k_{16}	1	AS1720.1 Clause 4.4.3.2
	k_{17}	1	AS1720.1 Table 4.12
	n	N/A	
Design capacity		5779.8 N	
Therefore		1.7 per metre	
		579.3	
	say	**550 mm cts**	
Note that if bottom plate is		MGP12	
		45 mm thick	
Then		JD3	
Characteristic bolt capacity		12800 N	
Design bolt capacity		9484.8 N	
	and spacing	951 mm	
	say	**900 mm cts**	
Fastener shear capacity			
Ramset AnkaScrew	hole dia	16 mm	
	embedment	120 mm	
		26.6 kN 'Indicative working load' in shear	

Therefore Timber performance governs

Check slab edge distances for bracing near first floor perimeter, and check fastener spacing into the concrete

Next steps

This presents one option for tie-down and shear. There are many other options. Consider other suppliers of proprietary fasteners. There are many more options if forces are lower. Consider the shear wall to shear wall connection for the upper floors. Possible screwed or rod connectors. Carry out stud design checks under axial loads associated with stability actions along with other actions.

Ensure the engineering outputs (drawing notes, specification etc) mention methods for dealing with tolerances, particularly in order to achieve a level interface between the concrete and timber structure.

Timber-framed global overturning

Global Overturning

Wind moment: 13,942 kNm on wide face (wind in Y)

Consider the central portion of the building (highlighted orange) as a pseudo core. In this area the stability elements are likely to be consdiered as working together globally to resist over-turning by:

- stiff diaphragm linking shear walls
- overlap of shear walls in plan
- multiple internal walls that will contribute to the rigidity of the system
- floor cassette spans that 'bridge' some of the discontinuities between shear walls
- stiffness and strength of the external walls (highlighted) is assumed sufficient to distribute global tie-down loads along the length.

Consider global overturning resisted by the external walls highlighted orange above.

Building depth		22.5 m	
Push/pull (tension/comp)		619.7 kN	
Width of external wall activated		14.0 m	
Therefore push/pull		44.3 kN/m	
Stud dead load at FL1	G	61.6 kN/m	See stud design worked example

Consider combination

0.9G + W

Therefore net tension	- kN/m	COMPRESSION

There is no net tension under the wind case.

Therefore the design of the walls for local shear-wall forces and out-of-plane wind loads is sufficient to also resist global tie-down actions.

Detail connection between shear walls and load bearing walls in the envelope to carry the shear associated with the tie-down requirement.

Timber-framed global overturning

Global Overturning

Consider the additional compressive load into the envelope studs due to the global overturning.

Refer to stud design worked example for detail:

Stud spacing	450 mm
Compression due to overturning wind load	44.3 kN/m
Load per stud	19.9 kN

Consider combination $1.2G + W + \psi_c Q$

External wall studs	G	Q	W	ψ_c
		kN/stud		
Storey FL7	2.84	1.96		
Storey FL6	3.55	2.03		
Storey FL5	3.55	2.03		
Storey FL4	3.55	2.03		
Storey FL3	3.55	2.03		
Storey FL2	3.55	2.03		
Storey FL1	3.55	2.03		
Total	**24.1**	**14.1**	**19.9**	**0.4**

$1.2G + W + \psi_c Q$ 54.52 kN per stud

in this case	k_1	1	AS1720.1 Table 2.3
Stud design worked example includes stud design for			
Normal loads		50.2 kN per stud	
	where k_1	0.80	AS1720.1 Table 2.3
Long-term loads		36.3 kN per stud	
	where k_1	0.57	AS1720.1 Table 2.3
Equivalent stud design load for $k_1 = 1.0$		> 60 kN per stud	

Therefore studs are okay to resist additonal axial load from global overturning

Next steps:

The global tie-down based on the assumptions stated is sufficient. However, roof tie down will be required. The most robust solution to tie-down is to tie the perimeter with continuous threaded rod to the foundation, which would also improve the performance of general over-turning. This approach should be considered.

Futher checks include

- consider combined out-of-plane bending with global over-turning wind as appropriate
- ensure connections between shear walls and external walls are sufficient to distribute global tie-down forces.

Timber-framed diaphragm

The distribution of the forces into the lateral load-resisting system depends on the flexibility of the diaphragm (rigid or flexible). A diaphragm is considered to be flexible if its deformation is more than twice the average inter-storey drift at that level. For flexible diaphragms, the load can be determined by the tributary area approach.

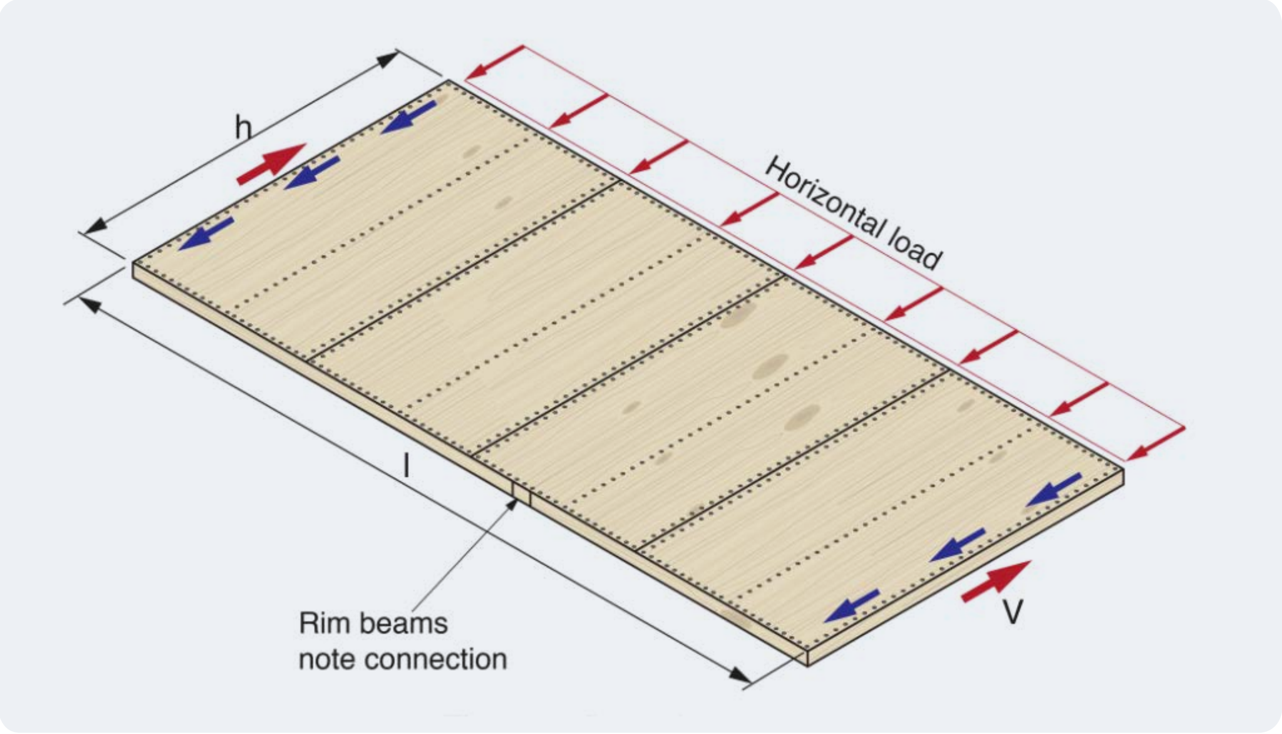
The *Manual for the Design of Timber Building Structures to Eurocode 5* published by the Institution of Structural Engineers suggests the following criteria be met in order to assume a timber-framed diaphragm is 'stiff' and will have sufficient strength.

- Characteristic wind pressure less than 1.5kPa (approximately equivalent to 2 kPa W_u to AS).
- Span:depth less that 2:1 in any wind direction.
- Span less than 12m between stability walls.
- Sheathing panel is minimum 15mm thickness for plywood or OSB, and 18mm for particleboard.
- Panel-to-panel junctions are nailed to a common joist, rafter, or batten.
- Minimum fasteners to be 3.1mm dia. ringed shank machine driven nails or 4mm dia. wood screws, length 2.5 times panel thickness, at max 150mm cts on panel edges.
- Maximum fastener centres within the panel to be 300mm.

With more investigation the engineer may consider this approach valid for the design of timber-framed diaphragms to AS 1720.1 within typical timber-framed construction practices.

Span: depth in the case of the worked example building is 1.5.

Floor diaphragms for the worked example building satisfy the criteria for a stiff diaphragm. This is the case for the majority of timber-framed mid-rise residential buildings with reasonably proportioned geometry and internal spaces. Refer the design guide for further information.



Timber-framed diaphragm

In order to verify the strength performance of the diaphragm, Eurocode 5 model for diaphragm strength checks can be used considering inputs based on AS1720.1 properties.

The conditions for application of the model are:

Limitations:

- l < 6b where l and b
- all panel edges are fastened either to joists or perimeter framing, or else to neighbouring panels, in accordance with guidelines set out in *Manual for the Design of Timber Building Structures to Eurocode 5*
- supporting battens are double skew-nailed at each end to a joist or the perimeter framing is provided as a means for the transfer of the tensile force across joints in rim beams.

The design checks assume the diaphragm is working as a deep I-beam in plan.

Three verifications are needed:

1. Shear strength of the 'web'.
2. Axial strength (tension governs) of the 'flanges' including connections.
3. Shear strength of the web connections.

1. 'Web' shear strength

Verify that

$$V_{d,i} \geq V_i^*$$

$$V_{d,i} = \varphi k_1 k_{12} k_{19} g_{19} f_s' A_s$$

where

Diaphragm depth	b	22.5 m	
Joist spacing		450 mm	
Plywood thickness		17 mm	
Grade F11	f'_s	5 MPa	AS1720.1 Table 5.1
	E	10500 MPa	AS1720.1 Table 5.2
	V_i*	69.2 kN	Total wind load at a typical storey assumed shared between two points (conservative)
	ϕ	0.85	AS1720.1 Clause 2.3
	k_1	1	AS1720.1 Table 2.3
	k_{19}	1	AS1720.1 Table 5.2(A)
	g_{19}	1	AS1720.1 Table 5.4
Shear area 2dt/3	A_s	255000 mm^2	
	k_{12}	See below	AS1720.1 I2.2.2
	g_{60}	0.3	AS1720.1 Table I1, 5 ply, and theta = 90
	s	7.9	AS1720.1 clause I2.2.1
	k_{12}	1.0	
	V_{di}	1083.75 kN > 69.2 kN	

Therefore okay

2. Axial strength of 'flanges'

Assuming diaphragm simply supported over full length

	l	34 m	Diaphragm length
Total load		138 kN	Wind load at a typical storey
		4.07 kN/m	as equivalent udl
diaphragm moment		587.8 kNm	conservatively assuming cantilvering from ‘stiff’ centre
Flange axial force		17.29 kN	Tension or compression in rim beams

Note that in this case the rim beams are perpendicular to the joist span. These are therefore trimming/blocking members with multiple connections. Consider likely connection required to transmit the axial force. Say cassette comprises: 240 mm deep truss joists with 190 mm deep trimmer/strong-back in MGP12 or better.

Rim beam grade	MGP12
Rim beam joint group	JD4

Consider using proprietary 'tie-down' as one possible option between aadjacent cassettes.

Say Pryda MPCPCH with screws into strong-back
Tie-down brackets horizontally mounted
M12 rod connects adjacent panels

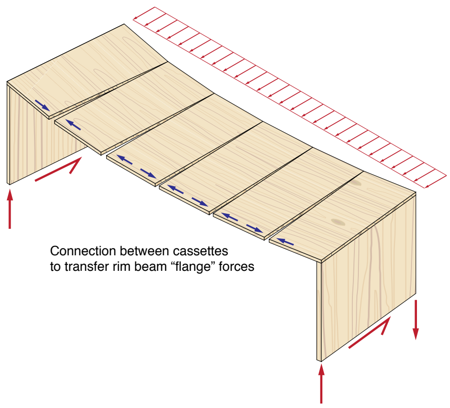
Design capacity	15 kN from product data

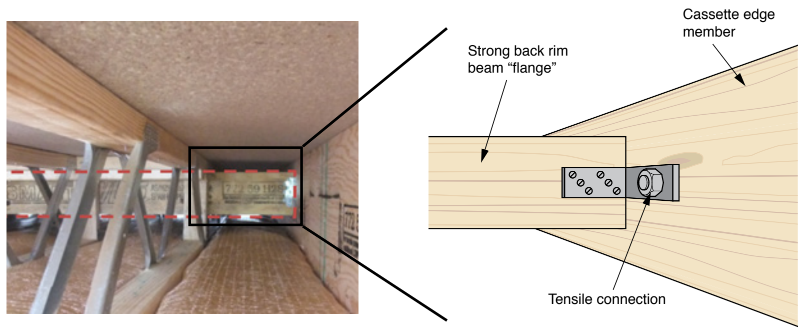
Check axial tension in rim beam

t_{member}	45 mm
d_{member}	190 mm

Tensile stress	1.75 MPa

Okay by inspection.





Timber-framed diaphragm

3. Shear strength of 'web' connectors

$$V_{d,i} \geq V_i^*$$

$$V_{d,i} = 2b\left[\frac{N_{d,j}}{s}\right]$$

The above formula is taken from the model adopted in EC5 and explained in the *Manual for the Design of Timber Building Structures to Eurocode 5* for strength design checks on floor diaphragms.
In this case the notation has been changed to align with AS1720.1 notation.

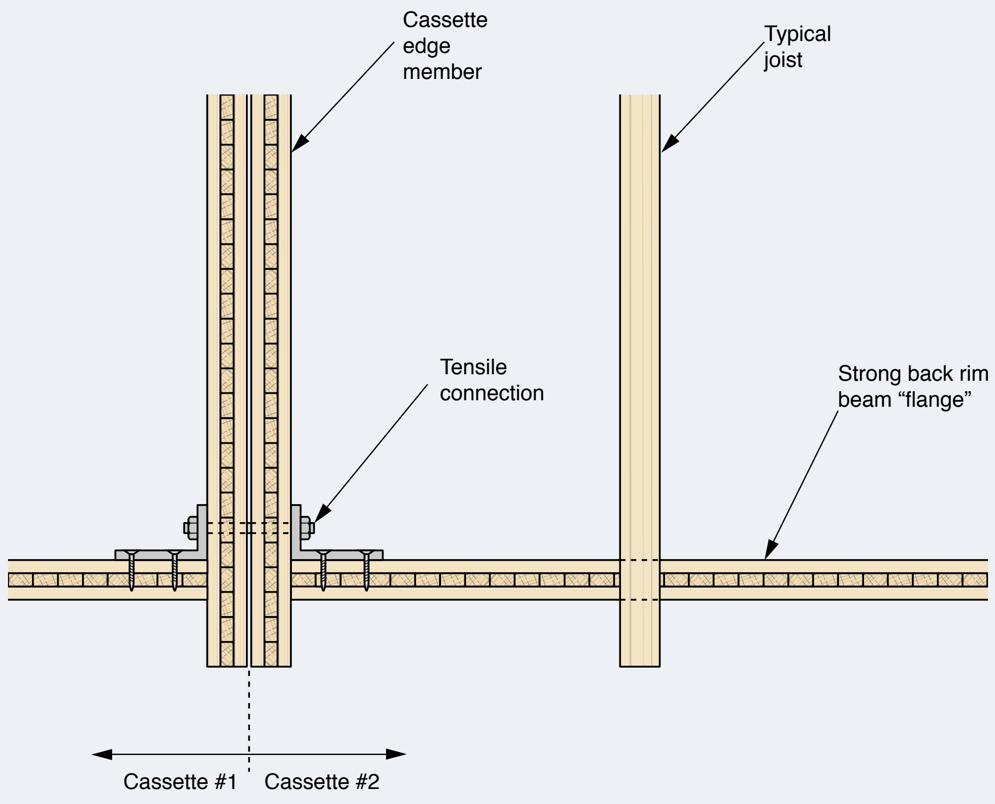
Consider no.10 screws at 150mm cts.

s	150 mm	
Q_k	1420 N	AS1720.1 Table 4.5(B), conservatively say JD5
ϕ	0.8	AS1720.1 Clause 2.3
k_1	1.14	AS1720.1 Table 2.3
k_{13}	1	AS1720.1 Clause 4.3.3.2
k_{14}	1	AS1720.1 Clause 4.3.3.2
k_{16}	1	AS1720.1 Clause 4.3.3.2
k_{17}	0.85	AS1720.1 Table 4.3 (A)
n	6	fasteners per m
N	1101N per fastener	AS1720.1 Clause 4.3.3.2

Therefore shear strength of 'web' connectors

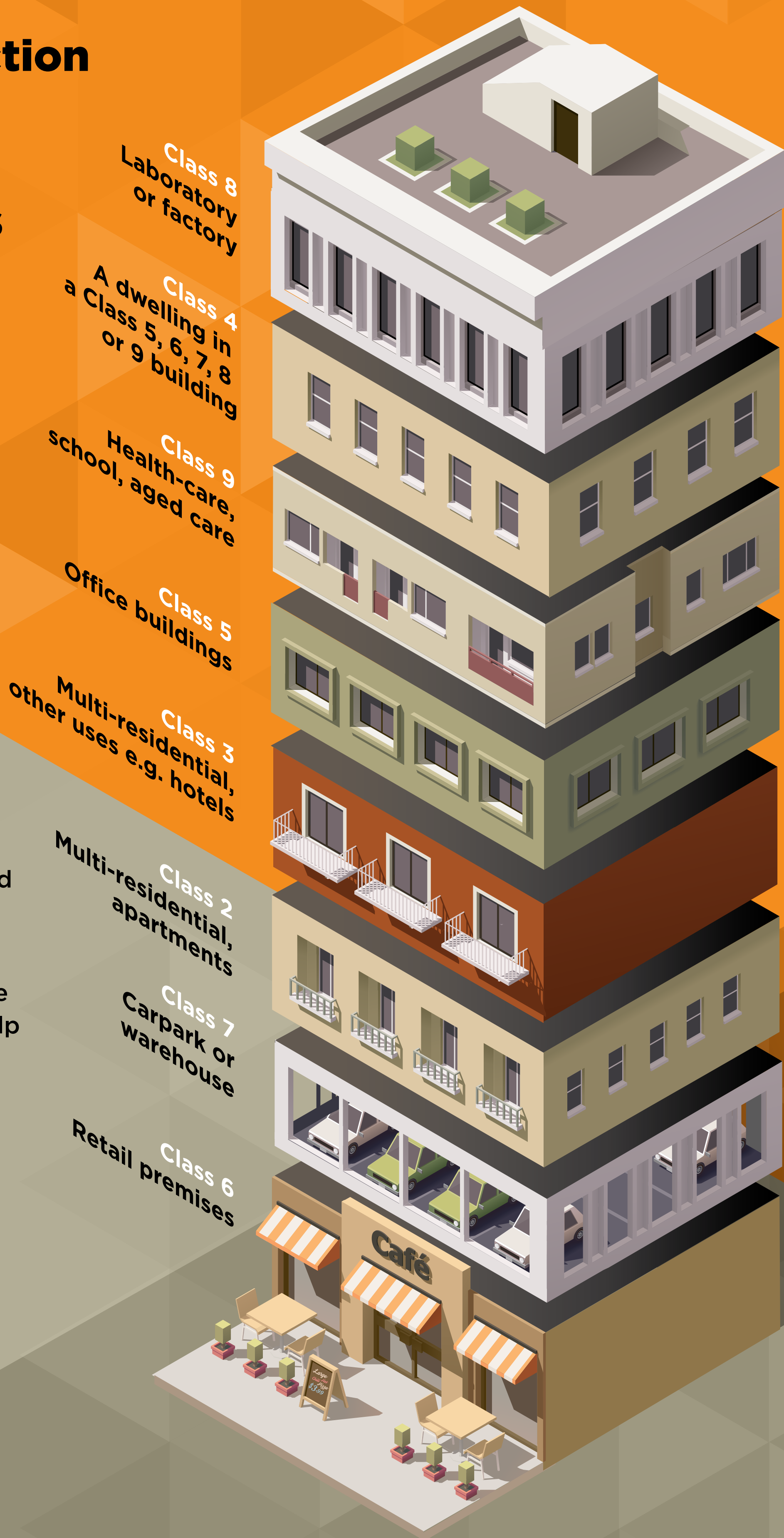
426 kN > 69.2 kN

Therefore okay.



A Class Act.

Changes to the National Construction Code mean that more classes of mid-rise buildings can deliver the benefits of wood.



Now more projects with an effective height of up to 25m or 8 storeys, can more easily gain the environmental, economic and safety benefits of construction with timber systems.

Plus, the WoodSolutions Mid-rise Advisory Team is available to help with free advice on optimising the design and build process.

You can discover more about designing and building with wood – whatever the size or purpose of your project - at woodsolutions.com.au.

WoodSolutions™
design and build
woodsolutions.com.au

50 A2

WoodSolutions™
design and build
woodsolutions.com.au

This guide was published in Dec 2019
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Mid-rise Timber Building Structural Engineering

Appendix 2: Worked Example for a CLT Mass Timber Panel Apartment Building

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Cover image: Latrobe University Student Accommodation Multiplex

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-75-1

Researchers:

Adam Jones
TPC Solutions (Aust) Pty Ltd
Level 2, 394 Little Bourke St
Melbourne, Vic, 3000

Nick Hewson and Jeremy Church
XLam
4-10 Bank Place

First Published: December 2019

Contents

Introduction: Worked Example Scope - CLT Mass Timber-Panel Mid-Rise Structure

The building utilised for this worked example is the WoodSolutions mid-rise model apartment building. It was designed to provide a basis for defining, comparing and presenting different mid-rise timber-based construction solutions and it is utilised in a range of different WoodSolutions mid-rise resources, where it provides a prototypical situation for modelling spatial, loading, fire and noise resistance conditions. WoodSolutions have referred to this building in several design guides.

More information on the mid-rise model apartment building can be found in *Design Guide #27 Rethinking Apartment Building Construction*. In this Appendix it is used to illustrate some of the structural analysis methods and approaches which may be adopted in the design of a mid-rise timber building using only Cross Laminated Timber (CLT) panels, optimising it for a specific product.

The worked examples:

- Provide guidance to the structural engineer to assist in understanding some of the key steps required in the design of timber mid-rise buildings but are not a full set of structural computations as will be required in a real project.
- Discuss key decision points and assumptions. The calculation steps are provided with a narrative to highlight the key steps. Simple calculation steps are not presented, nor are detailed steps copied from design standards.
- Are based on one set of engineering assumptions that may or may not be valid for a particular design.
- Present one possible approach to the engineering design. It is not intended that all the calculations necessary in design are presented here, or that the methods are the only valid methods.
- Make simplifying assumptions on architecture. The engineering design has not been completed as part of an iterative design process, and so broad assumptions on what is feasible have been made.

Had this worked example been completed as a real project, an iterative design process would have been utilised, and feedback to the client and design team would have been provided in order to inform discussion, e.g. on construction time, a particular facade finish, deflection criteria from the lift manufacturer, acoustic or thermal efficiency requirements beyond the code compliance.

As far as reasonably practical, hand-based methods of analysis have been used. Reference is made to computer analysis where appropriate. It is the intention of the worked examples to present the thought behind the process, which is best highlighted through hand calculations.

The worked examples in Appendices 1 & 2 present element design for both lightweight timber-framed construction and CLT construction respectively. The calculations of the different element types follow a similar format and so some crossreferencing is possible. But the systems are designed to different loads and assumptions and so cannot be easily swapped like-for-like.

Overview of the Design Process - Mass Timber Panel Mid-Rise Structure

As discussed in Section 1.2.2, while design of a building in practice is an iterative process, it does generally follow the following the three phases and design steps. This worked example illustrates computations for the steps identified below.

1

Phase 1: Preliminary Design

Step 1: Building layout and performance considerations – Early Contractor Involvement (ECI):

Project team to discuss and determine:

- overall building design and layout
- performance requirements for: structure, fire, acoustics, robustness, etc.

Step 2: Preliminary structural design – Early Supplier Involvement (ESI)

Engineer to provide preliminary advice on:

- the structural approach used,
- structural element layout (roof trusses, floors, walls, cores, bracing walls, etc),
- initial member sizing information for 'preliminary costings' to be developed,
- non-timber elements: concrete basement, transfer structures, etc.
- also consider as part of ESI process: the construction sequence; limits of transportation, lifting and manual handling; site restrictions; temporary bracing and propping required; building services and voids required.

2

Phase 2: Detailed Design

Based on decisions agreed in the preliminary design phase the actual design loadings can be determined. The major structural design steps are:

Step 3: Vertical load – Roof and floor cassette design

Step 4: Vertical load – Wall design

Step 5: Vertical movement design

Step 6: Lateral load – Stability design

Step 7: Check robustness

Step 8: Other details for consideration

Step 9: Engineering drawings and documentation for certification

3

Phase 3: Fabrication & Assembly

Step 10: Engineered timber systems fabrication (shop drawing review)

Step 11: On-site construction assembly supervision - certification

WoodSolutions Mid-rise Model Apartment - Building Overview

The WoodSolutions mid-rise model apartment building was designed to meet high-end consumer needs, including large and open room layouts. An emphasis was placed on characterising a building that could apply to many suburban/urban apartment situations across Australia. The model apartment typical floor plan is shown Figure A2.1, the building has a breadth of 34 m and a depth of 22.5 m, providing a 765 m² floor plate per level. Visualisations of the model apartment are provided in Figure A2.2. A section through the model apartment are provided in Figure A2.3. A summary of the design approaches utilised including relevance and reasons is provided in Table A2.1.

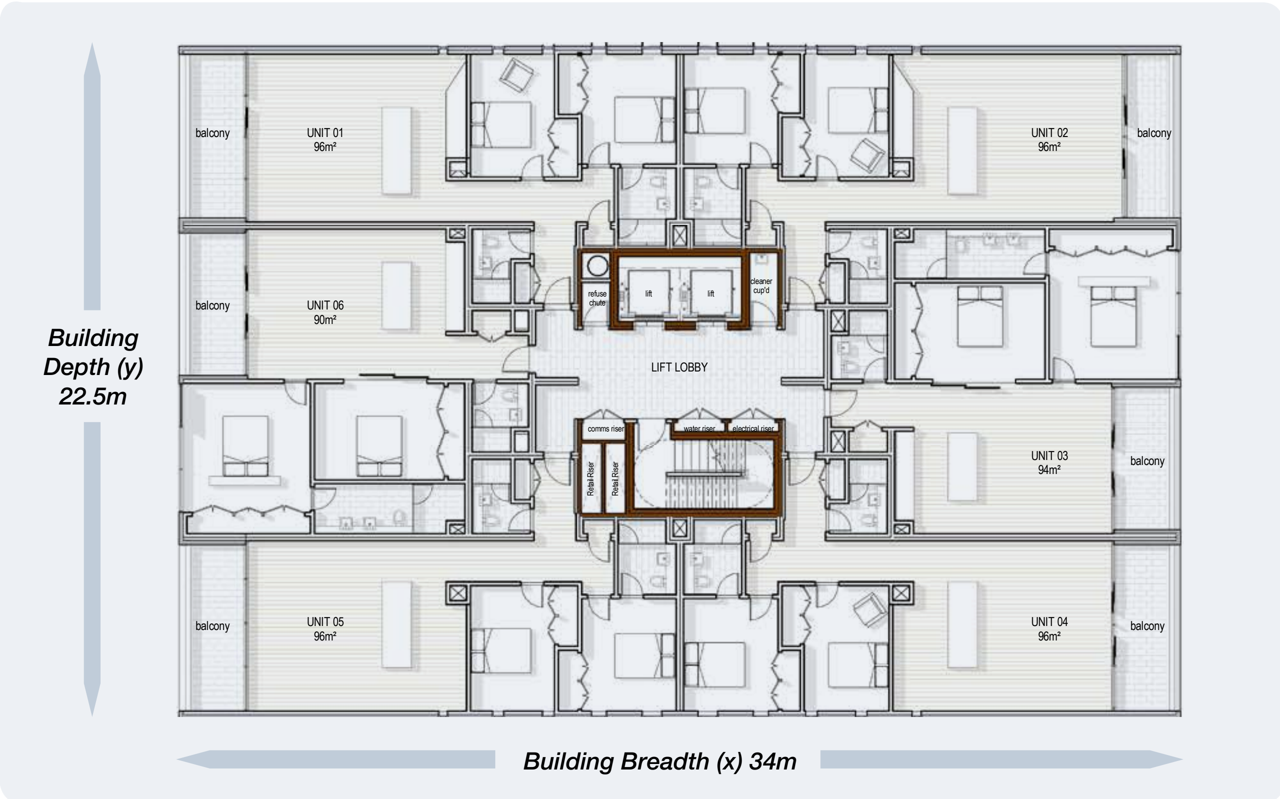


Figure A2.1: WS model apartment, typical floor plan.

Figure A2.2: WS model apartment visualisations.

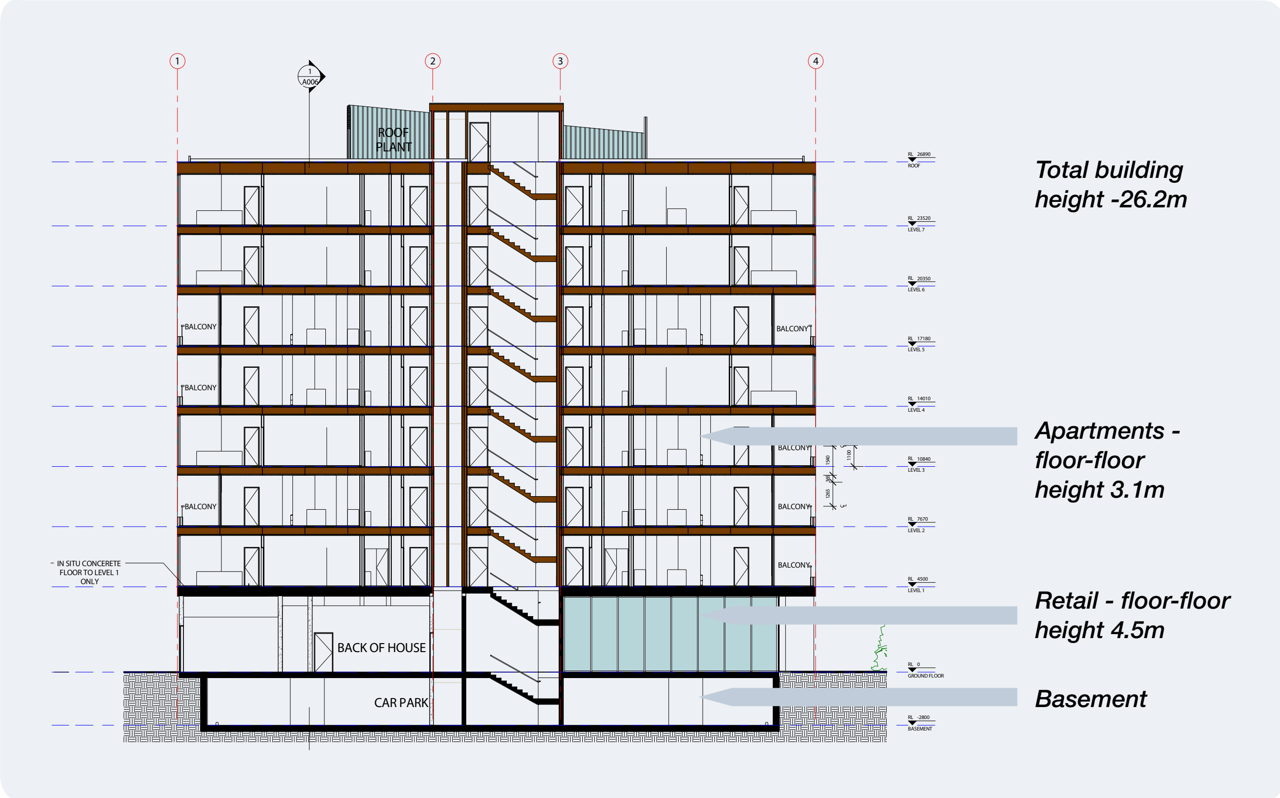


Figure A2.3: WS model apartment section.

Table A2.1: WS model apartment: Summary of design approaches utilised including relevance and reasons.

Item	Design Approach	Relevance and Reasons
Height	• 8-storey design height above ground level, including 7 apartment levels and 1 retail level. • A 26.2 m overall building height with an NCC effective height of 23.1 m. • A 3.1 m floor-to-floor height for the apartment levels and 4.5 m floor-to-floor height for the retail level.	• The apartment levels provide a habitable height plus depth for the structure and services within minimum NCC limits. • The retail level provides for a maximum depth 500 mm thick transfer slab above, i.e. used to transition loads from the timber to concrete parts of the building.
Area	• A floor plate area of 765 m^2. The apartment levels include 42 SOUs (94–96 m^2 each). • The retail level assumes three shops varying in area from 77–150 m^2. It also includes a foyer area, an entrance to basement car parking, utility meter rooms, an electrical substation and a waste area.	• Feedback and analysis indicate that many suburban mid-rise apartment buildings fit this scenario.
Key set out criteria	• Plant room, lift shaft or machinery room, public corridor, public lobby.	• The width of the building accommodates the size and set-out of the large, high-end apartments. • The grid layout accommodates car parking in the basement.
Building ownership and fire separation	• The building is considered to be strata titled, including the retail area on the ground floor.	• Strata title creates the need for each title to be defined as a separate Sole Occupancy Unit under the NCC, which creates fire and noise performance requirements.
Setbacks	• External wall distances are (at minimum) less than 1.5 m from the property boundary.	• The location of the building relative to other buildings or properties affects the façade fire resistance requirements.

Phase 1 Preliminary Design

Step 1: Building layout and performance considerations – Early Contractor Involvement (ECI)

The architect determines the massing, orientation, floor plan, façade design and building area functions, see Figures A1.1 – A1.3 and Table A1.1 for general specifications and details.

Determination of Performance Requirements and Preliminary Structural Advice

The architect and design team (including acoustic & fire consultants) have determined the following:

- FRL: Class 2 building, 26.2 m, 8-storeys ➔ Type A construction, therefore, requires an FRL of 90/90/90; therefore
  - walls and ceilings to be lined with 1 layer of 16 mm fire-rated plasterboard
- Acoustic Performance:
  - Walls: Airborne, $R_w+C_{tr} \geq 50$, between apartments use double leaf discontinuous walls (see Fig A2.4)

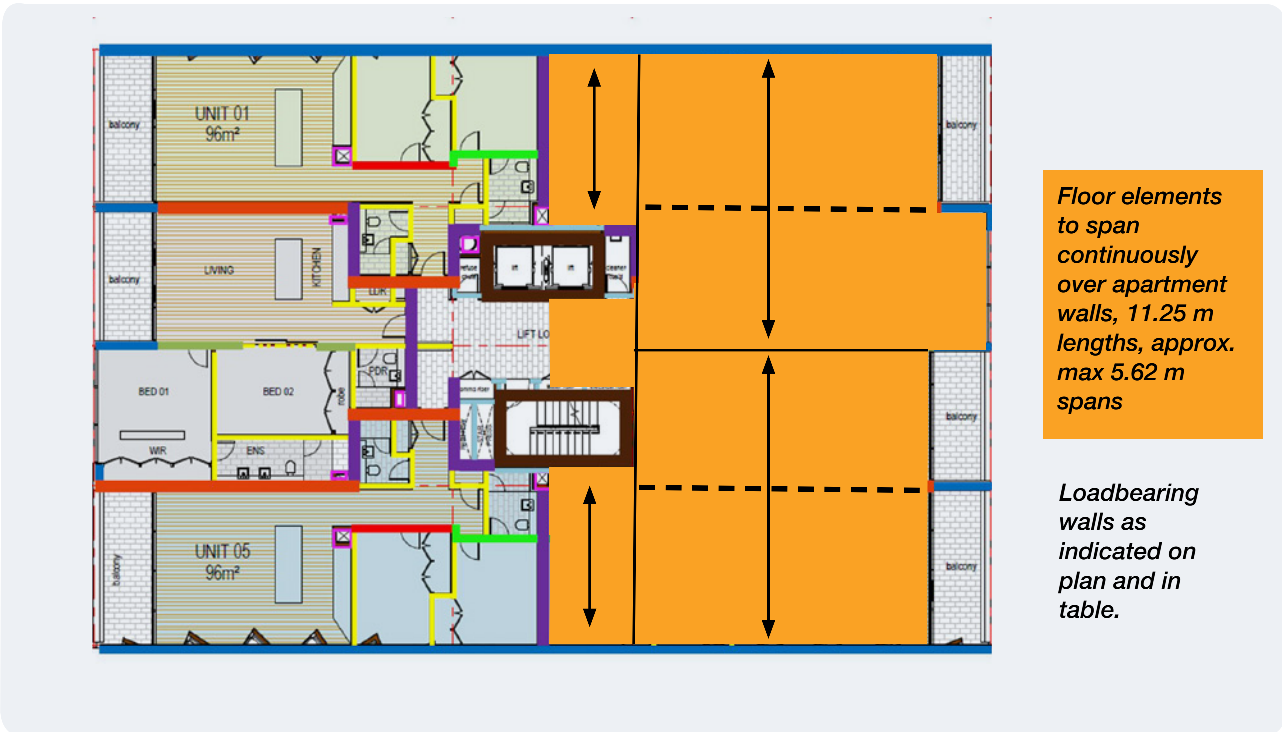


Figure A2.4: Floor plan showing proposed timber framed wall types and floor panel layout.

Floors: Airborne, $R_w+C_{tr} \geq 50$ (apartments), Impact $L_{n,w} \leq 55$ (AAAC 3 Star rating)

Proposed floor/ceiling system build up:

12 mm hardwood overlay flooring or carpet
Mass overlay, 2 x 18 mm Promat 40 kg/m² (see Table 3.2)
10 mm acoustic mat
225 mm deep CLT floor panel (approx. 6.0 m span)
1 x 16 mm fire-rated plasterboard
75 mm Glasswool insulation
185 mm suspended ceiling with resilient mounts
13 mm standard plasterboard
Total depth of floor = 497 mm (299 mm no suspended ceiling)

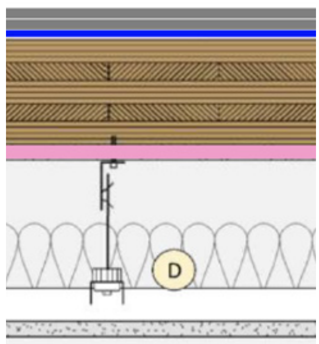


Figure A2.5: Floor build-up.

Preliminary floor member sizing

The walls between apartments are required to be discontinuous for acoustics, the CLT wall panel will be used as a loadbearing wall, and a second non-loadbearing stud wall leaf will be utilised to assist with acoustic performance.

The CLT floor panels will span continuously over the apartment walls, total length 11.25 m lengths, therefore approx. two 5.62m spans.

Preliminary sizing of floor cassettes can be obtained from

- Approximate span to depth ratio estimate (see section 3.5.1) for a 5layer panel: span: depth = 25-27, therefore 5,620/25 = 225 mm
- Tables in *DG# 46 Guide to Wood Construction Systems*, see Figure A2.6. Load multi-res, continuous, 225 mm thickness.

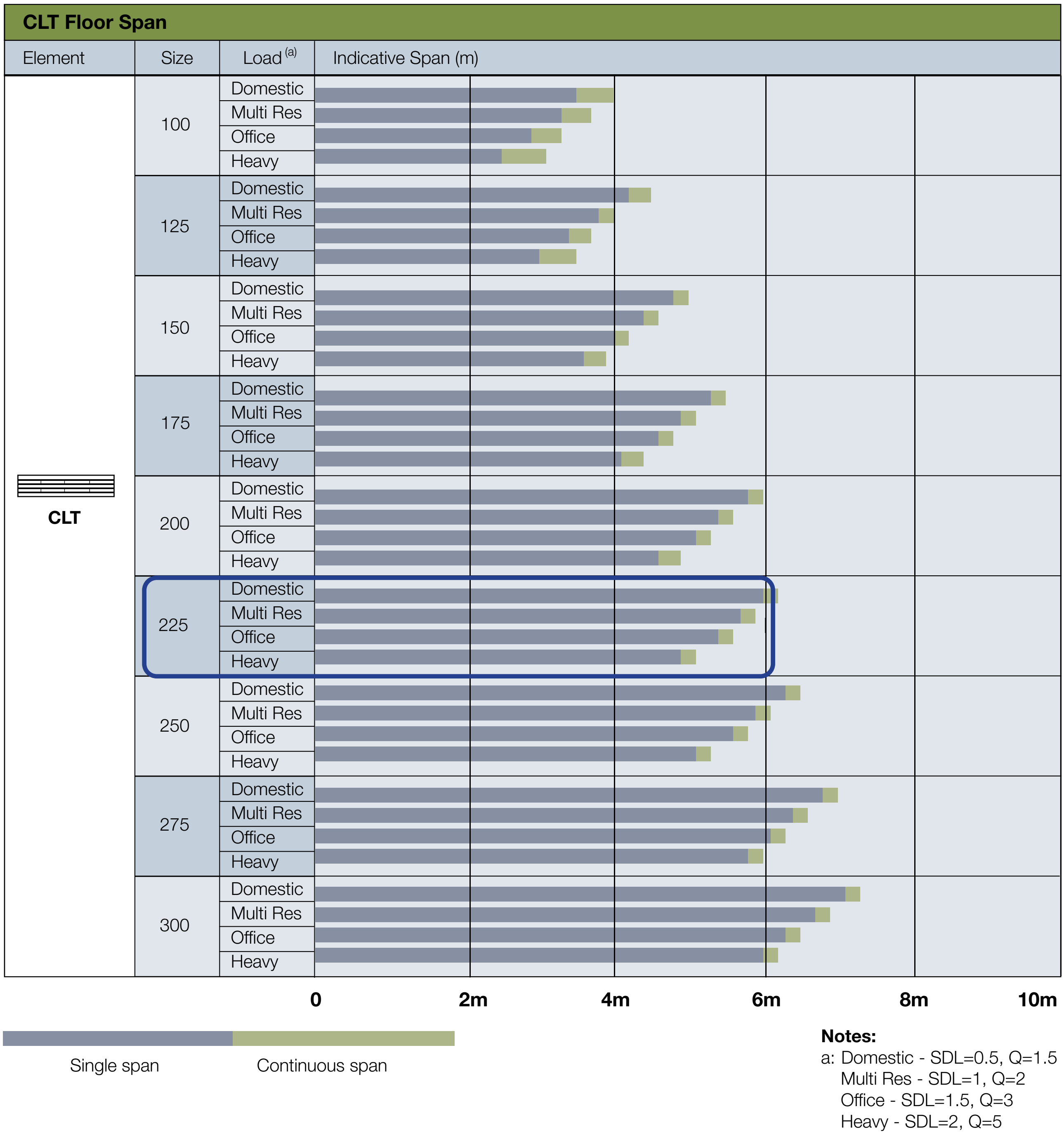


Figure A2.6: CLT floor panel size estimate using WSDG#46 CLT floor span table (Table 13).

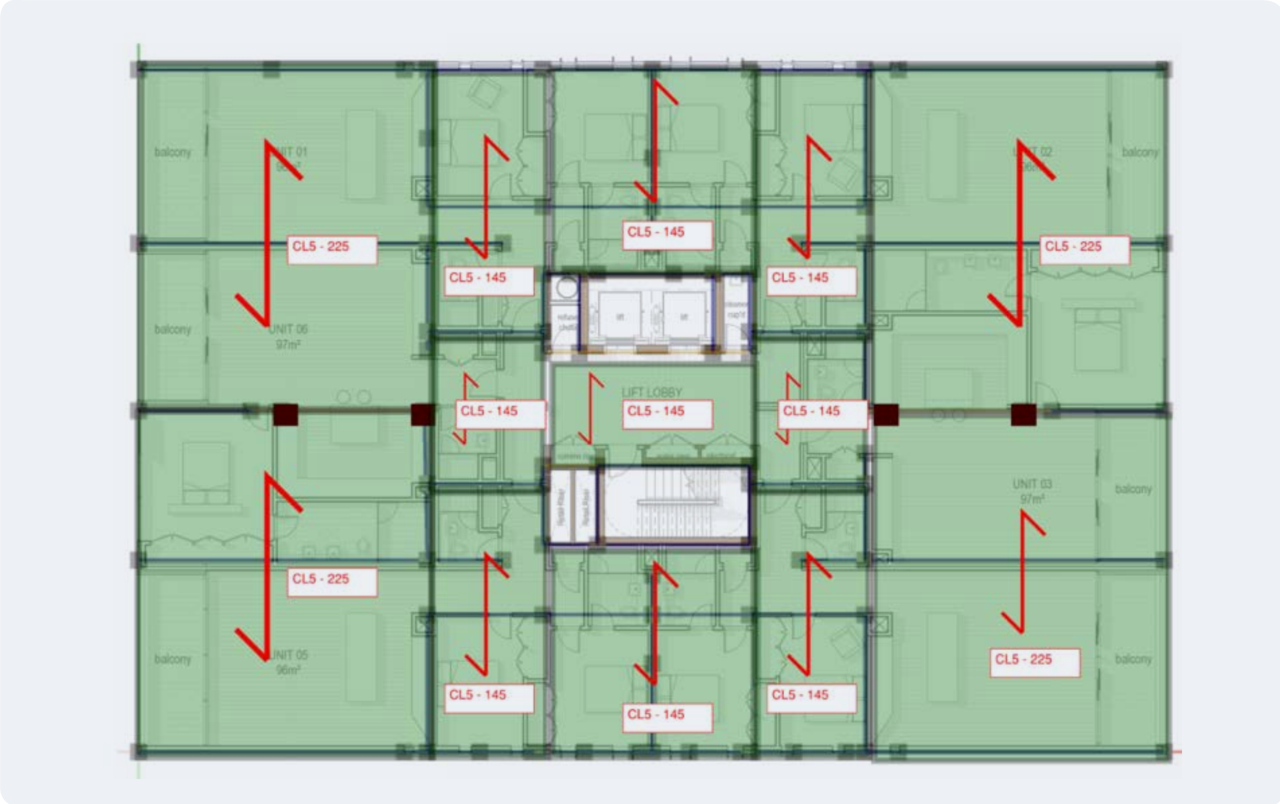


Figure A2.7: CLT floor panel size estimate using WSDG #46. Live load assumptions include: apartments 1.5kPa, balconies 2kPa and lift shaft lobby 4kPa.

Preliminary wall sizing (gravity loads)

The walls in the East/West direction will be taking both the gravity loads of the entire building, and lateral loads. The walls spanning in the North/South direction will be taking lateral loads only, and with the short lever arms tensile forces are most likely developed for these shear walls.

Figure A2.8: Wall breakup based on tributary breadths and restraint conditions.

Preliminary loading for the five different wall types yields the following loads for each wall by level:

G - Walls Loads	
2 x 16 mm fire-rated plasterboard	0.96 kN/m
Insulation	0.50 kN/m
CLT 115 Wall Self-weight (465 kg/m^3)	0.58 kN/m
Studs	0.10 kN/m
Services	0.3 kN/m
Other allowance (prelim stages)	0.5 kN/m
Total Wall Load	**3 kN/m**

G - Floor Loads	
2 mm carpet	0.96 kN/m
Acoustic layer	0.4 kPa
CLT 225 floor selfweight (465 kg/m^3)	1.05 kPa
13 mm fire-grade plasterboard	0.13 kPa
Frame work	0.05 kPa
75 mm glasswool batts (14 kg/m^3)	0.014 kPa
13 mm standard plasterboard	0.13 kPa
Internal walls (squashed load)	0.22 kPa
Other allowance (prelim stages)	0.50 kPa
Total	**2.6 kPa**

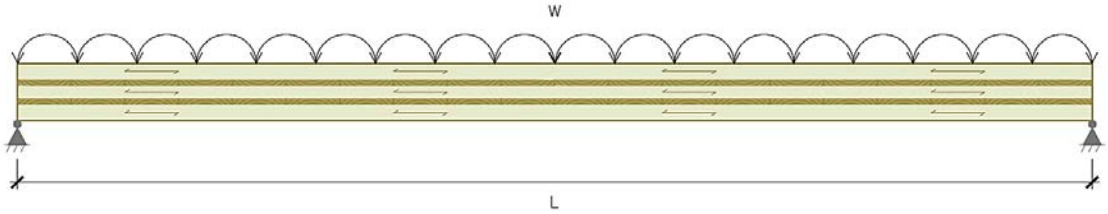
Q - Live Load	
Apartment	1.5 kPa

Table A2.2: WS model apartment: wall panel sizes based on suppliers' Design Guide.

	kN/m	WT1	kN/m	WT2	kN/m	WT3	kN/m	WT4	kN/m	WT5
Roof	20	CL3 - 85	39	CL3 - 85	34	CL3 - 85	41	CL3 - 85	4	CL3 - 85
7	40	CL3 - 85	79	CL3 - 85	68	CL3 - 85	83	CL3 - 85	7	CL3 - 85
6	59	CL3 - 85	118	CL3-105	102	CL3 - 85	124	CL3-105	11	CL3-105
5	79	CL3 - 85	158	CL3-105	135	CL3-105	166	CL3-105	14	CL3-105
4	99	CL3 - 85	197	CL3-105	169	CL3-105	207	CL3-105	18	CL3-105
3	119	CL3-105	236	CL3-115	203	CL3-105	248	CL3-115	22	CL3-115
2	139	CL3-105	276	CL3-115	237	CL3-115	290	CL3-115	25	CL3-115

CLT CL5/225 Calculate Deflection Using the Gamma Approach

For this worked example, the 'Gamma Method' is used to calculate the stiffness and hence deflection of the floor panel. Note the Gamma method calculates for simply supported conditions only. See Section 3 of the Guide for further details.



1. Geometry & Loading

Section 3.6 - Design fo CLT Floors

L	6000.00 mm	Gfloor	2.37 kPa
b_{eff}	1000.00 mm	LL	1.50 kPa

$$w_s = (G + 0.4Q)$$

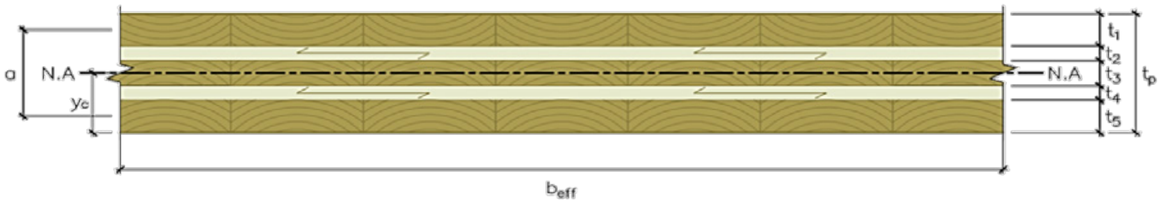
w_s 2.97 kN/m/m SLS

$$w_u = 1.2G + 1.5Q$$

w_u 5.09 kN/m/m ULS

2. Calculate Section Properties (Bending Stiffness)

3.6.2.2



	t_i (mm)	y_i (mm)	h_i (mm)	θ_i (deg °)	Grade	E_i (MPa)	G_i (MPa)
t_1	45	203	90	0	SG8	8000	533
t_2	45	158	45	90	SG6	200	40
t_3	45	113	0	0	SG6	6000	400
t_4	45	68	45	90	SG6	200	40
t_5	45	23	90	0	SG8	8000	533

Properties from XLam CLT Design Guide (Aus)

t_p 225.00 mm Total panel thickness

$$y_i = \sum_{n=i+1}^{5} (t_n) + \frac{t_i}{2}$$

$$y_c = \frac{\sum_{i=1}^{5} t_i E_i y_i}{\sum_{i=1}^{5} t_i E_i}$$

y_c 112.50 mm CLT section centroid (from bottom of panel)

$$h_i = |y_i - y_c|$$

$E_{90} = E_0/30$ $G_0 = E_0/15$ $G_R = G_0/10$

	$b_{eff}\ t_i^3/12$ (mm⁴)	$b_{eff}\ t_i\ h_i^2$ (mm⁴)	E_i (MPa)	E_iI_i (Nmm²)
t_1	7.59E+06	3.65E+08	8000	2.98E+12
t_2	-	-	-	-
t_3	7.59E+06	0.00E+00	6000	4.56E+10
t_4	-	-	-	-
t_3	7.59E+06	3.65E+08	8000	2.98E+12
				Σ = 6.00E+12

$$I_i = \frac{b_{eff}\ t_i^3}{12} + b_{eff} t_i h_i^2$$

$$EI_{eff} = \sum_{i=1}^{5} E_i I_i \quad = 6.0\text{E+12 Nmm}^2$$

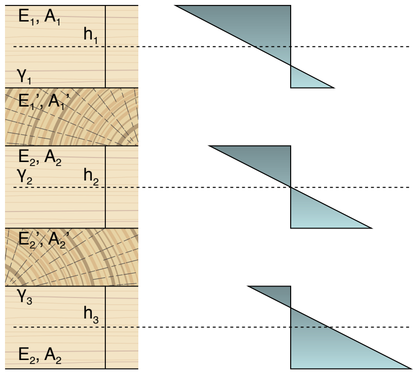
CLT CL5/225 Calculate Deflection Using the Gamma Approach

The structural performance of CLT is relatively unique due to the deformation caused by the crosslayers relative shear stiffness. This phenomenon, known as rolling shear reduces the effectiveness of the outer layers, and hence the section properties of the CLT panels.

The Gamma method will be used for this calculation, and is only applicable to 'Simply Supported' conditions. This is also known as 'mechanically jointed beam theory', where the middle transverse layer is considered a connection for the parallel layers.

3. Calculate Gamma for each layer *Eurocode 5*

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i}{G_R \cdot \frac{b}{d} \cdot l_{ref}^2}\right]^{-1}$$



	t_i (mm)	E_i	GZ (MPa)	A_i mm²	b/d	l_{ref}^2 mm⁴	γ_i
t_1	45.00	8000.00	-	4.5E+04	22.22	3.6E+07	0.90
t_2	45.00	200.00	40.00	-	-	-	
t_3	45.00	6000.00	-	4.5E+04	22.22	3.6E+07	1.00
t_4	45.00	200.00	40.00	-	-	-	
t_5	45.00	8000.00	-	4.5E+04	22.22	3.6E+07	0.90

4. Calculate Effective Stiffness for Panel

$$EI_{eff} = \sum_{i=1}^{n} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

	$b_{eff}\, t_i^3/12$ (mm⁴)	$b_{eff}\, t_i\, h_i^2$ (mm⁴)	γ_i	E_i	EI_{eff}
t_1	7.59E+06	3.65E+08	0.90	8000.00	2.69E+12
t_2	-	-	-	-	-
t_3	7.59E+06	0.00E+00	1.00	6000.00	4.56E+10
t_4	-	-	-	-	-
t_3	7.59E+06	3.65E+08	0.90	8000.00	2.69E+12
				$\sum = EI_{eff} = \sum_{i=1}^{n}\left(E_i J_i + \gamma_i E_i A_i a_i^2\right) =$	**5.42E+12**

5. Calculate Maximum Deflection

$$\Delta_{max} = \frac{5w(L)^4 j_2}{384 \times EI_{eff}}$$

j_2 2.00

Δ_{max} 18.51 mm

Maximum deflection at mid-span

L/Δ = 324 ≥ 300 therefore Okay

Although the span is continuous, the floor has been designed as simply supported at this early stage. It's considered potentially a good way to start designing as you can't be 100% certain how a chosen supplier. A conservative estimate for J2 is 2, although estimates by proHolz is 1.8 for this building type.

An alternative method for calculating CLT Stiffness is by using the Shear Analogy method, and is adopted in the USA/ Canada. In contrast to the Gamma Method, the Shear Analogy Method calculates the deformation due to bending and shear for each individual layer, for a range of boundary conditions. This method can account for a reduced stiffness of internal layers, as sometimes adopted by Australian CLT suppliers.

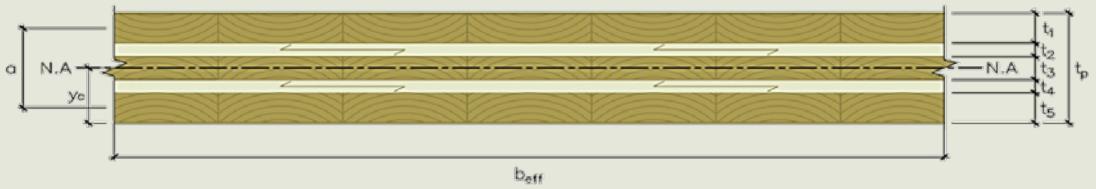
1. Geometry & Loading

L	6000.0 mm	G_{SDL+SW}	2.37 kPa
b_{eff}	1000.0 mm	LL	1.50 kPa

$$w_s = (G + 0.4Q)$$

w_s 2.97 kN/m/m SLS

2. Calculate Section Properties (Bending Stiffness) *FP Innovations CLT Handbook - 3.4.1*



	t_i (mm)	y_i (mm)	h_i (mm)	θ_i (deg °)	Grade	E_i (MPa)	G_i (MPa)
t_1	45	203	90	0	SG8	8000	533
t_2	45	158	45	90	SG6	200	40
t_3	45	113	0	0	SG6	6000	400
t_4	45	68	45	90	SG6	200	40
t_5	45	23	90	0	SG8	8000	533

Properties from XLam CLT Design Guide (Aus)

t_p 225.00 mm Total panel thickness

$$y_i = \sum_{n=i+1}^{5} (t_n) + \frac{t_i}{2}$$

$$y_c = \frac{\sum_{i=1}^{5} t_i E_i y_i}{\sum_{i=1}^{5} t_i E_i}$$

$$h_i = |y_i - y_c|$$

$$E_{90} = E_0/30 \qquad G_0 = E_0/15 \qquad G_R = G_0/10$$

	$b_{eff}\, t_i^3/12$ (mm⁴)	$b_{eff}\, t_i\, h_i^2$ (mm⁴)	E_i (MPa)	$E_i I_i$ (Nmm²)
t_1	7.59E+06	3.65E+08	8000	2.98E+12
t_2	7.59E+06	9.11E+07	200	1.97E+10
t_3	7.59E+06	0.00E+00	6000	4.56E+10
t_4	7.59E+06	9.11E+07	200	1.97E+10
t_3	7.59E+06	3.65E+08	8000	2.98E+12
				Σ = 6.04E+12

Properties and relationships will vary depending on CLT supplier

$$I_i = \frac{b_{eff}\, t_i^3}{12} + b_{eff} t_i h_i^2$$

$$EI_{eff} = \sum_{i=1}^{5} E_i I_i \quad = 6.0\text{E}+12 \text{ Nmm}^2$$

3. Calculate Section Properties (Shear Stiffness) *FP Innovations CLT Handbook - 3.3.2*

α 180.00 mm Distance between centroids of outer lamella

	t_i (mm)	G_0 (MPa)	G_R (MPa)
t_1	45.00	533.33	-
t_2	45.00	-	40.00
t_3	45.00	400.00	-
t_4	45.00	-	40.00
t_5	45.00	533.33	-

t_i /(2Gb_{eff}) (mm²/N)	t_i /(Gb_{eff}) (mm²/N)
4.2E-05	-
-	1.13E-03
-	1.13E-04
-	1.13E-03
4.2E-05	-

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{t_1}{2G_1 b_{eff}}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i b_{eff}}\right) + \left(\frac{t_n}{2G_n b_{eff}}\right)\right]}$$

GA_{eff} 1.32E+07 N Effective shear stiffness

4. Calculate Section Properties (Shear Analogy Method) *FP Innovations CLT Handbook - 3.4.1*

$$\Delta_T = \Delta_b + \Delta_s$$

$$where\ \ \Delta_b\ is\ deformation\ due\ to\ bending$$
$$where\ \ \Delta_s\ is\ deformation\ due\ to\ shear$$

$$\Delta_b = \frac{\kappa_b w L^3}{EI_{eff}} = \frac{5wL^4}{384EI_{eff}}$$

$$\Delta_s = \frac{\kappa_s w L}{GA_{eff}} = \frac{\gamma w L^2}{8GA_{eff}}$$

$$\therefore \Delta_T = \frac{\kappa_b w L^3}{EI_{eff}}\left(1 + \frac{K_s EI_{eff}}{GA_{eff}L^2}\right) = \frac{\kappa_b w L^3}{EI_{eff}}(\alpha + \beta)$$

$$where\ \ \alpha = 1, \beta = \frac{K_s EI_{eff}}{GA_{eff}L^2}\ \ \&\ \ K_s = \gamma\left(\frac{\kappa_s}{\kappa_b}\right), K_s\ is\ solved\ for\ several\ cases\ in\ Section\ 2.1.3\ CLT\ H/B\ (USA)$$

$$\therefore EI_{app} = \frac{EI_{eff}}{(\alpha + \beta)} = \frac{EI_{eff}}{\left[1 + \frac{K_s\, EI_{eff}}{GA_{eff}\, L^2}\right]}$$

γ 1.20 shear coefficient form factor (6/5 for rectangular sections)

K_s 11.50 loading condition factor (11.5 for uniformly distributed and simply supported end fixity)

EI_{app} 5.24E+12 Nmm² *Apparent stiffness*

EI_{eff} (Gamma) 5.42E+12 Nmm² *Gamma method*

Difference 3.2%

5. Calculate Maximum Deflection *FP Innovations CLT Handbook - 3.4.1*

$$\Delta_{max} = \frac{5w(L)^4}{384 \times EI_{app}}$$

j_2 2.00 *Maximum deflection at mid-span*

Δ_{max} 19.13 mm

L/Δ = 314 ≥ 300 therefore Okay

Although the span is continuous, the floor has been designed as simply supported at this early stage.

The Dynamic performance of a floor is governed by 3 factors: stiffness (stiffer floors perform better), mass (heaver floors perform better) and damping floors with additional layers (furniture, etc) perform better. For the rest of the design we usedthe stiffness calculated using the Gamma approach.

Refer to Section 3.6 for the detailed description on the vibration of floors. In this section we design for vibration using the modification on Eurocode 5, as outlined in 3.6.5.

1. Calculate Dynamic Frequency *Section 3.6.5.2*

$$F_1 = \frac{\pi}{2l^2}\left(\frac{(EI_{eff})}{m}\right)^{1/2}$$

EI_{eff}	5.42E+06 kNm^2
m	237.03 kg/m^2
L	6.00 m
F_1	6.60 Hz

Acceleration check required

If the natural frequency is below 8Hz, then an acceleration check is required.

2. More detailed analysis of acceleration (required only if natural frequency is below 8 Hz)

Funamdental Freq	Fourier Coefficient	Forcing Frequency
$3.4 < f_{1\ <4.6}$	0.20	ff
$4.6 < f_{f\ <\ 5.1}$	0.20	ff
$5.1 < f_{f\ <\ 6.9}$	0.006	ff
$f_{f\ >\ 6.9}$	0.006	6.90

$$F_1 = 0.4\frac{P_o a_1 f_1}{m_{gen}}\frac{1}{\sqrt{\left(\left(\frac{f_1}{f_F}\right)^2 - 1\right) + (2D\frac{f_1}{f_F})^2}} \leq 0.05\ m/s^2$$

P_0	700.00	N, mass of one person
a_f	0.01	Fourier coefficient
ff	6.60	Natural frequency
f_F	6.60	Forcing frequency
D	1.50	Damping (1.5% for light finish)

$$M_{gen} = m \times 0.5 \times b_{eff}$$

M_{gen}	118.51 kg
a	0.03

OK

3. Check the deflection of the floor under a 1 kN point load, <1.5mm

$$B_{eff} = \frac{L}{1.1}\sqrt[4]{\frac{EI_T}{EI_L}}$$

Calculate the overall stiffness of the transverse layer. This is assumed to be simply supported and therefore considered conservative

	$b_{eff}\ t_i^3/12$(mm^4)		$b_{eff}\ t_i\ h_i^2$ (mm^4)		E_i (MPa)		E_iI_i (Nmm^2)
t_2	7.6E+06		9.1E+07		0.0E+00		5.92E+11
t_4	7.6E+06	0.0E+00	9.1E+07	0.0E+00	6000.00	0.00	5.92E+11
							Σ = 1.18E+12

$$\Delta_{1kN} = \frac{PL^3}{48EI_{eff}b_{eff}} < 1.5mm$$

EI_T	1.18E+12	Transverse stiffness of panel
EI_L	6.00E+12	Longitudinal stiffness of panel
B_{eff}	3.64 m	
Δ	0.24mm	Okay

	f'_b (MPa)	f'_s (MPa)	f'_r (MPa)	E_0 (MPa)	E_{90} (MPa)	G_0 (MPa)	G_R (MPa)
SG8	12.00	3.80	1.20	8000.00	266.67	533.33	53.33

Refer to Xlam Australia Pty Ltd Design Guide for material properties

Refer to Section 3.6 for the detailed description on the vibration of floors. In this section we will design for vibration using the modification on Eurocode 5, as outlined in 3.6.5.

1. Calculate Section Bending Capacity

Section 3.6.2

$$\varphi M = \varphi_b k_1 k_4 k_6 k_9 k_{12} f_b Z_{eff}$$

φ_b 0.85 k_9 1.00 k_{12} 1.00 k_6 1.00 k_4 1.00

$$Z_{eff} = \frac{EI_{eff}}{E_1} \times \frac{2}{t_p}$$

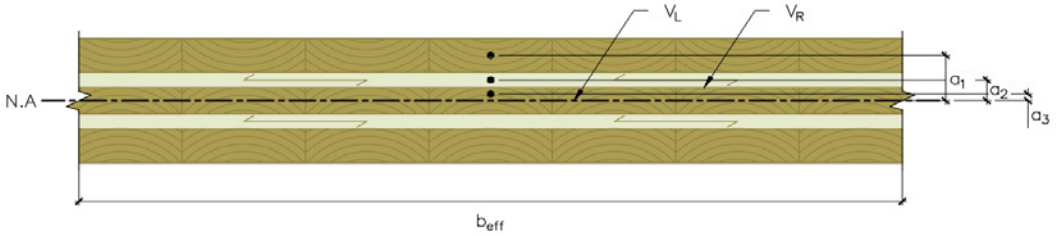
EI_{eff} 5.4E+12 using EIeff calculated from the Gamma method
Z_{eff} 6.0E+06 mm^3 Effective section modulus

	LC1	LC2	LC3	LC4		
k_1	0.57	0.57	0.80	1		
φM	34.99	34.99	49.11	61.38	kNm/m	Bending capacity
M^*	14.40	16.85	22.92	19.55	kNm/m	Applied bending moment ($wl^2/8$)

Load Case 1: 1.35G
Load Case 2: 1.2G + 1.5Qw_l
Load Case 2: 1.2G + 1.5Q
Load Case 4: 1.2G + W + Q

$M^*/\varphi M$ ≤ 1.0 therefore Okay

2. Calculate Section Shear Capacity



$where\ \tau_v = f'_s or\ f'_r$

Shear failure can occur in both *rolling shear* and *longitudinal shear*

Section 3.6.2

$$V_d = \varphi k_1 k_4 k_6 f_s' A_s$$

φ_s 0.85 k_4 1.00 k_6 1.00

$$A_{eff,mid} = \frac{K_{CLT}}{\left(E_1 t_1 z_1 + \frac{E_3 t_3^2}{8}\right)}$$

Where K = EIeff

$$A_{eff,rolling} = \frac{K_{CLT}}{(E_1 t_1 z_1)}$$

f'_s	3 MPa		f'_s	0.7 MPa
z_1	90 mm		z_1	90 mm
E_1	8000 MPa		E_1	8000 MPa
T_1	45 mm		T_1	45 mm
E_3	6000 MPa			
T_3	45 mm			

$A_{eff,mid} =$ 1.6E+05 mm^2 $A_{eff,mid} =$ 1.7E+05 mm^2

Vd,long 407.19 kN *Vd,rolling* 99.46 kN

$\varphi V = \min(\varphi V_L, \varphi V_R)/k1$ 99.46 kN rolling shear governs

Refer K factors Section 1

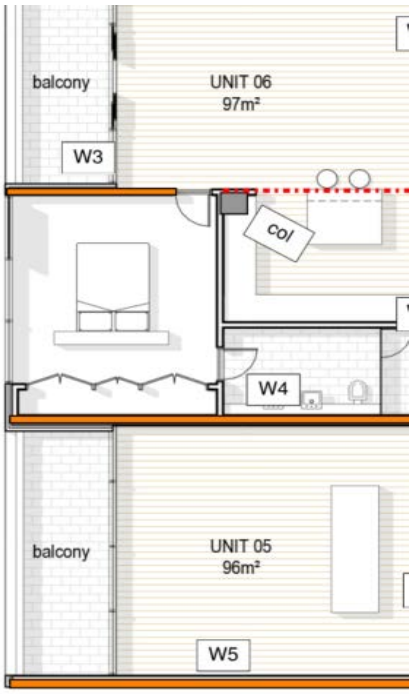
	LC1	LC2	LC3	LC4
k_1	0.57	0.57	0.80	1
φV_R	56.7	56.7	79.6	99.5 kN
V^*	9.60	11.23	15.28	13.03

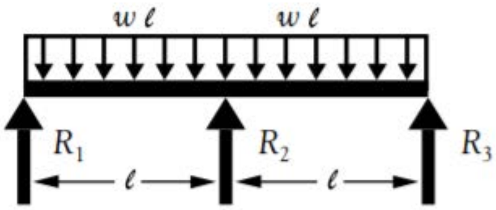
Load Case 1: 1.35G
Load Case 2: 1.2G + 1.5Qw_l
Load Case 2: 1.2G + 1.5Q
Load Case 4: 1.2G + W + Q

$V^*/\varphi V$ ≤ 1.0 therefore Okay

The various approaches to wall design for Australia and Internationally are outlined in Section 4 of the Guide.

This worked example design Australian product using appropriate Australian Standards. Note, when designing CLT walls it is important to consider the adjacent openings, for doors and windows for additional load spreads. The two walls that will be calculated are W4 and W5, an internal and external span. The wall W4 will be designed for axial compression and vertical movement. The wall W5 will also be compared with W4 with a differential movement computation.





1. Determine critical wall

L 5.60 m

W4 Calculation

Adjacent Openings 0.00 m

$R2 = \frac{10l}{8} =$ 7.00 m (per m) Trib breadth

W5 Calculation

Adjacent Openings 0.00 m

$R1 = \frac{3l}{8} =$ 2.10 m Trib breadth

2. Determine wall loading

Roof Loads

G 2.13 kPa
Q 0.50 kPa
Levels 1

Typical Floor

G 2.37 kPa
Q 1.50 kPa
Levels 6

Transfer Concrete

G 14.75 kPa
Q 1.50 kPa
Levels 1

Wall Self-weight

G 2.89 kN/m

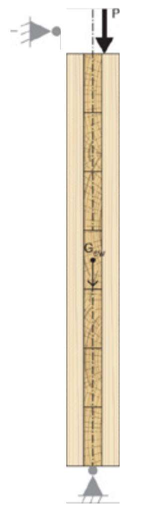
3. Determine wall loading at each floor

	W4	
	G (kN/m)	Q (kN/m)
Roof	17.8	3.5
7	37.3	14.0
6	56.8	24.5
5	76.3	35.0
4	95.7	45.5
3	115.2	56.0
2	134.7	66.5
L1 -Trans	238.0	77.0

Ground

W5	
G (kN/m)	Q (kN/m)
4.5	1.1
22.8	6.7
42.3	17.2
61.8	27.7
81.2	38.2
100.7	48.7
120.2	59.2
165.7	69.7

1. Geometry & Loading



L	2875.00	mm	
b_{eff}	1000.00	mm	

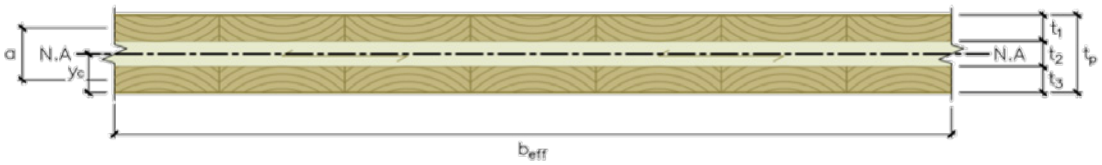
$$e_T = t_p/15$$

e_T	7.67	mm	
	0.13		
	0.92		
P_G	134.71	kN/m	
P_Q	66.50	kN/m	

$$w_u = 1.2G + 1.5Q$$

w_u	265.02	kN/m	ULS

2. Calculate Section Properties (Bending Stiffness)



	t_i (mm)	y_i (mm)	h_i (mm)	θ_i (deg °)	Grade	E_i (MPa)	G_i (MPa)
t_1	35	97.5	40	0	SG8	8000	533
t_2	45	57.5	0	90	SG6	200	40
t_3	35	17.5	40	0	SG8	8000	533

t_p 115 mm Total panel thickness

$$y_i = \sum_{n=i+1}^{3} (t_n) + \frac{t_i}{2}$$

$$y_c = \frac{\sum_{i=1}^{3} t_i E_i y_i}{\sum_{i=1}^{3} t_i E_i}$$

y_c 57.50 mm CLT section centroid (from bottom of panel)

	$b_{eff}\, t_i^3/12$ (mm⁴)	$b_{eff}\, t_i\, h_i^2$ (mm⁴)	E_i (MPa)	E_iI (MPa)
t_1	3.57E+06	5.60E+07	8000	4.77E+11
t_2	7.59E+06	0.00E+00	200	1.52E+09
t_3	3.57E+06	5.60E+07	8000	4.77E+11
			$\sum =$	9.55E+11

$$I_i = \frac{b_{eff}\, t_i^3}{12} + b_{eff} t_i h_i^2$$

$$EI = \sum_{i=1} E_i I_i \quad = 9.5\text{E}+11 \text{ Nmm}^2$$

The EI effective, with gamma value of 1 can be used for strength checks. Therefore gamma is not calculated for this wall design.

3. Calculate Gamma for each layer

$$\gamma_i = \left[1 + \frac{\pi^2 E_i A_i}{G_R \cdot \frac{b}{d} \cdot l_{ref}^2}\right]^{-1}$$

	t_i (mm)	E_i	G_R (MPa)	A_i mm^2	b/d	l_{ref}^2 mm^4	γi
t_1	35	8000	-	3.5E+04	28.57	8.3E+06	0.77
t_2	45	200	40.00	-	-	-	
t_3	35	8000	-	3.5E+04	28.57	8.3E+06	0.77

4. Calculate Effective Stiffness for Panel

$$EI_{eff} = \sum_{i=1}^{n} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

	$b_{eff}\ t_i^3/12$ (mm^4)	$b_{eff}\ t_i\ h_i^2$ (mm^4)	γi	E_i	EI_{eff} Nmm^2
t_1	3.57E+06	5.60E+07	0.77	8000.00	3.75E+11
t_2	-	-	-	-	-
t_3	3.57E+06	5.60E+07	0.77	8000.00	3.75E+11
$\sum = EI_{eff} = \sum_{i=1}^{n} \left(E_i J_i + \gamma_i E_i A_i a_i^2\right) =$					7.5E+11

Gamma is 1 for strength

5. Calculate Section Axial Capacity

$$\varphi N = \varphi_b k_1 k_4 k_6 k_{12} f'_c A_c$$

AS1720.1 - CL 3.3.1.1

φ_b	0.85	k_4	1.00
k_6	1.00	f'_c (MPa)	18.00

E_i	8000.0 MPa	*EI (eff)*
A_c	7.0E+04 mm^2	(A_c = ($T1$ x $E1$ + $T2$ x $E2$ + $T3$ x $E3$)/$E1$)
		For a 3 layer panel

$$S_5 = 0.3\ g_{13}\ g_{28}\ L \left(\frac{EA_{eff}}{EI_{eff}}\right)^{0.5}$$

AS1720.1 - CL E4.4.1.2

g_{13}	1	for CLT walls
g_{28}	1.00	
S_5	20.89	

$$\rho_c = 11.39 \left(\frac{E}{f_c'}\right)^{-0.408} r^{-0.074}$$

AS1720.1 - E2(3)

r 0.25 ρ_c 1.05 $\rho_c \times S$ 21.91

$$\rho_c \cdot S \leq 10 \quad \rightarrow \quad k_{12} = 1.0$$

$$10 \leq \rho_c \cdot S \leq 20 \quad \rightarrow \quad k_{12} = 1.5 - 0.05 \cdot \rho_c \cdot S$$

$$\rho_c \cdot S \geq 20 \quad \rightarrow \quad k_{12} = \frac{200}{(\rho_c \cdot S)^2}$$

k_{12} c 0.42 *slenderness coefficient for columns*

	LC1	LC2	LC3	LC4	
k_1	0.57	0.57	0.80	1	
φN	254.3	254.3	356.9	446.1	kN/m
N^*	181.9	201.6	228.2	188.3	kN/m
	72%	79%	64%	42%	

Load Case 1: 1.35G
Load Case 2: 1.2G + 1.5Q$_{W_l}$
Load Case 2: 1.2G + 1.5Q
Load Case 4: 1.2G + W + Q

6. Calculate Section Axial Capacity

$$\varphi M = \varphi_b k_1 k_4 k_6 k_9 k_{12} f_b Z_{eff}$$

φ_b 0.85 k_4 1.00 f'_b *(MPa)* 12.00 k_6 1.00 k_9 1.00

$$Z_{eff} = \frac{EI_{eff}}{E_1} \times \frac{2}{t_p} =$$ 2.1E+06 mm^3 *with gamma of 1 for strength checks*

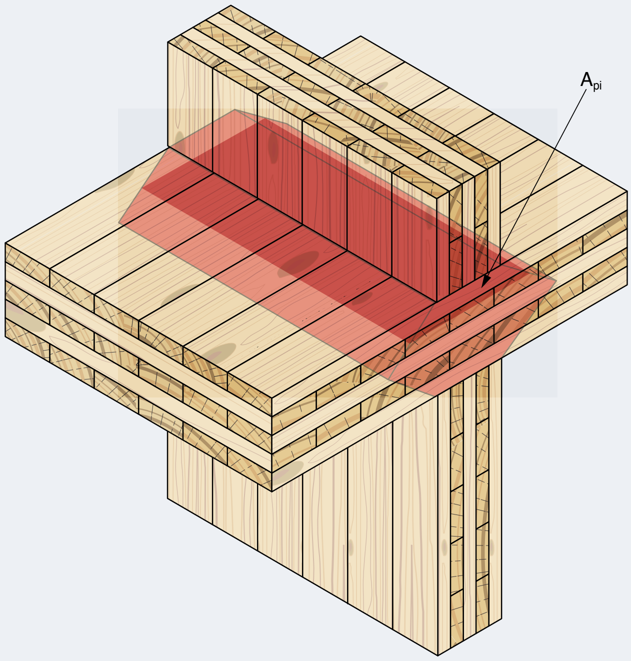
k_{12} b	1.00	*slenderness coefficient for floor, assumed no torsion*			
k_1	0.57	0.57	0.80	1	

$M^* = N \times e_T$	LC1	LC2	LC3	LC4	
φM	12	12	17	21	kNm
$\boldsymbol{M^*}$	1.39	1.55	1.75	1.44	kNm
	12%	13%	10%	7%	

7. Calculate Combined Actions

$\frac{\sigma_b^*}{f'_{b,d}} + \frac{\sigma_c^*}{f'_{c,d}} =$	83.1%	92.1%	74.3%	49.0%	OK
$\left(\frac{\sigma_b^*}{f'_{b,d}}\right)^2 + \frac{\sigma_c^*}{f'_{c,d}} =$	72.9%	80.9%	65.0%	42.7%	OK

1. Calculate loaded cross-sectional area of elements, perpendicular to grain in storey



$$A_{p,i} = Max\ (b_4 + \frac{d_3}{4}, k_7 b_4) \times b_3$$ Section 4.33

b_4	115.00 mm	*panel thickness above*
d_3	225.00 mm	*wall thickness below*
k_7	1	*length of bearing of member*
b_3	1000 mm	
$\boldsymbol{A_{pi}}$	1.7E+05 mm^2	

2. Calculate perpendicular to grain compressive strength

$$\varphi N_{d,p} = \varphi k_1 k_4 k_6 k_7 f'_p A_p$$

$f'p$ 8.9 MPa *Refer supplier supplier for properties (XLam Design Guide Version 1.0)*

φ	0.8		k4	1
k_6	1		k7	1

	LC1	LC2	LC3	LC4
k_1	0.57	0.57	0.80	1

G (L3)	115.2	kN
Q (L3)	56.0	kN

	LC1	LC2	LC3	LC4	
N*	155.6	160.7	171.9	160.7	kN
$\varphi N(d.p)$	4058.4	4058.4	5696.0	7120.0	kN

Load Case 1: 1.35G
Load Case 2: 1.2G + 1.5Qwl
Load Case 2: 1.2G + 1.5Q
Load Case 4: 1.2G + W + Q

1) Geometry & Loading

Refer to Section 4.3 for Building Shortening

L	2875.00 mm	G	134.7 kN
d_{floor}	225.00 mm	Q	66.5 kN
$w_u = G + 0.4Q$	161.31 kN		

2. Calculate Shrinkage Parallel to Grain

$$\delta_{s,\ell} = U_\ell(\Delta mc)L$$

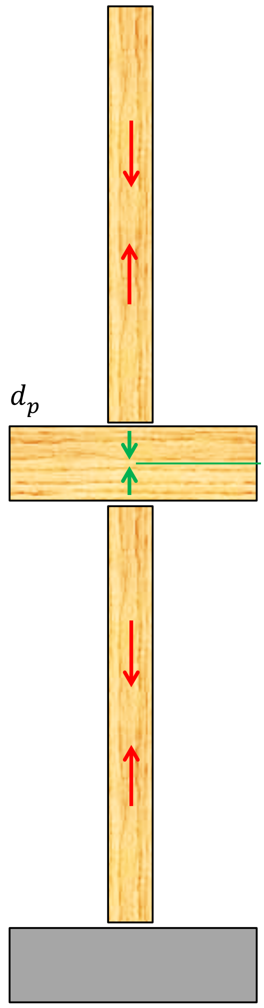
u	0.0027	*Tangential moisture movement, radiata pine*
u_l	0.0000675	*mm/mm/%change in MC*
(Δmc)	3 %	
$\delta_{(s,l)}$ =	0.58 mm	

3. Calculate Shrinkage Perpendicular to Grain

Refer to Section 4.3.1

$$\delta_{s,\ell} = U_p(\Delta mc)d_p$$

u	2.70E-03	*Tangential movement, radiata pine*
up	0.0027	*mm/mm/%change in MC*
(Δmc)	4 %	*mm/mm/%change in MC*
$\delta_{(s,l)}$=	2.43 mm	



4. Calculate Parallel to Grain Deformation and Creep

$$\delta_{c,\ell} = \sum_{floors} \frac{j_2 N_{c,i} L_i}{E_i A_p}$$

j_2	2.00	
$N_{c,l}$	161.31 kN	
L_i	2875.00 mm	
E_i	8000.00 MPa	
$Ap_{arallel}$	7.00E+04 mm^2	(A = ($T1$ x $E1$ + $T2$ x $E2$ + $T3$ x $E3$)/$E1$) For a 3 layer panel
$\delta(c,l)$=	1.66 mm	

5. Calculate Perp to Grain Deformation and Creep

$$\delta_{c,p} = \sum_{floors} \frac{j_2 N_{c,i} d_{2,i}}{E_{p,i} A_{p,i}}$$

j_2	2.00
$N_{c,l}$	161.31 kN
d_p (outer)	135.0 mm
E_i (outer)	266.67 MPa
d_p (inner)	90.00 mm
E_i (inner)	200.00 MPa
A_{pi}	1.7E+05 mm^2
$\delta_{(c,l)}$=	1.80 mm

6. Settlement or Embedment of Joints

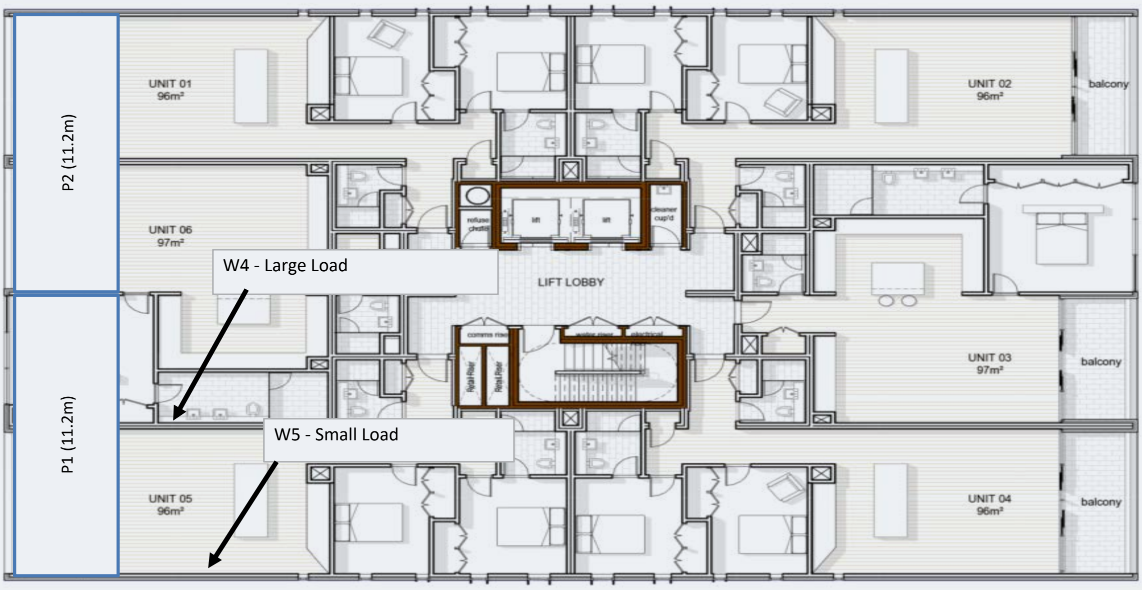
$$\delta_j = n_{joints}\delta_{gap}$$

n_{joints}	2.00
δ_{gap}	0.60 mm
δ_j	1.20 mm

$\boldsymbol{\delta_{Total} = \delta_{s,\ell} + \delta_{s,p} + \delta_{c,\ell} + \delta_{c,p} + \delta_j}$ = 7.67 mm **Transfer structure not considered in this calculation*

CLT Wall Panel - Differential Shortening Example

The differential shortening, for this example is calculated between W4 and W5, which upon inspection seem to be the CLT walls with the maximum and minimum loads respectively. Best practice is to determine the shortening of each wall within the building via spreadsheet or design software.



1. Calculate for W4

	w (kN)	$\delta(s,l)$ mm	$\delta(s,p)$ mm	$\delta(c,l)$ mm	$\delta(c,p)$ mm	$\delta(j)$ mm	$\delta(t)$ mm
Roof	19.22	0.58	2.43	0.20	0.21	1.20	4.62
7	42.90	0.58	2.43	0.44	0.48	1.20	5.13
6	66.59	0.58	2.43	0.68	0.74	1.20	5.64
5	90.27	0.58	2.43	0.93	1.01	1.20	6.15
4	113.95	0.58	2.43	1.17	1.27	1.20	6.65
3	137.63	0.58	2.43	1.41	1.54	1.20	7.16
2	161.31	0.58	2.43	1.66	1.80	1.20	7.67

1. Calculate for W5

	w (kN)	$\delta(s,l)$ mm	$\delta(s,p)$ mm	$\delta(c,l)$ mm	$\delta(c,p)$ mm	$\delta(j)$ mm	$\delta(t)$ mm
Roof	4.90	0.58	2.43	0.05	0.05	1.20	4.32
7	25.46	0.58	2.43	0.26	0.28	1.20	4.76
6	49.14	0.58	2.43	0.50	0.55	1.20	5.27
5	72.82	0.58	2.43	0.75	0.81	1.20	5.77
4	96.51	0.58	2.43	0.99	1.08	1.20	6.28
3	120.19	0.58	2.43	1.23	1.34	1.20	6.79
2	143.87	0.58	2.43	1.48	1.61	1.20	7.30

3. Determine differential movement

	W4 $\delta(t)$ mm	W5 $\delta(t)$ mm	$\delta(diff)$ mm
Roof	4.62	4.32	0.31
7	9.76	9.08	0.68
6	15.40	14.34	1.05
5	21.54	20.11	1.43
4	28.20	26.39	1.80
3	35.36	33.18	2.18
2	43.03	40.48	2.55

CLT buildings act like a honey comb structure, with each CLT wall resisting the lateral forces within the building. The CLT core panels will not be designed to act compositely, and act as individual walls.

It is important to input the correct panel lengths, as they are manufactured to determine the correct relative stiffness of each wall.

For this worked example, a hand computation via excel, and a computer analysis will be undertaken.

The computer analysis will break each panel up individually to determine the relative stiffness of each panel and the total stiffness of the building.

The hand computation undertaken via Excel, willl be used for model verification purposes, and therefore a much more simplistic approach will be undertaken breaking all of the shear walls into a combination of 2.5m, 4.5m and 8m panel widths as shown in the following figure.

In order to determine the lateral load distribution for each wall, the relative stiffness of each shear wall must be calculated. The stiffness (K) is a function of unit force (kN) per unit deformation (mm).

The deformation of the shear wall is a function of bending deformation, shear deformation, expansion of the tie rods and displacement in one of the two joints between wall and ceiling.

For the purpose of this exercise, the stiffness of three walls will be checked and input. This 'hand computation' is only for demonstration and verification of the computer software, and each wall should be checked individually for design.

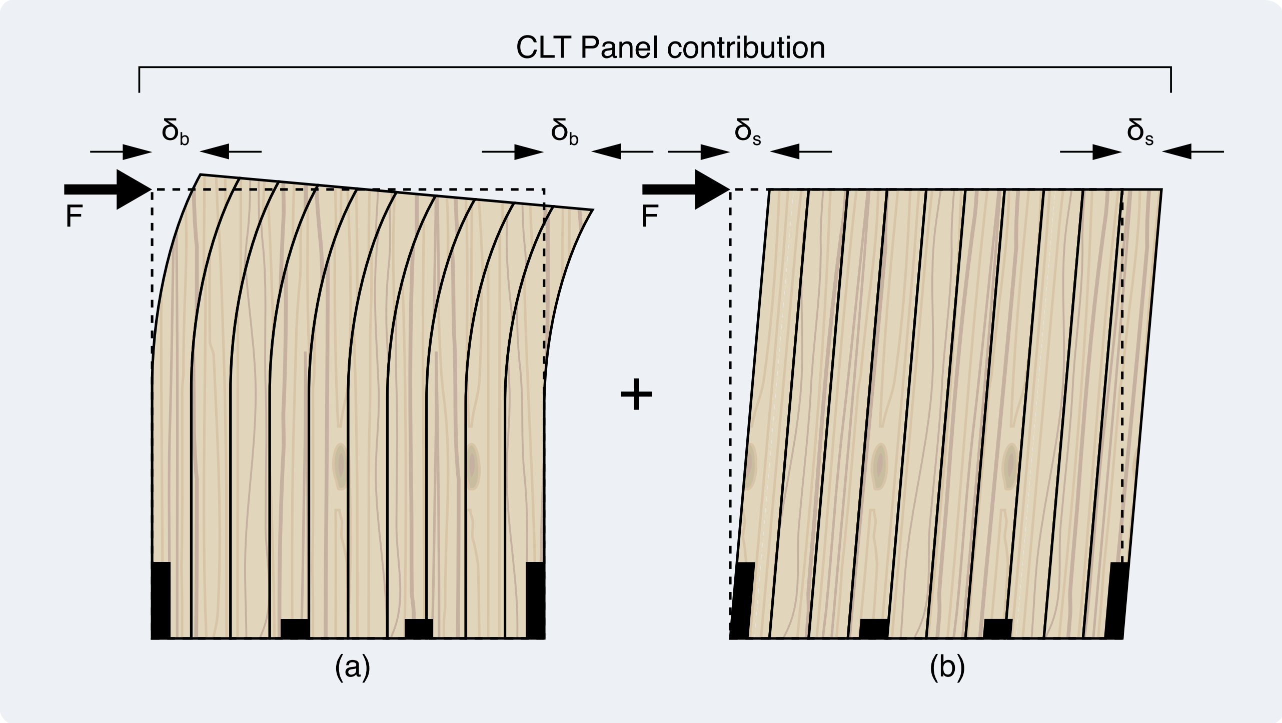
Computer software can be used as a more accurate tool for structural analysis of lateral loads.

In order to determine the lateral load distribution for each wall, the relative stiffness of each shear wall must be calculated. The stiffness (K) is a function of unit force (kN) per unit deformation (mm).

The deformation of the shear wall is a functino of bending deformation, shear deformation, expansion of the tie rods and displacement in one of the two joints between wall and ceiling.

For the purpose of this exercise, the stiffness of three walls will be checked and input. This 'hand computation' is only for demonstration and verification of the computer software, and each wall should be checked individually for design.

Fk **1kN**



1. Calculate in-plane bending deformation of the shear wall *pro-Holz - CLT Structural Design 10.5.3*

$$I = \frac{d_0 b^3}{12}$$

	Type 1	Type 2	Type 3		
b	2500	4500	8000	mm	Total panel length (maximum break-up)
d_0	90	90	90	mm	Depth of CLT layers, in plane
I	1.17188E+11	6.83438E+11	3.84E+12	mm^4	
E	7333	7333	7333	MPa	Average Elastic Modulus (Ext 8000 MPa, Intl 6000 MPa)

$$\delta_m = \frac{F_k h^3}{3EI}$$

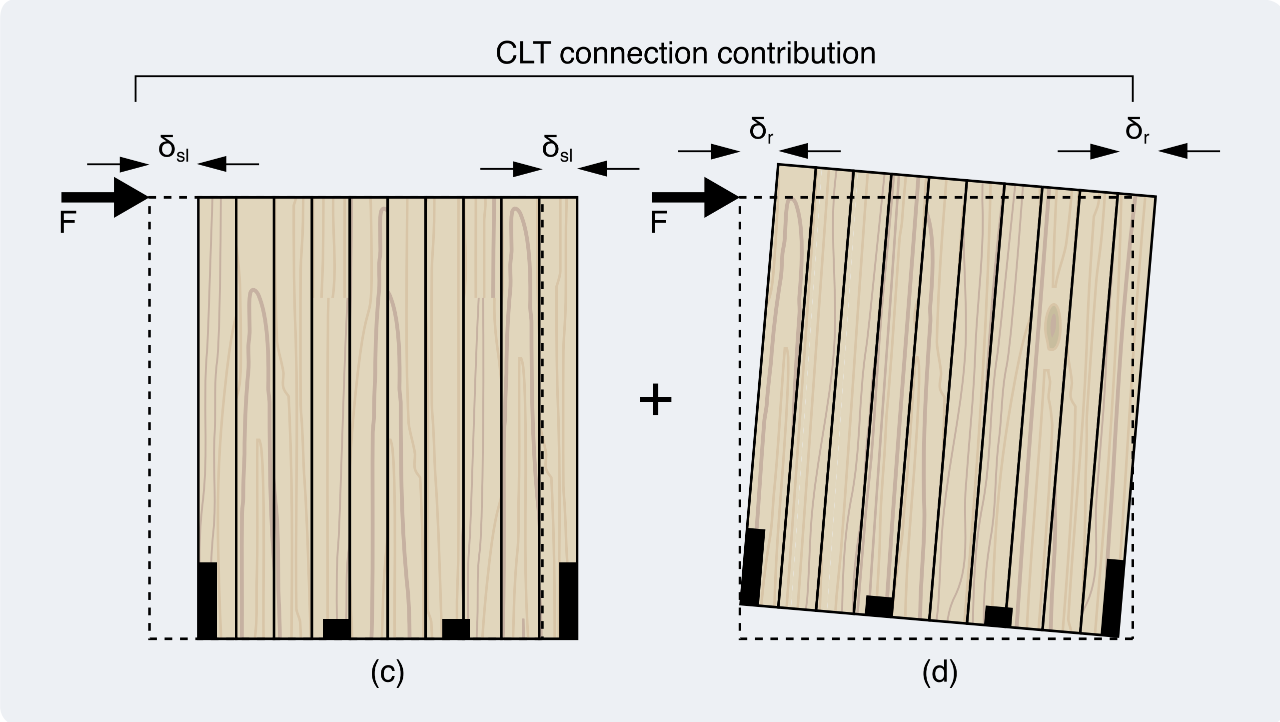
F_k	1000	1000	1000	N
h	2875	2875	2875	mm
δ_m	0.0092	0.0016	0.0003	mm

2. Calculate shear deformation of the shear wall *pro-Holz - CLT Structural Design 10.5.3*

$$\delta_v = \frac{F_k h}{G A_z}$$

$G_v = 0.75 \times G_{o,mean}$ A= $d_{gross} b$

G_{mean}	533	533	533	MPa	*Properties from supplier handbooks (G = 533 for E = 8000 product)*
G_v	400	400	400	MPa	
A_s	225,000	405,000	720,000	mm^2	
δ_v	0.032	0.018	0.010	mm	



3. Calculate expansion of the tie rods

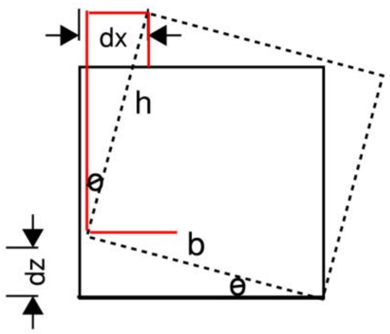
pro-Holz - CLT Structural Design 10.5.3

$$F_k = \frac{(F_t \times b)}{h}$$

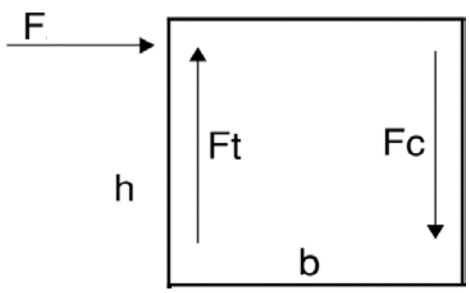
Vertical stiffness can be found in connection suppliers handbook - which needs to be converted into a horizontal stiffness via trigonometry.

c_z	12381	N/mm	*vertical stiffness from connection supplier*
n	2		*number of expansion rods per end of panel*
			Convert vertical stiffness (cz) into horizontal stiffness

$$d_x = \frac{hd_y}{b}$$



$$c_z = \frac{F_k}{d_x}$$



$$\delta_z = \frac{F_k h^2}{b^2 c_z n}$$

δ_z	5.34E-02	1.65E-02	5.22E-03	mm

4. Displacement in one of the two joints between wall and ceiling (Titan)

pro-Holz - CLT Structural Design 10.5.3

$$\delta_f = \frac{F_k}{nc_f}$$

c_f	11240	11240	11240	N/mm	Connection stiffness from supplier handbooks -
n	3.0	5.0	8.0		number of fixings per panel (@1000mm)
δ_f	0.03	0.02	0.01	mm	

The deformation of the shear wall is a function of bending deformation in the shear wall, shear deformation in the shear wall, expansion of the tie rods and displacement in one of the two joints between wall and ceiling.

5. Determine total displacement, per unit force

pro-Holz - CLT Structural Design 10.5.3

$$\delta_t = \delta_m + \delta_v + \delta_z + \delta_f$$

	W1	W2	W3	
δ_t =	0.12	0.05	0.03	mm/kN

6. Check relative contribution of each panel (optimisation possible)

	% Contribution					
	δ_t (mm)	δ_m	δ_v	δ_z	δ_f	Total
T1	0.12	7.42%	25.73%	42.99%	23.87%	100.00%
T2	0.05	2.95%	33.12%	30.74%	33.19%	100.00%
T3	0.03	1.06%	37.54%	19.60%	41.80%	100.00%

Note that for shorter walls, the expansion of the tie rods reduces the stiffness the most - due to the shorter lever arm.

7. Calculate stiffness of each wall type

	Wall Length (mm)	K (N/mm)
T1	2,500.00	8,048.53
T2	4,500.00	18,651.08
T3	8,000.00	37,584.32
T3+T1	10,500.00	45,632.85

CLT Wall Stiffness and Centre of Stiffness

The following section calculates the load distribution, which is generic and not unique to CLT. The lateral load distribution for this worked example is calculated via a hand model, with rigid diaphragms assumed.

For stiffness calculations, the walls have been assumed to be broken up into a combination of 2.5 m, 4.5 m and 8 m panels. Note this is a simplification as the purpose of this worked example is to show the process only and verify the adopted computer software.

1. Calculate stiffness for each wall

Wall No	L (m)	Model Input	Direction	yi (m)	xi (m)	K (N/mm)	Kx (N/mm)	Ky (N/mm)
W1	8.4	T3	x	22.4	0	37584	37584	0
W2	11.2	T3+T1	x	16.7	0	45633	45633	0
W3	3.3	T1	x	11.2	0	8049	8049	0
W4	11.2	T3+T1	x	6.3	0	45633	45633	0
W5	8.4	T3	x	0	0	37584	37584	0
W6	8.4	T3	x	22.4	0	37584	37584	0
W7	11.8	T3+T1	x	16.7	0	45633	45633	0
W8	3.3	T1	x	11	0	8049	8049	0
W9	11.8	T3+T1	x	6.3	0	45633	45633	0
W10	8.4	T3	x	0	0	37584	37584	0
W11	4.5	T2	y	0	9.5	18651	0	18651
W12	4.5	T2	y	0	16.9	18651	0	18651
W13	4.5	T2	y	0	23.90	18651	0	18651
W14	3.3	T1	y	0	9.5	8049	0	8049
W15	5.0	T2	y	0	23.9	18651	0	18651
W16	4.9	T2	y	0	9.5	18651	0	18651
W17	3.3	T1	y	0	23.9	8049	0	8049
W18	4.5	T2	y	0	9.5	18651	0	18651
W19	4.5	T2	y	0	16.9	18651	0	18651
W20	4.5	T1	y	0	23.9	8049	0	8049
CA-1	6.7	T2	x	6.8	0	18651	18651	0
CA-2	2.4	T1	y	0	13.4	8049	0	8049
CA-3	2.4	T1	y	0	20.1	8049	0	8049
CA-4	6.7	T2	x	9.2	0	18651	18651	0
CB-1	2.4	T1	y	0	13.4	8049	0	8049
CB-2	5.7	T2	x	13.1	0	18651	18651	0
CB-3	5.7	T2	x	15.5	0	18651	18651	0
CB-4	2.4	T1	y	0	19.1	8049	0	8049
	164.1m					∑	**423570**	**186897**

2. Calculate centre of stiffness of the building

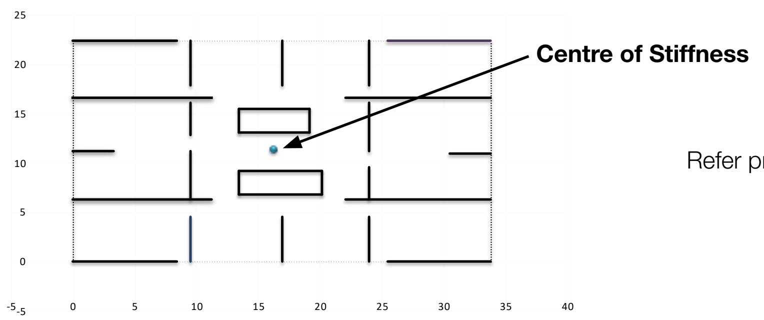
$$Y_{c.o.s} = \frac{\sum Y_i K_{xi}}{\sum K_{xi}} \qquad X_{c.o.s} = \frac{\sum X_i K_{yi}}{\sum K_{yi}}$$

$\sum K_{xi}$ 42,3570.0 N/mm $\sum K_{yi}$ 186897 N/mm

$\sum Y_i K_{xi}$ 4.8E+06 $\sum X_i K_{yi}$ 3.0E+06

Y cos 11.3 m **X cos 16.3 m**

Hand Model - Wall Inputs and Centre of Stiffness



Refer previous pages for Wall Labels

CLT Wall Panel - Seismic Load Distribution

In this section, load distribution through the bracing walls is determined based on wall stiffness. This analysis also considers the torsional effect associated with eccentric loading required in seismic design.

1) Calculate torsional constant, J

Wall No	d_y	d^x	k_y x d^{x2}	k^x x d_y^2
W1	11.1	0.0	5605544.81	0
W2	5.4	0.0	1322449.19	0
W3	-0.1	0.0	109.56	0
W4	-5.0	0.0	1148442.62	0
W5	-11.3	0.0	2388590.30	0
W6	11.1	0.0	2291101.41	0
W7	5.4	0.0	1089201.16	0
W8	-0.3	0.0	3769.03	0
W9	-5.0	0.0	945885.14	0
W10	-11.3	0.0	2388590.30	0
W11	0.0	-6.8	0	371837.39
W12	0.0	0.6	0	6781.36
W13	0.0	7.6	0	2637824.15
W14	0.0	-6.8	0	861669.17
W15	0.0	7.6	0	465248.34
W16	0.0	-6.8	0	861669.17
W17	0.0	7.6	0	465248.34
W18	0.0	-6.8	0	861669.17
W19	0.0	0.6	0	13665.30
W20	0.0	7.6	0	465248.34
CA-1	-4.51	0	380488.48	0
CA-2	0	-2.89	0	67548.87
CA-3	0	3.80	0	116403.42
CA-4	-2.11	0	83562.59	0
CB-1	0	-2.89	0	67548.87
CB-2	1.78	0	59315.15	0
CB-3	4.18	0	326398.04	0
CB-4	0	2.80	0	63235.08
Total	-0.83	5.24	18033447.87	7325597.01

For each direction, the Earthquake actions are applied at + or i 0,1b (as per AS1170.4), from the nominal centre of mass. The eccentricity will be applied to calculate the most adverse torsional moment.

A 1kN load will be applied as the nominal load to calculate the distribution. Once the distribution is determined, the real Earthquake loads (previously calculated) can be applied to the building.

$$J = (dx_i^2 K_{iy} + dy_i^2 K_{ix}) =$$

J 25359044.89 m^6

Applied Loads

Fx Load 1 kN

Fy Load 1 kN

e (x) 3.375 m

e (y) 2.24 m

2) Calculate the lateral load distribution based on a 1kN load

	Wall No	1kN in X Direction			1kN in Y Direction		
		Fsx	Ftx	Fty	Fsy	Ftx	Fty
T3	**W1**	8.9%	3.7%	0.0%	0.0%	5.5%	0.0%
T3+T1	**W2**	10.8%	2.2%	0.0%	0.0%	3.3%	0.0%
T1	**W3**	1.9%	0.0%	0.0%	0.0%	0.0%	0.0%
T3+T1	**W4**	10.8%	-2.0%	0.0%	0.0%	-3.0%	0.0%
T3	**W4**	8.9%	-3.8%	0.0%	0.0%	-5.7%	0.0%
T3	**W5**	8.9%	3.7%	0.0%	0.0%	5.5%	0.0%
T3+T1	**W6**	10.8%	2.2%	0.0%	0.0%	3.3%	0.0%
T1	**W7**	1.9%	0.0%	0.0%	0.0%	0.0%	0.0%
T3+T1	**W8**	10.8%	-2.0%	0.0%	0.0%	-3.0%	0.0%
T3	**W9**	8.9%	-3.8%	0.0%	0.0%	-5.7%	0.0%
T2	**W10**	0.0%	0.0%	-1.1%	10.0%	0.0%	-1.7%
T2	**W11**	0.0%	0.0%	0.1%	10.0%	0.0%	0.1%
T2	**W13**	0.0%	0.0%	1.3%	10.0%	0.0%	1.9%
T1	**W14**	0.0%	0.0%	-0.5%	4.3%	0.0%	-0.7%
T2	**W15**	0.0%	0.0%	1.3%	10.0%	0.0%	1.9%
T2	**W16**	0.0%	0.0%	-1.1%	10.0%	0.0%	-1.7%
T1	**W17**	0.0%	0.0%	0.5%	4.3%	0.0%	0.8%
T2	**W18**	0.0%	0.0%	-1.1%	10.0%	0.0%	-1.7%
T2	**W19**	0.0%	0.0%	0.1%	10.0%	0.0%	0.1%
T1	**W20**	0.0%	0.0%	0.5%	4.3%	0.0%	0.8%
T2	**CA-1**	4.4%	-0.7%	0.0%	0.0%	-1.1%	0.0%
T1	**CA-2**	0.0%	0.0%	-0.2%	4.3%	0.0%	-0.3%
T1	**CA-3**	0.0%	0.0%	0.3%	4.3%	0.0%	0.4%
T2	**CA-4**	4.4%	-0.3%	0.0%	0.0%	-0.5%	0.0%
T1	**CB-1**	0.0%	0.0%	-0.2%	4.3%	0.0%	-0.3%
T2	**CB-2**	4.4%	0.3%	0.0%	0.0%	0.4%	0.0%
T2	**CB-3**	4.4%	0.7%	0.0%	0.0%	1.0%	0.0%
T1	**CB-4**	0.0%	0.0%	0.2%	4.3%	0.0%	0.3%
	Sum	1.000	0.000	0.000	1.000	0.000	0.000

$$F_{shear,x} = \frac{F_x K_{ix}}{\sum K_{iy}}$$

$$F_{torsion,x} = \frac{(F_x e_y) y_i K_{iy}}{(dx_i^2 K_{iy} + dy_i^2 K_{ix})}$$

$$F_{torsion,y} = \frac{(F_y e_x) x_i K_{ix}}{(dx_i^2 K_{iy} + dy_i^2 K_{ix})}$$

$$F_{shear,y} = \frac{F_y K_{ix}}{\sum K_{iy}}$$

Formulas from:

http://site.iugaza.edu.ps/sshihada/files/2012/02/Handout-5-13.pdf

1. Calculate fundamental period of building *AS1170.4, CL7.2*

$$T_1 = 1.25\ k_t h_n^{0.75}$$

k_t	0.05	All other structures
h_n	26.2	m
$T1$	0.72	s

2. Calculate horizontal static shear force *AS1170.4, CL7.2*

$$V = \left(\frac{K_p Z C_h(T_1) S_p}{\mu}\right)\ W_t$$

K_p	1	1/500 years
Z	0.08	Melbourne
Site Soil	Ce	
$C_h(T_1)$	1.81	
S_p	0.67	(shear wall)
u	3	
V	0.03239584 x W_t	

Section 2.2.4 - Earthquake Loads

3. Calculate seismic weight for each floor *AS1170.4, CL5.4*

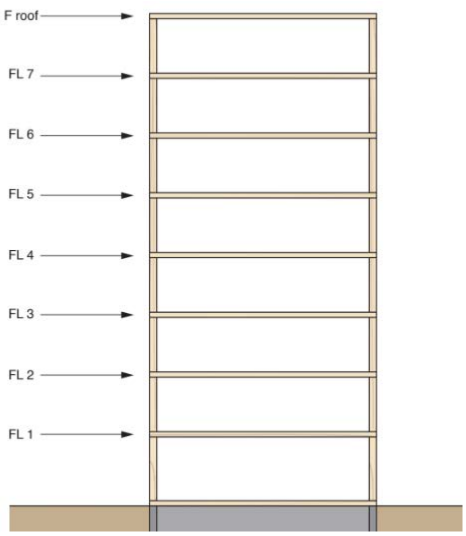
$$w_i = \sum G_i + \sum \omega_c Q$$

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n}(W_j h_j^k)}\left[k_p Z C_h(T_1)\frac{S_p}{\mu}\right]\ \times W_t$$

w_c	0.3	
h_i	3.1	m
A	756	m²

	w_i (kN)	h_i (m)	w_i x h_i	w_ih_i/w_jh_j	F_i (kN)
R	2,182	26.2	57,163	19%	133
7	2,409	23.1	55,639	18%	129
6	2,409	20.0	48,172	16%	112
5	2,409	16.9	40,705	14%	95
4	2,409	13.8	33,239	11%	77
3	2,409	10.7	25,772	9%	60
2	2,409	7.6	18,305	6%	43
L1 -Tran	4,986	4.5	22,437	7%	52
Total	21,619		301,432		

R	133 kN
7	129 kN
6	112 kN
5	95 kN
4	77 kN
3	60 kN
2	43 kN
1	52 kN



Gclt walls
2.61 kN/m
164.10 m

Gclt floors
2.17 kPa

Qroof
0.5 kPa

Qapt
1.5 kPa

Gconc
14.75 kPa

The calculated Earthquake action is applied at ht position +/- 0.1b, from the nominal centre of mass. Apply the eccentricity in the same direction at all levels, and orient to produce the maximum 100% and 30% loads. The load cases are simplified by taking the absolute number for each wall.

For this scenario, the Absolute values are used to reduce the overall number of load cases.

3. Calculate load on each wall for load cases specified in AS1170.4 (based on 1kN)

LC1	ABS(100% Tx) + ABS(30% Ty)	Calculate for 1kN only
LC2	ABS(100% Ty) + ABS(30% Tx)	Calculate for 1kN only

Wall No	LC1 - x	LC1 - y	LC2 - x	LC2 - y
W1	0.14	0.00	0.06	0.00
W2	0.13	0.00	0.03	0.00
W3	0.01	0.00	0.00	0.00
W4	0.13	0.00	0.03	0.00
W5	0.14	0.00	0.06	0.00
W6	0.14	0.00	0.06	0.00
W7	0.13	0.00	0.03	0.00
W8	0.01	0.00	0.00	0.00
W9	0.13	0.00	0.03	0.00
W10	0.14	0.00	0.06	0.00
W11	0.00	0.02	0.00	0.20
W12	0.00	0.03	0.00	0.19
W13	0.00	0.04	0.00	0.20
W14	0.00	0.01	0.00	0.08
W15	0.00	0.04	0.00	0.20
W16	0.00	0.02	0.00	0.20
W17	0.00	0.02	0.00	0.08
W18	0.00	0.02	0.00	0.20
W19	0.00	0.03	0.00	0.19
W20	0.00	0.02	0.00	0.08
CA-1	0.05	0.00	0.01	0.00
CA-2	0.00	0.01	0.00	0.08
CA-3	0.00	0.01	0.00	0.08
CA-4	0.04	0.00	0.00	0.00
CB-1	0.00	0.01	0.00	0.08
CB-2	0.04	0.00	0.00	0.00
CB-3	0.05	0.00	0.01	0.00
CB-4	0.00	0.01	0.00	0.08

Critical Load (kN)
0.14
0.13
0.01
0.13
0.14
0.14
0.13
0.01
0.13
0.14
0.20
0.19
0.20
0.08
0.20
0.20
0.08
0.20
0.19
0.08
0.05
0.08
0.08
0.04
0.08
0.04
0.05
0.08

Note, each wall has been checked to determine load attracted for a 1kN force.
Next, the real loads are applied to the building to determine the force acting on each wall.

Note, each wall has been checked to determine load attraced for a 1kN force. Next, the real loads are applied to the building to determine the force acting on each wall.

4. Calculate loads for each wall at each floor

Forces for the wind loads

F (kN) Eq	R	7	6	5	4	3	2	Cumulative*
	133	**129**	**112**	**95**	**77**	**60**	**43**	**648**
W1	19	18	16	13	11	9	6	92
W2	18	18	16	13	11	8	6	90
W3	3	2	2	2	1	1	1	12
W4	18	18	15	13	11	8	6	89
W5	19	19	16	14	11	9	6	93
W6	19	18	16	13	11	9	6	92
W7	18	18	16	13	11	8	6	90
W8	3	2	2	2	1	1	1	13
W9	18	18	15	13	11	8	6	89
W10	19	19	16	14	11	9	6	93
W11	27	26	23	19	16	12	9	132
W12	27	26	22	19	15	12	9	130
W13	27	26	23	19	16	12	9	132
W14	12	11	10	8	7	5	4	57
W15	27	26	23	19	16	12	9	132
W16	27	26	23	19	16	12	9	132
W17	12	11	10	8	7	5	4	57
W18	27	26	23	19	16	12	9	132
W19	27	26	22	19	15	12	9	130
W20	12	11	10	8	7	5	4	57
CA-1	7	7	6	5	4	3	2	36
CA-2	12	11	10	8	7	5	4	56
CA-3	12	11	10	8	7	5	4	56
CA-4	7	6	5	5	4	3	2	32
CB-1	12	11	10	8	7	5	4	56
CB-2	6	6	5	5	4	3	2	31
CB-3	7	7	6	5	4	3	2	35
CB-4	12	11	10	8	7	5	4	56

5. Calculate the Interstorey drift (seismic) *AS1170.0, Table C1*

$\sum Kxi$ 4.2E+05 N/mm for the entire level

$\sum Kyi$ 1.9E+05 N/mm

	Fi (kN)	F Cumul	Drift x (mm)	Drift y (mm)
R	133	133	0	1
7	129	262	1	1
6	112	374	1	2
5	95	469	1	3
4	77	546	1	3
3	60	606	1	3
2	43	648	2	3
	Max		2	3
	Total		7	16

Limit, H/300 10.3 mm Okay

1. Calculate site wind speed

AS1170.2, CL2.2

$V_{sit,B} = V_R M_d (M_{z,cat} M_s M_t)$

V_r	45	Region A3
M_d	1.0	
$M_{z,cat}$	1.0	
M_s	1.0	
M_t	1.0	
V_{sit}	45 m/s	

2. Determine the site wind pressure

AS1170.2, CL2.4

$p = 0.5 p_{air} V_{des}^2 C_{fig} C_{dyn}$

p_{air}	1.2 kg/m^3
C_{dyn}	1.0
p	1215 $\times$ C_{fig}

3. Determine aerodynamic shape factor

AS1170.2, CL5.2

$C_{fig,ext} = C_{p,e} K_a K_c K_l K_p$

$C_{fig,int} = C_{pi} K_{c,i}$

Windward Wall	
$C_{p,e}$	0.8
K_a	0.8
K_c	1
K_l	1
K_p	1

Leeward	
$C_{p,e}$	-0.5
K_a	0.8
K_c	1
K_l	1
K_p	1

Internal		
C_{pi}	-0.3	or 0
K_{ci}	1	

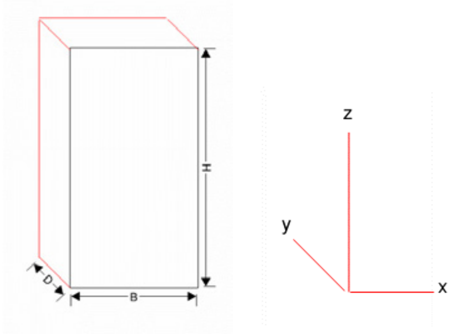
$C_{fig,\,ext}$	0.64		$C_{fig,\,ext}$	-0.4		$C_{fig\ int}$	0	or -0.3
p	0.77	kPa		-0.48	kPa		0	kPa
$p\ net$	1.26	kPa						
						Critical		

The total load tributary area is above 100m^2 - therefore k_a is assumed to be 0.8.
Assuming K_c as 1 is on the conservative side.

Wind Load Distribution

1. Area of load, per floor

Floor to floor height (h)	3.1	m
Breadth (b)	22.5	m
Depth (d)	34	m
Trib Area (x) $= hi \times B$		
Area / floor (wind x)	69.75	m^2
Area / floor (wind y)	105.4	m^2



2. Calculate Wind loads for Each Floor

FL	A m^2 (x)	A m^2 (y)	F_{wi} x (kN)	F_{wi} y (kN)
R	34.9	52.7	44.1	66.6
7	69.8	105.4	88.1	133.2
6	69.8	105.4	88.1	133.2
5	69.8	105.4	88.1	133.2
4	69.8	105.4	88.1	133.2
3	69.8	105.4	88.1	133.2
2	69.8	105.4	88.1	133.2

Fcum x	F cum y
44.1	66.6
132.2	199.8
220.3	333.0
308.5	466.1
396.6	599.3
484.7	732.5
572.9	865.7

3. Calculate inter-storey drift per floor

**Assuming rigid diaphragm*

$\sum Kxi$ 423570.0741 mm/N

$\sum Kyi$ 186897.3 mm/N

	Drift x (mm)	Drift x (mm)
R	0.1 mm	0.4 mm
7	0.3 mm	1.1 mm
6	0.5 mm	1.8 mm
5	0.7 mm	2.5 mm
4	0.9 mm	3.2 mm
3	1.1 mm	3.9 mm
2	1.4 mm	4.6 mm
Max Drift	1.4 mm	4.6 mm
Total Drift	6.5 mm	22.1 mm

**Using the cumulative loads of each level*

Limit, L/300 10.3 mm Okay

The inter-storey drift limit is subject to the building cladding and linings.
The limit L/300 is an absolute limit that should not be exceeded.

Wall No	kix	1kN x%	kiy	1kN y%
W1	37584	8.9%	0	0.0%
W2	45633	10.8%	0	0.0%
W3	8049	1.9%	0	0.0%
W4	45633	10.8%	0	0.0%
W5	37584	8.9%	0	0.0%
W6	37584	8.9%	0	0.0%
W7	45633	10.8%	0	0.0%
W8	8049	1.9%	0	0.0%
W9	45633	10.8%	0	0.0%
W10	37584	8.9%	0	0.0%
W11	0	0.0%	18651	10.0%
W12	0	0.0%	18651	10.0%
W13	0	0.0%	18651	10.0%
W14	0	0.0%	8049	4.3%
W15	0	0.0%	18651	10.0%
W16	0	0.0%	18651	10.0%
W17	0	0.0%	8049	4.3%
W18	0	0.0%	18651	10.0%
W19	0	0.0%	18651	10.0%
W20	0	0.0%	8049	4.3%
CA-1	18651	4.4%	0	0.0%
CA-2	0	0.0%	8049	4.3%
CA-3	0	0.0%	8049	4.3%
CA-4	18651	4.4%	0	0.0%
CB-1	0	0.0%	8049	4.3%
CB-2	18651	4.4%	0	0.0%
CB-3	18651	4.4%	0	0.0%
CB-4	0	0.0%	8049	4.3%
Total	423570	100.0%	186897	100.0%

The lateral load attracted for each wall, is proportional to the stiffness of the individual wall to the building stiffness in the same direction.

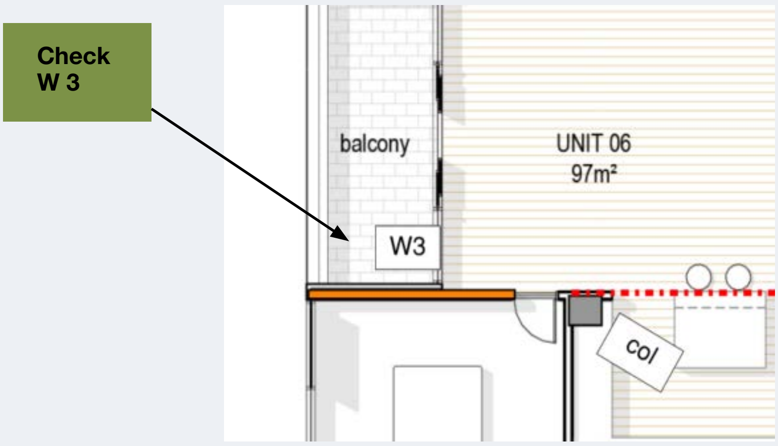
$$F_{shear,x} = \frac{F_x K_{ix}}{\sum K_{ix}}$$

$$F_{shear,y} = \frac{F_y K_{ix}}{\sum K_{iy}}$$

The lateral wind loads are calculated below. The Earthquake loads are larger at the upper levels and assumed to be critical. Both cases needs to be checked fully in practice.

4. Calculate individual wall loads (wind)

	R	7	6	5	4	3	2	Cumulative*
F (kN) Wx	44	88	88	88	88	88	88	573
F (kN) Wy	67	133	133	133	133	133	133	866
W1 - x	4	4	4	4	4	4	4	27
W2 - x	5	5	5	5	5	5	5	33
W3 - x	1	1	1	1	1	1	1	6
W4 - x	5	5	5	5	5	5	5	33
W5 - x	4	4	4	4	4	4	4	27
W6 - x	4	4	4	4	4	4	4	27
W7 - x	5	5	5	5	5	5	5	33
W8 - x	1	1	1	1	1	1	1	6
W9 - x	5	5	5	5	5	5	5	33
W10 - x	4	4	4	4	4	4	4	27
W11 - y	7	7	7	7	7	7	7	47
W12 - y	7	7	7	7	7	7	7	47
W13 - y	7	7	7	7	7	7	7	47
W14 - y	3	3	3	3	3	3	3	20
W15 - y	7	7	7	7	7	7	7	47
W16 - y	7	7	7	7	7	7	7	47
W17 - y	3	3	3	3	3	3	3	20
W18 - y	7	7	7	7	7	7	7	47
W19 - y	7	7	7	7	7	7	7	47
W20 - y	3	3	3	3	3	3	3	20
CA-1 - x	2	2	2	2	2	2	2	14
CA-2 - y	3	3	3	3	3	3	3	20
CA-3 - y	3	3	3	3	3	3	3	20
CA-4 - x	2	2	2	2	2	2	2	14
CB-1 - y	3	3	3	3	3	3	3	20
CB-2 - x	2	2	2	2	2	2	2	14
CB-3 - x	2	2	2	2	2	2	2	14
CB-4 - y	3	3	3	3	3	3	3	20



1. Determine resultant actions at wall

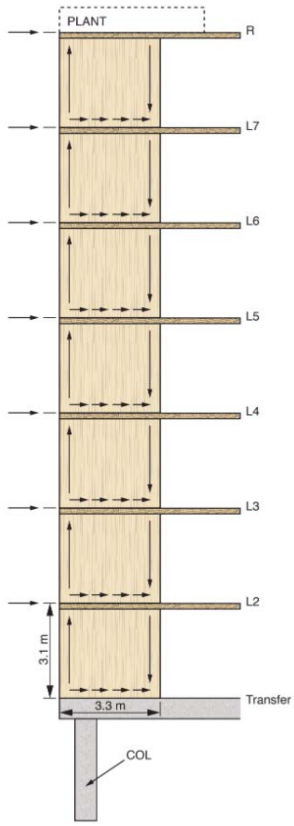
H 3.1m Floor to floor height
B 5.4m W3 Panel Breadth

Fw= $\frac{M_{3-3}}{b}$

V*

	F (kN)	Cumulative (kN)
R	3	3
7	2	5
6	2	7
5	2	9
4	1	10
3	1	12
2	1	12

M* (kNm)	T=C (kN)
8	2
23	6
46	12
73	19
106	27
142	36
180	46



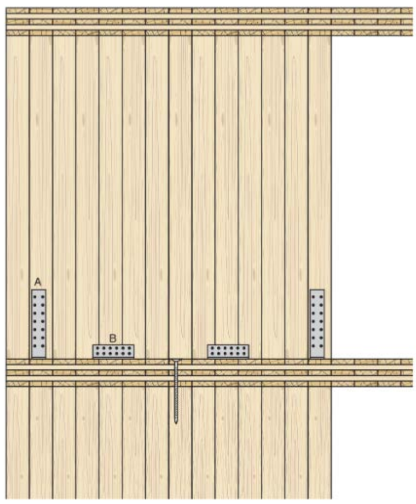
G	2.2	kPa
A	24.0	m²

2. Check for tension

	G (kN)	0.9G - Fw
R	52	50
7	104	98
6	156	145
5	208	190
4	260	233
3	313	276
2	365	318

*No tension in wall

Note, computer model confirms no tension in East/West direction



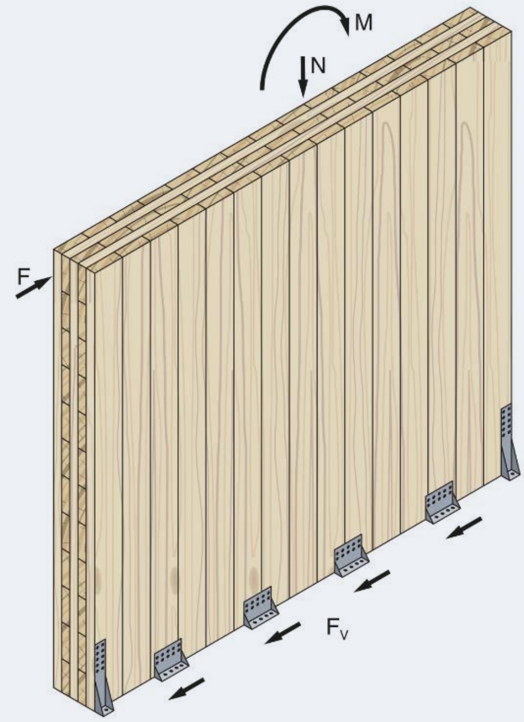
3. Critical load cases

L2 - Timber/Timber Critical Connections
*V** 11.6 kN
*T** 0 kN

Transfer - Timber/Concrete Critical Connections
*V** 12.4 kN
*T** 0 kN

CLT Shear Wall

The distribution of the forces to the shear walls typically depends on the flexibility of the diaphgragm. A rigid diaphragm can be assumed as described in the main section of the CLT Design Guide. Shear walls distribute the forces of the diaphragms in-plane.



Diaphragm Dimensions **Shear wall dimensions**

in plane

tp	90	mm
b	3900	mm
H	3100	mm

1. Calculate actions

V* (F)	12	kN	*cumulative*
M*	38	kNm	

	t_i (mm)	y_i (mm)	h_i (mm)	θ_i (deg °)	Grade	E_i (MPa)	G_i (MPa)
t_1	45	102.5	40	0	SG8	8000	533
t_2	35	62.5	0	90	SG6	200	40
t_3	45	22.5	40	0	SG8	8000	533
	125						

	f'^{b} (MPa)	f'_s (MPa)	f'_r (MPa)	E_0 (MPa)	E_{90} (MPa)	G_0 (MPa)	G_R (MPa)
SG8	14	3.8	1.2	8000	266.66	533.33	53.33
SG6	10	3.8	1.2	6000	200	400	40

1. Calculate the section properties of the in-plane bending

Reference

E 8000 MPA

$$I = \frac{d_0 b^3}{12}$$

b 90 mm
d 3900

I_{eff} 4.44893E+11 mm^4 In plane moment of inertia
E_{ieff} 3.55914E+15 mm^4

2. Calculate Section Bending Capacity

AS1720.1, CL3.2

Z 228150000 mm^3

$$\varphi M = \varphi_b k_1 k_4 k_6 k_9 k_{12} f_b Z_{eff}$$

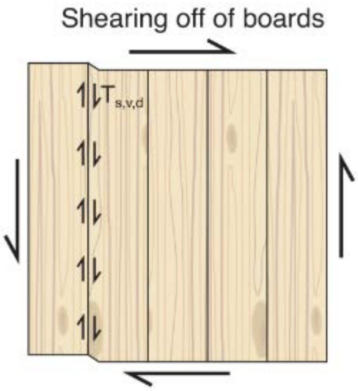
φ_b 0.85 k_1 1 k_4 1
k_6 1 k_9 1 k_{12} 1

φM 2715 kNm
M^* 38.43 kNm

Okay

1. Shearing-off failure of the boards along a joint

pro-Holz - CLT Structural Design 5.8



a. Calculate shear capacity along joints

$$f'_{v,d,i} = \phi \cdot k_1 \cdot k_4 \cdot k_6 \cdot f'_{v,i}$$

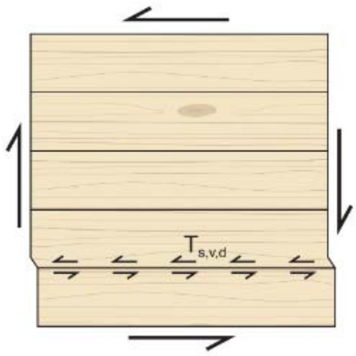
φ_s	0.8	k_1	1
k_6	1	k_4	1

f'_{vi}	1.2	MPa	
$f'(v,d,i)'$	0.96	MPa	characteristic value in shear (rolling shear strength)

$$\tau_{TD} \leq f_{v,di}$$

$$\tau_{TD} \leq \frac{T}{A_{s,net}}$$

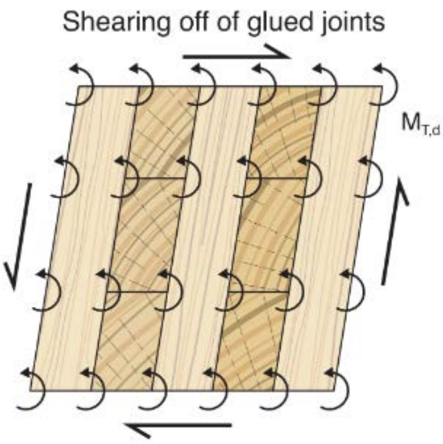
$$A_{snet} = \min(A_0, A_{90})$$



b. Calculate torsional stress

V^*	12	kN	
t_p	35	mm	Ao critical (transverse layer thinner)
L	4	m	
A_o	136500	mm²	area across length of panel
$\tau_0 = \tau_{90} =$	0.09	MPa	shear stress along the panel
	9.5%	OK	

2. Calculate for the internal torsion stress



a) Calculate torsional shear capacity

φ_s	0.8	k_1	1
k_6	1	k_4	1

f'_{vi}	1.2	MPa	
$f'(v,d,i)'$	0.96	MPa	characteristic value of torsion strength, for crossing surfaces (rolling shear strength)

$$\tau_{TD} \leq \frac{M_T}{\sum I_p} = \frac{M_T}{n_k \frac{a^4}{6}} \frac{a}{2} = \frac{3M_T}{n_k a^3}$$

$$n_k = n_s \times n_f$$

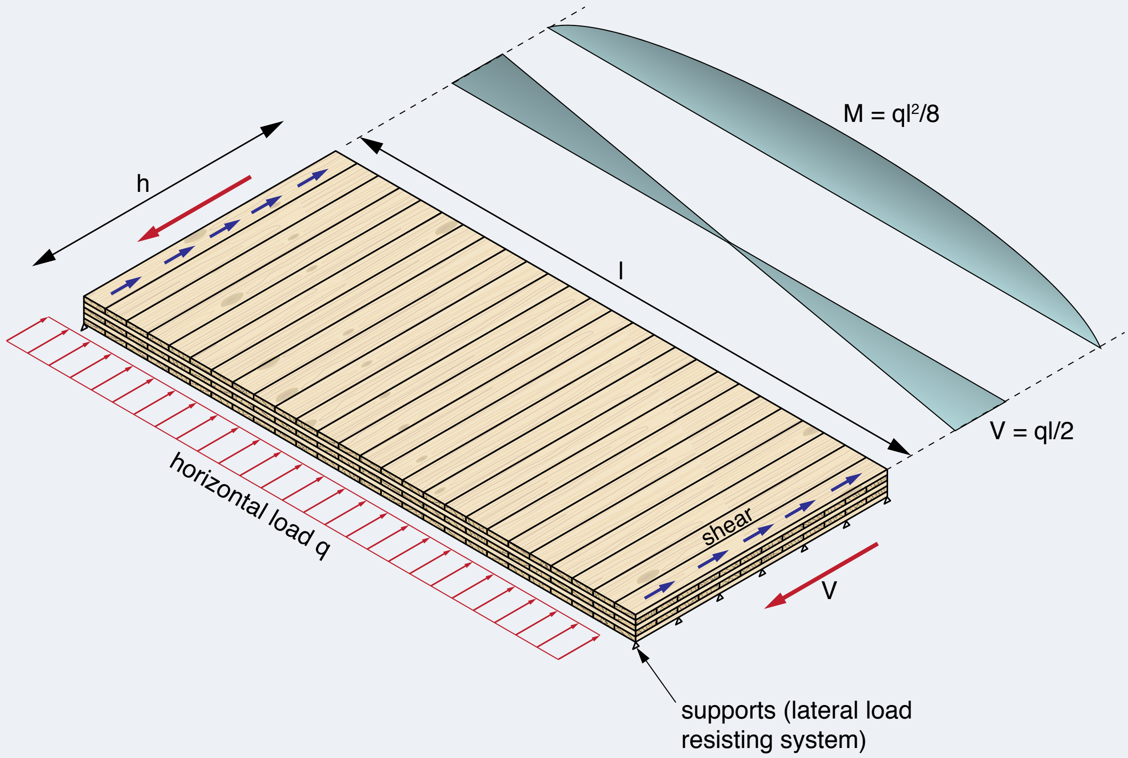
$$\tau_{TD} \leq f_{v,Td}$$

a) Calculate torsional shear capacity

h_p	3.1	m	
a	140	mm	confirm with manufacturer. (Xlam's typically min 140 for this calc)
I_p	$a^4/6$		polar moment of inertia, of a square intersection field
n_s	3		Number of glued joints between layers
n_f	490		Number of intersection fields
n_k	1470		
M_{td}	180.0	kNm	Max moment
τ_{TD}	0.13	MPa	
	13.9%	Okay	

The distribution of the forces into the lateral load-resisting system depends on the flexibility of the diaphragm (rigid or flexible). A diaphragm is considered to be flexible, if its deformation is more than twice the average inter-story drift at that level. For flexible diaphragms, the load can be determined by the tributary area approach.

Looking at the calculated loads, the diaphragm between W1 and W2 appears to be critical and will be designed for.



	t_i (mm)	y_i (mm)	h_i (mm)	a1	Grade	E_i (MPa)	G_i (MPa)
t1	45	202.5	90	-22.5	SG8	8000	533
t2	45	157.5	45	-67.5	SG6	6000	40
t3	45	112.5	0	0	SG6	6000	400
t4	45	67.5	45	45	SG6	6000	40
t5	45	22.5	90	-22.5	SG8	8000	533

	f'_b (MPa)	f'_s (MPa)	f'_r (MPa)	E_0 (MPa)	E_{90} (MPa)	G_0 (MPa)	G_R (MPa)
SG8	12	3.8	1.2	8000	266.66	533.33	53.33
SG6	10	3.8	1.2	6000	200	400	40

The diaphragm loads are calculated by combining the reactions of all of the shear walls along the diapgragm, and then applying them as an equivalent line load. The diaphragm will be checked for L7, deemed twith critical Earthquake loads.

1. Shearing-off failure of the boards along a joint

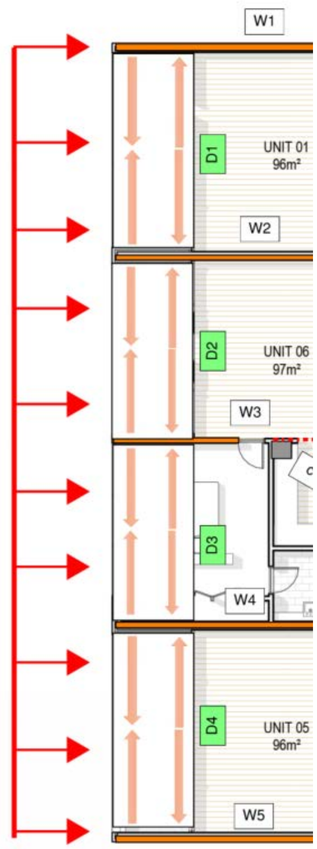
Resultant forces

W1	18.4	kN
W2	18.0	kN
W3	2.5	kN
W4	17.7	kN
W5	18.5	kN
Total Load (kN)	56.6	kN
Wall Length	22.5	m

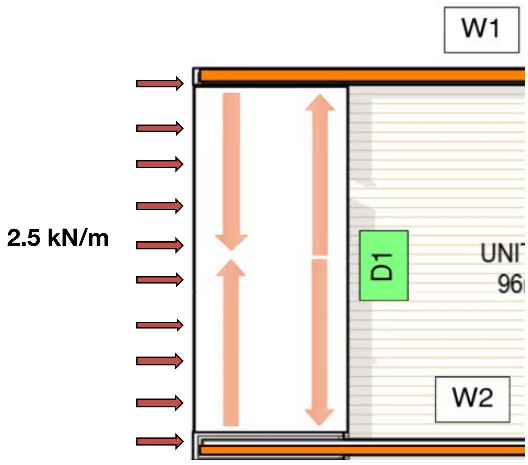
Equiv line load **2.5** **kN/m** *across D1, D2, D3, D4*

Diaphragm Dimensions

tp	135	mm	*3 layers of 45 mm in-plane*
b	2400	mm	*panel cross length*
L	5300	mm	



The line load is then applied to the bottom diaphgram bay.



2. Calculate diaphragm actions

M*	8.83	kNm
V*	6.66	kN

Reference

1. Calculate the section properties of the diaphragm

E 8000 MPA

$$I = \frac{d_0 b^3}{12}$$

b	135		
I_{eff}	1.5552E+11	mm^4	*In plane moment of inertia*
E_{ieff}	1.24416E+15		

2. Calculate Section Bending Capacity *AS1720.1, CL3.1*

Z 129600000 mm^3

$$\varphi M = \varphi_b k_1 k_4 k_6 k_9 k_{12} f_b Z_{eff}$$

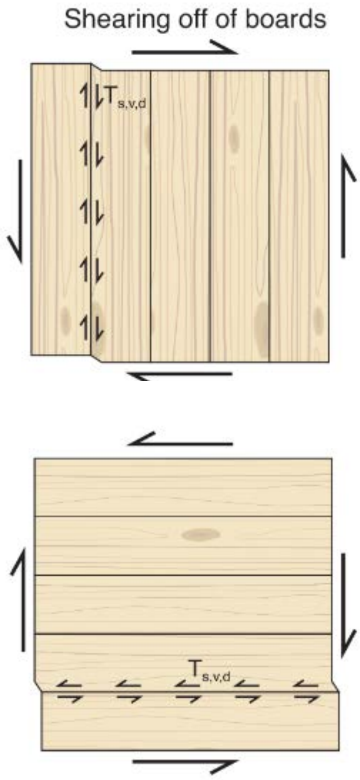
φ^b	0.85	k_1	1	k_4	1
k_6	1	k_9	1	k_{12}	1

φM	2715	kNm
M^*	38.43	kNm

Okay

1. Shearing-off failure of the boards along a joint

pro-Holz - CLT Structural Design 5.8



a. Calculate shear capacity along joints

$$f'_{v,d,i} = \phi \cdot k_1 \cdot k_4 \cdot k_6 \cdot f'_{v,i}$$

φ_s	0.8	k_1	1
k_6	1		

f'_{vi}	1.2	MPa	
$f'(v,d,i)'$	0.96	MPa	characteristic value in shear (rolling shear strength)

$$\tau_{TD} \leq f_{v,di}$$

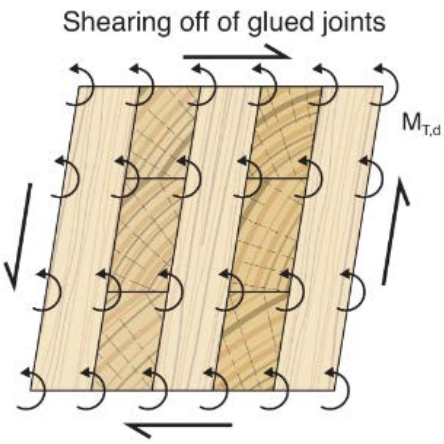
$$\tau_{TD} \leq \frac{T}{A_{s,net}}$$

$$A_{snet} = \min(A_0, A_{90})$$

b. Calculate shear stress along the joints

V^*	7	kN	
t_p	90	mm	Ao critical (transverse layer thinner)
L	4	m	
A_o	216000	mm^2	area across length of panel
$\tau_0 = \tau_{90} =$	0.03	MPa	shear stress along the panel
	3.2%	OK	

2. Calculate for the internal torsion stress



a) Calculate torsional shear capacity

φ_s	0.8	k_1	1
k_6	1		

f'_{vi}	1.2	MPa	
$f'(v,d,i)'$	0.96	MPa	characteristic value of torsion strength, for crossing surfaces (rolling shear strength)

$$\tau_{TD} \leq \frac{M_T}{\sum I_p} = \frac{M_T}{n_k \frac{a^4}{6}} \frac{a}{2} = \frac{3M_T}{n_k a^3}$$

$$n_k = n_s \times n_f$$

$$\tau_{TD} \leq f_{v,Td}$$

a) Calculate torsional stress

h_p	3.1	m	
a	90	mm	
I_p	$a^4/6$		polar moment of inertia, of a square intersection field
n_s	3		Number of glued joints between layers
n_f	1186		Number of intersection fields
n_k	3558		
M_{td}	20.7	kNm	Max moment
τ_{TD}	0.02	MPa	
	2.5%	Okay	

CLT - Connection Summary

There are multiple appraoches to CLT connection design. What is presented in this worked example is just one method, and it must be noted that the full complexity of the design isn't explored. Engineers need to also design for additional cases including robustness, fire and temporary support cases.

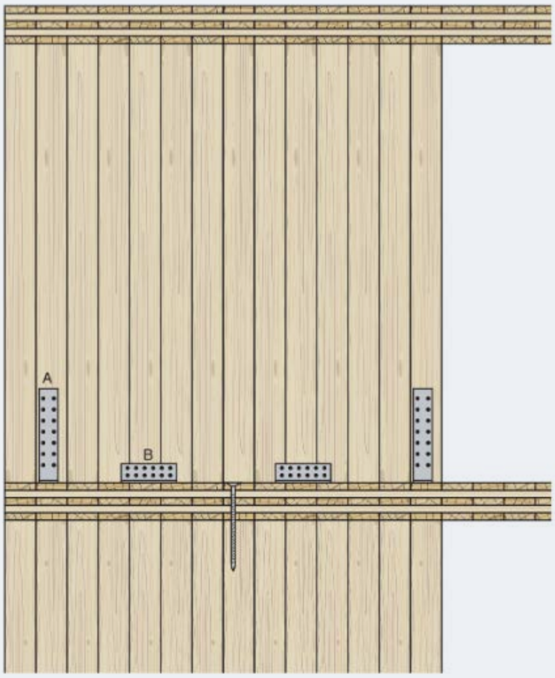
The configuration selected for this worked example is illustrated below.

The distribution of the forces in the panel is outlined in the illustration below.

Tensile forces - the tensile force, induced by the moment on each floor is resisted by the tensile brackets indicated by 'A' below.

Shear forces - are transferred from the diaphragm to the shear walls by the angle brackets indicated by 'B' in the illustration below.

The cumulative shear and tensile forces - are then transferred from floor to floor by screws, indicated by Illustration A and B below.



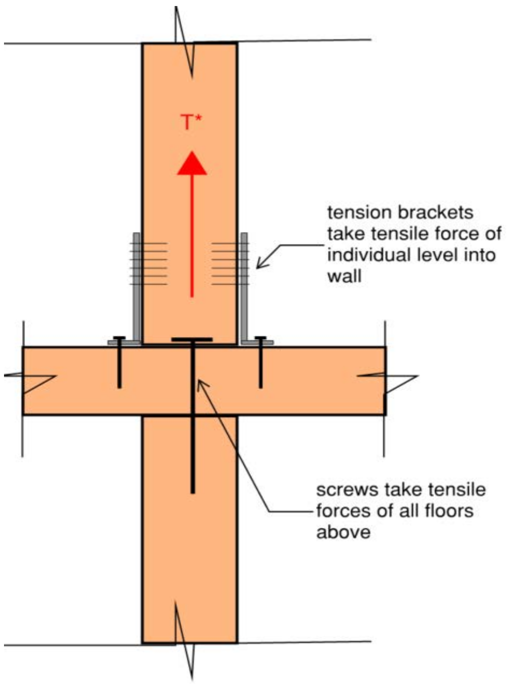


Illustration A

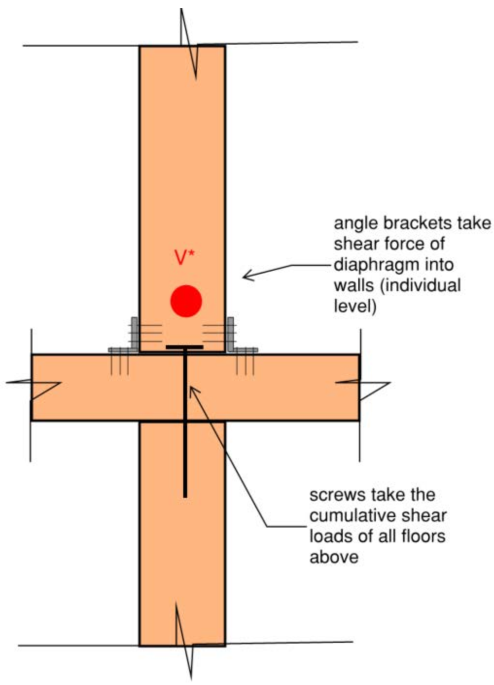


Illustration B

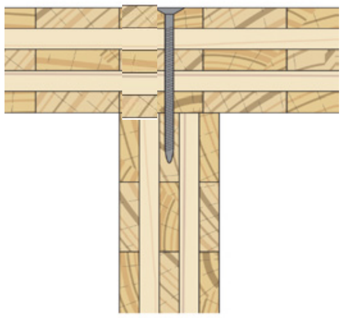
The characteristic value, or withdrawal resistance is determined by:

1. Withdrawal failure of threaded part in wall member (threaded part)

2. Head pull-through failure in floor member (head side), which is a maximum of the head pull through, and withdrawal in the floor member)

3. Tensile failure of steel

tp 225 mm



(a)

Fastener nom diameter d 8mm

Countersunk head with washer

tp 175 mm

1. Determine withdrawal capacity in the wall section (threaded part of screw) *Eurocode 5 (EC5)*

a) $f_{h,k,1} = \dfrac{n_{eff} f_{ax,k} d_1 l_{eff}}{1.2cos^2\emptyset + sin^2\emptyset} \times \left(\dfrac{\rho_k}{350}\right)^{0.8}$ withdrawal of timber 1

n_{eff}	1		
n_{eff}	1.00		
$f_{ax,k}$	12	N/mm²	characteristic value of withdrawal, from supplier (Spax EC5 V09.2015)
$d1$	8	mm	outer thread diameter
l_{eff}	200	mm	penetration depth (in the wall system only)
p	465	kg/m³	
ϕ	0	degrees	side grain
$f_{ax,rk}$	20.1	kN	through wall panel

2. Determine withdrawal capacity in the floor section (a, withdrawal, and b, head pull-through) *Eurocode 5 (EC5)*

a) $f_{h,k,1} = \dfrac{n_{eff} f_{ax,k} d_1 l_{eff}}{1.2cos^2\emptyset + sin^2\emptyset} \times \left(\dfrac{\rho_k}{350}\right)^{0.8}$ withdrawal of timber 2

Note the withdrawal mechanism check isn't required for partially threaded screws, as there is no withdrawal capacity apart from the head (there is no threaded section through the floor)

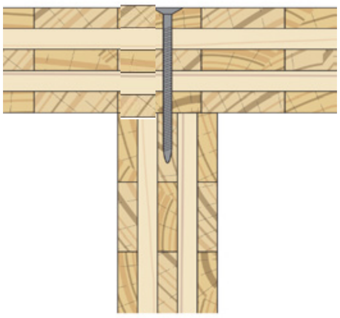
b) $f_{h,k,1} = n_{eff} f_{head,k} d_h^2 \left(\dfrac{\rho_k}{350}\right)^{0.8}$ head pull-through

n_{eff}	1		
n_{eff}	1.00		
p	465		
d_h	20	mm	
$f_{head,k}$	14	MPa	
$f_{h,k,1}$	7.03	kN	
$f_{h,k,1}$ adopted	7.03	kN	withdrawal value

f,head k values (N/mm²)

d	Countersunk	Countersunk with washer
<16mm	27 - dh	29-dh
16<d1<22mm	11-0.2(dh-16)	13
22<d1<32mm	11-0.2(dh-16)	16-0.5(dh-16)

tp 225 mm



(a)

d	10	mm	Fastener nom diameter
d_s	4.88	mm	Shank diameter
d_{eff}	5.37	mm	Fastener eff diameter

tp 175 mm

*8 x 425 (225+200) mm Partly Threaded Washer Head Screw

1. Calculate critical shear of the 6 different failure mechanisms (Johanson Theory)

a) Embedment Failure of Timber 1

$$F_{v,Rk,a} = f_{h,1,k} \cdot t_1 \cdot d$$

$$f_{h,k,1} = \frac{0.082 p_k d_1^{-0.3}}{2.5cos^2\emptyset + sin^2\emptyset}$$ *embedment strength of timber, non pre-drilled (parallel to grain of CLT)*

p_k	465 kg/m^3	*timber density*
d_1	8 mm	
ϕ	90 degrees	
$f_{(h,k,1)}$	20.43 MPa	
t_{pa}	225 mm	
$F_{(v,Rk,a)}$	36.8 kN	

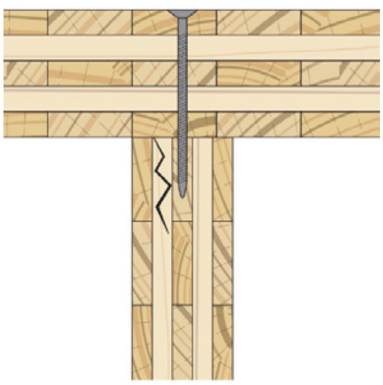
b) Embedment Failure of Timber 2

Spax Design Guide - EC5

$$F_{v,Rk,b} = f_{h,2,k} \cdot t_2 \cdot d$$ *embedment strength of timber, non pre-drilled (parallel to grain of CLT)*

$$f_{h,k,2} = 20d^{-0.5}$$ ***in wall (parallel to grain)*

d	8 mm
t_{pb}	200
$f_{(h,k,1)}$	21.2 MPa
$F_{(v,Rk,b)}$	33.9 kN

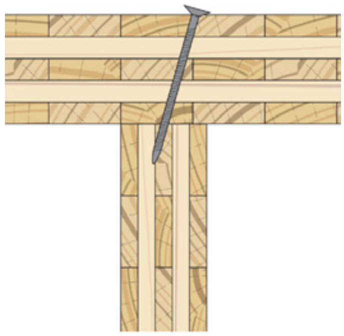


(b)

c) Joint embedment failure of timber 1 and 2

Eurocode 5 (EC5)

$$F_{v,Rk,c} = \frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right] + \beta^3 \left(\frac{t_2}{t_1}\right)^2} - \beta \left(1 + \frac{t_2}{t_1}\right) \right] + \frac{F_{ax,Rk}}{4}$$



(c)

$f_{(h,k,1)}$	20.43336106	MPa	
t_1	225	mm	
t_2	175	mm	
t_2/t_1	0.78		
d	8	mm	
$F_{ax,r}$	7.03	kN	

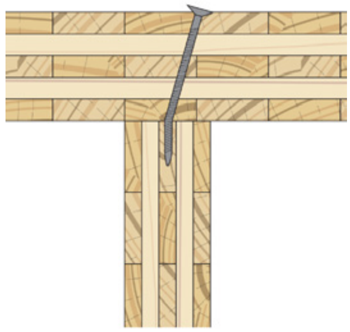
$$\therefore \beta = \frac{f_{h,2,k}}{f_{h,1,k}}$$

β	1.04	embedment ratio
Rope	1757.259754 N	rope effect ($F_{ax,Rk}/4$)
$F_{(v,Rk,c)}$	25.51 kN	

d) Plastic failure of Screw and Embedment Failure in Timber 1 (floor component)

Eurocode 5 (EC5)

$$F_{v,Rk,d} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta \right] + \frac{F_{ax,Rk}}{4}$$



(d)

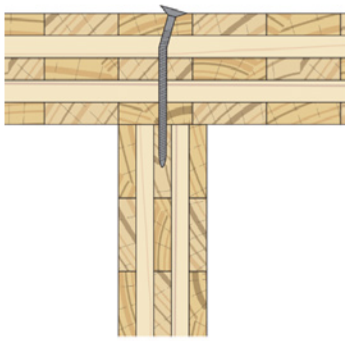
$M_{y,Rk}$	20000	Nmm	characteristic value of yield moment (as defined in ETA)
$F_{v,Rk,d}$	14.80	kN	

e) Plastic failure of Screw and Embedment Failure in Timber 1 (wall component)

Eurocode 5 (EC5)

$$F_{v,Rk,e} = 1{,}05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta} \cdot \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta \right]$$

$F_{v,Rk,e}$ 12.20 kN



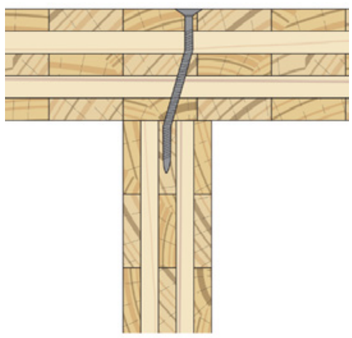
(e)

f) Double Plastic Failure of Screw

Eurocode 5 (EC5)

$$F_{v,Rk,f} = 1{,}15 \cdot \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4}$$

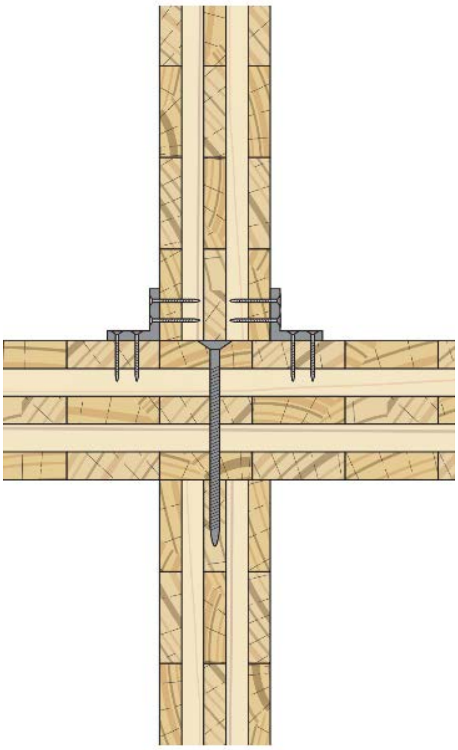
$F_{v,Rk,f}$ 4.73 kN



(f)

A	B	C	D	E	F	G
36.78	33.94	25.51	14.80	12.20		4.73
kN	kN	kN	kN	kN		kN

Qk 4.73 kN *Critical shear failure*



Detail B

2. Design capacity as according to AS1720.1 *AS1720.1*

$$R_{d,j} = \phi \cdot k_1 \cdot k_{13} \cdot k_{14} \cdot k_{17} \cdot n \cdot Q_k$$

φ_b	0.85		k_{14}	1
k_{13}	1		k_{17}	1
n	1	screws		
		Lateral load case		
k^1		1.14		
$R_{d,j}$	4.58	kN		
V^*	12	kN	*note lateral load case only, at L3 junction*	
Wall Length	3.9			
V^*/m	3.0			
Min Spacing	1542	mm	*min spacing calculated*	

Adopt 300mm spacing

1. Design angle bracket fastener in shear *AS1720.1, CL3.1*

$\emptyset N_{dj} = \emptyset k_1 k_{13} k_{14} n Q_k$

φ_s	0.8		
k_1	1.14		*AS1720.1*
k_{13}	1		*side grain*
k_{14}	1		*single shear*
Q_k	70	kN	*From connection manufacturer (Rotho Titan TTF 200selected)*
n	1		

ϕN_{dj}	63.8	kN
N* Fastener	11.6	kN
Required Fasteners	**1**	

Image: Soundproofing Solutions DocPlater.Net

Note - no tension. Pull-out screws with 150mm embedment included for robustness purposes

2. Design pull out bolt in tension

$$\emptyset N_{dj} = \emptyset Q_k$$

φ_s	0.8		
Q_k	31.4	kN	*From connection manufacturer (Rotho Titan WHT 340 Selected)*

ϕN_{dji} 25.12 kN/connection

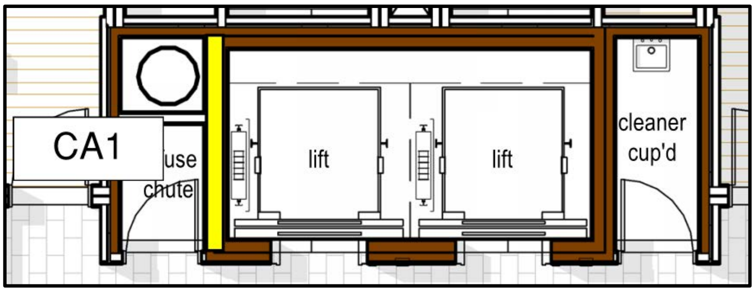
*No tension - Okay

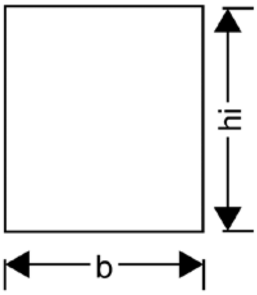
This connection design example is very simplistic and not comprehensive. It doesn't include:

- Design for temporary condition
- Fire design
- Robustness (e.g double spanning floors)

WoodSolutions intends to release a comprehensive Design Guide specifically for connections.

Check CA1





1. Determine resultant actions at wall

hi 3.1 m
b 2.7 m

$$Fw = \frac{M_{3-3}}{b}$$

	F (kN)	Cumulative V* (kN)
R	7	7
7	7	14
6	6	21
5	5	26
4	4	30
3	3	33
2	2	36

M* (kNm)	T=C (kN)
23	8
67	25
131	48
210	78
303	112
406	150
516	191

G	2.6	kN/m
G	7.0	kN/floor

*Note, no floor spans onto core wall. Therefore, dead load from wall SW only.

2. Check for tension

	G (kN)	0.9G - Fw (kN)
R	7	-1
7	14	-11
6	21	-27
5	28	-50
4	35	-77
3	42	-108
2	49	-142

-142.0 kN No tension

3. Critical load cases

L2 - Timber/Timber Critical Connections
V* 33 kN
T* -108.20 kN **Critical cumulative load taken*

Transfer - Timber/Concrete Critical Connections
V* 36 kN
T* -141.98 kN

1. Design screws in shear

V^*	25.7	kN	*Cumulative load at L5 (entire wall)*
$R(d,j)$	4.58	kN	*Capacity per screw*
L	2.7	m	
S	481	mm	*Min spacing for screws (shear)*

2. Design fastener in shear

ϕVdj	63.8	kN	
V^*	26	kN	
N	1		*Minimum fasteners required*

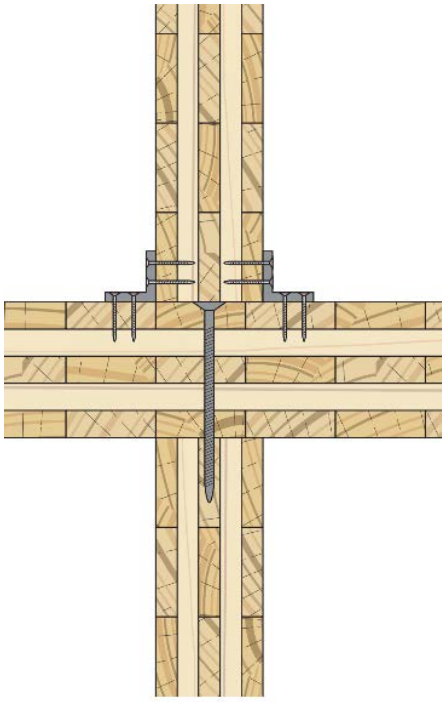
3. Design screws in tension

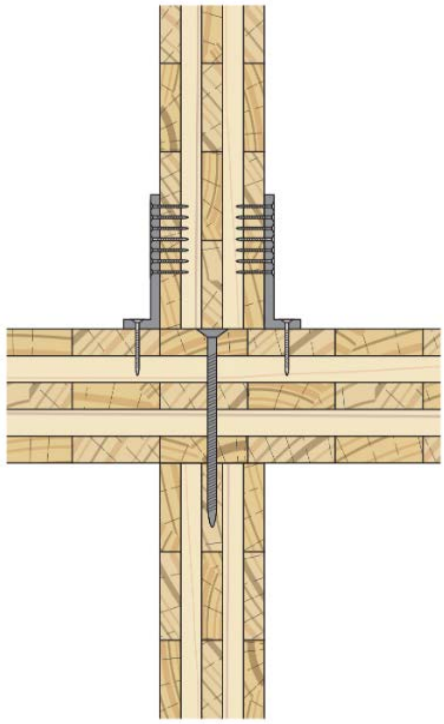
ϕTdj	5.6	kN	
T^*	-49.8	kN	
N	9.0		*"Minimum screws each face required (for tension)"*

Adopt 100mm spacing Okay

4. Design hold-down bracket for tension

ϕTdj	25.1	kN	
T^*	-49.8	kN	
N	2		*Minimum fasteners required*





Step 7 : Check Robustness

Not included in WE

51

WoodSolutions™
design and build
woodsolutions.com.au

This guide was published in Nov 2019
Please check woodsolutions.com.au/publications to ensure you have the latest edition

Cost Engineering of Mid-rise Timber Buildings

Technical Design Guide issued by Forest and Wood Products Australia

WoodSolutions Technical Design Guides

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

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

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

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

More WoodSolutions Resources

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

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

Wood Solutions™
design and build
woodsolutions.com.au

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

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

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

ISBN 978-1-925213-71-3

Authors
WoodSolutions: Laurence Ritchie and Paolo Lavisci
Rider Levett Bucknall: Natasha Carter and Trent Kennedy

Acknowledgements
We wish to acknowledge the significantly useful contribution and support received from
Nick Milestone (TRADA, UK)
Philip Zumbrunnen (Eurban, UK)
Josh Russell and Jules Tribuzio (Multiplex, AUS)
George Konstandakos and David Luttrell (Lendlease, AUS)
Keven Durand (Nordic, CAN)
Milos Slavik, (Rothoblaas, AUS)
Daniel Wright, (ASH, AUS)
Duncan Mayes, (Timberlink, AUS)
Craig Kay, (Tilling, AUS)

First Published: November 2019

Contents

Contents

1 Introduction

This guide provides reference data and methodology advice for cost engineering activities directly and indirectly associated with the design, procurement and installation of wood structures in Australia, especially with reference to mid-rise buildings (four or more levels).

There has been a rapid growth in the use of Engineered Wood Products (EWPs) across the property industry. Since the delivery of Australia's first mid-rise contemporary wood building – Forté Living from Lendlease – structural wood products have been used to construct a variety of buildings across the country. This trend has been supported by amendments to the National Construction Code (NCC), which since 2016 has provided a Deemed-to-Satisfy (DtS) solution for timber construction to an effective height of 25 metres (from 2019 applicable to all building classes), and has been further backed by the WoodSolutions free advisory program in this field.

While many detailed guides have been published on the design and maintenance of timber structures, there has been relatively little focus on the specifics of costing and ultimately building them. This guide has been prepared to address the specific cost-related knowledge and approach that needs to be considered throughout the development process, with respect to the Australian Cost Management Manual and other relevant publications.

Written in conjunction with Rider Levett Bucknall, a leading independent organisation in cost management, quantity surveying, project management and advisory services, this guide has been divided into sections associated with the typical activities of a cost engineer. While each section is complete on its own, the reader will gain most benefit by considering them to be inter-related, and it is recommended that the document is read as a whole.

This guide refers to projects that are based on either individual Wood Products or systems, or their combinations. Further information on the nature, performances and design of such products and systems can be found, with a comprehensive and comparison-based approach, in *WoodSolutions Technical Design Guide #46 Wood Construction Systems*. Other guides within the WoodSolutions library offer more detailed information specific to a given application.

This guide identifies and explains the typical differences between the costing of wood-based projects with respect to those using other systems, as described by the six main categories shown in Figure 1.2. Drawing on examples from a database of relevant completed projects, the text discusses several considerations under each category of difference, suggesting how they may be best measured and allowed for. Table 1.1 summarises what these differences have typically been shown to include.

Figure 1.1: The office built by Lendlease at 25 King Street, Brisbane, embodies the advantages of timber construction for all six categories described in Figure 1.2 (Lendlease)

Figure 1.2: The main advantages of using Wood Products with respect to other systems.

Table 1.1: Summary of the main advantages of using Wood Products.

Differential value	• Cost-effective elements are possible for some optimised designs, due to the high strength-to-weight ratio of Wood Products. • Fewer variations and defects are typically achieved thanks to tighter dimensional tolerances. • Increase in net sellable or rentable areas in some cases, with reduced wall depths for equivalent structural, fire and acoustic combined performances.
Improved safety	• Larger working platforms and pre-installed edge protection are typical. • Simpler and easier anchoring for safety harness in most points. • Easier handling and fixing, with smaller and lighter tools. • No hot works or welding on site.
Faster delivery	• Follow-on trades can start immediately, with no props and curing time, because the floors are readily stable and load-bearing. • Higher number of panels for each truck delivery. • Accelerated construction programs can lead to lower cost of finance. • BIM-ready with shop drawings typically from manufacturers.
Lower preliminaries	• Lifting is quicker and can be achieved with smaller cranes and/or shorter equipment rental times and related costs. • Shorter on-site programs resulting in reduced costs of temporary works. • Scaffolding can be limited or even avoided in some instances. • Storage areas are reduced in size and can be easily organised on floors. • Site accommodation is reduced as the crews are smaller, with more work happening off site.
Reduced foundations	• Lighter above-ground structure with respect to reinforced concrete, reducing foundation size for the same sized superstructure, and significantly improving designs in weak soils and vertical extensions over existing buildings. • Higher built volumes have been possible in some projects, a big plus for the developer.
Lower impacts	• Less noise, dust, vibrations and truck movements result in less disruption of neighbourhoods and existing activities and tenants. • Credits for CO_2 storage (carbon sink) and/or renewable materials are sometimes applicable. • Demonstrated benefits on occupants' health and wellness may justify higher rental fees.

Figure 1.3: The Monash Peninsula student accommodation in Frankston, a Passive House design for 150 beds and related amenities from JCB Architects and AECOM (engineers), was completed in 2018 by Multiplex only 16 months after being engaged for the D&C contract, with a track record of three months for assembling the carpentry, zero site accidents and significant time savings for the installation of services and interior finishes. Images: http://www.jcba.com.au and Multiplex.

1.1 Definitions

Several terms relevant either to timber construction or development in general have been used in this guide. While an in-depth explanation of many of these terms is available in *WoodSolutions Technical Design Guide #46 Wood Construction Systems*, a brief set of definitions has been provided here for ease of reference.

Bracing Panel: A panel resisting shear loads in either vertical or horizontal planes that adds overall stiffness to a structure.

Computer Numerical Controlled (CNC) Machine: An automated electro-mechanical device that manipulates shop tools using computer programming inputs. In the fabrication of Engineered Wood Products, CNC machines are used to cut elements into the designed shapes through cutting, drilling, routing, or other processes, typically with millimetre precision.

Connectors: The items used to connect one element to another. Typically made of steel, aluminium or timber, connectors include products such as: screws, bolts, nails, dowels, nail plates, angle brackets, flat plate brackets, and bespoke designs.

Crew: The group of people involved in the installation of engineered timber products. Typically, a crew will consist of 4–8 people, with work tasks including setting out/organising, installing/landing panels, and nailing/fixing.

Cross-laminated Timber (CLT): A panel composed of layers of solid wood boards, typically 12-45 mm thick and 40-300 mm wide, finger-jointed, face-flued (and edge-glued in some instances), each layer at 90° to the next. CLT panels are typically 57 mm – 320 mm thick and made up of 3, 5, 7 or more layers. Panels are available in 1.25 to 3.5 metre widths and up to 16 metre lengths. More detailed information on CLT can be found in *WoodSolutions Technical Design Guide #16 Massive Timber Construction Systems: Cross-laminated Timber (CLT)* and in the producers' technical literature.

Deemed-to-Satisfy (DtS) Solution: A design that follows the Deemed-to-Satisfy Provisions in the NCC, which includes materials, components, design factors and construction methods that, if used, are deemed to meet the Performance Requirements.

Fire Protected Timber: A timber element that, through the application of a non-combustible fire protective lining, has an FRL appropriate for that element.

Glued-laminated Timber (Glulam): A timber element consisting of a number of strength-graded, kiln-dried, finger-jointed laminations, face bonded together with adhesives. Elements can be manufactured to practically any length, size or shape. Beams are often manufactured with a built-in camber to accommodate dead load deflection or curved to follow the design. A wide variety of strength grades and section sizes are available, with depths from 90 mm to more than 1,000 mm, and thicknesses from 40 mm to more than 135 mm.

Laminated Veneer Lumber (LVL): Engineered Wood Product made from peeled veneers that are bonded together with adhesive under heat and pressure to form panels. Most veneers are oriented so their grain runs parallel, leading to high capacities of compression parallel to the grain. LVL can either be used as a massive timber element (e.g. a panel, beam, or column) or cut into smaller sections for framing.

Lift: The operation (or amount of time) for a crane on-site to hook a load, lift it to the desired location, unhook it, and return the hook to the starting position.

Lightweight Framing (or Stud Framing): Timber frame construction assembled from lightweight timber studs and plates with either wood panel or metal strap bracing, and using fasteners like nails, staples or screws as connectors. Individual timber products used in framing generally have at least one dimension measuring 45 mm or less, but multiple or staggered studs can be used to increase design strength and stiffness properties.

Massive Timber: In the NCC, an element not less than 75 mm thick as measured in each direction formed from solid or laminated components.

Modified Resistance to the Incipient Spread of Fire (MRISF): As per definition for RISF, but allows for the delay in temperature rise experienced with massive timber elements under fire load by specifying the minimum amount of time taken for the surface of the timber element to reach 300°C for the system to comply with the DtS Solution.

Resistance to the Incipient Spread of Fire (RISF): In the NCC, the ability of the ceiling or wall membrane to insulate the space between the ceiling or wall and the adjacent sole occupancy unit, so as to limit the temperature rise of materials in this space to a level which will not permit the rapid and general spread of fire throughout this space.

Tabulated rates: Costing rates made available in tabular form in commercially published guides such as Rawlinson's Construction Cost Guide, or the Australian Institute of Quantity Surveyors' 'Building Cost Index'.

Timber: Short synonym for 'structural wood component' or 'Engineered Wood Product (EWP)'.

Take-off: A detailed measurement of materials and labour needed to complete a construction project.

Variation: Alteration to the scope of works in a construction contract in the form of an addition, substitution or omission from the original scope of works. This can occur because of technological advancement, statutory changes or enforcement, change in conditions, geological anomalies, non-availability of specified materials, or simply because of the continued development of the design after the contract has been awarded. Also known as a variation instruction or change order.

Figure 1.4 – The Kambri precinct of Australian National University (Canberra, 2019) incorporates two large mass timber construction buildings: the 450-bed student accommodation and the five-storey collaborative teaching building. Designed by BVN (architects) and constructed by Lendlease, the student accommodation was also the first building in Australia to use a new prefabricated facade system from Inclose that enabled 13-metre-long fully complete brick facade sections to be installed within 20 minutes. Images: Lendlease

2 Database

The content of this guide has been developed and supported by the observations and experiences recorded from 26 significant projects, sourced from both Australian and international markets. Special care has been taken to ensure that all the international projects were completed in a similar work environment to that in Australia, with a major focus on Occupational Health & Safety (OH&S), a similar magnitude of labour costs, hire costs, and so on.

While all identifying characteristics of these projects have been withheld for confidentiality reasons, their relevant metrics are described in Table 2.1.

Table 2.1: Fundamental metrics of the reference projects built with Engineered Wood Systems.

#	Type	Levels	Crew	Crane(s)	m^3 EWP	Lifts	m^2 GIA	Lifts/d	GIA/d
1	Residential	6	8	2	1,510	936	4,682	16	78
2	Residential	8	5	1	1,082	994	3,072	20	61
3	Residential	8	8	2	1,853	1,299	6,154	16	77
4	Residential	8	8	1	1,515	1,323	4,154	19	59
5	Residential	7	7	4	3,930	1,514	12,276	14	76
6	Residential	7	5	1	1,506	1,310	3,883	17	49
7	Residential	10	7	2	4,649	2,810	14,001	14	68
8	Residential	6	5	1	533	514	16,84	16	42
9	Residential	8	5	2	3,170	1,586	9,797	14	86
10	Residential	9	5	2	2,967	1,622	8,650	16	82
11	Residential	8	5	2	2,735	1,486	8,032	16	85
12	Residential	7	5	2	1,667	839	4,887	16	81
13	Residential	9	5	1	1,000	759	2,431	13	41
14	Residential	18	9	1	2,233	n.a.	15,120	n.a.	336*
15	Residential	9	5	1	926	n.a.	2,890	n.a.	107
16	Residential	4	12	2	2,208	1,557	5,824	28	106
17	Residential	6	12	2	1,750	935	6,500	17	96
18	Residential	9	8	1	2,700	1,376	9,478	16	110
19	Residential	7	7	1	2,300	1,534	9,330	21	128
20	Residential	8	7	1	3,084	2,258	13,166	25	145
21	Residential	7	9	1	2,525	1,541	9,896	19	124
22	Residential	12	6	1	3,084	2,346	10,220	14	59
23	Commercial	7	5	1	1,162	503	4,149	14	40
24	Commercial	10	n.a.	1	6,270	3,097	14,900	24	115
25	Commercial	7	8	1	2,950	1,750	7,910	19	88
26	Commercial	7	8	1	4,700	1,750	10,000	17.5	100

Note: Residential includes also similar floorplans such as aged care, hotel and student accommodation.
** this project experienced significantly faster installation times due to extensive planning and an efficient design*

The projects referred to in Table 2.1 vary in height, floor area, usage and site setup. The average project in this database is eight storeys with a Gross Internal Area of 7,811 m^2, and requires an installation team of just seven people to complete the structure at a rate of 94 m^2 per day. These metrics represent the average of a sample and may not be directly applicable to non-standard projects. When segregated by building use, these metrics refer to:

- ***residential*** projects on average extending to eight storeys with a Gross Internal Area of 7,551 m^2, and requiring seven people to install at a rate of 95 m^2 per day
- ***commercial*** projects averaging seven storeys, with a Gross Internal Area of 9,240 m^2, and required on average an installation team of seven to achieve an installation rate of about 86 m^2 per day.

While there are a number of factors that could explain these slight differences in productivity, it has widely been observed that the most significant factor in a project's overall productivity is its number of storeys. Evidence suggests that installation rates are slowest where precisely cut timber elements interface with less accurate in situ concrete surfaces or steel elements, often requiring packing and levelling to achieve a tight assembly fit or an even footing. Once the first level is complete, timber projects experience an immediate and substantial increase in productivity, with install rates quickly making up for any time lost in the first level. Variance in install speeds can also be due to design complexity, size of elements, typology of building, climactic conditions, number of cranes, level of prefabrication, or a range of other variables.

Figure 2.1: The Brock Commons student accommodation (Vancouver, CAN – 2017) is an 18-storey post and slab structure in glulam and CLT with concrete cores and a prefabricated facade, designed by Acton Ostry Architects. It was considered by the local Fire Brigade to be "the safest building of this type in town" because of encapsulation of most of the Engineered Wood structure which, together with the prefabricated façade, Urban One Builders erected in only 66 days with a crew of nine.
Images: Top left Seagate Structure; top right Neil Taberner; bottom: Michael Elkan.

3 Estimating

The process of cost estimation differs little between timber and other construction systems. Nevertheless, there are several key areas for consideration in any estimate of a timber structure to ensure that the outcome represents the design as accurately as possible:

- Reference to ***tabulated rates*** is seldom available and, when it is, typically has quite large safety factors because the evolution of the products (with many being proprietary products instead of commodities) and the local conditions (with the influence of logistics) account for significant differences
- Price ***variations over time*** are typically smaller than for other materials and/or may have a different origin. Depending on the location of the supplier of timber elements, costs may also change subject to the season in which they are purchased. For example, peak supply period for European suppliers is between April and December and purchases within this period may attract a higher rate than those in the off-peak period (European winter). This cost differential is less pronounced in Australia, as construction typically proceeds year round.
- ***Preliminaries and workmanship*** are normally a significant part of the building costs and are sometimes 'hidden' within other line items. For timber structures these may be the main source of cost efficiency. We suggest they are always specifically and accurately analysed and reported.
- A key function of the estimator role, the take-off is quickly becoming more automated with the growth of building information modelling. With engineered timber elements produced and fabricated in accordance with a digital design file, timber construction is highly adaptable to modern construction environments utilising building information modelling for design, programing and estimating.

This section highlights some of the factors any estimator should be aware of when preparing an estimate for a timber project.

3.1 Engineered Wood Components

3.1.1 Products and Systems

Subject to the loads and spans designed for, Engineered Wood can be utilised in either panelised form (as is typical with cross-laminated timber (CLT), floor cassettes or stud-framed wall panels) or in linear sections (as is common with both glue-laminated timber and laminated veneer lumber (LVL) used for beams, columns or joists).

Timber panels are widely used as loadbearing walls and floor elements and are therefore well suited to repetitive 'honeycomb' designs, such as student accommodation, hotels and some multi-residential designs. Complementary to this system, linear products can be used as a column or beam to provide larger open spaces, and widely used in projects requiring large open floor plans (with spans up to 12 m under office loads).

The estimator should consider the following when either preparing or checking a take-off:

- Different timber-based products and systems are sometimes fully interchangeable (same performances and therefore, for instance, same structural depth). In other cases they differ to such a point that the costs/benefits associated with a substitution must be considered. This is currently typical with proprietary products like CLT, LVL and I-joists, that have different lay-ups, or with studs, joists and glulam beams that have a range of structural grades for the same cross sections. In these cases, assuming a generic rate/m^2 or /m^3 could be misleading. Further information and comparisons of different products and systems can be found in *WoodSolutions Technical Design Guide 46 - Wood Construction Systems.*
- A system is not worth just the sum of its components, as both the amount and cost of labour for its assembly will significantly differ when this is performed on-site or off-site. Also, the associated costs of transport and installation will vary.
- Product certification (both for performances and for sustainability) follows different standards, which are typically quite similar, but not identical. An experienced designer or consultant will provide enough information about the applicable standards and the acceptable tolerances and deviations. If this is not the case, the estimator needs to request the relevant information before making their own assumptions.

The influence of these elements will be detailed further in this chapter.

While a late-stage detailed estimate relies on an elemental take-off of the project, this level of detail is not available for early-stage estimates. At this stage of a project an estimator can provide an order of magnitude estimate based on a rule of thumb that has been developed in the industry, and is observable in the projects listed within this guide's database. The rates in Table 3.1 are provided as a high-level reference and can be interpolated when the structure is a mix of the types assumed for reference, then a parametric costs/m^3 obtained from a supplier can be applied. Interestingly, these rates can be reduced with an optimised design and the use of high-strength wood components, that may provide a cost-efficient alternative to lower grade materials.

Table 3.1: Fundamental metrics of the reference projects built with Structural Wood Systems. Note that these metrics are relative to supply only in 2018-19 (including connectors but excluding installation). There are indicative and should only be applied at a high level basis. For updated, more accurate and project specific costs please contact the relevant supplier.

m^3/m^2	Structure type	\$/m^2
0.20-0.30	Light frame residential designs (e.g. 4-6 storey)	\$300-450
0.30-0.40	Post and beam open plan with panelised floor system	\$550-650
0.27-0.32	Massive timber residential, standard design (e.g. 4-6 storey)	\$350-\$450
0.32-0.40	Massive timber residential, standard design (e.g. 7+ storey)	\$400-\$500

Figure 3.1: The Murray Grove residential building (London, UK, 2009) is an iconic CLT structure from WaughThistleton architects and has been a real ice-breaker that inspired many design teams. CLT brought significant savings in the program: only 49 weeks instead of the 72 estimated for the equivalent concrete building. A mobile crane eliminated the need for a tower crane and scaffolding was needed only to fix the cladding. A five-man crew accomplished the entire superstructure erection in 27 working days. The architects recently wrote: "If built today, it could use 30% less timber due to modern analytical tools." Images: http://waughthistleton.com

3.1.2 Wood species and finishes

Engineered timber elements are typically produced out of softwood plantations or sustainably managed forests, a fully renewable resource grown around the world under various climates and soils conditions. Alternatively, they can be produced out of sustainably sourced hardwoods, capable of achieving smaller sectional depths for the same loading capacity and featuring a colour and texture specific to the species of timber. The results are different, although generally comparable when the standards for product testing and certification are. However, in certain cases it is quite difficult to compare and substitute one species with another.

Timber elements can be supplied at a range of visual grades, varying from industrial non-visual through to premium visual finish. In addition to this, some suppliers offer a rough brushed finish, which removes the soft fibres on the surface and provides an 'exposed grain' texture. As one may expect, higher standard finishes and increased levels of processing typically attract higher costs.

Beyond their natural finishes, all timber elements can be painted or stained as required, however, it is important to note that applied finishes may require maintenance in exterior uses and re-application at 2-10 year intervals. For further information refer to *WoodSolutions Technical Design Guide #13 Finishing timber externally* in conjunction with product-specific manufacturer documentation.

3.1.3 Cutting and Fabrication

Engineered timber elements are produced in a safe factory environment and can have penetrations and connections pre-cut by a Computer Numerical Control (CNC) machine to a tolerance of just +/-1 mm, allowing a perfect fit on site every time. Appendix 1 describes the guidelines for the quality of execution of timber structures that are typically used in Europe, based on established practises.

While CNC cutting is a highly beneficial aspect of off-site timber construction, CNC machine time is expensive and poorly nested or un-optimised designs with a lot of cutting may be quite costly. Basic design optimisation and simplification can typically make a significant difference.

An astute estimator will check the consistency of the cost rates with respect to the amount of fabrication and may choose to quantify the latter separately, so it can be double-checked and/or value managed by those using the results.

3.2 Connectors

A commonly overlooked area, the connectors involved in the delivery of a timber structure, can be substantial in cost. With a wide variety available, from screws to nails, bolts, steel or timber dowels and complex proprietary products, the choice of timber connectors can affect not only the cost, but also the speed at which a structure is installed and fixed, and the productivity on site.

This is also true for the bracketry associated with timber construction, as simple angle brackets may be much faster to install than hidden dowels or plates that have a different fire performance and appearance.

An experienced estimator will check the consistency of the cost rates with respect to the type and amount of fixing materials and may choose to quantify the latter separately.

3.3 Acoustic and Thermal Performances

Designed and built correctly, timber systems are capable of achieving high standards of acoustic and thermal performance in both wall and floor/roof sections. It is important to note that this outcome has not only been predicted by advanced modelling programs, but has also been proven in several laboratory and on-site acoustic tests in Australian projects (see *WoodSolutions Technical Design Guides #22, #23, #24 and #44)*.

To be successful, a project must address three main acoustic measures being: airborne noise (Rw), impact noise (Ln) and structure-borne or flanking noise. These measures can typically be addressed through two methods including: the addition of mass to a timber element to insulate against airborne noise; and the introduction of a resilient layer or structural separation to minimise the transfer of vibration.

For example, it is common for floors to feature an acoustic build-up on top of the load-bearing wood element, such as a resilient matting product under a dry mass (e.g. CFC sheet, aerated concrete, wet-area plasterboard or particleboard), or a screed (which has the disadvantage of introducing a wet trade). Projects complying with the Deemed-to-Satisfy requirements for mid-rise timber construction also require a lining of fire-rated plasterboard to the underside of the load-bearing wood elements, and this can aid in the addition of mass. See Section 3.4 for more information about the fire requirements of floors. For walls, the use of two wall panels separated by a 20 mm air gap has been proven to provide the highest standard of acoustic performance.

The issue of structural borne (or flanking) sound is common across all materials and methods of construction, however the highly dissipative and prefabricated nature of timber construction allows this to be addressed better than with stiffer, heavier materials. This measure can be easily accounted for by placing a resilient strip in loadbearing joints between panels, effectively minimising the transfer of vibration from one section to another. Figure 3.2 shows this strip, which can be pre-applied to the top or bottom edges of a wall panels before installation on site.

Similarly, thermal comfort and durability require a number of ancillary products (thermal insulation, vapour control membranes, sealing tapes and foams) that either directly or indirectly form a significant part of the take-off.

An experienced estimator will check the consistency of the cost rates with respect to the amount of materials used for the acoustic and thermal treatment of the structure and may choose to quantify separately the latter, so they can be double-checked and/or value managed by those using the results.

Figure 3.2: Example of acoustic strip in loadbearing connections and of resilient flooring batten

3.4 Fire Safety

The NCC has different requirements according to the Building Class and height, with two pathways for compliance: the application of specific Deemed-to-Satisfy (DtS) provisions or a Performance Solution (PS) through fire engineering.

To comply with the NCC's DtS requirements for mid-rise timber construction, a timber building over three storeys and under or equal to 25 metres in effective height (from the ground floor to the top walking surface) must meet four main criteria:

- All structural timber must be fully encapsulated by sufficient fire-protective linings to meet the required Fire Resistance Level (FRL).
- Sprinklers are required throughout the project.
- Any insulation used in a fire-protected cavity must be non-combustible.
- Cavity barriers must be installed in any vertical cavities between Sole Occupancy Units (SOU).

WoodSolutions Technical Design Guide #37 – Mid-rise Timber Buildings provides comprehensive guidance to the application of DtS provisions.

Where seeking compliance with the DtS provisions, all structural timber must be enclosed in a fire-rated lining so that it meets the prescribed FRL. The fire-protective linings typically required to achieve the FRL of 90/90/90 commonly found in Class 2 and 3 buildings are summarised in Table 3.2. While the DtS requirements require all vertical elements to be fully enclosed in fire-protective material, this requirement only extends to the underside of flooring elements. This means that while the suspended floors of a DtS compliant project must be protected from underneath, there is no requirement for treatment to the top of the floor panel.

Table 3.2: Indicative fire-protective linings required to achieve a 90/90/90 FRL. (This is project specific and may differ in your case. Always confirm the requirement for fire-rated linings with your consultants before allowing for it in your project).

Element	Material	Typical Lining
Wall/Column/Beam	Mass Timber	1 x 16 mm fire-rated plasterboard (or similar)
Floor	Mass Timber	1 x 16 mm fire-rated plasterboard (or similar) to underside only
Wall	Lightweight Frame	2 x 13 mm fire-rated plasterboard (or similar)
Floor	Lightweight Frame	2 x 16 mm fire-rated plasterboard (or similar) to underside only

While the DtS provisions require that all mid-rise timber projects are fitted with a sprinkler system, the 2019 update to the NCC extended this requirement to all building materials and classes for buildings of four storeys or more.

Any project either exceeding 25 metres in effective height or not complying with these criteria will need to follow a fire-engineered Performance Solution. This process can often result in a more optimised and efficient design and should be considered as a possibility at the outset of any project. Specific information, with an example relative to a mixed-use building, is provided in *WoodSolutions Technical Design Guide #17 – Alternative Solution Fire Compliance – Timber Structures* (being updated at the time of writing).

Finally, adequate fire safety is obtained when penetrations are treated as required to achieve adequate separation. A number of ancillary fire protection products, such as caulking and fire collars, are either directly or indirectly associated with a timber structure (as well as with other materials) and form an important part of the take-off.

An experienced estimator will check the consistency of the cost rates with respect to the amount of fire-rating materials and may choose to quantify separately the latter, so it can be double-checked and/or value managed by those using the results.

3.5 Preliminary Costs

The cost category of Preliminaries can often achieve some of the most significant savings across a project. Timber has several inherent properties that make it easier, faster and safer to build with, resulting in reduced time-related and other costs, as discussed here.

3.5.1 Time-related costs

Depending on a number of factors not limited to the design, the experience of the builder and the layout of the site, timber projects have been shown to reach practical completion up to 30% faster than the alternative in reinforced concrete. This observation has been reported and published for Projects 7, 9, 13, 14, 15, 16, and 18. This has also been the case in overseas markets familiar with timber construction.

While a significant portion of this total time saving can be attributed to early services rough-in (due to the absence of formwork or propping), the majority is achieved in the speed of structural assembly – commonly seen to reach completion in just half the time experienced with other materials. This speed and efficiency comes from the fact that Engineered Wood Products are lighter, easier and safer to handle and install than other components of equivalent size and performances.

This is significant when considering ongoing time-related costs such as:

- hiring times of the crane(s), sheds and other plant and equipment
- installation and maintenance of temporary services, facilitated by easier fixing, pre-drilled holes, and a reduced program overall
- wages of the workforce, which are reduced in both number and time on-site
- insurances premiums, which are related to the duration of the work and its risk (lower for wood products)
- permits and certifications, which often attract significant fees and are valid for a limited period of time.

The potentially reduced program offered by timber structures can also enable time critical projects to be realised. This can be a crucial consideration for many projects, including schools and student accommodation buildings, for which funding periods can be set and term dates and new intake levels fixed.

Figure 3.3: Lendlease designed and built the Forté Living apartments in Melbourne in 2012, as a first significant experience in the use of CLT for residential mid-rise construction. A crew of five completed the carpentry with a remote-controlled light crane in only 10 weeks, with minor disturbance to the neighbourhood. The high dimensional stability and low weight of the structure is a significant advantage during construction but also for the quality and durability of the finishes, which require a reduced level of maintenance compared with a building of comparable location, size and construction cost. Images: WoodSolutions and https://www.architectureanddesign.com.au

3.5.2 Other costs (not time-related)

The prefabricated, lightweight nature of timber elements allows for a number of efficiencies in any project. Beyond the duration of on-site works and its associated benefits, the nature of engineered timber components often allows significant economies in the type and/or size of the equipment involved in site management and logistics. Also, with much work completed off-site, the few on-site functions can become an 'assembly' operation, leading to:

- ***Fewer on-site staff***: With structural elements produced in an off-site location, timber projects require just 5-8 installers (plus the crane crew) on site to complete the structural stage of a project. This significant reduction in on-site staff attracts reduced trade costs in addition to the many benefits discussed further below.
- ***Reduced on-site infrastructure***: Timber projects typically see a dramatically reduced reliance on common site infrastructure such as scaffolding, formwork, pumps, and props – the installation of which adds not only cost, but also time to assemble and remove. This benefit was reported for all projects in our database.
- ***Less waste***: The off-site manufacture of timber components has been proven to result in up to 90% less waste during the structure stage, and reduced waste in the finishes stage after. A benefit of prefabrication, this may result in significantly reduced bin hire and tipping costs, as reported for projects 7, 9, 13, 14, 15 and 18 in our database.
- ***A smaller crane***: The low density of timber elements results in floor and wall panels that weigh just 20% of their alternative in pre-cast concrete. This characteristic means that, subject to site constraints, projects can be completed without the use of high-capacity tower cranes. Instead, it is common for timber projects to utilise mobile cranes, self-erecting cranes and, where site constraints render it necessary, small tower cranes. Builders may also consider other lifting methodologies such as 'spider' cranes, mobile gantry cranes or other means that solve project-specific problems and increase productivity, thanks to the reduced weight of components.
- ***Improved site safety***: Significant timber projects have demonstrated several safety benefits associated with timber construction, including an increased ratio of supervisors to workers, cleaner and tidier sites, reduced acute joint strain associated with hammer drilling, the ability to pre-fit edge protection before lifting floor panels (essentially eliminating live edges), the absence of welding operations, etc. A 'zero lost time injury' result was reported for projects 10, 11, 12, 13, 14, 15, 17, 24, 25 and 26 in our database.

- ***Fewer deliveries***: With a typical delivery truck holding between 15 and 30 panels or structural elements, timber projects can experience up to 80% fewer deliveries than those utilising reinforced concrete as the main structure. This not only reduces disruption to the surrounding community, but also reduces costs associated with traffic control and permits. While all projects in our database experienced this benefit to different magnitudes, a reduction of 80% was reported in projects 5, 7, 9, 13, 14, 15, 16, 24, 25 and 26.
- ***Cheaper follow-on trades***: Services and finishes are typically installed faster in a timber structure than in concrete or brick due to easier fixing with smaller and lighter battery-operated tools, thus improving both the safety and the productivity of the workers. This benefit was reported for almost every project in our database, and has been suggested to result in competitive pricing when an experienced trade is quoting for a timber-based project.

3.6 Project Program

Timber projects can be faster to build on site than the alternative in concrete. While time-savings can often be explained by the pre-determined installation sequences and pre-planning of other on-site processes, evidence suggests that this increased site productivity is a result of the workers feeling safer. Subject to a number of site-specific factors (e.g. site conditions, number and size of loading docks, number and type of crane) and design specific factors (e.g. whether the design features load bearing walls, post and flat plate slab, or a post and beam typology) it has been observed that a typical timber project can be assembled on site at a rate of approximately 80-100 m^2 per standard working day, or 400-500 m^2 per five day working week. This rate reflects the average installation rate of the projects in the database (with outliers removed).

For example, Project #1 in Tale 2.1, a six storey timber structure with a floor area of approximately 1000 m^2 per level took 12 weeks to achieve structural completion. While it has been observed that the first floor can be quite slow to install (due to the interface between the concrete slab and the millimetre perfect timber elements), full speed is quickly achieved in the following floors.

3.7 Labour

A benefit of the dry, prefabricated nature of timber construction, timber projects often require a structural installation crew of just 5-8 people plus the crane crew. Using only battery-powered impact drives this crew can install columns, beams, walls, and floor panels at the rates identified above. Of this crew, it is necessary that just 3-4 of the installers are qualified carpenters as many tasks associated with timber installation are simple and highly repetitive. Typical installation times for off-site prefabricated timber panels are 16-20 lifts per day, allowing 15 minutes per panel for fixing and taping.

Both installation of the prefabricated timber elements and subsequent works are easier, quieter and safer, reducing or completely avoiding wet trades and reducing the number of personnel required to erect the superstructure by around 50-70%. Typically the following approximate time savings were found for various follow-on trades in the projects listed in our database:

- services (MEP) – about 30-50% faster
- dry liners – about 20-30% faster
- window & door installers – about 20-30% faster
- insulation installers – about 20-30% faster
- cladding installers – about 20-30% faster.

3.8 Foundations/consolidations

The natural light weight of timber typically means that projects with an engineered timber structure are 30-50% lighter (completed project weight) than the same design in concrete. Depending on the soil conditions on site, this may result in significant reductions in the size and depth of the footings and/or the consolidation works of the existing structures, in case of a vertical extension.

In some cases, the feasibility of a development has become positive only thanks to the lower weight of the timber structures, which has allowed to build a significantly higher volume over an existing structure, with minor consolidation works and much less disturbance to the existing tenants.

Figure 3.4: The short delivery time requirement for the Aveo Norwest building, completed in Sydney in 2018, made it an ideal project for the CLT design by Jackson Teece (architects and interior designers) and TTW (engineers), which enabled Strongbuild to complete it 13 weeks earlier than with the concrete program. The AVEO Norwest building is in a way the first of its kind, breaking the mould of the typical mass timber designs for residential buildings we have seen internationally over the past decade. In order to conform to the client's brief for this particular project there could be no obvious compromise in apartment layouts or building design in favour of specific prefabrication or mass timber construction requirements. The planning and the design of the building, including the curved and seemingly cantilevered balconies, required an innovative hybrid construction approach utilising CLT slabs and walls, Glulam beams and structural steel. The design had to achieve a balance between the desired architectural design outcome and construction rationale and had to push the boundaries of what was then perceived as possible in mass timber design and construction technology. It was a client decision to prioritise a specific desired architectural expression, through a specific budget allowance. Images: Brett Boardman and Strongbuild.

4 Procurement, Purchasing and the Supply Chain

The procurement routes and processes associated with timber construction are already commonplace within the industry, however it is important to note that some methodologies are better suited to it than others. This section discusses the most effective procurement and purchasing approaches and provides a brief overview of the main supply chains – both local and international.

4.1 Contracting Approach

Modern construction projects follow a wide variety of contracting approaches based on the project type, experience of the client and other factors. While all approaches can be suitable for timber projects, it is important to understand the potential benefits and drawbacks of each to ensure that the selected path is the right one. This guide discusses three commonly used approaches: traditional lump sum, early contractor involvement, and design and construct contracts. While they aren't discussed here, other forms of contract (e.g. construction management) can also be successful with timber projects.

4.1.1 Traditional Contracts

Traditional or lump sum contracts are common throughout the industry for projects of all sizes. Perhaps the simplest form of all contracting models, this approach sees the rigid separation of design and construction. In this process, the client appoints a team of consultants who complete the design of the project, generating all construction drawings, details, specifications, and often a Bill of Quantities. With this complete, the head contractor is selected through competitive tender and project is constructed based off the tender drawings.

This fragmented approach is well understood but is becoming less popular on large-scale and complex projects for a number of reasons ranging from the extended time frames to the reduced cost-efficiency compared to other types.

Timber construction can be paired with this contracting approach; however this is most successful where the design team is experienced with timber or works closely with a preferred supplier throughout the design phase as an inexperienced design team is likely to deliver a poorly optimised design. Once they are engaged, it is sometimes possible for the contractors to procure the timber structure while earthworks/basement works are underway, with the first timber elements typically available on site 'just-in-time' for their install.

4.1.2 Design and Construct Contracts

Perhaps the most popular form of construction contract, design and construct (D&C) contracts see the client engage a design team to complete 50-80% of the design at which point the responsibility for the design and construction of the project is transferred to a selected contractor. As part of this process it is common for the design team to be novated across to the contractor.

With the contractor in control of both the completion of the design and the construction of the project they are able to optimise the project for buildability and efficiency, ultimately delivering the same quality of project in a shorter duration, and often at a lower price.

This contracting approach is well suited to engineered timber construction as it allows for the engagement of a timber supplier before the design is complete, offering some potential to optimise the design for further efficiencies. Examples of this basic level of optimisation may include the selection of the most suitable mix of Engineered Wood Products and systems, varying the size and strength of selected elements, and the detailing of simple connections to improve on site productivity.

4.1.3 Early Contractor Involvement

While not a contracting approach per se, Early Contractor Involvement (ECI) can prove to be highly beneficial in large or complex timber projects. The first stage in what is typically a two-stage process, ECI sees the head contractor involved in the early design of the project. This input allows the contractor to offer design optimisation advice from the very start, ensuring the project is as efficient as possible. This input may involve the discussion of structural systems, spans, exposed finishes and much more. Perhaps the most valuable opportunity offered by the first stage of the ECI is to have a supplier involved from the start of the project. This allows for the design to be completed with the sizing, grading, pricing, and any limitations of the supplier in mind, generating a better outcome for all stakeholders.

With the ECI complete, the project can then proceed to the second stage and the appointment of the main works contractor. This process can either take place through competitive tender, however it is also common for this contract to be negotiated with the contractor in charge of the ECI.

4.2 Supplier Services

While the majority of suppliers specialise in the manufacture and delivery of a specific product, many are willing to add value where possible, often providing design support services or sourcing and even installing other complementary products. For example, the supplier may develop specific and optimised structural engineering calculations, shop drawings, provide all the accessory materials under one contract to optimise procurement and logistics, and may even be able to pre-install non-timber elements such as vapour membranes, windows, doors and so on. The addition of these services in a safe, clean, well-lit factory environment will often prove to be highly beneficial, minimising work on site, improving the standard of finish, reducing the number of contracts entered into, and ultimately reducing risk.

Regardless of the contract or procurement model, it is advisable for the client and design team to engage with potential suppliers of the timber superstructure at the earliest possible date. This will allow for valuable technical input to be offered at a time when it is easiest to implement. This early communication will also allow the supplier to ensure that they hold enough stock or product to supply the project, ensuring a positive experience for all.

4.3 The Supply Chain

The global supply chain for timber elements and connectors is well established and continues to grow at a significant rate. With almost all Engineered Wood Products sourced from the ever increasing area of sustainably farmed plantations, the international supply chain has enough fast-growing fibre to sustain the growth of the timber into the coming decades. This section discusses the state of the timber supply chain in Australia and overseas.

4.3.1 Australia

The Australian supply chain is well established, with plantations, mills, distributors and fabricators. Several of these businesses have been operating for more than 100 years, showing both environmental and financial sustainability. This mature supply chain is well placed to service both the traditional detached house market and the growing market of mid-rise stud frame structures, and has recently seen the emergence of several suppliers with the infrastructure to pre-install membranes, linings, floor coverings, windows and doors, thus delivering components enabling the builders to quickly achieve airtight building envelopes.

The Australian supply chain is also quickly growing to meet the needs of designs requiring mass timber construction. With established Glulam producers recently upgrading their facilities, mature LVL manufacturers gearing their product to the mid-rise market, and CLT manufacturers and fabricators already supplying significant projects, the Australian mass timber production capacity is growing quickly.

Purchasing from local suppliers has multiple benefits, ranging from the ease of inspection and Quality Assurance, to flexibility in design, to supporting the local economy and jobs in rural areas.

Australian produced Engineered Wood Products are commonly available in both softwoods and hardwoods. At the time of writing, the most common softwood used is *Pinus Radiata*, a fast-growing pine with a pale yellow tone. Engineered Timber suppliers specialising in hardwoods generally work with their closest resource but can also produce custom elements in a variety of timber species.

4.3.2 Overseas

It is now common practice for head contractors to consider suppliers established and producing in other countries for a variety of elements in a construction project (e.g. glazing, finishes). This approach is well known to any procurement manager, who consider financial costs, exchange rates, differences in legal terms, etc, however special consideration should be given when dealing with Wood Products:

- Always check the product's performance specifications, sizes and dimensional tolerances, certification and testing documents, appearance, packaging, custom rules, etc, against the project specification (or the consultant's brief) as it may be necessary to make different criteria compatible. Although there is a well-established set of trade and technical relationships (e.g. with New Zealand, North America and Europe) that make products compatible, sometimes little details may differ and the approval process may then become time consuming.
- If not buying from a local distributor, preferably adopt a 'pull' approach in logistics with the freight forwarder located in Australia, as they are experienced in importing products through local ports and customs. Buying 'ex-works' is sometimes attractive but the amount of associated work may result in unexpected difficulties and delays.

- Always remember that transporting a timber-based product on a ship exposes it to changing climatic conditions. Although there are plenty of positive experiences, the adequate protection and the prevention of unexpected movements is quite important to avoid defects in terms of dimensional stability and surface appearance. Typically, qualified overseas suppliers have suggestions that come from experience.
- Element sizing may be limited by the size of the transport vessel. While open top and special size containers and racks are available, the benefit of being able to transport larger elements does not always out-weight the cost premium associated with the specialised containers.
- Allow for a storage facility and labour to unload the containers, check the materials and let them reach the equilibrium moisture content with the local climate, before installing them.
- Fumigation may be required for all imports from certain regions at specific times of year. Confirm requirements with your supplier and local customs as early as possible, as the fumigation process must be allowed for in both program and cost estimates. This is now a routine operation and can be completed cost efficiently, but still must be considered.
- Loading containers takes time and wood elements have to be adequately protected and strapped to minimise damage during shipping.With all the right conditions in place, procurement times of 1-2 months from New Zealand or 2-3 months from Europe and North America are typical, including shop drawings, manufacturing and shipping.

Figure 4.1: The Green (Melbourne, 2014) is a 5-storey development designed by SJB Architects (planning) and Point Architects (documentation), scaling up the traditional stud frame design and adapting it to a new set of requirements. This 57-apartment project was approved with a Performance Solution and set an example for the Deemed-To-Satisfy rules that were approved in May 2016 in the National Construction Code. Developed and built by Australand, the project reported an overall 25% cost saving with respect to the alternative concrete-based program. The advantages of timber construction in this project, according to Irwin Consult (engineers) were: 1) lighter overall building loads on columns, foundations and ground floor transfer beams; 2) reduced building mass and stiffness resulting in lower lateral design earthquake loads in comparison to concrete buildings; 3) prefabrication of walls and floor elements to achieve a construction program equal to that of equivalent concrete buildings; and 4) use of a labour force experienced with this type of construction using materials that are commonly available and economical. Images: https://citta.com.au and WoodSolutions.

5 Risk Management

Risk management is an important facet of any construction project. The correct identification, assessment, contingency and control of project risks is key to ensuring the success of projects. While many risks are common across all building materials and systems, this section identifies where timber may be different and what the cost engineer or contractor should be aware of when assessing a project.

The risks associated with timber construction are generally similar to those encountered when purchasing prefabricated elements, such as glazed curtain wall units. First and foremost, it is important that design resolution and clash detection are completed before shop drawings and production. While minor on-site adjustments are very simple and easy to carry out with timber-based products, last minute, large-scale changes can be costly and inefficient.

5.1 Quality Risk

As with any product or material, it is important for the contractor to ensure that all elements produced are compliant with the specified design. Appendix 1 provides extended and more specific guidelines on how this should be planned and implemented.

Quality Assurance is a simple task with Engineered Wood Production and fabrication because these are normally highly automated processes in which all materials are tested, recorded and tracked throughout the production line.

Engineered Timber producers are required by the relevant Certification Body to engage in surveyed testing and quality assurance procedures to confirm the full compliance with the design requirements. This starts with strength grading of every single element and proceeds with controls on the bonding process, the dimensional tolerances and every other performance-related parameter. A copy of the track records of the activities and checks performed on a batch ready for delivery (or a summary document) can be requested from the supplier.

Tags, marks or other means of identification will assure the traceability of every element for QA and/or chain-of-custody purposes.

It is interesting to note that the Eurocodes associate a lower safety factor (meaning the materials are considered as having lower risks) to Engineered Wood Products than to other structural products, because:

- Every single element that goes into the building is strength graded with non-destructive and calibrated tools (not just a sample extracted from the batch and therefore not going to be installed, while the rest is considered equivalent).
- The sizes and the equations used in both the design checks and the laboratory tests are exactly the same.

5.1.1 Dimensions

Structural elements can be machined to their exact final dimensions using a CNC machine. At this stage, the CNC machine can also cut any penetrations, route channels for cables, and complete any other cutting to meet the design. The modern CNC machines run by timber element fabricators cut to within +/-1 mm tolerance, and so the accuracy of the element sizing and cuts is governed by the quality of the shop drawings. While timber suppliers have quality control procedures in place to ensure that their drawings accurately reflect the design they received, it is important that the contractor confirms this and regularly cross-checks their output.

5.1.2 Construction site

While timber projects typically have reduced exposure to the on-site risks commonly associated with construction, there are a number of risks that may have a greater impact for timber projects than others. With the fabrication of structural elements occurring in an off-site facility, on site activities centre around the assembly of the structure and the rate at which this can occur.

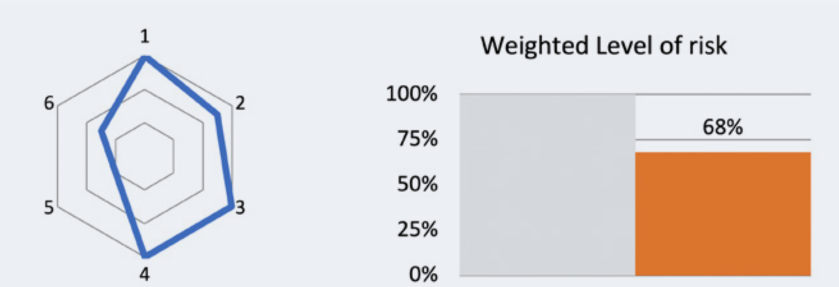
Analysis of the projects within our database shows that the most significant factors potentially affecting this key metric can be condensed into three categories: design (D), on-site access and activities (O), and environmental (E) factors. These categories each comprise several weighted parameters, for instance those identified in Table 5.1, which can each be scored in line with the notes in the ‘guide’ column to achieve an overall risk rating. Parameters and scores can be displayed and analysed with various stakeholders, for instance as shown in Figure 5.1.

Table 5.1: Example of risk parameters contributing to overall project installation rate

Code	Parameter	Weighting	Guide
D1	Gross Installation Area	0.3	Smaller means lower risk
D2	Number of Storeys	0.9	Less means lower risk
D3	Timber Volume (m^3)	0.8	Less means lower risk
D4	Steel Share Factor (kg steel/m^2 timber)	0.6	Less means lower risk
D5	Percentage of Visual Timber (%)	0.8	Less means lower risk
D6	Building Shape Complexity	0.85	Simpler is better
O1	Ease of Site Access	0.85	Larger entries are better
O2	Delivery Route	0.55	Less traffic is better
O3	Site Location	0.6	Less built up is better
O4	Trucks per Week	0.9	More is better
O5	Scaffolding and Building Access	0.7	Easier access is better
O6	Cranes	0.9	Less reliance is better
O7	Lay down area ratio (m^2 timber/m^2 lay down area)	0.7	Lower ratio is better
E1	Installation Season	0.5	Fewer rain days is better
E2	Geographical Location (Wind)	0.85	Less wind is better
E3	Wind Speed (m/s)	0.85	Lower speeds are better
E4	Protected Areas	0.2	More protected is better
E5	Flood Zone	0.1	Less flood zone is better

Figure 5.1: Example of risk assessment matrixes and graphs (http://www.eurban.co.uk/).

Building Risk Parameters							
	1	2	3	4	5	6	
	GIA (m2)	No. of Storeys	Timber Volume (m^3)	Steel Share Factor (kg/m^2)	Percentage of Visual Timber (%)	Building Shape Complexity	Weighted Building Risk
Weight	0.3	0.9	0.8	0.6	0.8	0.85	
Level of Risk	5	4	5	5	1	2	69%



5.1.3 Finishes

The client should seek and approve a physical sample of the desired finish before entering into a supply agreement, as finish grades may vary. Where a 'Visual Grade' finish has been specified, it is best practice to ensure that the face to be exposed is properly protected with an anti-mould and UV-resistant product until it has been installed and protected from UV exposure (e.g. the building envelope has been completed). Adequate detailing can be beneficial where visual grade surfaces interface with non-galvanised steelwork, as rust will stain the timber face in the event of rain.

On unprotected wood surfaces, a moisture content level that exceeds 15-16% for a significant period may start the appearance of blue stains from mould, which is significantly different from rot (no structural damage) and may have the following origin:

- surface moisture from high humidity in the air (windows open in foggy or rainy days, water spilled on the floor)
- excess of a waterborne surface finish, which has no anti-mould
- micro-particles from welding, sawing metal or plasterboard that could have happened close to the wood element.

The suppliers of coating systems will provide specific indications. As a general recommendation, it is advisable to:

- select a finish that has an anti-mould in it
- apply one coat of it either before shipping panels/beams from the factory or when individually de-stacking them from the truck or container on site (this will also protect the visual grade surfaces from dust and foggy days)
- apply a second coat after installation.

5.1.4 Weather exposure

While it is not a problem for non-visual grade elements to be exposed to the weather for short periods, it is important to prevent pooling of water for extended durations. It is best practice to protect the end grain of structural elements from water wherever possible, however if the timber structure is exposed to rain, it must be allowed to reach a moisture content of lower than 16-18% before encapsulating linings are installed. Slightly higher, localised moisture content levels are possible when breathable linings are used and adequate drying is foreseen over a short time. Measurements with a calibrated hygrometer are easy to perform and assure there is no risk of excess moisture being held behind the finishes.

Although protection methods will largely vary with location, period of installation and design, a typical mid-rise timber project in Australia would require the following actions, not significantly different from those considered when installing tilt-up concrete components:

- maintaining panelised elements dry as long as possible during transport and site storage
- taping of floor-wall joints as early as possible
- protection of the top edge of the walls with sarking
- use of vertical risers to promptly evacuate rainwater
- closing the window and similar openings as early as possible, eventually using temporary sarking
- use of tarpaulins on floor and roof elements during prolonged rainy periods or construction stops.

Appendix 1 provides extended and more specific guidelines on how to deal with temporary exposure to rainy weather during the construction period.

5.2 Logistical Risk

The selected supply route may affect the logistical risk associated with a project because international freight attracts a higher risk than road transport. Beyond this, other factors that may affect the success of the project include the following.

5.2.1 Supplier capacity

Regardless of the location of the supplier, it is important to confirm their capacity to produce, process, pack and deliver when required by the project's program. This is significant, as 'all up' deliveries, where all panels are sent in just a few batches will likely incur significant storage and sorting costs while waiting for installation on site. With this in mind, it is best practice to follow the 'just-in-time' philosophy, with each delivery of elements arriving on site when it is needed, with the right moisture content.

5.2.2 Packing order

The packing order of timber elements may influence on site productivity significantly, especially if elements are packed in a container (although this is also relevant for truck deliveries). Best practice is to ensure that when the elements are packed for the delivery on site, this is completed in the reverse order of installation, eliminating double handling and maximising productivity.

5.3 Financial Risk

When the level of off-site manufacturing increases, suppliers may require a substantial deposit well in advance of delivery. This should be evaluated and balanced with respect to the time saving in final delivery that, in most cases, mitigates the financial risk.

Where sourcing product from overseas, consider the stability of exchange rates and how their fluctuation may affect the supply contract.

Consider the required payment timeframes and the point at which the ownership (and therefore responsibility) of the goods transfers to the purchasing party.

5.4 Certification and Approval

Independent testing and/or certification often forms an important activity before materials are shipped or delivered them to the construction site. Their duration is not always predictable and may be considered within the risk analysis.

An efficient approach to reducing the risk is getting the certifier involved as early as possible and compiling a set of documents and drawings/information models that are consistent with the complexity of the project, in order to:

- define a set of technical requirements for the execution of a timber structure
- ensure that the designer gives the contractor all the relevant technical information for the execution of the structure, and that they are transferred to the contractor
- specify conditions to be fulfilled before the works begins
- list controls suggested at delivery, during and at the end of the execution, to assure that the specified quality is achieved.

A guideline for such an 'execution specification' is available in Appendix 1.

5.5 Delays and Variations

Building construction projects are often carefully planned. Despite this, some projects are completed late and/or generate unforeseen variations claims. If delays are compared to budget over-runs, the former play a much greater role in affecting the profitability of a construction project. A commercial construction project that overruns its budget by 50% but is finished in time earns only 4% less than the one that keeps both schedule and budget. In contrast, if a construction project stays in budget but exceeds its schedule by half a year, its earnings may drop as much as 33% [1]. Furthermore, time performance has been identified as the most important criteria for defining whether a construction project has been successful or not, surpassing both cost and quality performance [2].

Variations in timber construction are much like those in any other form of building. As experienced on most projects, decisions made at an early stage have the highest impact for the lowest cost; conversely later decisions attract a higher cost for a reduced impact.

While this theory holds true, some structural typologies can be seen to be more flexible than others. For example, post and beam or post and slab designs typically deliver large open spaces within which non-loadbearing partition walls can be altered as required with minimal concern of compromising the structure. In comparison, the honeycomb structure of projects utilising loadbearing wall panels (common in hotels, student accommodation and some multi residential buildings) can often be altered once built, however this can be a more complex process and require approval from the project's structural engineer prior to execution.

As wood-based projects require design to be largely resolved before the elements can be produced, projects often experience fewer variations than is common with other materials that have larger tolerances and require more site work.

Finally, in general terms, significant advantages can be found in a fast and predictable building process, like the one which arises from a well-planned and detailed design, where the accuracy of Engineered Wood Products can play a major role.

[1] O. Port, Z. Schiller, R. King, D. Woodruff, S. Phillips and J. Carey, A smarter way to manufacture, Business Week, no. 30, pp. 110-115, 1990.

[2] S. Khosravi and H. Afshari, A success measurement model for construction projects, in International Conference on Financial Management and Economics, Singapore, 2011.

6 Design Optimisation

A well-optimised project utilises the best material or system needed to deliver the level of performance required for a reduced price. This concept is common across all building materials and systems and, where applied correctly, can deliver high-performing, high-quality and cost-effective projects. This section details some common methods of design optimisation in timber structures followed by a case study in which several of these are applied.

The risks associated with timber construction are generally similar to those encountered when purchasing prefabricated elements, such as glazed curtain wall units. First and foremost, it is important that design resolution and clash detection are completed before shop drawings and production. While minor on-site adjustments are very simple and easy to carry out with timber-based products, last minute, large-scale changes can be costly and inefficient.

6.1 Structural Design Optimisation

6.1.1 Material and system

The first consideration when looking to optimise a timber project should always be the composition of the structure. While designers often may like to specify the same material for all the elements in timber construction, this isn't always the optimised solution. For example, a five-storey apartment block with few spans over 5 m is likely to be well suited to a lightweight frame solution (stud frames), with mass timber elements utilised where needed (e.g. in the core, some floors and potentially some structural walls at lower levels). An office building requiring large open floor spaces may be best suited to a Glulam or LVL column and beam structure, with CLT or LVL suspended floor plates. An optimised design typically uses a range of Engineered Wood Products, all to their specific best use.

The choice of the structural material and system is governed by a number of factors not limited to building use, required spans and the vertical load path through the building. Indeed, a well-optimised project may utilise a combination of materials and systems. Regardless of their form, all timber systems behave in a similar fashion under varying environmental conditions and therefore are simple to design, install and maintain together. This has been demonstrated around the world in projects that utilise mass timber elements (both panels, beams, and columns) in areas of significant loading, and lightweight wall and floor elements where lower loads are encountered.

The prefabricated nature and 'just-in-time' delivery of timber elements facilitates the installation. With all timber elements produced and machined off-site and delivered to the site in the order of their installation, concerns over the coordination of element install are effectively mitigated. This is particularly relevant where section sizing decreases or structural material changes (e.g. from mass timber panels to lightweight panels) as the building ascends, as all previous elements with a different size have already been installed.

Last but not least, Engineered Wood Products may be combined with other products within an optimised hybrid structure.

6.1.2 Load path and transfer

An optimised timber project should require minimal load transfer in the timber structure, with all vertical loadbearing elements 'stacked'. Where loadbearing wall panels or columns are required over a large span area (e.g. a loadbearing wall in the apartment above with a living room in the apartment below) this can typically be accounted for with the use of a high strength Glulam or LVL beam, however this may affect the ceiling height in this specific area.

Where timber structures are over a carpark, retail space or a building use requiring a specific grid, it is common for this load transfer to take place via a concrete slab. In practice, this has led to having the structure up to the most significant load transfer completed in concrete, with the timber structure above. This is common in multi-residential developments that feature retail and hospitality tenancies at the ground level, with the concrete transfer slab providing structural transfer, as well as fire and acoustic separation.

6.2 Fire Engineering

While the NCC provides a Deemed-to-Satisfy (DtS) solution for timber construction to an effective height of 25 metres, a DtS compliant design may not be the most cost-efficient design. Depending on a building's usage and risk profile, involvement of a fire engineer and the adoption of a performance solution can often result in simplified construction processes, and ultimately reduced costs.

Deemed-to-Satisfy requirements must allow for all buildings in all circumstances, and therefore consider the fire protection required for a 'worst case scenario' project. In contrast to this, a performance solution is able to consider a project in its own specific context and assess the real risk posed to its users.

Unlike a DtS solution, the fire engineering process considers the charring rate of timber – a natural and well-understood phenomenon where the surface of a timber element exposed to fire will char, ultimately insulating the structural core of the element for a period of time. Consideration of this factor in fire modelling has been seen to result in reductions to the number of layers, or extent of fire protective linings called for in mass timber projects. It is common for the savings made possible by this process to significantly outweigh the extra cost associated with engaging a fire engineer.

6.3 Acoustics

As discussed earlier in this guide, timber elements which separate residential units often require acoustic treatment to deliver high standard internal environments. This field is often over-complicated and can ultimately be simplified to two main requirements: (1) the addition of mass to a timber element to insulate against airborne noise, and (2) the introduction of a resilient layer or structural separation to minimise the transfer of vibration. With these principles in mind, this section will discuss floor and wall elements separately.

Acoustic modelling and testing can provide an effective opportunity for design optimisation or value management, limiting overdesign and time-consuming construction practices by, for example, using dry trades only and materials that can be immediately walked on by different trades.

6.3.1 Floor elements

The NCC 2016 requires that floor elements between Sole Occupancy Units (Class 2 and 3 buildings) achieve a minimum acoustic rating of 50 for airborne noise (R_w+C_{tr}), and a maximum rating of 62 for impact noise (L_{nw}). There is no minimum requirement for separation between commercial spaces (Class 5).

This requirement is easy to achieve where a suspended ceiling is utilised in the design, as a gap equal to or exceeding 100 mm and a resilient mounted suspended ceiling has been shown to greatly improve acoustic properties of a floor element. Where this is the case, a simple above floor build up including a resilient acoustic matting product topped with 30-40 mm of high-density material (e.g. a wet concrete screed, 2 x 15 mm compressed fibre cement boards, magnesium oxide board, particleboard, etc) has been shown to deliver a compliant system.

Where the design intends to expose the underside of a timber panel, the mass and resilience that would otherwise be on the underside of the element must be added to the top build up. In this circumstance, it is effective to specify a resilient batten (Figure 3.1) to maximise acoustic performance.

6.3.2 Wall elements

The NCC requires that wall elements between Sole Occupancy Units achieve a minimum acoustic rating of 50 for airborne noise (R_w+C_{tr}), with no requirement for impact rating (Class 2 and 3). Again, there is no minimum requirement for acoustic separation between commercial spaces (Class 5).

Similar to floor elements, the acoustic properties of timber walls are determined by both the mass of linings and the level of separation or discontinuity between the wall faces. The optimal wall design is commonly seen in timber frame, and comprises two wall frames aligned to parallel and separated by 20 mm. These frames are lined with a dense fire-protective lining on the external faces only, and offer complete discontinuity between faces (and therefore very good acoustic properties). This wall type can also be achieved with massive timber panels or a combination of both lightweight and mass timber.

While discontinuous walls deliver the highest standard of acoustics, a close second can be seen in staggered stud walls. Staggered stud walls achieve a moderate level of discontinuity within a single wall panel, resulting in a faster install than experienced with discontinuous walls. This wall type is not suitable for all sections of a party wall (the NCC requires complete discontinuity in some areas between Sole Occupancy Units), but its proper specification and use can result in savings to both program and cost.

6.4 Finishes

Ever popular within the design community, exposed structural timber elements are indeed beautiful. While many look to expose as much of the structure as possible, it is important to consider the costs that this may add to the project, and potentially look to alternatives to achieve a similar finish.

Visual grade timber elements require higher levels of processing, and therefore attract a higher cost out of the factory. The exposed faces may also require special protection in transport and installation, adding to the total building cost, and attract additional maintenance costs.

While not always practicable, structural elements with visual grade finishes can be substituted with an 'industrial grade' structural component to which a non-structural 'visual grade' element like plywood or a solid wood panel is attached during the finishes stage of construction.

6.5 Design for Assembly

It is important to consider the buildability of a structure and whether the construction sequence can be simplified to improve on-site processes and productivity. For example, while a loadbearing wall 'honeycomb' structure may seem logical for a short span project, this may experience a slower install rate than if a short span post and slab design were utilised. Typically, early contractor Involvement is used to collaboratively define the best option.

Where large lay down areas are available, both the designer and construction team should consider the opportunity to pre-assemble certain groups of elements at ground level before they are lifted and secured into their final position. This on-site prefabrication process is well suited to complex, repetitive, or 'box' sections such as bracing or core elements, and has proven to significantly reduce the total number of crane lifts in a project, resulting a reduced build time over all.

Figure 6.2 - Blackdale Residences (Norwich, UK, 2016) designed by LSI Architects and Ramboll (engineers) for the University of West Anglia, comprises 514 apartments built to BREEAM Excellent standards. With a tight build program, offsite construction techniques ensured on-time delivery and high quality, also through BIM level 2 with full 6D asset information incorporating time, cost, virtual modelling and all aspects of life-cycle facility management. The Senior Project Manager described it as his 'best project ever'. Images: https://ramboll.com and https://b4ed.com

7 Feasibility

While the feasibility process for a timber project is similar to that for a traditionally built structure, there are some key contrasts that must be considered. Differences between these systems range from familiar factors of prefabrication such as varied payment structures, to potential benefits such as increased Gross Floor Area and the 'unlocking' of previously unviable sites.

7.1 Payment Structures

Modern timber construction involves the on-site assembly of engineered timber elements that have been produced and fabricated off site. This process sees much of the work typically performed on site moved to safe, well lit, well supervised, indoor locations, and so much of the expense is incurred off site. Depending on the scope of their contract, Engineered Wood Product suppliers and fabricators typically request 50-100% payment before elements are installed on site. While no different to the delivery of a unitised curtain wall from overseas, this is an expense that must be expected early in a project.

7.2 Reduced Interest Expenditure

As identified in this guide, projects featuring timber structural elements typically experience a reduced on-site program. This expedited process has many benefits, including the significant amount of cost savings afforded by the reduced number of interest payments required before settlement. Timber projects can vary in productivity depending on the complexity of the design, however it is common in Europe and North America for timber projects to proceed up to 30% faster than an alternative in concrete. These savings will increasingly be seen also in Australia, as the use of Engineered Wood Products matures in the market.

7.3 Payback Periods

The payback period associated with a major construction project is determined by many factors, including the site, the size and type of building, the condition of the market, and the standard of finish achieved by the building. While highly finished timber buildings may attract a similar if not slightly higher cost than the alternative in traditional systems, the many benefits of timber workplaces have seen higher rentals than would have otherwise been the case.

Conversely, compliant standard timber buildings can be cheaper to build than the concrete or steel alternative of the same standard while achieving a similar rental income. Ultimately, payback periods associated with a timber building can be slightly shorter than experienced in the traditionally built alternative, however this can vary depending on the factors identified above.

7.4 Opportunities

The use of Engineered Wood Products can have a significant impact on the overall success of a project, when there is an early and shared understanding and what it may enable the developer, designer, builder, agent and, ultimately, the owner to do. The following is a non-exhaustive list of considerations and suggestions from those who have experienced it first hand and contributed to our Database of successful projects.

7.4.1 Optimised design

When considering the use of Engineered Wood Products as the primary structural frame, the architect and structural engineer need to carefully evaluate the product capability, referring to their consultants, suppliers and advisers as required. A lot of information is usually available from the product manufacturers. For an experienced designer, one of the major strengths of timber structures is the possibility to combine the solution to the structural, fire, thermal, acoustic and aesthetic requirements within a single material or assembly.

A pure conversion of a concrete frame to a timber frame, without any variation of geometries and depths is normally possible, but it will often prove to be inefficient. Therefore, specific tools for optimising the design are freely available, including WoodSolutions Technical Design Guides (notably Guide #46 – Timber Construction Systems) and commercially available structural design software.

Aiming for a repetition of vertical penetrations in the walls will optimise the flow of the service installations and reduce material off-cuts, with significant time and cost benefits.

Adopting clear spans that do not exceed the material's optimum capabilities (9 m for commercial buildings and 6 m for multi-residential buildings) will result in a design which is not only optimised for structural performances, but also from the point of view of supply and logistics.

Using the highest possible number of timber panels as load-bearing walls will reduce the need for long spanning joists or primary beams (however this may affect installation times).

Deviation from the line loading of the walls floor to floor should be avoided, as any off-sets or eccentricity of the line loads will require thicker wall panels and/or stiffening in the floor in localised areas, thus creating inefficiency in both frame design, the procurement and the installation.

Stacking wet areas, among many other advantages, offers the opportunity to use them as additional cores, when their walls are load-bearing timber panels that can be easily connected to each other, rather than being dead load over the floors. Also, a reduced floor depth is possible in this case, with the additional benefits of easily achieving a set-down in the wet area and a reduction in the materials needed because of a reduced span.

These and other opportunities are most likely to arise when a project proceeds with a timber structure in mind from the early draft design. Experienced architects and engineers typically report significant savings from their growing skills and the evolution of a dynamic sector.

7.4.2 New sites

As a raw material, timber is considerably lighter than both steel and concrete, typically 20% of the weight for equivalent structural components. Once completed with all fire rating, acoustic build ups, services, finishes, furniture, and all other components required to deliver them, timber buildings are still significantly lighter than their concrete counterparts, with a rule of thumb suggesting the completed timber building will weigh 40-50% less than the equivalent concrete alternative.

This is significant in that is opens up new possibilities. The light weight of timber structures means that they can be built in locations previously deemed unviable with traditional systems. Whether above an existing building, on poor or contaminated soil, or over a subway tunnel, timber projects pose the answer to many difficult sites.

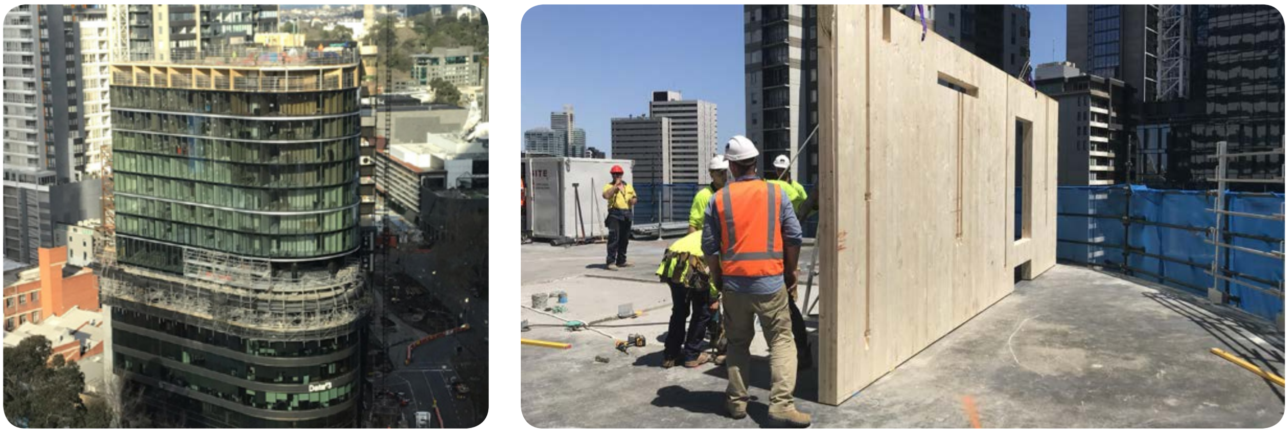


Figure 7.1: A 10-storey vertical extension to an existing six-storey office, this Melbourne hotel designed by BatesSmart for Hume Partners Property would not have been possible without timber. With the existing office building only capable of accommodating an extra six storeys in concrete, the project team turned to a CLT and Glulam solution, finding the lighter weight system allowed an extra four storeys for a total height of 16 storeys. Images: http://atelierprojects.com.au/

7.4.3 Increased net saleable area

An outcome of the highly efficient and high-performing 'honeycomb' structural typology, projects featuring load bearing wall elements have been observed to achieve a higher floor plate efficiency than their traditional alternatives. With floors spanning 4-7 m and floor-to-floor heights rarely exceeding 3.1 m, multi-residential, student accommodation, and hotel projects are perfectly suited to the 'honeycomb' structure of panelised timber construction.

Panelised timber construction utilises wall panels as load-bearing elements, eliminating the need for columns. In a well-optimised design, party walls – being either discontinuous or staggered stud walls – are utilised as load bearing. These wall types are structurally efficient, and are often at least 40 mm thinner than the non-load-bearing alternative utilised in traditional projects (these are often up to 300 mm thick to allow for columns), with the same or superior acoustic and fire performance.

While 40 mm doesn't sound much, when multiplied by the hundreds of metres of party wall typically found in a multi-residential, student accommodation or hotel project, this can total tens of square metres, and potentially hundreds of thousands of dollars of extra revenue.

Figure 7.2: Dalston Works by Waugh Thistleton Architects (London, UK, 2017) is a landmark project in the use of timber construction in high-density urban housing. The 10-storey, 121-unit development is made entirely of CLT, weighing a fifth of a concrete building of this size, and reducing the number of deliveries during construction by 80%. Due to its reduced weight, the building is taller than was ever thought feasible using the same sub-structure on the site, the difference being very significant: 35 additional flats were made possible by using wood structures. Images: http://waughthistleton.com

7.4.4 Rental yields

Timber projects featuring exposed structural elements have been seen to achieve higher rental yields than traditionally built structures. While there are many potential reasons for this, tenants have been attracted by several proven benefits of exposed timber in office environments (see 8.6 Biophilic design), as well as the branding opportunity as an organisation that cares about the environment.

7.4.5 Goodwill and branding

Timber projects are widely identified as having a smaller impact on the environment than those using steel or reinforced concrete. With certified Engineered Wood Products manufactured from sustainably farmed, highly efficient plantations grown specifically for harvesting, timber construction can be confidently labelled as a sustainable practice.

Not only does the production of timber elements require significantly less power generation than other materials (and most of what is needed is provided by the sun), but trees absorb carbon dioxide, locking it up in the timber itself. This means that timber elements ultimately remove more CO_2 from the environment than their fabrication produces, making it the only truly green and sustainable construction material. This title is not lost on the informed public, and through popular media outlets has seen timber buildings earn a reputation as being beneficial for the environment.

Organisations seeking to differentiate themselves and transparently demonstrate their investment in the environment have been some of the first to both develop, and move into significant timber structures, and this trend is expected to continue for the foreseeable future.

More information about how to integrate sustainability features into costing and feasibility is presented in Chapter 8.

8 Life Cycle Costing

Timber projects can be simple and potentially cost effective to build but how much maintenance do they require and how do they perform in operation? And, can the carbon sequestration benefit be quantified and costed?

While there are still relatively few local examples to answer these questions, this section draws on the available local data as well as relevant evidence from international projects to do so. The list of international projects is significant and every-growing, with Engineered Wood Products established in markets around the world. This popularity is documented well in Waugh Thistleton Architects' 100 UK CLT Projects – showcasing 100 significant buildings in the United Kingdom alone.

8.1 Operation

Engineered timber elements are typically cut to a tolerance of 1 mm and sealed with tape during their assembly, meaning that, where used as external walls, they can deliver an airtight envelope to the building (subject to good window and door fittings). While just 20% the weight of reinforced concrete, timber elements have a very low thermal conductivity, good thermal inertia and high water vapour permeability. Combined, these are the properties explain why timber construction is the first choice for a 'passive house' design. While this means a significant reduction in the need for heating and cooling energy requirements, it requires a mechanical and automated means of providing the requested air exchange or an 'active' behaviour from the occupants operating its openings.

8.2 Durability

If designed and built correctly, timber projects can stand for hundreds of years. While this has been well demonstrated by religious structures overseas, it is also the case in Australia with a nine-storey timber frame structure standing in central Brisbane since 1913 (Figure 8.1).

To ensure the long-term durability of a timber structure it is vital to consider the location and the way to which it will be exposed to the weather. While it has been shown that internal and weather protected elements are stable without maintenance, structural members exposed to the outdoor environment require regular inspection and re-application of any finishes.

Internally, timber finishes exposed to UV rays may discolour over time however this has no impact on the structural performance of the element.

WoodSolutions *Technical Design Guide #5 Timber service life design - design guide for durability* is a useful tool for anyone seeking to understand the better way to minimise risk and associated costs in a given project.

Figure 8.1: Perry House in Brisbane, a semi-tropical climate, was built in 1913 with a timber structure and masonry cladding. It was then the tallest building in Brisbane. An extra storey was added in 1923. Perry House is still believed to be the tallest timber structure in the Southern Hemisphere that's built on wooden foundations. Its timber structures were kept in the 1996 renovation and are still efficient and beautiful in its new life as the Royal Albert Hotel. Its contemporary equivalent (Figure 8.2) adopts a CNC-machined GLT and CLT structure and is clad with a glazed curtain wall, but shares with Perry House the same care in design and constructions, for a wonderful and future-proof result that makes it a valuable asset. Images: https://www.royalalbert.com.au/

Figure 8.2: Aurecon (engineers) sought to create a world-class working environment when searching for new office space in Brisbane. The timber option they designed with BatesSmart (architects), built by Lendlease in 2018, created a 10-storey office environment that supports the health and wellbeing of its users and has met high rating standards from the Green Building Council, NABERS and WELL. Images: https://www.aurecongroup.com and WoodSolutions.

8.3 Maintenance

After an initial settling period where gravity loads, construction tolerances and moisture-related shrinkage or swelling act to different extents according to each project's specific conditions, engineered timber buildings are known to experience minimal seasonal movement in their service conditions, generating very little cracking in the finishes and virtually no disruption to vertically reticulated services. This is attributed to the high dimensional stability of timber at different temperatures, with very minimal expansion and contraction occurring parallel to the grain. Also, the lower weight and higher ductility of timber structures provide a better behaviour with respect to any wind loads, or seismic and soil movement events that are within the expected range of design actions.

Mid-rise Australian projects have been observed to require equal to lower maintenance when compared to similar buildings in the same vicinity constructed of heavier and/or more brittle materials. Remarkable examples are Melbourne's Forte' Living and the Library at the Dock, both of which have reported reduced maintenance requirements compared to a typical building of their class.

In-situ water damage repairs during building use, for instance after a leak from the hydraulic systems or appliances, as well as other repairs and modifications due to changes in the installations or the fit out, can be easily achieved with minimum disruption, noise and execution times.

Moreover, timber buildings can easily be designed for 'zero structural damage' and/or high levels of robustness with respect to actions beyond the codes and the project specifications, so it is possible, fast and economical to provide structural consolidation and repair after a fire, quake or high wind has caused some significant effects. This is an often underestimated quality of timber construction, that is nonetheless very interesting for a Cost Engineer able to use it to provide a significant benefit to the building owners and occupants.

8.4 Demolition and Disposal

At the end of a building's operational life, timber elements can be reclaimed and reused for other purposes. Where the timber structure is carefully dismantled, its elements may be re-used in another structure. Where not usable for a structural purpose, timber elements can be re-purposed in engineered timber board products, or at a later stage as fuel in a biomass burner.

8.5 Carbon Credits

Many organisations are managing their greenhouse gas emissions in order to:

- gain a competitive advantage in a rapidly evolving low-emissions context that is already rewarding energy efficiency and will soon need to extend to reducing emissions
- demonstrate corporate responsibility by becoming carbon neutral (e.g. reduce emissions as much as possible and compensate for the remainder by investing in carbon offset projects).

Buildings owned or occupied by an organisation are a primary tool to achieve cost savings from improved energy productivity or other operational efficiencies, while responding to the demand from investors and tenants for sustainable and energy-efficient accommodation. Increasing the use of wood products in a building is generally acknowledged as a significant contribution towards these targets by almost every rating scheme. As operational energy emissions are decreasing through improved energy efficiency and use of renewable energy, the embodied impacts of structural materials is increasing proportionally to a buildings impacts.

In Australia, at the time of writing, NABERS and the Green Building Council provide some rating benefits for timber construction systems, although not directly related to quantified greenhouse gas emissions or embodied energyand are in the process of making substantial changes in addition to the points in the star rating tool, especially for embodied carbon. The Federal Government has developed the voluntary National Carbon Offset Standard for Buildings with best-practice guidance on how to measure, reduce, offset, report and audit emissions that occur as a result of the operations of a building (i.e. emissions generated from the day-to-day running of the building) to target carbon neutrality. Emissions from energy (including energy embodied in construction materials) are not considered part of a building's operational carbon account and are therefore not covered by the current version of the Standard, which already foresees that "Embodied energy from construction materials and processes may be considered for future versions of the standard."

In Europe, the EN 16449 standard *Wood and wood-based products. Calculation of the biogenic carbon content of wood and conversion to carbon dioxide* provides a reference for different tools used in design, rating and procurement schemes. The standard specifies that the calculation method can be used in building design and Environmental Product Declarations. The calculation is based on the atomic weights of carbon (12) and carbon dioxide (44):

$$P_{CO2} = \frac{44}{12} \cdot cf \cdot \frac{\rho\omega \cdot V\omega}{1 + \frac{\omega}{100}}$$

With:

P_{CO2} The energy use at the end-of-life of the product (kg). In scientific terms, it's the biogenic carbon oxidiszed as carbon dioxide emitted by the product into the atmosphere

cf The carbon fraction of the wood-based products (0.5 is the default value).

ω Tthe wood moisture content (12% is the typical value).

$\rho\omega$ The density of the wood-based products at the above moisture content (kg/m^3).

$V\omega$ The volume of wood-based products at the above moisture content (m^3).

From a cost engineering perspective, being able to correctly quantify the carbon amount, embodied energy and emission reduction of a project is therefore already very important and may become critical in the near future. Next to the 'direct' effects proportional to the amount of wood used, the 'indirect' effects (e.g. the reduction of foundations and emissions from truck deliveries) will allow a project to reap the full potential from the applicable schemes and benefits.

To support designers and specifiers, WoodSolutions has made available a series of environmental product declarations (EPDs), developed through an extensive stakeholder consultation process (https://www.woodsolutions.com.au/articles/environmental-product-declarations).

Software tools for integrated life-cycle analysis (i.e. https://legep.de/?lang=en or https://www.oneclicklca.com/)support the project teams in the design, construction, quantity surveying and evaluation of new or existing buildings. Their databases contain the description of all elements of a building, their life cycle costs based on standard references and the calculation rules of the applicable certification schemes. All information is structured along life cycle phases (construction, maintenance, operation, cleaning, refurbishment and demolition). The environmental assessment comprises the material flows (input and waste) as well as an effect-oriented evaluation. The database is hierarchically organised, starting with the LCI-data at the bottom, building material data, work-process description, simple elements for material layers, composed elements like windows, and ends with macro-elements like building objects. The data can be used either bottom up or top down. Elements at each level contain all necessary data for cost, energy and mass-flow and affect evaluation.

8.6 Biophilic Design

The health and happiness benefits associated with biophilic design are well known and demonstrated, as summarised in a report from Planet Ark (Wood – Nature Inspired Design – freely available from www.makeitwood.org/healthandwellbeing). Surveyed Australians appear to be innately drawn towards wood. The results indicate that wood elicits feelings of warmth, comfort and relaxation and creates a link to nature.

Multiple physiological, psychological and environmental benefits have been identified for wooden interiors and how they provide a healthier, happier environment:

- ***Offices***: Productivity can be increased, and sick leave decreased, thanks to improvements to a person's emotional state and level of self-expression, resulting in significant benefits in concentration and efficiency.
- ***Education***: Reduced blood pressure, heart rate and stress levels result in increased rates of learning, improved test results, concentration levels and attendance, reduced impacts of ADHD (Attention Deficit Hyperactivity Disorder).
- ***Healthcare***: In a clinical study, the physiological effects described above resulted in post-operative rates of recovery improved by 8.5%, while pain medication was reduced by 22%.
- ***Retail***: The presence of vegetation and landscaping has been found to increase average rental rates on retail spaces with customers indicating they were willing to pay 8-12% more for goods and services.
- ***Residential***: Research in North America found that 7-8 % less crime is attributed to areas with access to nature and can command an increase of 4-5% in property prices.

In Australia, these benefits have already been shown to translate into higher rental returns and selling prices, in a growing number of cases.

Indoor environment quality not only benefits the current occupants but also enhances the value of a property in the long term and reduces the costs of ordinary maintenance, as occupants tend to keep what they appreciate and change what they don't.

The wellness advantages are being progressively reflected in the evolution of building standards and rating organisations, both in Australia and internationally, e.g. by the GBCA, WELL, Living Future and several other rating systems and networks.

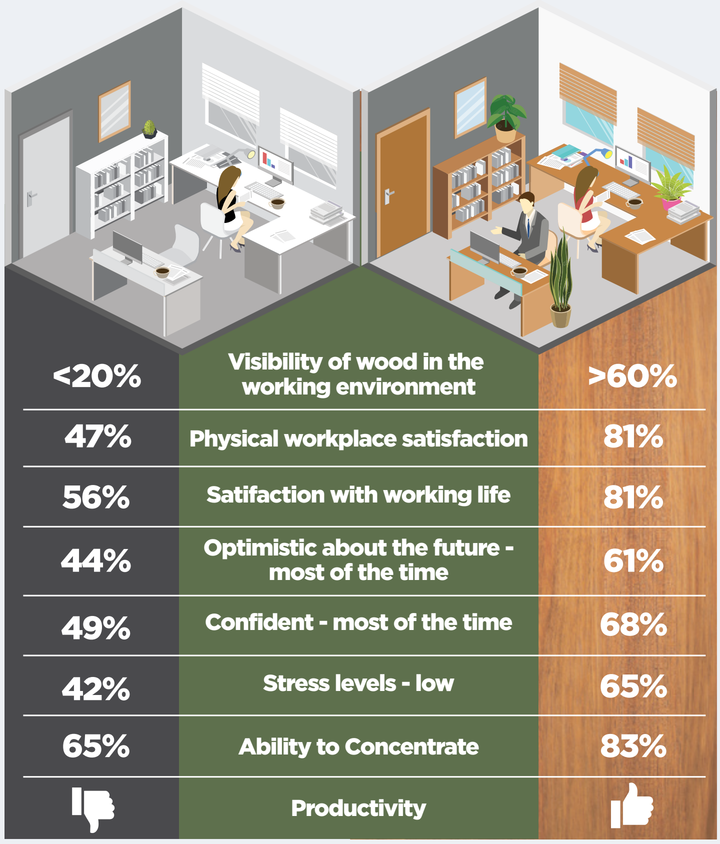


Figure 8.3: Results from an independent survey of 1,000 office workers in Australia show a clear relationship between visible wood and workplace satisfaction and productivity.

9 Case Studies

The case studies illustrate the principles discussed in this guide through examples which, although realistic, state of the art and referring to real projects, cannot represent the variety of cases and conditions that are found in practice. However, their value is in guiding through the different steps and activities that are typical of Cost Engineering and explaining their interdependence through the resulting numbers. In particular:

- ***9.1 Design Optimisation*** describes a value management activity on an existing timber-based design, with reference to updated objectives. It illustrates the costing of individual Engineered Wood components, based on few EWP inputs and simple % or parametric (/m^2, /m^3) rates for connectors, labour, CNC machining, transport.
- ***9.2 Estimating of a Whole Construction Cost*** shows how to take into account all the variables that can have an influence, and their relationship, into analysing the feasibility of a proposed development precinct.

Considering that numbers vary with time and location, and that some of what is reported in the case studies is subject to confidentiality, percentages are a better indicator that actual dollar values. Once the method is clear, the result will be as accurate as needed.

Considering that numbers vary with time and location, and that some of what is reported in the case studies is subject to confidentiality, percentages are a better indicator that actual dollar values. Once the method is clear, the result will be as accurate as needed.

9.1 Design

This case study aims to apply the concepts discussed in Sections 3 (Estimating) and 6 (Design Optimisation) to a typical residential mid-rise project, like the one which was first developed for use in *WoodSolutions Technical Design Guide #27 Rethinking Apartment Building Construction* (TDG#27) and later served as a basis for other initiatives such as our Demo Building at Holmesglen TAFE, and the worked examples in *WoodSolutions Technical Design Guide #50 Structural Engineering Timber Buildings*. Developed in collaboration with a group of consultants, with a typical floor plan featuring a variety of spans and in-set balconies, TDG#27 compares the costs of building an 8 storey multi-residential structure constructed entirely in one of three main structural systems: lightweight frame (stud frame), mass timber (CLT) and reinforced concrete.

This case study optimises the TDG#27 design approach a step further as per a typical value management process, in search of more cost savings, easier construction, and improved acoustic and fire performance. The most efficient way to achieve these targets is to apply the design optimisation concepts discussed in Section 6, specifying different Engineered Wood Products where they are best utilised. As shown hereafter, an efficient residential mid-rise timber design typically would feature lightweight framing for the top 3-4 levels, with massive timber components below.

9.1.1 Initial design

TDG#27 presents two timber designs, each of which demonstrates an efficient way to build using a single structural system (stud frames or CLT) within a platform frame approach (e.g. a 'load bearing wall' construction, with most walls aligning vertically to achieve an efficient load path and the floors sitting on top of each wall).

Where walls are required to be discontinuous for acoustic compliance this is achieved through the use of two wall panels separated by a 20 mm gap (while it is common for both panels to be loadbearing in stud frame construction, CLT construction typically requires just one of these panels to be load bearing while the other can be installed off the critical path of the program). Compliant acoustic separation of floors is achieved through the use of a wet 40 mm screed over a 10 mm rubber mat on the floor surface, and an underside comprising one layer of 16 mm fire-rated plasterboard, plus a 13 mm layer of standard grade plasterboard mounted on furring channels for mass timber, or alternatively two layers of resiliently mounted 16 mm fire-rated plasterboard for lightweight systems. Graphic representations of these two systems can be seen in Table 9.3 in Section 9.1.3.2.

The initial design of this structure is compliant with Deemed-to-Satisfy requirements of the National Construction Code, meaning that:

1. All structural timber is enclosed in fire-protective layers to achieve the required FRL.
2. All insulation within fire-protected cavities is non-combustible.
3. There are sprinklers throughout the project.
4. Cavity barriers are installed as required in areas.

With all loadbearing timber elements encapsulated by fire-protective linings, it is assumed that any mass timber within the structure features an industrial non-visual finish.

9.1.2 Areas for improvement

The first and most significant optimisation is, in this case, a revision of the structural design, optimised for its overall installed cost and not just for the material cost. While it is found that CLT provides the optimal solution for the higher loads experienced in the lower floors, the loadbearing function on the top four floors can be efficiently performed by stud framing. Therefore, the 105 mm and 85 mm thick CLT loadbearing walls found on the top three levels of the mass timber design were redesigned as stud frames, and all the panel sizes were optimised for transport and lifting.

Conversely, in the stud frame design it is commonly observed that the use of more and/or larger dimensioned studs at lower levels can be cost efficient than simply specifying a mass timber panel. This optimisation can have flow on effects, with the more structurally efficient mass timber panels resulting in a thinner wall panel than high-capacity load-bearing studs (plus fire-protective linings), and therefore more net sellable area.

This concept of designing for the installed condition can also be applied to floor panels, designed for their effective spans where unaffected by sizing constraints and adopting continuous spans wherever possible, rather than making all floor panels equal throughout the structure. While it isn't common to mix lightweight and mass timber floor elements on a single floor (although this is possible, e.g. if an exposed ceiling is requested for the lounge or bedroom), floor depth variance within single level allows the designer to maximise the available space for services reticulation and increased ceiling heights. This flexibility in appearance and structural floor depth may be expensive to achieve with concrete, but timber allows for high levels of design variance with very little penalty on cost and installation time.

While this cost balance between stud and mass timber elements is directly influenced by the timber and fabrication costs, it is also affected by the fire-rating requirements of the two systems (e.g. where lightweight elements may require two layers of fire-rated plasterboard for each fire-protected side, mass timber elements typically only require one).

Again, the 'installed cost' concept can be applied to the connectors utilised in a project. Connector costs and installation times can vary significantly, and as such it is valuable to optimise their design while this is being done for the structural elements.

With reference to acoustics, while the wall systems specified in the initial design are best practice, the floor systems offered an opportunity for further optimisation, as it features the same acoustic system for both mass and lightweight timber, with a 50 mm build up on top of the floor and furring channel mounted ceiling underneath. The build-up includes a 10 mm acoustic mat and 40 mm screed, providing both mass and resilience to the floor system. This design provides compliant acoustic ratings with modelling suggesting airborne ratings (R_w+C_{tr}) of 51, and an impact rating (L_{nw}) of 55 for both systems. These ratings render both the mass and lightweight systems compliant, however a value management approach is possible and was implemented.

There is also an opportunity to engage a fire engineer to model the behaviour of a fire in the building and identify areas for further design optimisation utilising a Performance Solution. While this will not be applicable to all buildings in all locations, this step often results in a simplified construction process and cost savings which typically far exceed the consultant's fees.

9.1.3 Optimised design

Structural optimisation

As shown in Table 9.1, a review of the structural design has resulted in an optimised mix of Engineered Wood products, with the structure made up of massive timber elements in the lower floors and stud-framed walls and cassette floors in the higher floors. In this case, the core remains as mass timber throughout the structure. To find the point at which it is most cost effective for the structure to transition from lightweight frame to massive timber we analysed the structural design and costing rates in conjunction, through a series of iterations.

Table 9.2 illustrates a case in which when using stud frame at lower floors (Level 5 here) the total costs to fabricate and fire rate may exceed the total costs associated with the CLT system, while the reverse is true from Level 6 up. The use of stud framing at higher levels is often an effective design optimisation, as panels with simple offsite fabrication, open on one side for the installation of services, can be transported and erected in the same manner as mass timber panels, providing equivalent performances and typically attracting a lower cost.

Table 9.1: Structural Optimisation Process (note that only one party wall type is shown here for illustrative purposes, but the value management was run on all the wall types).

Code	Element	Initial Design (Stud Frame)	Initial Design (CLT)	Optimised Design (mixed EWPs)
8	Wall	MGP10 90 x 35 @ 600 crs	CL3-85	MGP10 90 x 35 @ 600 crs
	Floor	499 mm deep cassette	CL5-225	412mm deep cassette
7	Wall	LVL 90 x 35 @ 600 crs	CL3-105	LVL 90x35 @ 600 crs
	Floor	499 mm deep cassette	CL5-225	412mm deep cassette
6	Wall	LVL 90 x 45 @ 600 crs	CL3 - 105	LVL 90 x 45 @ 600 crs
	Floor	499 mm deep cassette	CL5-225	412mm deep cassette
5	Wall	LVL 2 x 90 x 45 @ 600 crs	CL3 - 105	CL3 - 105
	Floor	499 mm deep cassette	CL5-225	CL5-225 (CL3-145 where applicable)
4	Wall	LVL 2 x 90 x 45 @ 600 crs	CL3 - 125	CL3 - 125
	Floor	499 mm deep cassette	CL5-225	CL5-225 (CL3-145 where applicable)
3	Wall	LVL 2 x 90 x 45 @ 600 crs	CL3 - 145	CL3 - 145
	Floor	499 mm deep cassette	CL5-225	CL5-225 (CL3-145 where applicable)
2	Wall	LVL 3 x 90 x 45 @ 600 crs	CL3 - 150	CL3 - 150
	Floor	499 mm deep cassette	CL5-225	CL5-225 (CL3-145 where applicable)

Table 9.2: Rate Comparison for two wall types at Levels 5 and 6, where the structural design requirements could be achieved with both options (Note: values applied for Melbourne in April 2019, ask the suppliers for an update).

Wall Type	Lightweight	Massive	Lightweight	Massive
Level	5	5	6	6
Wall Type	LVL 2 x 90 x 45 x 2 @ 600 crs	CL3-105	LVL 2 x 90 x 45 @ 600 crs	CL3-105
Bracing (one side only)	17 mm plywood		12mm plywood	
Costs ($/m^2)				
Loadbearing Element (stud frame or CLT panel)	$104	$157.50	$80	$157.50
Bracing	$50	-	$30	-
Insulation between studs	$10		$10	
Acoustic stud wall		$60		$60
Fire-rated plasterboard				
2 x 13mm FRPB (each side)	$120		$120	
1 x 16mm FRBP (each side)		$60		$60
Total Rate ($/m^2)	**$284.00**	**$277.50**	**$240.00**	**$277.50**
Selected for use?	×	✓	✓	×

To optimise the floor elements, cassette depths have been re-assessed, with a thinner floor cassette possible on multiple spans and depths further reduced where shorter spans are encountered.

Finally, all connectors have been reviewed and optimised based on availability, buildability and productivity (e.g. preferring connectors with fewer nails or screws to reduce the time spent installing each item).

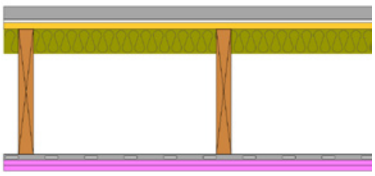
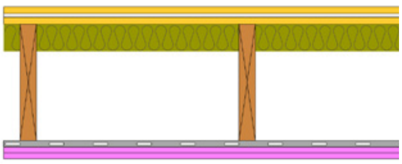
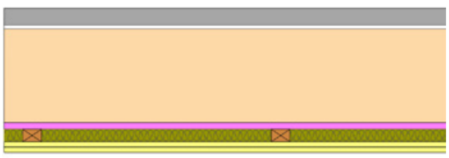
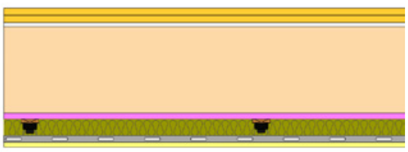
Acoustic optimisation

With wall systems already maximally optimised, we can focus on the optimisation of floor systems. The floor solution provided in the initial design has historically been a popular one, with builders correctly identifying a wet screed as an inexpensive way to add the required mass to timber floors.

While this system is indeed cheap to install, there are downsides associated with adding a wet trade to what is otherwise a dry site. First and foremost, the use of a wet screed calls for stringent moisture control to ensure that any moisture affecting the timber is able to escape (rather than pooling and causing durability issues). In addition to this, however, the use of wet screeds limits access to the screeded area while curing, limiting on-site productivity for this duration. With these issues in mind, a number of builders have moved toward achieving a similar effect with the use of dry boards. Whether compressed fibre cement, magnesium oxide, or even particleboard, the application of dry board products to the top of the timber floor (typically sitting on top of an acoustic mat) has been shown to deliver both the extra mass and resilience required to achieve acoustic ratings beyond compliance. What's more, other works can continue as these dry boards are installed, ensuring that the builder can take full advantage of timber's 'no-prop' characteristic.

To illustrate this, in this case study we replaced the 40 mm concrete screed on 10 mm of acoustic matting with one layer of 19 mm particleboard on 10 mm of acoustic matting (or 12 mm softboard) on the lightweight cassette, and two layers of 19 mm particleboard on 10 mm of acoustic matting on the mass timber panel. As shown in Table 9.3, this change provides a modest improvement in acoustic performance of the floors, even providing a decrease in floor depth for the cassette option. Note that while this floor system is effective in multi-residential projects, commercial projects can achieve similar ratings with the use of access floors.

Table 9.3: comparing acoustics and depth of system.

System	System Depth	Rw+Ctr	Lnw
Initial Design (cassette) 	50 mm (acoustics) + 382 mm (structure) + 32 mm (fire) + 35 mm (acoustics) = **499 mm**	51*	40-55 (depends on floor covering)
Optimised Design (cassette) 	29 mm (acoustics) + 319 mm (structure) + 32 mm (fire) + 32 mm (acoustics) = **412 mm**	55*	49* (no floor covering)
Initial Design (mass) 	50 mm (acoustics) + 225 mm (structure) + 16 mm (fire) + 50 mm (acoustics) = **341 mm**	51	55 (no floor covering)
Optimised Design (mass) 	48 mm (dry acoustic solution) + 225 mm (structure) + 16 mm (fire) + 50 mm (acoustics) = **339 mm**	51*	54* (no floor covering)

*INSUL Prediction **Based on Auckland Uni CLT Acoustic Test (with 140 mm CLT)

Fire engineering

The high level of this case study makes it difficult to develop a fire design different than using the DtS provisions and standard details, however if a site was specified and the design developed a little further it is likely that fire-protective linings and fire rating details for penetrations would be simplified.

9.1.4 Impact of optimisations

While direct factors such as reducing the volume of timber used or the number of connectors specified are easily quantified, less tangible factors such as the increase in productivity due to the use of dry boards rather than a wet screed or the simplification of fire rating details cannot be ascertained until the project is completed. In many cases this process may render the project more buildable, ultimately improving construction efficiency and reducing the total build time.

The structural optimisation has delivered thinner, lighter panels that may allow the builder to utilise a smaller crane. Finally, while the wall design of the base case is well optimised, it is common for a review of this design to result in a thinner wall panel with similar acoustic performance. This reduced area results in a variety of benefits, ranging from reduced loads on footings to the delivery of larger rooms and a higher quality product than otherwise achievable. These findings, and the overall impact of this optimisation, are displayed in Table 9.4 and Figure 9.1.

Table 9.4: Estimated impact of optimisation process in comparison to base CLT design. Cost savings identified in this case study relate to a design and layout completed with timber systems in mind, and a requirement for multiple building use classes. This is distinct from the second case study which refers to a building with the design initially completed for concrete and adapted to timber, and no transfer slab required.

Category	Estimated impact
Total Project Cost	-2% compared to initial CLT design
Comparison to concrete cost as estimated in WoodSolutions Technical Design Guide #27	-8% (formerly 6%)

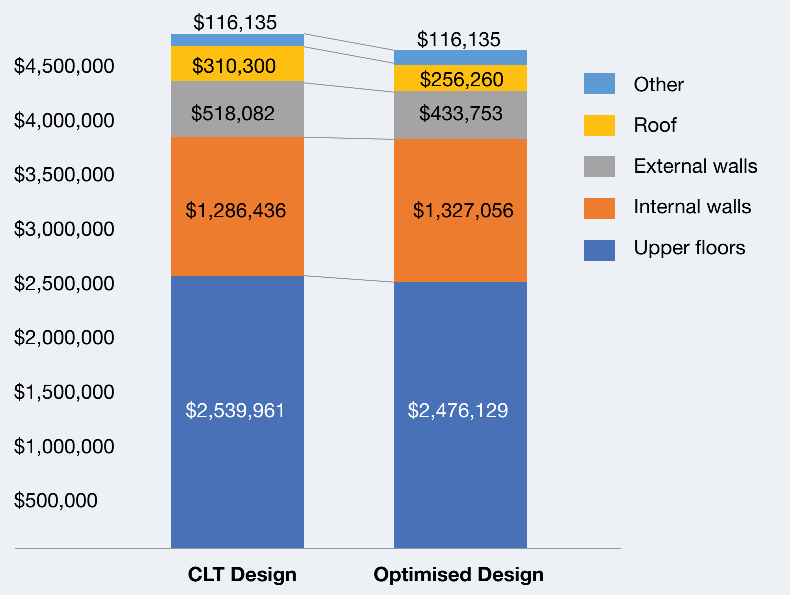


Figure 9.1: Graphical representation of the cost impact of design optimisation ('other' refers to costs not affected by the optimisation such as staircases and columns).

9.1.5 Estimating for design optimisation

Integral to the design optimisation process is the repeated checking for cost efficiency and viability. This section demonstrates how this checking was undertaken for this case study project with focus on how the prices for the walls, floors, and preliminaries were developed.

Before beginning the take-off it is important to ensure that the required level of design detail is available or that adequate assumptions are reported. For timber elements, this information may include the type of Engineered Wood product to be used, dimensional sizing, strength grade, level of treatment (if any), and quality of finish, and for connectors may include type of connector, and where required number of screws or nails required for compliance with certified structural design. Section 3 provides more information on each of these details. In this case study, we are aware of the type and strength of timber products required, and as the design is intended to be compliant with the Deemed-to-Satisfy requirements we can assume that the industrial non-visual finish is specified to any engineered products.

To illustrate the estimating procedure for a floor element in this case study, consider a section of the lightweight floor. The total installed cost of this element can be broken into two categories: supply costs and installation costs. The supply cost is best determined through the acquisition of quotes that allow for material and fabrication, however, where fabrication isn't possible this can be allowed for separately. For example, a cassette fabricator may be willing to quote for the supply and fabrication of the joists and floor membrane, but may be unwilling or unable to quote for the installation of the acoustic floor build-up. In this case, the floor build-up can be added on site, with the costing process proceeding as identified in Table 9.5.

Table 9.5: Supply cost for optimised lightweight cassette. (Values applied for Melbourne in April 2019, ask the suppliers for an update).

Detail	Quantity	Unit	Rate	Total
Floor cassette (including 300 deep, MGP10 chord floor joists and particleboard flooring)	1	m^2	\$135	\$135
Acoustic underlay (S&I)	1	m^2	\$15	\$15
19 mm particleboard (S&I)	1	m^2	\$25	\$25
2 x 16 mm fire-rated plasterboard installed on furring channel (installed on site)	1	m^2	\$81	\$81
Supply and fabrication cost for cassette:				**\$256**

With the off-site fabrication of the cassette already allowed for,, it is important that only the cost for the transport and site installation of the cassette is added. Installation rates can vary, depending on the complexity and efficiency of the project's design, however start by applying the installation rate of about 500 m^2/week observed from the database of completed projects identified in Section 1. This rate also allows for the installation of the walls within this floor area. We can simply divide the installed floor area by the approximate installation rate and multiply this by the expected number of workers, their working hours and their hourly rate. To this total number, add an allowance for contingency, overheads and margin, and this can be used to represent an approximate installation cost for the structure.

For example, if we assume this structure is 4,500 m^2 in area we can calculate an approximate installation program of nine weeks. Assuming that the installation of this structure will require seven workers (as observed from the database), working for eight hours a day, five days per week for this duration, we can calculate a net cost for installation. To this number we may choose to add 15% for contingency, overheads and margin, giving the gross installation cost for the structure (excluding any temporary works, propping or other project specific requirements). This number can then be divided by the installed floor area to give us a unit rate for install.

9.2 Estimating a Whole Construction Cost

In 2018, Rider Levett Bucknall undertook a benchmarking case study of a well-optimised project (Caulfield Village Precinct 1 Building 2A) developed by Beck Property Group, Melbourne. Completed in August 2016 by Probuild, the five-storey project comprises 65 apartments. Its initial specifications included a concrete flat plate slab with concrete columns and precast concrete core walls and stairs, and a façade featuring glazing, brick veneer and lightweight cladding.

Rider Levett Bucknall undertook costings based on the following revised specifications: in situ concrete podium; suspended slabs and roof comprised of a lightweight cassette system; Cross Laminated Timber (CLT) lift, stairs and stair shaft walls; and load bearing intertenancy and corridor walls comprised of timber stud. The revised specifications reduced the construction program by five weeks. An additional two weeks were eradicated as the curing process was eliminated, allowing finishing trades to follow on more quickly. In addition, the columns in the basement and the size of the pad footings were reduced, and columns throughout the upper levels were eradicated.

This benchmarking case study demonstrated that a switch to timber construction could have resulted in an overall saving of 2.2% and motivated Beck Property Group to request an estimate on one of the buildings designed for Caulfield Village Precinct 2, currently under development. The procedure and results are described hereafter.

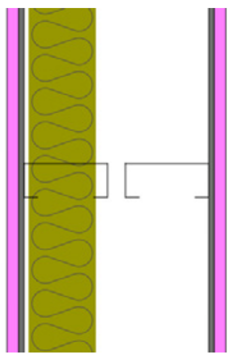
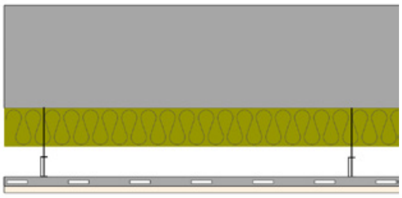
9.2.1 Case study project

This case study was an eight-storey apartment building with a total of 98 apartments and a Gross Floor Area of 7,817 m^2. The structure features a tight floor to floor height of 3,050 mm, with finished ceiling heights set at 2,700 mm above floor level. Originally designed in reinforced concrete, the project's façade includes large areas of exposed precast concrete, as well as areas of both glazing and lightweight cladding. The design features large balconies on each level, with most floor spans measuring around 5 m.

9.2.2 Initial concrete design

First designed for traditional concrete systems, the structure involved post tensioned flat plate slabs and in-situ columns, with an in situ concrete core. The significant dead load of the superstructure is transferred to an alternative column grid at the first floor via a 400 mm deep transfer slab with 500 mm deep in situ concrete beams (giving a total transfer slab depth of 900 mm). The design features non-loadbearing lightweight steel frame partitions and a lightweight dropped ceiling, with acoustic performances as identified in Table 9.6.

Table 9.6: Acoustic ratings of baseline floor and wall systems.

System	Build-up	Rw+Ctr	Lnw
Lightweight Steel Party Wall 	13 mm fire-rated plasterboard 6 mm cemintel wallboard 93 mm steel stud 20 mm gap 93 mm steel stud 6 mm cemintel wallboard 13 mm fire-rated plasterboard	51*	-
Lightweight Steel Dropped Ceiling 	200 mm concrete slab Rondo suspended ceiling systems w/ resilient mounts 11 kg/m^3 insulation Furring channel 13 mm plasterboard	56*	55-60 (no floor covering)

*CSR Redbook

9.2.3 Optimised timber design

An optimised timber design was developed, following the fundamental optimisation principles outlined in Section 7 and discussed further in Section 10. This design involved a massive timber structure to the fourth level, above which lightweight floors and walls extended to the full height of the building. As in the design of the case study project in Section 10, the central cores of the project are to remain as massive timber throughout. A massing of the project can be seen Figure 9.2.

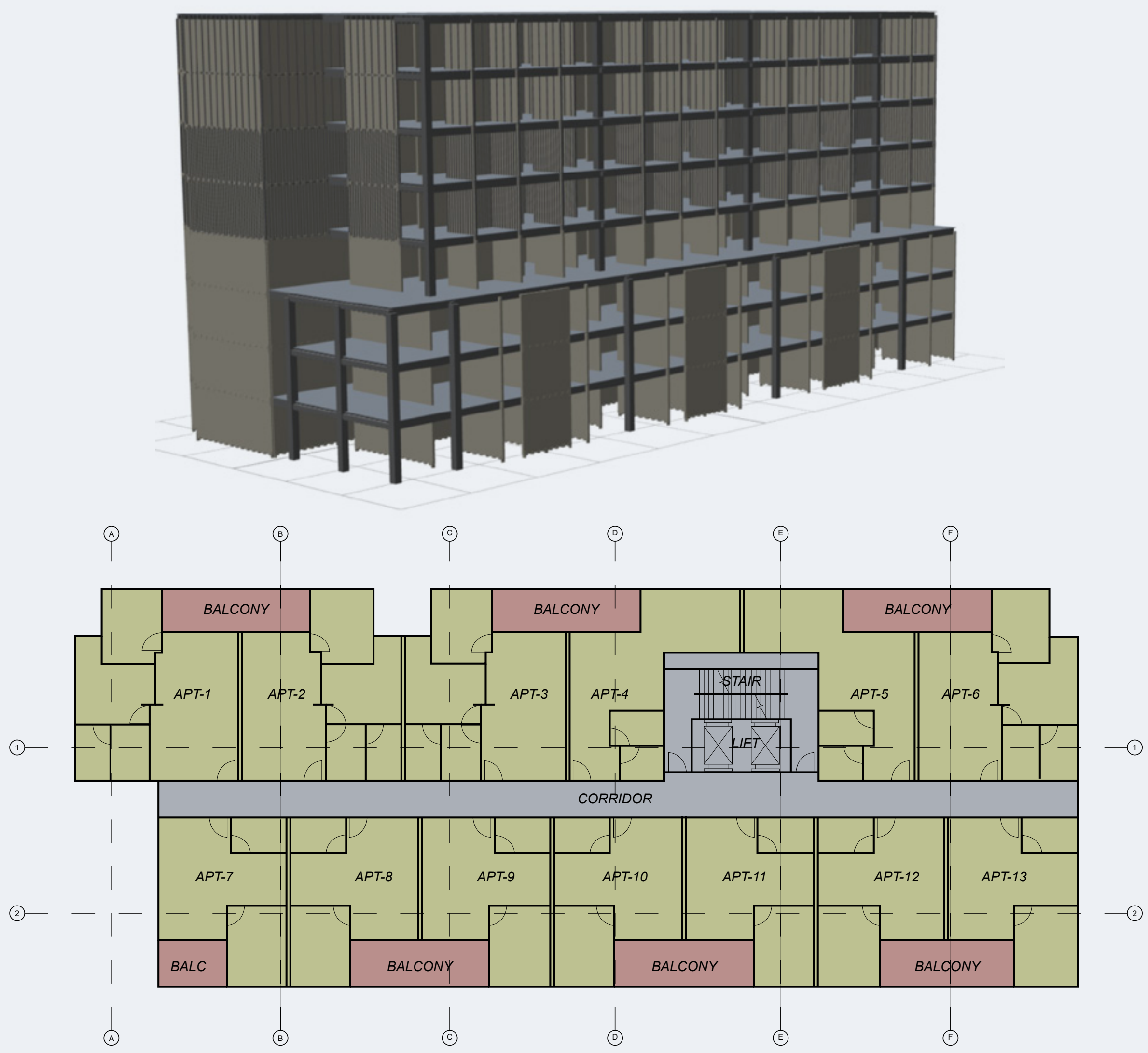
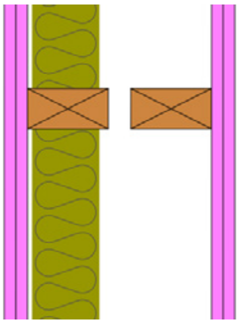
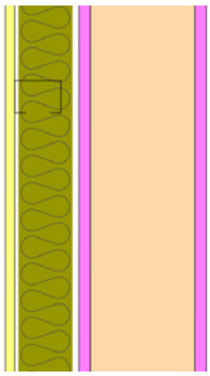


Figure 9.2: Massing and typical plan of case study project.

This new design was calculated to impart about 30% less dead load on the basement carpark and footings than the comparison in concrete, allowing for the significant transfer slab to be re-designed and simplified, and carpark columns and pad footings to be reduced in size by 25%.

The design of the acoustic systems also followed a similar path to what was observed in Section 10.1 of this report. A high standard of acoustic separation was achieved through the use of discontinuous wall construction for all party walls. Where the structure is timber framed, this is easily achieved via two parallel load bearing timber frames, with fire-rated linings applied to the outside faces. This system is effective as it delivers the required loadbearing capacity, a high standard of acoustic insulation, and the necessary fire protection within a single system. Where the structure is massive timber, acoustic discontinuity is achieved via a single load bearing mass timber panel, and a separate non-loadbearing frame. In this system, fire-rated plasterboard is directly applied to the massive timber element, with the stud frame available for the easy reticulation of electrical services. Examples of these systems can be seen in Table 9.7.

Table 9.7: Acoustic ratings of optimised wall systems.

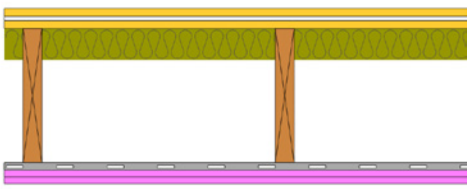
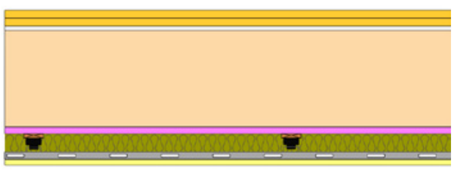
System	Build-up	Rw+Ctr
Lightweight 	2 x 13 mm fire-rated plasterboard 90 mm stud 20 mm gap 90 mm stud 2 x 13 mm fire-rated plasterboard	53*
Massive 	13 mm acoustic plasterboard 64 mm steel stud 20 mm gap 16 mm fire-rated plasterboard CLT element 16 mm fire-rated plasterboard	56**

*CSR Redbook ****WoodSolutions Technical Design Guide #44*

Similar to the walls, the acoustic system utilised for the floors varies slightly between lightweight and massive flooring systems. A strict limit of 350 mm is applied to all floor depths (inclusive of structure, fire rating and any acoustic treatment), requiring special attention to achieve this while delivering a high standard of acoustic performance. For the lightweight cassettes, acoustic compliance is achieved through the addition of an acoustic mat and one layer of 19 mm particleboard to the top of the system, and ensuring that the required two layers of 16 mm fire-rated plasterboard are fixed to the floor joists via acoustic isolation mounts. Mass timber floors feature a slightly more robust acoustic topping of two layers of 19 mm particleboard on top of a 10 mm acoustic mat (or 12 mm softboard), with the required layer of fire-rated lining direct fixed below, and a dropped ceiling of varying depths (dependent on the structural depth) below this. Examples of these systems can be seen in Table 9.8.

Note that while further efficiencies could likely be identified through a fire engineering process, this case study considers it under Deemed-to-Satisfy conditions.

Table 9.8: Acoustic ratings and depths of optimised floor systems.

System	System Depth	Rw+Ctr	Lnw
Lightweight 	29 mm (acoustics) + 257 mm (structure) + 32 mm (acoustics) + 32 mm (fire) = **350 mm**	54*	55* (no floor covering)
Massive 	248mm (dry acoustic solution) + 160mm (structure) + 16mm (fire) + 100mm (dropped ceiling) + 13mm (plasterboard) = **337 mm**	52*	53* (no floor covering)

*INSUL Prediction

9.2.4 Results

The results of this case study are as determined by Rider Levett Bucknall in consultation with the industry. In the completion of this exercise the cost consultant identified several main areas of cost variance, including both reductions and increases to certain cost categories. The following section interrogates each of these categories, noting the true value of timber construction in larger mid-rise developments.

Substructure

In this case study, the light weight of the timber superstructure has been shown to result in needing fewer and smaller footings. Beyond this, the reduced dead load also allows for all basement columns to be reduced in dimension by about 25%. This reduction in size not only saves material, but also saves labour hours, with smaller footings requiring less time in excavation, lining, steel fixing, and concrete placement. On the other hand, for this specific project, it was not possible to extend the benefits to the transfer slab, as there is no transfer grid. In mixed-used project where the residential part is above a commercial tenancy, this would have been possible.

Structure

This case study reveals that by using an optimised structural system, the cost associated with the structure can be comparable to, or less than, the initial design. Both allowances for internal and external walls increased in the timber design, as in this scenario many of the walls are loadbearing and therefore must be robust, with a higher degree of fire protection. Note that with all loads now transferred via the loadbearing wall elements, the structural columns initially specified throughout the project can be avoided, resulting in a significant cost impact.

Program and preliminaries

In consultation with major contractors and with reference to the database of timber projects in this guide, Rider Levett Bucknall developed a conservative indicative program for the construction of this case study project in both concrete and timber construction systems. In comparison, the timber design is expected to be delivered two weeks earlier than the concrete alternative, resulting in reduced time-related costs. The main impact of this reduction can be seen in the reduced preliminary costs. While this faster delivery has many other benefits (e.g. reduced interest expenditure, faster settlement times, etc), these have not been allowed for in this comparison.

Wall and ceiling finishes

In this estimate, the cost categories of both wall and ceiling finishes saw a reduction in total cost. This variance can be attributed to an allowance for these costs in the ‘Internal Walls’ and ‘Upper Floors’ cost categories, and therefore doesn’t represent a true reduction in the cost of finishes.

Summary of results

With these areas of cost variance in mind, an indexed comparison has been provided for the two designs in Table 9.9. All costs that remained the same across the comparison have been grouped in the ‘Other Costs’ category for simplicity. This cost exercise has identified a positive impact in the move to an optimised timber design, suggesting a total construction cost reduction of 2.51% could be achieved if this change were to occur. This cost saving does not allow for other potential savings associated with interest or holding costs.

Table 9.9: Indexed cost comparison of concrete and timber designs.

Element Category with cost difference	Indexed cost in initial design	Indexed cost in revised design	Percentage change in indexed cost
Preliminaries	10.63	9.92	-6.67%
Substructure	2.23	1.68	-24.86%
Columns	2.14	0.00	-100.00%
Upper Floors	11.14	9.16	-17.75%
Staircases	0.76	0.55	-28.26%
Roof	3.07	2.78	-9.66%
External Walls	15.48	16.78	8.35%
Internal Walls	9.46	12.35	30.50%
Wall Finishes	2.81	2.15	-23.45%
Ceiling Finishes	3.11	2.78	-10.60%
DD Contingency	2.44	2.36	-3.06%
Builder's Margin	2.92	2.84	-2.86%
Other Costs *	33.80	33.80	0%
Total	**100.00**	**97.14**	**-2.86%**

* Note: 'Other Costs' category includes all fields unchanged by varying the structure including external doors, internal screens, internal doors, floor finishes, special equipment, sanitary fixtures, sanitary plumbing, gas, ventilation, AC, fire protection, electrical, communications, transport systems, BWIC, ex storm and drain, and special provisions.

9.2.5 Feasibility and Sensitivity Analysis

The WoodSolutions Mid-rise Advisory Program has developed a discounted cash flow modelling tool to assess the potential financial viability of projects and model their sensitivity to movements in financial measures. For this case study, this sensitivity analysis was performed to determine the impact of project program on gross profit. With ancillary costs, approval timeframes, and sales values assumed, this feasibility analysis suggests that if this building were to be constructed at the full estimated cost and sold at a price estimated based on current market conditions, the profit realised would exceed typical developer requirements.

Furthermore, the sensitivity analysis identified that variance of construction cost and sales revenue are highly influential on the total profit margin, with any savings in construction costs or increases in revenue directly resulting in extra profit for the project. This is important when considering whether to pursue further design optimisation, as smart design using the right elements in the right locations may lead to further profits for the project.

10 Appendix 1: Guideline for the execution of timber structures

Contents

Introduction

This Guideline applies to the execution of timber structures to achieve the intended level of quality, safety and serviceability during their whole Life Cycle. Its main goals are to:

- define a set of technical requirements for the execution of a timber structure
- ensure that the designer transfers to the contractor all the relevant technical information for the execution of the structure
- specify conditions to be fulfilled before the works begins
- list controls suggested at delivery, during and at the end of the execution to ensure that the specified quality is achieved.

Before the work begins, make available for the contractor a set of documents and drawings/information models giving all the information required for the execution of the work, which includes the appropriate references to applicable Codes and Standards. This set of documents is referred to as the Execution Specification.

1 Scope

This Guideline contains provisions for the execution of load-bearing timber structures and applies to:

- timber structures built fully on site from individual components (e.g. stud frames)
- timber structures assembled with elements and modules prefabricated off site
- manufacture of timber elements and modules where no applicable product standard exists
- permanent and temporary timber structures.

This Guideline does not apply to:

- temporary parts of timber used only for equipment or construction aids for the execution (e.g. formwork and scaffolding)
- specification, production and conformity of the timber components
- safety and health aspects of execution or safety claim against third parties.

This Guideline does not regulate contractual and responsibility matters related to the construction work.

2 Terms and Definitions

Structure: Everything that is constructed or results from construction operations. It is a combination of connected parts designed to carry loads and provide adequate rigidity.

Connection: The point at which two or more structural members meet and are joined to transfer load effects.

Connector: Connecting element that transfers forces between two or more structural components.

Anchoring length: The depth of penetration of the portion where the tip is:

- the depth of penetration into the part that receives the tip (single section connection) or
- the thickness of the centre part (double section connection).

Critical moisture level: The predetermined level that, if exceeded, could cause damage associated with microbial growth.

Control: Evaluation of compliance by observation and assessment on the production site supported by measurement, testing or interpretation where appropriate.

Quality plan: Specification of the procedures and associated resources to be applied when and by whom to a specific object within a quality management process.

Temporary structure: Structure designed for a short design working life.

Assembly plan: Documents including drawings/information models, technical data, tolerances, weather protection measures, the order of work, execution methods, assembly procedures and other requirements necessary for assembly.

Assembly tolerance: Geometrical tolerances relating to location, verticality, horizontality or other characteristics of the assembly of a structure.

Prefabricated timber element/module: Timber element or module that is built on a place other than the final location of use, either at the factory or on site.

Manufacturing tolerance: Permissible deviation for the dimensions of a timber component, timber element or timber module arising from the production of the component, element or module.

Execution Specification: Documents that include all drawings/information models, technical data and requirements necessary for the execution of a specific project.

Compound construction tolerance (the 'box principle'): The sum of the tolerances for the actual position of a point, line or surface of the component and its basic position on the construction site.

Permitted deviation: Permitted algebraic differences between the limits of size and the corresponding reference size.

Tolerance: Difference between the upper and lower limits of size.

Note: Geometrical tolerances for prefabricated timber components, elements and modules are:

- manufacturing tolerances as defined in the relevant product standards
- assembly tolerances.

Tolerance is an absolute value without designation. It is expressed, however, usually by "sum of ± permitted deviations", so the value of tolerance is implied.

Timber component: Part of a timber structure that itself may be composed of several components.

Execution: All activities in the physical implementation of the work, including procurement, storage, gluing, mechanical fastening, transportation, stabilising measures, assembly of prefabricated timber elements/modules and control and documentation.

Execution Class: Classified set of requirements specified for the execution of the works as a whole or an individual component.

Note: Execution Classes differentiates requirements based on varying reliability expectations, complexity of the project and degree of new technology being used.

Consequence Class and Reliability Class: A classification of structures based on the risk associated with a collapse, referring to potential consequences. The Consequence Class of a structure determines its Reliability Class, which influences the design assumptions but also the levels of supervision, inspection and execution.

As an example, the following are the definitions adopted in the Eurocodes:

Class	Description	Examples
CC1 and RC1	Low consequence for loss of human life, and economic, social or environmental consequences small or negligible	Agricultural buildings that people do not normally enter (e.g. for storage), greenhouses
CC2 and RC2	Medium consequence for loss of human life, economic, social or environmental consequences considerable	Residential and office buildings, public buildings where consequences of failure are medium
CC3 and RC3	High consequence for loss of human life, or economic, social or environmental consequences very great	Grandstands, bridges, public buildings where consequences of failure are high (e.g. a concert hall)

3 Documentation and Quality Management

3.1 Assumptions

Before the execution begins, meet the following:

a) Accurately design the structure or the structural part, including prefabricated timber elements and modules.

b) Make the Execution Specification for the works available at the construction site.

c) For work that builds on previously performed work, ensure documentation showing that the work is within the permitted deviation, e.g. for foundations.

3.2 Execution Classes

Specify the Execution Class to be used in the Execution Specification, on the basis of the expected reliability requirements, the complexity of the work and the degree of new technology being used. Execution Classes differentiate requirements for documentation, competence and control of execution and may refer to the whole structure, to structural parts or to specific materials and technologies used in the execution.

Table 1: Guidance on the selection of the Execution Class.

Consequence Class, Reliability Class / and other associated requirements	Execution Class[a]
CC1/RC1 (low)	1
CC2/RC2 (medium)	2
CC3/RC3 (high)	3
[a] A more stringent Execution Class than the CC/RC would indicate may be selected, for example, because of a high degree of complexity and the use of new technology.	

3.3 Documentation

3.3.1 Execution Specification

The Execution Specification received for the works includes the following elements:

- project specification that contains information and requirements for the specific project with reference to this Guideline, relevant standards and regulations
- Execution Classes for the work
- any special control requirements
- tolerance classes for the work
- drawings/information models and other technical documents necessary for the execution including which products are to be used
- requirements for handling and storage, including weather protection during storage and execution
- assembly plan for prefabricated timber elements and modules, and where appropriate for other wood components
- moisture control plans.

3.3.2 Documentation of execution

Perform registrations and documentation in accordance with Tables 2, 3 and 4.

Measurements showing that the performed work meet the tolerance requirements of clause 8, should normally be recorded and documented, even when it is not required. The timing of control should be considered, especially if control is difficult at a later stage.

Where special documentation of the execution is required, specify the nature and extent of the documentation in the Execution Specification.

3.4 Quality Management

3.4.1 General

Where a quality plan is required in the Execution Specification, make it available on site. There may be one quality plan that covers all activities or one ‘high level’ plan complemented by separate plans for the various phases and activities to be performed.

To ensure compliance with the requirements of the Execution Specification and the requirements of regulations and agreed standards, the work should be performed with:

- the necessary competence and qualifications of the personnel
- suitable equipment
- adequate resources (e.g. staff number and time).

It is assumed that the execution will follow regulations and standards such as those involving all the mandatory OH&S and environmental aspects of the construction work.

The project and site management are responsible for organising the work to ensure:

- correct and safe use of equipment and machinery
- satisfactory quality of materials
- the execution of the structure meets the requirements
- coordinated assembly of prefabricated elements and modules according to the assembly plan
- safeguarding of structure until the handover of the works.

3.4.2 Execution Control

General

The requirements for control are set using one of three Execution Classes (Table 2).

An execution control involves checking that the execution, products and materials are according to the Execution Specification.

This Guideline does not define any provisions related to the degree of independence of the personnel performing the control.

The Execution Specification may specify additional provisions for the control.

Weather protection measures

The weather protection measures can be preventive or corrective. The higher the possible impact of moisture damage, the higher the level of preventive measures that is needed.

Moisture control plans should cover the entire construction process for timber structure and conform to building material manufacturers' instructions. Quality procedures may be established where moisture control plans are included.

Unless otherwise agreed, the contractor is responsible for the preparation and follow-up of moisture control plans, including their documentation.

The Execution Specification should specify the matters of significance for the moisture control.

The supplier(s) of all the moisture-sensitive building components specify the moisture level in the components at delivery and necessary weather protection measures in order to avoid exceedance of the critical moisture level.

Moisture Control plans

Make moisture control plans where:

- relevant moisture sources are considered
- the possible consequences of moisture damage are assessed
- moisture level in the different phases of the construction work is specified
- measurement methods are specified
- weather protection and necessary protection measures are indicated
- measures to limit moisture damage in the structure are specified.

Moisture control plans may include the following:

1) Basic information of the project such as:

- address and coordinates of the construction site
- name of the construction site manager
- the name of the person responsible for follow-up of the moisture control plan.

2) Overview of wood materials and products to be used on the construction site.

3) Limit of moisture content in wood upon delivery to the construction site, during erection and at completion.

4) Inspections on site and name of the person(s) who perform the inspections.

5) Possible moisture sources at the construction site (rain, snow, groundwater, etc).

6) Weather protection measures chosen for the construction phase and an estimate of the necessary duration of protection.

7) Protection of timber components on site:

- during storage
- during assembly (as given by the level of protection)
- drying methods for wood above critical moisture level.

8) Controlled drying of structures down to the level of use of the building:

- risk assessment and prevention of wetting, including rain
- the project's vulnerability to unfavorable weather and exceptional circumstances
- determination of moisture level in the wood, drying methods, drying times and suitable drying conditions
- organisation of drying conditions
- effects on the building site's time schedule (alternative plans).

9) Moisture measurement plan:

- measurement method; perform moisture controls with a calibrated instrument and preferably following a standard measurement method
- time schedule
- documentation
  - type of measuring instrument
  - air temperature
  - measurement points and positions
  - measurement date
  - measurements values, any corrections and final values
  - assessments and conclusions
- the responsible person.

Control of materials and products at delivery and after storage

At delivery, check the identification of received materials and products against the specification in the Execution Specification. It should also be checked at delivery and after any storage for any damage. Control requirements at delivery, for compliance with the Execution Specification, are given in Table 2.

Table 2: Control of materials and product on construction site at delivery and after storage.

Subject	Execution class 1	Execution class 2	Execution class 3
Materials for temporary stabilizing and corrective measures	Spot checks of compliance with the requirements of 7.1	Systematic control of compliance with the requirements of 7.1	
Fittings and connectors	Spot checks	Systematic control	
Timber components	Spot checks of compliance with the requirements of 5.1 and 5.2	Systematic control of compliance with the requirements of 5.1 and 5.2.	
Other components	Spot checks	Systematic control	
Handling and storage on site	Spot checks of compliance with the requirements of 5.3 and 5.4	Systematic control of compliance with the requirements of 5.3 and 5.4	
Documentation	Not required	Required	

At an initial inspection, verify and document that the materials and products delivered to the construction site are according to the Execution Specification. If the order or the order confirmation is controlled and found to be in accordance with the Execution Specification, the delivery may be checked against the order or order confirmation.

A control at delivery should include the following checklist:

- amount and quality of the products according to the Execution Specification
- assembly instructions are provided, clear and sufficient
- there is no visible damage
- the moisture content of wood-based and other moisture-sensitive products is correct.

Errors, defects and damage should be documented and signed by both parties before the delivery ticket is signed, and reported to the responsible manager for the follow-up of deviations.

Control of execution

Control requirements for compliance with execution specifications are given in Table 3 and Table 4.

Table 3: Subjects for control of execution.

Subject	Execution class 1	Execution class 2	Execution class 3
Control of the manufacturing and erection	Control of compliance with requirements in 5.2.2 and Chapters 6, 7 and 8 that are important for further execution.	Control of compliance with requirements in 5.2.2 and Chapters 6, 7 and 8.	
Control upon completion[a]	Control of compliance with requirements in 5.2.2 and Chapters 6 and 8.		
Assembly of prefabricated timber components, timber elements and timber modules	Control of compliance with the requirements of 7.2.		
[a] If control upon completion is not possible, the structure or structural part should be controlled during erection. This applies for instance when the structure is enclosed.			

Table 4: Type of control and documentation of control.

Subject	Execution class 1	Execution class 2	Execution class 3
Scope	Visual inspection of all works. Random control measurements	Visual inspection of all works. Systematic and regular control measurements of important works. Possible additional requirements for control according to the Execution Specification.	Visual inspection of all works. Detailed control of all works of importance for the bearing capacity and durability of the structure. Additional requirements for control as specified in the Execution Specification
Type of control	Self-inspection	Self-inspection. Internal systematic control.	Self-inspection. Internal systematic control. Extended control.
Documentation of performed control	Not required	Required	
Documentation of as-built structure	Not required	Control of compliance with Execution Specification required	

Control of execution verifies and documents that the execution is in accordance with the Execution Specification.

Record, document and resolve deviations in accordance with 4.3.3.

A control of the execution should include the following checklist:

1) Control that the work done by others is sufficient and within tolerances so that it is ready for further work, such as:
   - correct position of the building (map references, position of the corners)
   - casted slabs, floors and foundations (height, position, diagonal measures and any surface deviations).
2) Control of supports.
3) Control of temporary bracing.
4) Control of external walls.
5) Control of roof.
6) Control of the assembly of prefabricated elements or modules.
7) Moisture control.

3.4.3 Measures in case of deviations

Handle deviations according to the quality system of the contractor (or their client, if applicable).

Where the control reveals deviations, take appropriate measures to ensure that the structure is suitable for the purpose.

Examine the following conditions in the order listed:

a) the significance of the deviation on further execution and appropriateness to the designed purpose

b) measures necessary to make the component acceptable

c) the necessity of replacement of a non-repairable component.

When required in the Execution Specification, the deviation should be corrected in accordance with a procedure specified in that specification or as agreed.

3.4.4 Competence requirements

General

Requirements for competence are specified in the following points:

- project leaders
- construction site manager, where relevant
- assembly manager for prefabricated timber components, elements and modules, where relevant
- carpenter
- inspection manager of internal systematic control, where relevant
- inspector for internal systematic control, where relevant.

The project leader, inspection manager for internal systematic inspection and assembly manager for prefabricated timber components, elements and modules are to be be present and available where the work is performed, to the extent found necessary.

The same person can cover multiple management tasks if competence requirements are met and documented.

Project leader

The project leader has the top professional supervision of all parts of the works, including scaffolding, element assembly, transport, erection and finishing operations, depending on what is relevant for the specific project, also when done by subcontractors, hired workers or chartered business enterprises.

The project leader needs to have:

- understanding of loads and structural behaviour during the construction period and in the final state
- thorough knowledge of building technology, choice of construction methods and equipment
- understanding of how moisture and weather conditions affect execution
- understanding of the necessary requirements for timber properties
- understanding of required control of the works.

The project manager needs to understand the principles of the performed work operations, and have knowledge of critical aspects of the execution. For assembly of elements, the project leader will have undergone training in the relevant technological field.

When working in Execution Class 1 and 2, the project leader needs to have, at least:

- relevant craft certificate or equivalent qualifications
- necessary relevant supplementary education
- the experience necessary for the work.

When working in Execution Class 3, the project leader needs to:

- be an engineer with special qualifications for leadership of execution of timber structures or have equivalent knowledge
- have necessary relevant supplementary education
- have documented relevant experience in execution of timber structures in execution class 3.

Construction site manager

The daily work is carried out under the management of construction site manager with:

- relevant craft certificate or equivalent qualifications
- necessary relevant supplementary education
- work experience in similar type of work.

Assembly manager for prefabricated timber components, elements and timber modules

The assembly manager will lead the assembly work at the construction site according to the assembly plan.

The assembly manager needs:

- proven skills on structural behaviour, materials used, required weather protection, securing of elements and requirements for working at heights
- necessary knowledge of stability, stays, mooring and lifting
- proven experience of similar work.

Carpenter

The carpenter needs to:

- have craft certificate or equivalent qualifications
- know the requirements, rules and regulations applicable to assembly work
- have a basic understanding of concepts such as stability, stays, mooring and lifting.

Control manager for internal systematic control

The internal systematic control of the execution will be carried under an overall supervision of the Control Manager with experience in technical control, good insight into what is critical work operations and good insight into what is critical for the structural functionality.

The qualification requirements to the Control Manager of a work are corresponding to the requirements of the project leader. Experience both from the construction site, as a designer and with control work is relevant practice.

Where the Inspection Manager delegates parts of their tasks, the person delegated needs to have equivalent qualifications for the particular field.

Where a subcontractor performs parts of the works, the qualification requirements for the control manager of the subcontractor's control manager are equivalent to the one that leads the subcontractor's work.

Inspector for internal systematic control

The Inspector for internal systematic control needs to have adequate theoretical and practical knowledge to perform the allocated tasks, and a good understanding of the work to be performed.

4 Timber Materials

4.1 General

4.1.1 Product properties

Mark products for identification and, where required in the Execution Specification, the planned position in the structure.

Products must be in accordance with the Execution Specification and satisfy relevant product standards. Any surface treatment and structural protection must be in accordance with the Execution Specification and the manufacturer's instructions. Surfaces visible in the completed structure should not be exposed to sun and moisture, which would lead to an unattractive appearance.

4.1.2 Prefabricated elements and modules

Elements and modules manufactured in a factory are, until the delivery at the construction site, covered by the relevant product standards (prefabricated products). If the elements or modules are built on the construction site, the requirements in the relevant product standards apply, but from the time these are transported away from the place of production, the requirements of this standard apply. For prefabricated elements and modules not covered by any product standard, the provisions for manufacturing in this standard apply.

4.2 Moisture

4.2.1 Moisture level upon delivery on site

Where the Execution Specification contains requirements for moisture content upon delivery, control the moisture level of the delivered materials using the measurement methods referred to in the quality plan.

Control materials on delivery according to the execution class (see Table 2).

Unless otherwise agreed, the following moisture level is normally assumed upon delivery:

- solid timber for studs, plates, joists and trusses (non-pressure-treated): 10 to 16%
- solid timber for studs, plates, joists and trusses (pressure-treated): 16 to 24%
- glulam and Cross-laminated timber: 10 to 16%
- LVL, OSB, plywood and particleboard: 8 to 14%.

4.2.2 Moisture level during execution and upon completion

During transport, storage and erection at the construction site, keep the moisture level in the materials under control with weather protection and by controlling the moisture level in accordance with the moisture control plan. Make measurements where the humidity is highest or in areas with the highest risk of biological growth. It may be necessary to measure the moisture in the timber several times during the execution. Upon completion, control and document the moisture level.

Closure of timber structure (establishment of barrier layers) is not allowed until the desired moisture level is achieved, typically lower than 18%. It is assumed that other materials enclosed inside the structure are sufficiently dry.

For structures that dry very slowly after the closure, such as walls below ground level, compact timber roofs and sleeper floors with an impermeable coating, the critical moisture level may be lower than 18%. For such structures, a separate risk assessment with regard to critical moisture level should be provided in the Execution Specification.

At high moisture levels, assess what measures will be taken, such as further moisture measurements, drying measures or replacement of wood. Where there is visible mould growth, implement measures before closure of the structure.

Moisture control plans may differ from the Execution Specification where well justified, for example by changes in conditions on the construction site. Document the deviation. Such deviations must not reduce the quality of the completed structure.

4.3 Handling

Upon delivery and before installation, control handling so that there are no defects or malfunctions of materials or products (scratches, distortion from incorrect stacking, dirt, grease…).

Plan handling and carry out risk assessments in order to prevent damage.

Make available a lifting plan for heavy or complex lifting operations.

4.4 Storage and Weather Protection

Store products in accordance with the manufacturer's instructions.

In storage, keep the base flat and stable. Have at least 100 mm clearance from the terrain. On moist terrain, establish a barrier layer with necessary fall for runoff to protect against evaporation from the ground. Ensure good air circulation. Stack timber materials in such a way that damage does not occur at the bearings and there is no danger of overturning.

Cover the structures as follows, depending on the specified weather protection requirements, to avoid undue wetting:

a) No protection required

- relevant for only short periods (e.g. during erection and before sealing the roof) with no requirement to the appearance or where surface protection or other means provide protection
- structures with effective runoff (such as vertical structural surfaces).

b) Shielded and ventilated

- structures with required wood moisture to be to under 18%
- relevant for storage at the construction site
- structures erected over a longer period in areas with the risk of heavy rainfall
- the base does not contribute to wetting by capillary action.

c) Protection with tents or a temporary roof built over the roof

- structures with required wood moisture to be to under 12%
- relevant for structures with special requirements for moisture level and appearance.

Select weather protection during storage based on an assessment of the risk of wetting.

5 Connections

5.1 General

Follow the Execution Specification and the manufacturer's instructions during installation of fasteners.

Perform connections according to the Execution Specification.

Generally, the moisture level in the wood during assembly should not differ significantly from the moisture level in the connection as a whole.

Make connections so that the surfaces are planar, congruent and get good rest. Do not place connections on active faults, such as cracks, knots and dull edges, where this has an impact on the capacity or function of the connection.

Assemble structures in such a way that connections are not overloaded. Replace connections that have been skewed, cracked or that are poorly fitted.

Do not allow dirt and water collecting pockets in connections caused by the execution.

Where pre-drilling for mounting of fasteners is necessary, but not specified in the Execution Specification or in this standard, report the deviation to the designer in order to ensure that the strength and rigidity of the connection are achieved.

Ensure the connection is corrosion-resistant and fire-protected as specified in the Execution Specification.

Fasteners must be in accordance with the Execution Specification, which complies with relevant product standards.

5.2 Nail Connections

Connections require at least two nails.

Unless otherwise specified in the Execution Specification:

a) Perform skew nailing and skew screwing with approximately 45° angle with a minimum distance equal to 10x nail diameter *d* to the end (see Figure 2).

b) Nails are normally nailed perpendicular to the fibre direction and in a depth so that the nail head is flush with the surface of the wood.

c) Cross-loaded nails require a minimum anchoring length of 8x nail diameter *d*.

d) The diameter of the pre-drilled hole should not exceed 0.8x nail diameter *d*.

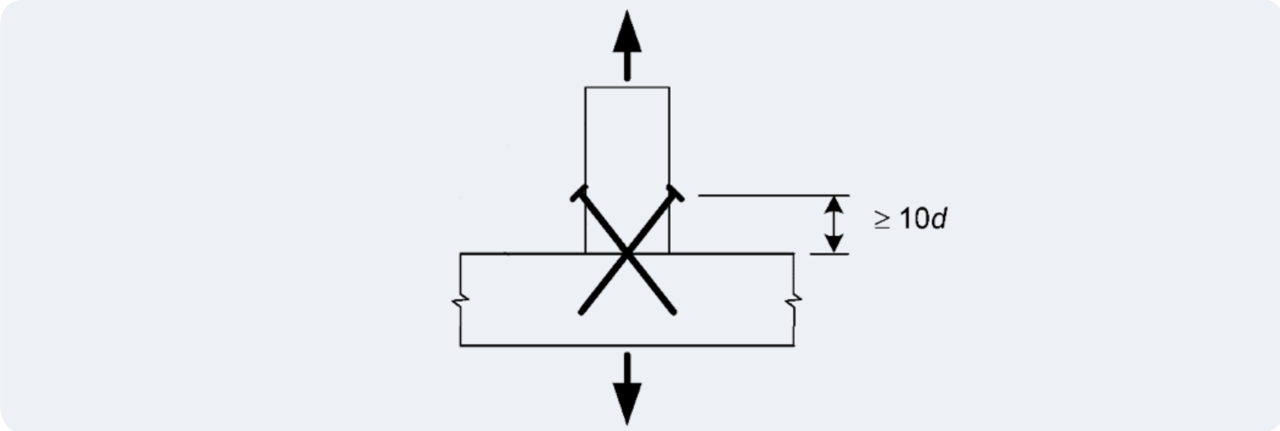


Figure 2: Skew nailing of connections.

5.3 Screw Connections

Pre-drill according to the manufacturer's instructions. Where instructions are not available, the holes may normally be pre-drilled with the following requirements:

- the hole for the shaft should have the same diameter as the shaft and the same depth as the length of the shaft
- the hole for the threaded part should have a diameter equal to 0.70x the shaft diameter
- guiding holes and pre-drilling holes for self-tapping screws require a diameter less than the internal thread diameter of the screw.

Pre-drilling can prevent cracking and injuries during execution. Pre-drilled holes may be appropriate to ensure proper placement and orientation of the screw. Test drilling may be used to check the suitability of the selected technical solution.

Unless otherwise specified in the Execution Specification, the anchoring length of screws needs to be at least 6x the screw diameter d.

Avoid overpulling of screws.

5.4 Bolt Connections

Select the bolt length so that at least one full thread round is above the nut, measured from the nut outer surface to the end of the bolt after tightening.

Ensure the diameter of the bolt hole in the wood is not any more than 1 mm larger than the bolt diameter d.

Washers against wood need a side length or diameter of at least 3x bolt diameter d and a thickness of at least 0.3x bolt diameter d. Mount washers under the bolt head and nut with complete contact surface.

Screw nuts on the corresponding bolt and tightened them so that the parts fit tightly. Repeat the tightening if necessary when the timber has reached its equilibrium moisture to ensure the structure's capacity and rigidness.

Avoid overpulling of bolts.

5.5 Dowels

Pre-drilled holes in the timber require have a diameter smaller than the dowel.

5.6 Glued Connections

Glued connections, where the adhesive bond strength is a precondition for the capacity in the ultimate limit state, need to be in accordance with the Execution Specification and controlled in order to ensure that the reliability and quality of the connection are consistent with the technical specification.

Follow the adhesive manufacturer's recommendations with regard to composition, environmental conditions for application and curing, the moisture content of the components and other factors relevant for the proper application of the adhesive.

For adhesives that require post-curing in order to secure full strength to be achieved, postpone the application of load on the connection until full strength is achieved.

Use adhesives that are not corrosive and are chemically neutral in relation to the base materials.

Adhesives should not be hygroscopic and be durable in the environment in which they are used. Ensure they are resistant to ageing, and adapt the curing temperature to the properties of base materials.

6 Special Provisions for Execution

6.1 Temporary Stabilising and Correcting Measures

6.1.1 Basic requirements

Safeguard elements and structural parts before and during construction. Ensure temporary support and bracing. Before commencing the work, agree which measures are to be implemented and who is responsible. Carry out measures in such a way that the structure or structural part:

- can withstand all foreseeable loads it can be exposed to during the construction process
- is rigid enough to ensure compliance with the specified geometrical tolerances
- does not get defects.

Treat the form, function, appearance and durability of the permanent timber structure with caution during assembly, use and disposal of temporarily stabilising and correcting measures.

Where the Execution Specification specifies stabilising measures, report deviations to the designer and document the divergent measures.

Ensure materials for temporary stabilising and correcting measures comply with the relevant product standard and are fit for the purpose to ensure the safety and requirements of the structure are met. Where there is no such product standard, take into account the properties of the material.

6.1.2 Design and assembly

Consider whether it is necessary to develop a description of methods for the assembly and dismantling of temporary supports and bracings.

Unless otherwise agreed, the contractor is responsible for the design of temporary stabilising and correcting measures. In the design of temporary supports and bracings, consider deformations that may occur during the execution of the permanent timber structures.

Where the design of the final permanent structure assumes support, bracing or other measures to the structure before other parts are in place, state this in the Execution Specification.

Do not remove temporary stabilising and corrective measures before the permanent timber structures are sufficiently secured to:

- prevent damage during demolition, e.g. to people, equipment or structures
- carry the loads applied to the timber structure at this stage
- avoid damage due to climatic effects.

Where the order for the removal of temporary stabilising and corrective measures are important, methods should be described and included in the Execution Specification.

6.2 Assembly of Prefabricated Timber Components, Elements and Timber Modules

The assembly plan specifies the execution of the connections, necessary stabilising and bracing measures, tolerances for the execution, safety measures, requirements for weather protection, any sequence of work and other necessary measures in the assembly phase.

Make the assembly plan available on site.

The assembly may be initiated when the previous works are adequately controlled.

Where relevant, perform a control of the assembly before the further works make control difficult.

Make available assembly plans showing element positions, as well as the reach and capacity of the building cranes. When required, access and working scaffolds should appear in the assembly plan.

Carry out measures to ensure the stability of the supports during construction and to reduce the risk of damage to such supports, with special considerations to achieving a safe installation and avoiding accidents and injuries. For beams and floors, minimum edge distance and support width should be specified in a way that facilitates both installation and control.

Ensure assembly of prefabricated elements and modules conform with the assembly plan, the Execution Specification and the order of work in the work program.

During assembly, control the positions of the prefabricated elements, the accuracy of the geometry and the position of the supports, the joints and the total formation of the structure and make any necessary adjustments.

The completed structure must be within the maximum permissible deviations to avoid detrimental effects in terms of:

- load-bearing capacity and stability in transient and service stages
- service performance during the use of the structure
- placing compatibility for the erection of the structure and its non-structural components.

The compound construction tolerance (the box principle) requires that all points of the structure are within the specified theoretical position with a margin in any direction corresponding to the permitted deviation.

Specify Tolerance Classes for the assembly in the Execution Specification, and when selecting tolerance classes, consider the production tolerances and requirements for the finished surface.

Production tolerances for timber components, elements and modules must be in accordance with the permitted deviations specified in the relevant product standard. For prefabricated elements and modules where production tolerances are not given in a product standard, the values for permitted deviation for Tolerance Class 3 given in Table 5 and 7 may be used as production tolerances.

Assembly tolerances must comply with permitted deviations given in Tables 5, 6 and 7. Unless otherwise stated in the Execution Specification, Tolerance Class 1 applies. During the design phase, consider the assembly tolerances.

This Guideline does not specify requirements for the combination of geometrical tolerances and deformations in the structure due to the applied actions. Permitted deviations apply to the situation prior to the occurrence of deformations due to loads and time-dependent deflections, unless otherwise stated in the Execution Specification.

Long-term effects resulting from normal creep, dimensional changes due to moisture, assumed use or settings, should have been taken into consideration during the design and are not covered by the permitted deviations.

If the requirements of this Guidelines are to be applied beyond the conditions defined above, this must be described in the Execution Specification.

Specify any requirements for special tolerances in the Execution Specification and give the following information:

- any amendments to the permitted deviations given in this Guideline
- any further type of deviations to be controlled, together with defined parameters and permitted values
- whether these special tolerances apply to all relevant components or to particular components which are identified.

If using more lenient permitted deviations than Tolerance Class 1, it should be documented that the design assumptions are met for the completed structure.

If a certain geometrical deviation is covered by different requirements, the strictest requirement to permitted deviation applies. Where components are incorporated in a structure, requirements for permitted geometrical deviations for such components will be subordinate to the requirements of the completed structure.

The specified permitted deviations of beamlines and floor levels also apply to other horizontal and sloping structural parts.

Permitted deviations for the support width of prefabricated beams and floors are not specified in this standard. They should be specified in the assembly plan or as technical information for the prefabricated element.

Permitted deviations for surfaces between components where forces are intended to be transmitted by full contact bearing between the surfaces are not specified in this Guideline. State any requirements for such surfaces in the Execution Specification.

The requirements for tolerances refer to the dimensions given in the Execution Specification. Position tolerances in plane refer to the construction axis grid (or secondary lines) in the plane. Position tolerances in height refer to the construction axis grid in height, e.g. a transferred height benchmark. Specify any requirements for the construction axis grid in the Execution Specification.

Handle deviations from the specified tolerance range in accordance with 4.4.3. Deviations that have no significant consequence on the performance of the completed structure may, by agreement, be ignored.

Tolerance Class 1

- Normal tolerances for the primary structure.
- Primary structures where a secondary structure with more stringent requirements for permitted deviations is built on site allowing adaptation, for example filling in timber framework and joists.
- Structures or structural parts where there are no strict requirements for the properties in use, for example roof structures, garages, warehouses and agricultural buildings.
- Secondary structures without strict requirements for surface deviations, such as floors in agricultural buildings, warehouses and garages.

Tolerance Class 2

- Load-bearing elements.
- Primary structures to be adapted to prefabricated elements and modules, such as wall elements.
- Normal tolerances for secondary structures.

Tolerance Class 3

- When strict requirements are necessary.

Table 5: Geometrical assembly tolerances for structural components such as walls, columns, beams and roof trusses/rafters.

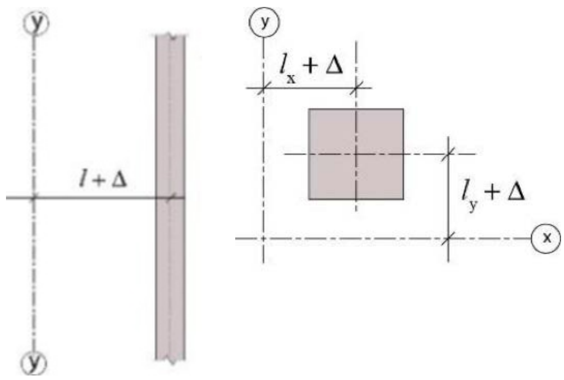
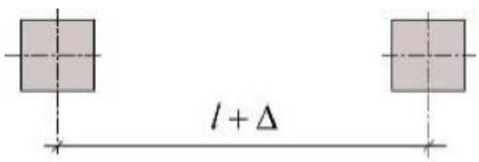
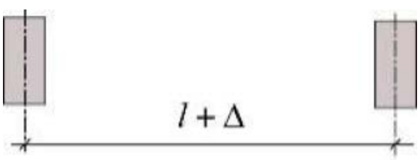
Type of deviation	Description	Permitted deviation		
		Tolerance Class 1	Tolerance Class 2	Tolerance Class 3
Position in plane relative to secondary lines		± 15 mm	± 10 mm	± 5 mm
Distance between individual beams, distance between individual columns and walls	 - columns:  - beams:	± 30 mm	± 20 mm	± 10 mm

Table 5 (continued): Geometrical assembly tolerances for structural components such as walls, columns, beams and roof trusses/rafters.

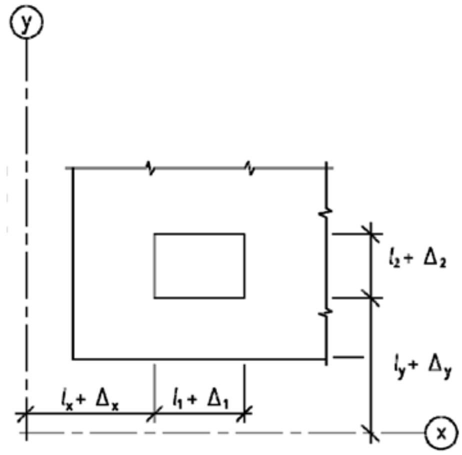
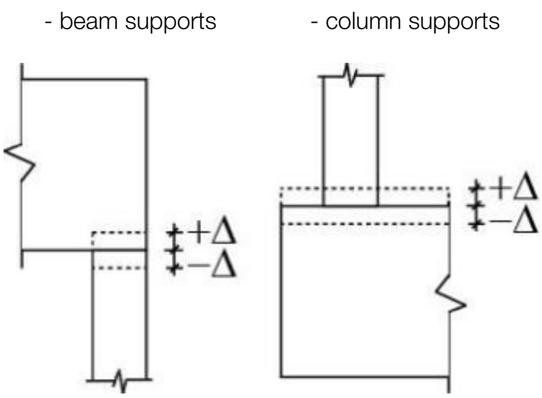
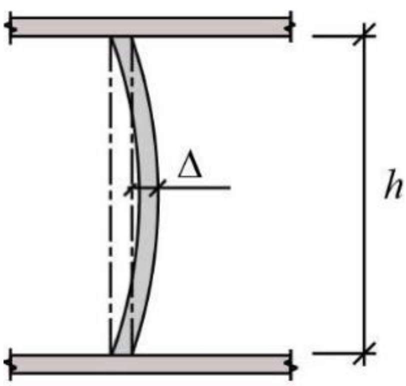
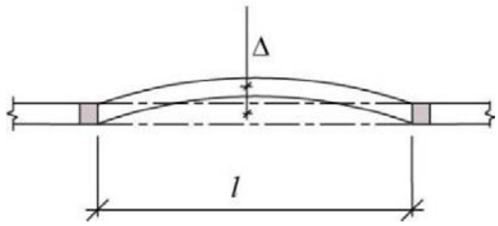
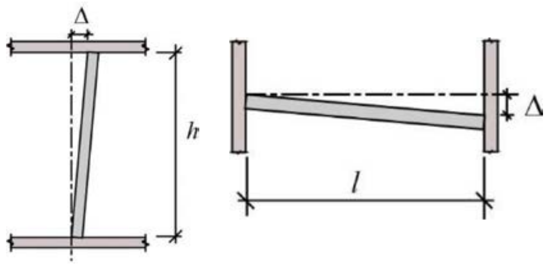
Type of deviation	Description	Permitted deviation		
		Tolerance Class 1	**Tolerance Class 2**	**Tolerance Class 3**
Distance between beams in frames, distance between roof trusses, distance between studs		± 15 mm	± 10 mm	± 5 mm
Dimensions of recesses for doors, windows or stairs etc. (Δ1). Position of recesses for doors, windows and stairs etc. (Δ2) [a]		± 15 mm	± 10 mm	± 5 mm
Vertical position of the support of beams, columns or roof trusses/rafters		± 15 mm	± 10 mm	± 5 mm
Curvature/ straightness of walls and columns		Minimum of ± 20 mm and 2 ‰ /3,3 ‰ of the height for glulam and solid timber, respectively	Minimum of ± 10 mm and 2 ‰ /3,3 ‰ of the height for glulam and solid timber, respectively	Minimum of ± 5 mm and 2 ‰ /3,3 ‰ of the height for glulam and solid timber, respectively

Table 5 (continued): Geometrical assembly tolerances for structural components such as walls, columns, beams and roof trusses/rafters.

Type of deviation	Description	Permitted deviation		
		Tolerance Class 1	Tolerance Class 2	Tolerance Class 3
Horizontal curvature/ straightness of beams or top/ bottom girders	 l = length between braced points	Minimum of ± 20 mm and 2 ‰ /3,3 ‰ of the length for glulam and solid timber, respectively	Minimum of ± 10 mm and 2 ‰ /3,3 ‰ of the length for glulam and solid timber, respectively	Minimum of ± 3 mm and 2 ‰ /3,3 ‰ of the length for glulam and solid timber, respectively
Total inclination from the designed plane for walls, columns, beams and floors.		Minimum of ± 15 mm and 5 ‰ of the height/ length	Minimum of ± 10 mm and 3 ‰ of the height/ length	Minimum of ± 5 mm and 1,5 ‰ of the height/ length

[a] The requirements in the table do not apply when the manufacturer of doors and windows specifies other tolerances.

Table 6: Type of control and documentation of control.

Type of deviation	Description	Permitted deviation
Connections	Gravity centre of connections	± 10 mm
	Position in plane for fasteners	± 10 mm
	Distance between fasteners	± 10 mm
	Inclination of fasteners	± 5°
	Edge distance	-5 / +10 mm
	Side distance	-5 / +10 mm

Table 7: Other geometrical assembly tolerances.

Type of deviation	Description	Permitted deviation		
		Tolerance Class 1	Tolerance Class 2	Tolerance Class 3
Compound construction tolerance (the box principle)	See other requirements in clause 8	± 50 mm	± 30 mm	± 15 mm
Joint between structural parts or elements	Discontinuities at laps etc.	± 10 mm	± 6 mm	± 3 mm
	The thickness of joints etc. Deviation from nominal value	± 10 mm	± 6 mm	± 3 mm