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Mid-rise Timber Building Structural Engineering

Technical Design Guide issued by Forest and Wood Products Australia

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Preface

In Australia, and internationally, there is a renaissance in the use of timber construction outside the traditional single storey and low-rise (up to three storeys) residential and multi-residential construction markets. New engineered wood products and systems, and changes to the National Construction Code (NCC) deemed-to-satisfy requirements, allow taller timber constructions to be used cost effectively over all building classes up to 25 metres effective height (mid-rise); or more (high-rise) using performance solution compliance.

Design teams are increasingly looking at timber construction options for mid-rise buildings and are rapidly learning about timber-based system optimisation and specification. One of the most important professional groups to timber's success in a project is the structural engineer. The Mid-rise Timber Building Structural Engineering Design Guide aims to provide the latest state-of-the-art design information for structural engineers to allow them to confidently undertake a mid-rise timber building design.

Acknowledgements

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Alastair Woodard
WoodSolutions

Design Process and Guide Layout

Typical structural design process for a mid-rise timber building

A typical process for design of a mid-rise timber building, from a structural engineering perspective is shown below. Design is generally an iterative process, rather than linear process shown here, however the following serves to spell out the general design required.

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Phase 1: Preliminary Design

- **Step 1**: Overall building layout and performance considerations
- **Step 2**: Preliminary structural design – including Early Supplier Involvement (ESI)

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Phase 2: Detailed Design

- **Step 3**: Vertical load – Roof and floor 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 engineering details for consideration
- **Step 9**: Engineering drawings and documentation for certification

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Phase 3: Fabrication & Assembly

- **Step 10**: Engineered timber systems fabrication (shop drawing review)
- **Step 11**: On-site construction assembly supervision – certification

The following pages provide a more detailed design summary flowchart, setting out these design steps in greater detail.

References are also provided to the chapters and sections in this design guide where relevant topic information is presented in greater detail.

Some additional detail is provided in the 'preliminary design phase' section to highlight for structural engineers the type of acoustic and fire-related discussions needed with other consultants during the initial conceptual phase. Wood-based building systems differ from those based on other materials and are but one component of an element system that also includes fire- and acoustic-related products.

Phase 1 Preliminary Design

Step 1: Building, layout and performance considerations

Consideration of overall building design and layout
Refer to Section 1.3

The architect identifies the building area functions, for example, sole occupancy units (SOU), common areas, corridors, stairs for access and egress, and the use category for any activities on the roof of the building. At this stage, load paths should be investigated and where misaligned load paths exist, they should if possible be removed.

Determination of Performance Requirements
Refer to Section 1.5

The client, architect and design team (including specialists such as acoustic consultants and fire engineers) determine the performance levels to be met for:

- fire resistance levels (FRL) from the NCC (minimum)
- sound and acoustics from the NCC (or more stringent market requirements)
- weatherproofing, energy efficiency and envelope performance from the NCC (or more stringent client requirements)
- material environmental specification performance (e.g. Green Star).

Fire performance requirements
Refer to Section 1.6.1

The architect (and fire engineers and BCA consultant) determine the building elements that require fire ratings (to form fire compartments) – floor/ceilings, SOU boundary walls. If load-bearing walls are used within an SOU they need to be fire rated (rarely acoustically rated). All DTS fire-rated elements are protected with layer/s of fire rated linings. Engineers need to consider this dead load impact on the design. *Note: Under the NCC Performance requirements, mass-timber elements could be designed for charring for fire protection.*

Acoustic performance requirements
Refer to Section 1.6.2

The architect and engineer determine the possible wall and floor/ceiling systems required to achieve acoustic performance as well as structural performance and buildability. The NCC provides minimum requirements, but clients will often request higher performance, particularly to achieve 'impact' noise requirements for apartment living). Walls between SOUs are likely to require double leaf 'discontinuous' systems to improve acoustic performance and reduce sound flanking transmission across walls. This influences the structural layout approach used, and the process to achieve lateral stability and diaphragm action.

Step 2: Preliminary structural design and early contractor involvement
Refer to Section 2.2 & 2.3

Based on early architectural planning drawings, information for preliminary costings is developed by the Engineer. Usually this involves providing preliminary advice on: the structural approach used (lightweight framing, mass-panel, post & beam, or optimised), structural element layout (floors, walls, cores, bracing walls, transfer structures, etc) and initial member sizing information**. Required loadings: dead loads, live loads, wind loads, and earthquake loads will be based on the advice from Step 1. It is also important at this point to consider: the construction sequence; limits of transportation, lifting and manual handling; site access and restrictions; temporary bracing and propping required; building services and voids required.

** For preliminary assessment, engineers may utilise *WoodSolutions Technical Design Guide #46 Wood Construction Systems* (design charts or span-to-depth ratios) or seek early advice from prefabricated timber system suppliers (ESI).

Phase 2 Detailed Design

Step 3: Vertical load – Roof and floor design

Roof Design
Refer to Chapter 3

- Confirm roof function and loadings: roof social common area, roof garden, plant/machinery areas, non-trafficable roof, etc.
- Design roof members for shear, bending, deflection, etc.
- Design connections, including tie-down.

Floor Design (generally, the same for all storeys)
Refer to Chapter 4

- Confirm fire and acoustic requirements for loading (possible multiple layers of fire-rated ceiling lining, floor acoustic overlay topping).
- Design floor members for strength and serviceability: bending, deflection, shear, vibration (likely to be critical factor), etc.
- Where off-site prefabricated floor cassette systems are to be used, consider discussing with prefabricator nailplate supplier technical staff.
- When this step is completed, the dead loads for the building can be refined and the analysis updated before the design of the wall elements.

Step 4: Vertical load – Wall design

Wall Design
Refer to Chapter 5

- Confirm fire and acoustic requirements for wall (possible multiple layers of fire-rated wall lining will influence dead load; and wall type required: discontinuous double leaf wall/single leaf wall will influence structural approach).
- Determine the floor to wall connection approach: platform framing (direct bearing on wall), semi-balloon framing (floor fixed to wall ledger – adds a design eccentricity).
- Design wall element members for compression, combined actions including out-of-plane bending as required, etc.
- Detail wall connections with floor.
- An iteration in design may be required if wall member sizes become costly or too large. Floor joist spans can be reduced to allow more economical solution. Conduct Step 3 again with smaller floor spans.

Step 5: Vertical movement design

Building Shortening
Refer to Sections 6.1, 6.2, 6.3

- Determine vertical movement for all floor and wall elements in vertical load path due to:
  – shrinkage of timber
  – parallel to grain deformation and creep
  – perpendicular to grain member crushing
  – settlement or embedment of joints
  – dimensional tolerances
  – support structure movement
  – differential movement.
- If vertical shortening is incompatible with proposed finishes, interfaces or cladding details, reconsider the wall system configuration.
- When this step is completed, the dead loads for the building can be refined and the analysis updated.

Step 6: Lateral load – Stability design

Stability Design ***Refer to Sections 7.1 to 7.7***

- Determine design loads for wind loads, and seismic loads.
- Determine distribution of horizontal loads to structure (stiffness assumptions required).
- Design resisting elements for: deflection (stiffness required), vibration, tension, compression, shear, torsion and combined actions, for both the critical tensile and compression cases, including:
  - core (show transfer of shear flow through core, proving composite action)
  - braced walls (vertical design loads in braced walls may increase due to horizontal load resistance)
  - horizontal floor diaphragms.
- Design all necessary horizontal stability connections:
  - floor diaphragm
  - floor to core connections (drag strips for example)
  - vertical tie-down rods (resisting global overturning)
  - timber building to concrete podium (showing transfer of loads to foundations).
- Undertake a computer-based structural analysis check of the building using appropriate FEM software to estimate overall deflections and performance. *Stability model can be completed using finite element modelling (FEM) software like Etabs, Microstran or Spacegass. Verification using hand computations will accompany the computer outputs.*

Step 7: Check robustness ***Refer to Chapter 8***

- Check structure for NCC Robustness requirements and modify details, increase connection performance or respective member sizing if required.

Step 8: Other details for consideration

- Verify the specification of the timber is appropriate given the use and hazard (see *WoodSolutions Technical Design Guide #05 – Timber service life design*).
- Check weatherproofing requirements to protect structural elements and ensure detailing is appropriate, particularly differential movement.
- Confirm that the risk of exposure of the structure to interstitial condensation is acceptable.
- Propose appropriate maintenance schedules and ensure that maintenance points are accessible.
- Check acoustic performance possibly affected by structural approach (ie flanking sound bridges).
- Check fire performance, design members for fire load case.
- Undertake foundation design (or pass on loading details to another consultant).

Step 9: Engineering drawings and documentation for certification

- Prepare all relevant engineering drawings/models.
- Prepare all relevant engineering specification information.
- Provide all required supporting design/fabricate/construct/cost information from timber system suppliers (if applicable).
- Prepare appropriate brief for checking engineer (if required).

Phase 3 Detailed Design

Step 10: Engineered timber systems fabrication

- Check and sign-off on shop drawings from timber system prefabrication suppliers (scope dependent).

Step 11: On-site construction assembly supervision

- Monitor on-site assembly (construction) process (if required).
- Sign-off (certify) correct installation (if required).

Aim and scope

This Mid-rise Timber Building Structural Engineering Design Guide aims to provide specific design criteria, assumptions and guidelines to assist structural engineers to make judgements and perform structural calculations and checks as part of the design of mid-rise timber buildings up to eight storeys in height.

The guide presents both 'whole-of-building' concepts and structural design considerations independent of structural system, and also specific structural element design calculation considerations. Appendices 1 and 2 provide worked examples for the design of the *WoodSolutions model apartment building* (seven storeys, timber over a concrete podium) for both:

- *A1: Timber-framed construction*, which includes lightweight timber elements such as: stud framing with wall plates, roof trusses, floor trusses/joists
- *A2: CLT Mass-panel timber construction*, which includes CLT floor plates, wall panels and lateral stability resisting elements.

Guide limitations and supporting information

As each individual project is unique, there is no one solution that gives the best outcome for the design of every mid-rise timber building. This guide does not seek to present the 'right design' but rather to illustrate the principles for applying the Australian Standards and other design documents relevant to mid-rise timber construction. Designers can utilise this knowledge to find a balance between structural efficiency, fire resistance, acoustic performance, ease of fabrication, constructability and cost in order to optimise the overall project delivery.

Generic *timber-framed* and *mass-timber* details are provided as examples only. Before applying them, designers should check that they are appropriate for use in their particular project. Where the guide refers to product information from manufacturers, designers should ensure the information is still current.

It is assumed that this guide is read alongside other reference information such as:

- National Construction Code (NCC),
- Australian Standards
- manufacturer's design literature for specific products
- the suite of WoodSolutions Technical Design Guides The relevant guides are listed inside the back cover and are available at www.woodsolutions.com.au.

The WoodSolutions Mid-Rise Advisory Program staff can provide further design support and advice, see the woodsolutions.com.au website for contact details.

1 Mid-Rise Timber Building Design Methodology

1.1 Introduction

The 2016 National Construction Code (NCC) introduced Deemed-to-Satisfy (DTS) provisions for '*fire protected timber*', allowing timber building construction in Class 2 (apartments), Class 3 (hotels) and Class 5 (office) buildings up to an effective height of 25 m (referred to in this guide as '*mid-rise*' timber construction). The 2019 NCC expanded these provisions to all Building Classes, further increasing the options to use DTS timber construction systems for schools, aged-care buildings and every other typology.

These NCC changes have opened up substantial opportunities for the cost-effective delivery of mid-rise timber buildings and, as such, generated significant interest from building professionals in optimised timber design.

This design guide has been developed to assist structural engineers with appropriate information to undertake the structural design of mid-rise timber buildings up to eight storeys from concept development through to specific detailed design considerations.

A number of structural timber system options are available for mid-rise timber buildings, providing a variety of ways to optimise the needs of a specific project. Figure 1.1 provides an overview of these options.

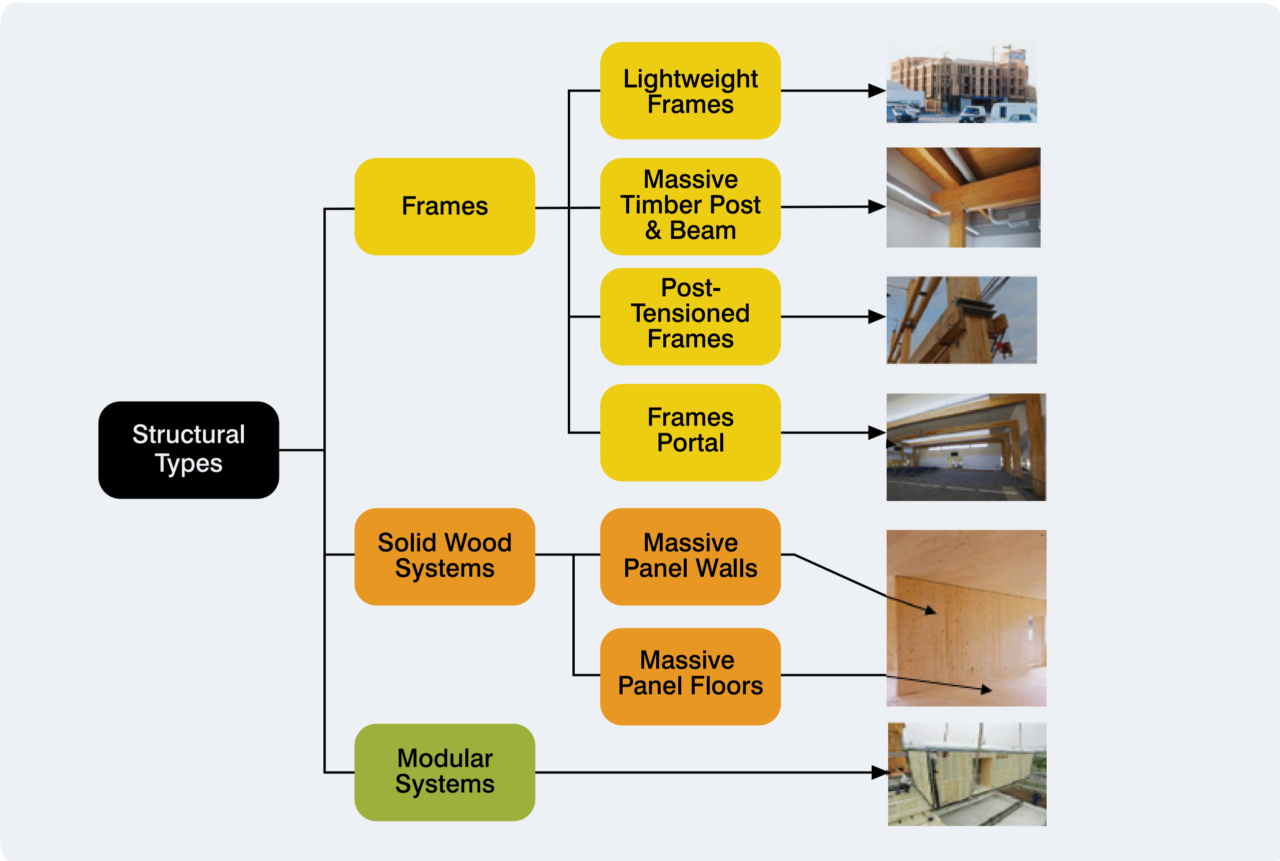


Figure 1.1: Summary of structural timber construction options for mid-rise timber buildings.

Mid-rise timber construction requires a 'systems-based' approach to meet structural, fire and acoustic performance requirements as well as other NCC performance requirements. In this chapter:

Section 1.2 examines the different structural timber products and systems available.

Section 1.3 describes the different building layout considerations that affect the structure.

Section 1.4 discusses design for manufacture and assembly with prefabricated timber systems.

Section 1.5 summarises the key NCC performance requirements that need to be considered.

Section 1.6 examines the fire and acoustics performance requirements in greater detail.

1.2 Timber products and systems for mid-rise timber construction

A wide range of structural timber products and systems are used in mid-rise timber construction for roofs, floors, loadbearing walls, shear walls, cores, lift shafts and stairwells.

Table 1.1 – summarises the range of ***structural sawn and engineered wood products commonly used in mid-rise timber construction***, providing: a description of the products, the applications they are generally used in, the grades and the typical product dimensions available.

One of the major benefits of timber construction over concrete construction is the fact that many of the building elements and systems can be prefabricated off-site under high quality factory conditions to high tolerance levels, then delivered just-in-time to site and assembled very quickly. This vastly improved speed of construction allows buildings to be finished much earlier, providing significant efficiencies and proven cost savings if done correctly.

Table 1.2 – summarises the prefabricated timber systems used in mid-rise timber construction, providing: a description of the prefab-systems and their common mid-rise applications, defining the typical dimensions and spans of these systems, and also who can supply these systems.

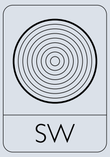
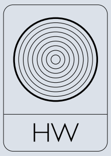
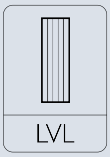
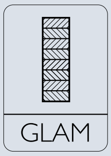
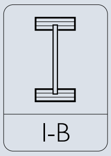
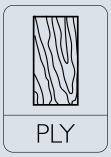
Readers are also referred to *WoodSolutions Technical Design Guide #46 Guide to Wood Construction Systems* which provides considerable additional detail on the timber products/systems available and also some indicative span tables for quick preliminary comparative analysis in the conceptual design phase.

In terms of supply of timber materials and systems:

- Structural sawn softwood (Note: MGP15 has limited availability, check for stock if specifying), sawn hardwood, and EWP's such as: LVL, I-beams, plywood and OSB are readily available nationally from timber merchants.
- Nailplate roof and floor trusses and open wall frames are available from all frame & truss manufacturers – prefabricated cassette floor systems and panelised wall frames are available from specialised frame & truss manufacturers.
- Glued laminated timber (GLT or Glulam) is available from a number of speciality local and international GLT manufacturers and local timber wholesalers and merchants.
- Laminated veneer lumber (LVL) mass-panels are manufactured locally and some imported mass panel product is available (check with wholesalers).
- Cross laminated timber (CLT) is manufactured locally and imported product is also available from a number of international companies.

For 'supplier contact' details visit www.woodsolutions.com.au

Table 1.1: Commonly used structural sawn and engineered wood products in mid-rise timber construction. (see also WoodSolutions Technical Design Guide #46 Guide to Wood Construction Systems).

Product	**Description**	**Common Application**	**Common grades**	**Common typical dimensions**	**Other**
Seasoned softwood 	Structural sawn timber mainly pine species	Wall framing for upper storeys, non-loadbearing walls, truss elements	MGP10, MGP12 (AS 1720.1)	Lengths up to 6 m Depths: 70,90,120,140,190 Thick: 35, 45 mm	Treated available MC <15%
Seasoned hardwood 	Structural sawn timber from hardwood species	Wall framing for middle to lower storeys, high strength wall plates and truss elements	A17, F17, F27 (AS 1720.1)	Lengths up to 6 m Depths: 70,90,120,140,190 Thick: 35, 45 mm	Treated available MC <15%
LVL (Laminated veneer lumber) 	Manufactured by gluing thin veneers with grain parallel to form beams or panels. Cross-banded LVL has a limited number of layers with grain perpendicular to the main grain direction	Wall and floor framing; billets can be glued together for use as panels in cores or floors.	Manufacturers' information	Lengths up to 12 m Panel (billet) width 1.2 m typ 2.5m max available Depths: 95–400 mm typical Thick: 35,45,63,75 mm	Treated available MC <15%
GLT (Glued laminated timber) 	Manufactured by gluing sawn timber laminates with grain parallel to form beams and columns	Beams and columns in post and beam construction	GL grades (AS 1720.1)	Lengths: 12 m stock, 27 m spec. Depths: variable 195–1,000 mm Thick: 65, 85, 115, 135 mm (typ)	Treated available MC <15% Camber possible
I-Beams 	Top and bottom flanges from sawn timber or LVL, glued to webs made from light gauge steel, plywood or OSB	Floor joists and floor cassettes	Manufacturers' information	Lengths: 8.4 m typ, 12.6 m spec. Depths: 200,240,300,360,400 Thick Flange: 45, 51, 63, 90 mm	Treated available
Plywood 	Manufactured by gluing thin veneers with alternate grain directions to form sheets	Bracing panels, flooring	Manufacturers' information	Panel lengths 2.4, 2.7 m Panel width 1.2 m, Thick: 3, 4, 6, 7, 12, 13, 15, 17, 19, 21, 25 mm	Treated available MC <15%
OSB (Oriented strand board) 	Manufactured by gluing and pressing timber flakes to form sheets	Bracing panels, flooring	Manufacturers' information	Panel lengths 1.2 m Panel width: 2,440, 2,745 mm Thick: 9.5, 18.5 mm	

Recommendations: Only use seasoned structural timber in mid-rise buildings as it more dimensionally stable than unseasoned timber. At the preliminary design stage, it may be useful to provide timber sizing in more than one grade or size to provide flexibility to the fabricator for pricing.

Designers should discuss the availability of grades and sizes with suppliers and fabricators before specifying products in the design. The availability of products varies between states and will change over time.

Table 1.2: Prefabricated timber systems used in mid-rise timber construction (see also WoodSolutions Technical Design Guide #46 Guide to Wood Construction Systems).

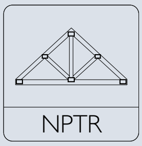
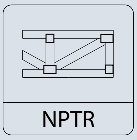
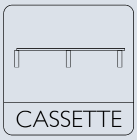
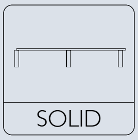
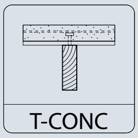
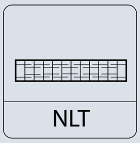
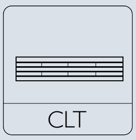
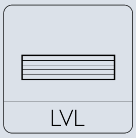
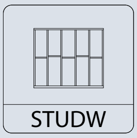
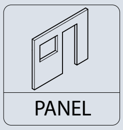
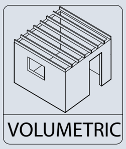
Product	Description	Common mid-rise application	Common dimensions	Typical Spans (2kPa LL)	Source of supply
Nailplate trusses triangular 	Engineered trusses utilising lightweight framing (35, 45 mm thick) and nailplate connectors	Roof systems	Up to around 3 m in depth	25 m+	Frame & truss sector
Nailplate trusses parallel chord 	Engineered trusses utilising lightweight framing (35, 45 mm thick) and nailplate connectors	Floor systems (singularly laid or utilised in floor cassettes)	Typically, up to 12 m long* Depths 150 mm to 550 mm	Fl joists, 450crs, 2 kPaLL 300mm deep: 5.5m 400mm deep: 6.0m	Frame & truss sector
Cassette floor panels 	Prefabricated engineered elements using floor joists or trusses overlain by timber flooring	Floor systems (very quick to install and safe)	Typically, up to 12 m long* 3 m wide* Depths 300-550 mm	Span/depth: 15–18 4-8 m	Frame & truss sector, speciality builders
Timber-timber composite floors 	Prefabricated floor cassettes using a heavy timber floor slab (and/or ceiling) connected compositely to floor joists	Floor Systems	Typically, up to 12 m long* 3 m wide* Joist depth 150–600 mm	Span/depth: 12–20 6-9 m	Frame & truss sector, speciality builders
Timber-concrete composite floors (TCC) 	Composite timber-concrete floor (conc acting in compression, timber in tension), connected by shear studs/keys	Floor Systems	Joist depth 150–600 mm Cast-in-situ or prefab	Span/depth: 12–20 5–10 m	Specialty builders
Nail laminated timber (NLT) 	Sawn timber nailed together to form larger mass panel elements	Floor systems, wall systems, shafts and cores	Typically, up to 12 m long* 3 m wide* 75–300 mm thick	Span/depth: 24–30 4–7m	Frame & truss sector, speciality builders
Cross laminated timber (CLT) 	Mass wood panels made by gluing layers of timber with the grain direction of alternating layers at right angles	Floor systems, wall systems, shafts and cores	Typically, up to 12 m long 3 m wide* 50 mm – 500 mm thickness	Span/depth: 24–30 4–7m	CLT manufacturers, speciality builders

Table 1.2: Continued

Product	Description	Common mid-rise application	Common dimensions	Typical Spans (2kPa LL)	Source of supply
Laminated veneer lumber (LVL) 	Mass wood panels made by peeled veneers with the grain of most veneers running in the same direction	Floor systems, wall systems, shafts and cores	Typically, up to 12 m long* 1.2 m wide billets Thick: 35, 36, 39, 46, 63, 75	Span/depth: 24–30 4-7m	LVL manufacturers, timber wholesalers
Open wall frames 	Prefabricated wall elements assembled by nailing vertical studs between horizontal plates, often panel braced	Wall systems	Variable	-	Frame & truss sector, speciality builders
Panelised wall fames 	Prefabricated wall elements can be lined one side (partially enclosed) or lined both sides (fully enclosed)	Wall systems, cores	Variable	-	Frame & truss sector, speciality builders
Volumetric Modules 	Prefabricated 3-dimensional rooms, that can be assembled as whole buildings or components stacked on one another	Fully finished modules (floors/walls)	13 m long, 4.2 m wide, 3.1 m high*	-	Speciality prefabricators & builders

* Size generally limited by transport restrictions: 12 m long, 3 m wide – no restriction

1.3 Mid-rise building consultant collaboration and layout considerations

There are multiple strategies for determining the layout of a mid-rise timber building, ranging between the following two extremes.

- Option 1 – Start with a structural timber system (or combination) and fit the architecture to that choice, possibly with an early engagement of the supplier(s) to consider the available dimensions and grades as soon as possible within the design brief. This will optimise the result in terms of 'best possible use' of the selected engineered wood products and systems.
- Option 2 – Start with the architecture and apply a structural optimisation approach later on. Although this may produce higher costs, system inefficiencies and more engineering challenges, it may be necessary to ensure compliance with some planning rules or site constraints.

A hybrid of Option 1 and 2 is recommended – an early and iterative collaboration involving all the project team usually provides the optimal solution. When this approach also includes the timber system prefabricator(s), and if known, the builder, it will be particularly beneficial for the structural engineer because it will readily address issues such as:

- *fire and acoustic performance requirements* that will affect the choice of structural approach and the materials and systems used (e.g. depending on the internal architectural occupancy layout, and acoustic performance requirements, these will affect the wall type and overall thickness required)
- *procurement and logistic considerations* on how the building elements can be supplied and assembled on-site will also influence the structural design, (e.g. utilisation of platform floor or roof cassette systems to speed up construction, crane type).

With smart design, a detail might be selected that may simultaneously satisfy a range of requirements: architectural, fire, acoustic, building-services and structural. There is no one single 'correct' solution, but early discussions with each party will help determine the optimal solution.

A number of recent very successful timber projects have also demonstrated the benefits of an early contractor involvement (ECI) process that can significantly de-risk the project with respect to cost, time, quality, constructability and commercial arrangements.

Table 1.3 summarises the typical key considerations for achieving an efficient structural building layout with timber structures.

Table 1.3: Building layout considerations.

Element	Considerations
Building profile	If possible, avoid high, narrow, slender building configurations as they are more challenging for lateral load resistance and can increase wind-induced vibration.
Roofs	Identify mechanical plant areas early (high live load) and the roof function. If it is to be used as a trafficable or green roof particular attention will need to be paid to loading and the waterproofing detailing for long-term performance.
Floor plan	Repeating the same floor plan on each storey will allow load bearing walls to align vertically in the building. This will avoid the need for costly transfer structures. If the floor plan is to be changed best to it be done on the top floor or two (i.e. penthouse); the lower down the building the change happens, the harder it is to deal with. Buildings with 'stacked' type walls are preferred. Arranging similar room uses in adjacent spaces minimises difficulties with sound transmission. Limiting large internal voids like atriums will have a beneficial impact on fire design and robustness.
Floors	Select floor span directions to optimise floor performance. This may be to suit floor cassette use and continuous joists, but generally span direction will be across the shortest room dimension based on position of loadbearing walls. Understand the requirements for mechanical services as these may limit options for floor systems.
Walls	Walls dividing apartments are usually most suitable for loadbearing functions and as shear walls. Internal apartment walls can be loadbearing, but this is less desirable as it limits future internal apartment refurbishments. When DTS provisions are applied, all loadbearing timber walls, either external or dividing occupancies, will require protection using fire-rated linings. Position walls to allow moderately consistent spans for the floors that are supported by them. Big changes in span between adjacent floor panels may give differential displacements at the junctions. Allow sufficiently long sections of load-bearing wall in each direction to maximise potential for bracing elements.
Cores	Core walls are also suitable both as loadbearing walls and as stiff elements to assist in resisting lateral wind and seismic loads. Positioning lift and stairs near the centre of the building will ensure that torsional response of the whole building is minimised for all load directions; or if two cores, position them symmetrically within the building. Coordinate cores carefully with requirements for building services distribution and eventually use the vertical alignments of wet areas to obtain additional cores by joining their walls through the floors (e.g. those supporting the risers).
Services	Look to run services vertically in strategically positioned dedicated vertical service shafts or risers that can be adequately fire protected. Where possible locate the risers around the vertical core for easier management and maintenance.
Basement car park	Layout the column grid to coincide with heavily loaded walls in apartments above as this reduces loads on transfer structures at ground or first storey level.
Concrete ground level slab	Minimise the number of penetrations to provide an effective termite barrier. Try to avoid a 'soft storey' for the ground level if the construction is different at that level.

Mid-rise timber buildings up to eight storeys (25 m effective height) may be constructed in a range of different ways. The optimised solution is likely to have a mix of construction methods.

- For residential mid-rise timber buildings up to six storeys, lightweight timber framed systems are likely to be most cost effective if the floor layout is regular, otherwise they can be a good option for the upper levels.
- For a seven or eight storey building the lower levels and cores may be best constructed from massive timber panels with lightweight timber framed systems above.
- The whole building might be constructed from mass-timber panels to simplify the on-site assembly process or other system design considerations.

Often with apartment projects the ground floor level may also be mixed Class (i.e. including retail). In this instance, concrete is often utilised for a podium structure, above which timber systems are then constructed (see Figure 1.2). The raised podium also provides a separation of the timber structure from the ground, thereby reducing ongoing water related or insect attack durability issues, while the lightweight nature of timber structures above can significantly reduce the costs of the foundations, basement and podium.

With construction in-the-ground, such as basement structures, where high water impact is likely, it is recommended that concrete is used rather than timber.

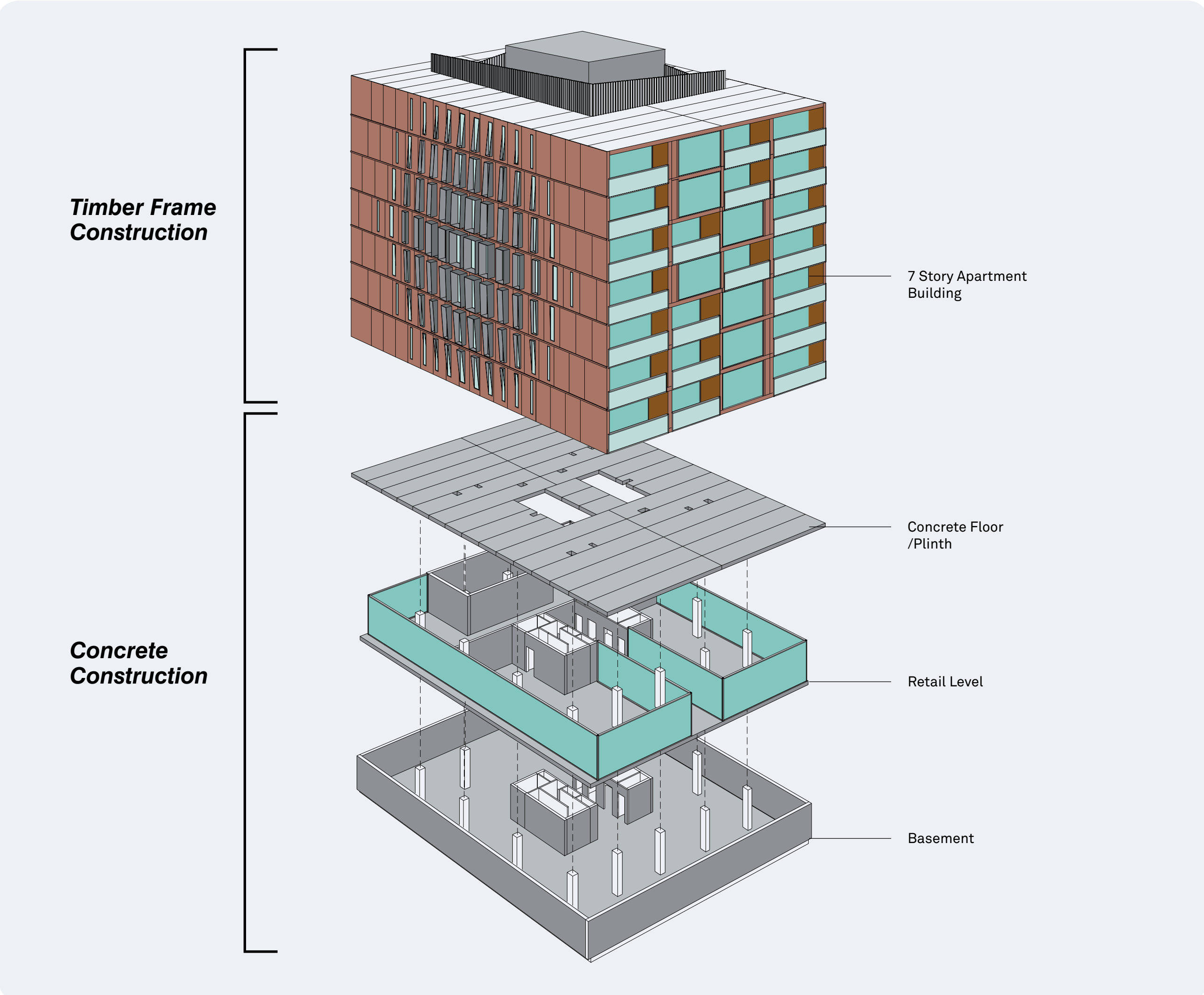


Figure 1.2: Typical timber apartment constructed over a concrete podium.

1.4 Design for Manufacture & Assembly

Design for Manufacture and Assemble (DfMA) is commonly used in the timber sector to describe a process of efficiencies gained through appropriate design of projects (the panelisation process) for both 'manufacturing' (an off-site processing plant) and 'assembly' (on a construction site).

In isolation, Design for Manufacturing refers to making the manufacturing and CNC machining of timber elements as simple as possible, without due regard to how it might be installed on-site. Whereas, Design for Assembly would focus on the erection and construction process without due regard for how long it might take to manufacture in the factory. Design for Manufacture and Assembly considers both manufacturing and on-site erection. When properly coordinated and executed, DfMA ensures that the overall process is as efficient as possible.

A detail requiring complex machining may be expensive to manufacture but might be very simple and fast to install on-site while the same could be true for the opposite. Individual projects will have different drivers and only by understanding these can the optimum solution be arrived at.

For engineers and designers, the design of a mid-rise timber building can require a different mindset to that which they are used to. It involves gaining an understanding of the process which goes into the building being installed on-site. There are different factors that go into the costs associated with a mid-rise timber building based on manufacturing, transport and assembly. The following summarises some of the key points for consideration

Manufacturing:

- *Raw material use – How much timber is in the element? Has a standard size been specified?* Non-standard products increase costs.
- *Manual handling and labour – How much labour in the factory is needed to handle the material?* Lots of small panels typically require more labour to lift than single larger panels.
- *CNC machine time* – The more complex details are the longer they take to cut. This is typically reflected in higher material costs (see Figure 1.3).

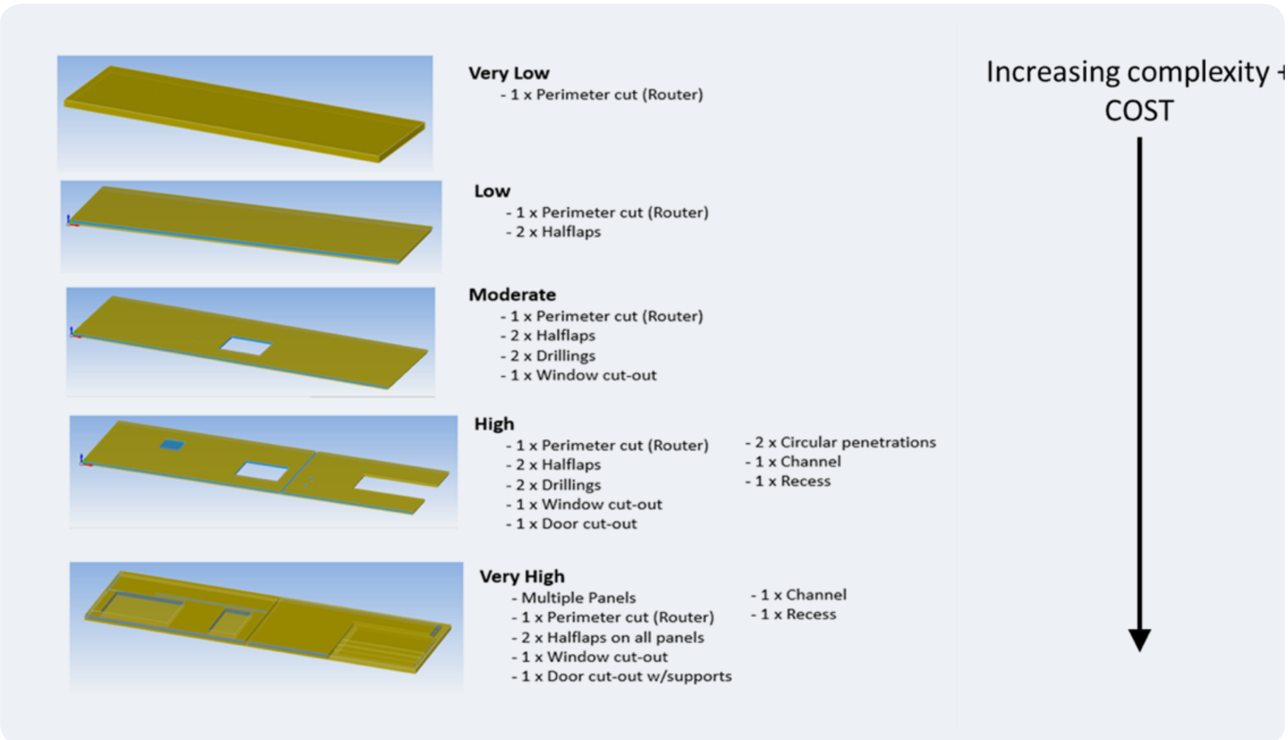


Figure 1.3: Cost impacts of in-factory panel machining.

Transport

- What size trucks can be brought to the site? Larger trucks may mean higher costs but often can be less than spreading across two smaller loads.
- *Do oversized loads need to be considered?* Access can be critical on a great number of projects and factoring this into the design at an early stage will yield the best results.
- *Is there a laydown area for materials are panels installed directly from the truck?* Prefabricated panels need proper and adequate storage and protection on-site.

Assembly

- *What size of panel can be lifted on-site?* Limitations on panel sizes will mean more lifting and more fixings.
- *What is the best way to minimise crane hook time?* This is the single largest factor that will affect construction time and cost.
- *How many fixings are required to be installed?* Over-specification of brackets and fixings will add not only material cost but also large amounts of labour.

While structural engineers are often trained to minimise material as a way to minimise costs, this may not be the best approach for mass timber panel buildings. It is often a case that the larger panels with openings cut into them may well prove to be ultimately more cost effective than a structure made up of lots of smaller panels framing around openings (see Figure 1.4).

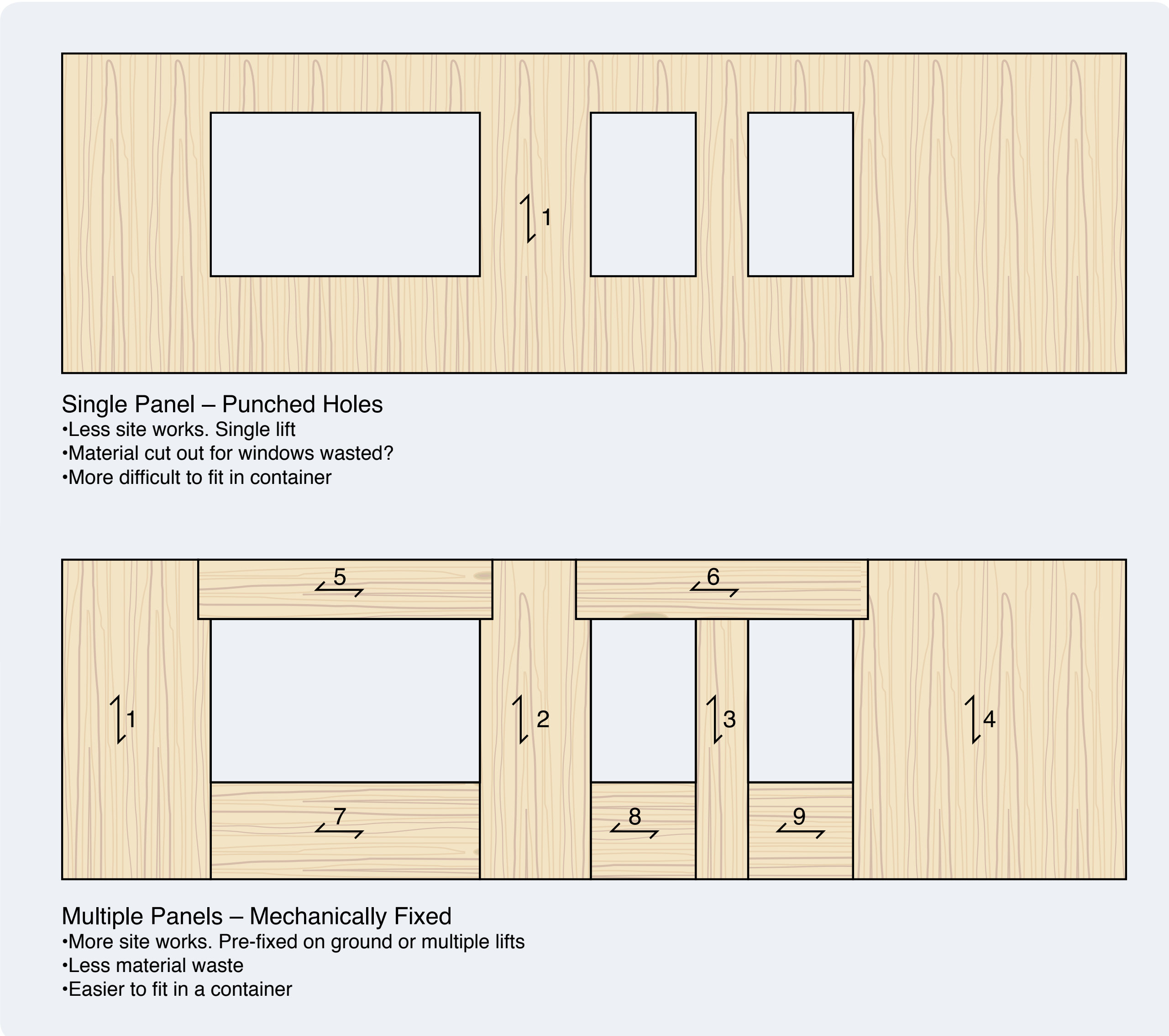


Figure 1.4: Mass timber panel manufacture and assembly considerations.

Best results are obtained by working closely with your prefabricated element provider as early as possible.

1.5 NCC Performance Requirements

The NCC requires that all buildings are designed and constructed to comply with the stated Performance Requirements. The NCC Performance Requirements can be met by using Deemed-to-Satisfy (DTS) provisions or a Performance Solution. Designers often use a combination of DTS and Performance Solution to deliver the most cost-effective overall building result.

Further details on the Performance Requirements and Deemed-to-Satisfy solutions for mid-rise timber buildings are provided in WoodSolutions Technical Design Guide #37 'Mid-rise Timber Buildings'

The NCC has performance requirements for structure, fire resistance, access and egress, services and equipment, health and amenity (including weatherproofing and sound transmission), and energy efficiency. Other Performance Requirements include architectural considerations such as massing, and space planning. It should be noted that customer/user requirements may go beyond the requirements of the NCC, particularly with acoustic performance, which is often specified above minimum NCC requirements.

Mid-rise timber buildings are assembled using 'systems'-based elements so, while this design guide focuses in the main on the structural provisions, there are other performance requirements of critical importance, particularly those for fire, acoustics, robustness and weatherproofing.

1.5.1 Structural performance

NCC Part B1 Structural Provisions sets out the performance requirements for buildings or structures during construction and use, it also defines the design actions which must be considered in service including frequently repeated or extreme loads and actions. The NCC lists AS/NZS 1170 series as DTS actions on the structure and AS/NZS 1720.1 as DTS structural resistance for timber. This design guide provides detailed design explanations with reference to these documents in the following chapters.

1.5.2 Fire performance

Section C of the NCC deals with Performance Requirements concerned with safeguarding people when a building fire occurs. Among the DTS provisions that address fire performance, NCC Clause C1.13 defines the concessions for Fire-Protected Timber. The 2016 NCC limited these provisions to Class 2, 3 and 5 buildings, and the 2019 NCC expanded the Clause C1.13 provisions to all Building Classes. Clause C1.13 (NCC 2019) states:

Fire-protected timber may be used wherever an element is required to be non-combustible, provided

- *a) the building is -*
  - *I. a separate building; or*
  - *II. a part of a building -*
    - *A. which only occupies part of a storey, and is separated from the remaining part by a fire wall; or*
    - *B. which is located above or below a part not containing fire-protected timber and the floor between / the adjoining parts is provided with an FLR not less than that prescribed for a fire wall for the lower storey; and*
- *b) the building has an **effective height of not more than 25 m**; and*
- *c) the building has a **sprinkler system** (other than a FPAA101D or FPAA101H system) throughout complying with Specification E1.5; and*
- *d) any insulation installed in the cavity of the timber building element required to have an FRL is **non-combustible**; and*
- *e) **cavity barriers** are provided in accordance with Specification C1.13.*

Further detail on the principles of fire design and satisfying the fire performance requirements is presented in Section 1.6.1.

1.5.3 Acoustic performance

There are multiple performance NCC requirements that relate to the transmission of airborne sound and impact generated sound between different functional areas in mid-rise timber structures (sole occupancy unit habitable rooms and non-habitable rooms, corridors, stairways, lift and stair shafts, plant rooms, lobby's, etc). Design, specification and detailing for appropriate acoustic performance is a key design function in mid-rise timber buildings to ensure consumer and user satisfaction. Further detail on satisfying the acoustic performance requirements is presented in Section 1.6.2.

1.5.4 Robustness

The NCC also now requires that a building 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. See *WoodSolutions Technical Design Guide #39 Robustness in Structures*.

1.5.5 Weatherproofing

The roof and external wall (including openings around windows and doors) must prevent the penetration of water that could cause unhealthy or dangerous conditions, or loss of amenity for occupants, and undue dampness or deterioration of building elements.

There are currently no DTS provisions in the NCC in relation to the weatherproofing of external walls. It is, however, critically important that water-resisting membranes/systems are used for timber construction and that they are vapour permeable (i.e. allowing timber building components to breathe), but do not permit water to penetrate (water barrier) through to the structural timber building elements. See appropriate membrane manufacturers' literature.

The timber product specification must be appropriate for the hazard to which the timber is exposed in-service in order to achieve the desired design life. See *WoodSolutions Technical Design Guide #05 Timber service life design*. The envelope build-up and details should be such that the hazard levels are as anticipated, for example, ensuring timber is not located with zones of interstitial condensation.

1.5.6 Building services

The strategy for the building services distribution will have influence on the structural solution adopted. Building services risers for vertical distribution and routes for horizontal distribution require coordination with the structure.

1.5.7 Energy efficiency

A building including its services, must have to the degree necessary, features that facilitate the efficient use of energy appropriate to the function and use of the building, and its location.

Refer to:
WoodSolutions Technical Design Guide #22 Thermal performance in timber-framed buildings
WoodSolutions Technical Design Guide #23 Using thermal mass in timber-framed buildings
WoodSolutions Technical Design Guide #24 Thermal performance in timber-framed residential construction

1.6 Principles of fire and acoustic performance with mid-rise timber construction

Structural engineers generally do not give great consideration to fire and acoustic performance, particularly with reinforced concrete building design. For mid-rise timber construction, however, fire and acoustic performance and specification are fundamentally important, as they can have a significant impact on structural element configurations and design loadings. This section provides an overview of some of the key principles that structural engineers need to be familiar with.

Achieving appropriate fire performance is critical to providing acceptable levels of fire safety to 'protect the lives' of building users. Achieving appropriate acoustic performance is similarly important because of its daily impact on the 'quality of life' of the building users, particularly in apartment buildings.

For 'fire-protected' timber elements, the NCC DTS fire provisions effectively require encapsulation of 'all structural timber elements' with appropriate fire-rated linings or claddings (no structural timber can be exposed). 'Non-fire-protected' timber elements, such as mass timber columns and beams, could be exposed if appropriately designed for fire resistance due to charring (refer to *AS1720.4 Timber Structures: Fire resistance for structural adequacy of timber members*). For mass timber exposed walls or floors an NCC Performance Solution will be required.

For acoustic performance, the NCC sets minimum limits for airborne and impact noise, however, stricter market requirements also apply, particularly for impact noise in apartment and hotel buildings. Acoustic performance is achieved through the use of resilient mounts, acoustic mats, mass overlays, separation of structural elements to resist flanking noise and vibrational transfer through members in direct contact with one another.

Figure 1.5 illustrates the practical fire and acoustic performance approaches within a timber framed floor/ceiling and wall connection between four separate sole occupancy units.

The fire performing elements are labelled in red, the acoustic performing elements in blue and the structural elements in black.

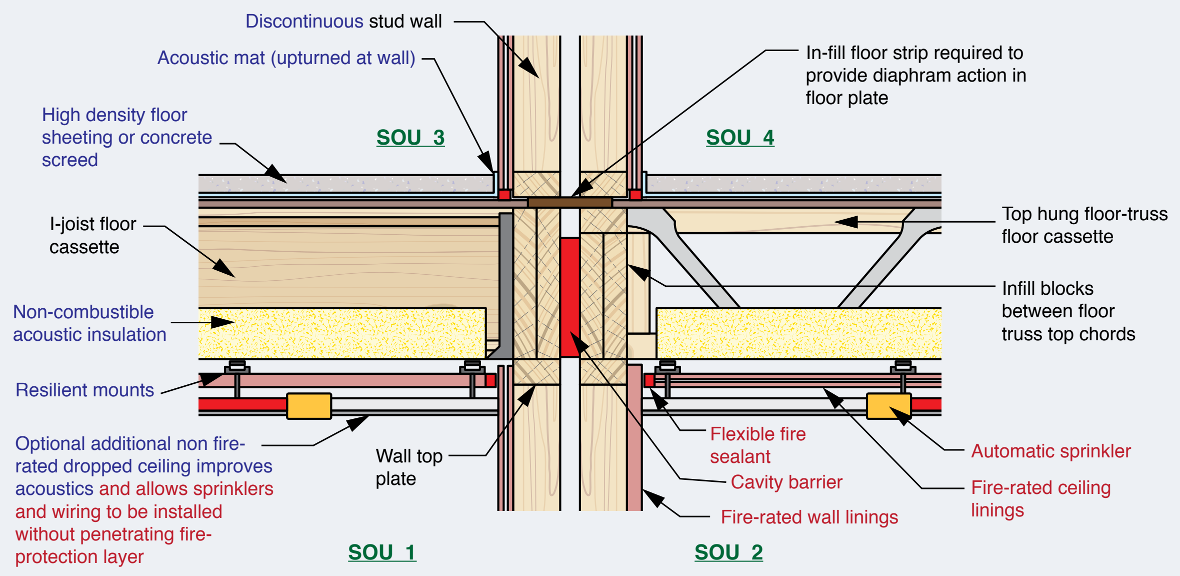


Figure 1.5: Floor/ceiling and wall connection illustrating fire, acoustic and structural elements.

1.6.1 Fire protection principles and NCC fire terminology and requirements

The critical performance requirement in the NCC for fire design is to protect life safety. In the case of mid-rise buildings, the requirement is that the fire must be contained within the fire compartment of origin, or that a fire does not get into the building from an outside source for a period of time so that the occupants of the building can safely escape and fire-fighters can extinguish the fire. For apartment buildings, the SOU is often the fire compartment. In this period the building must remain stable and not collapse. There is not a requirement to ensure business continuity, or a limit on the effort needed to bring the building back to serviceability. These more onerous conditions may be a client requirement.

Fire containment is achieved by enclosing each fire compartment (SOU for apartment buildings) with fire-rated barriers – walls and ceilings. This can be simply illustrated by considering an apartment building to be a series of self-contained fire rated box or ‘compartment’ of boxes as shown in Figure 1.6.

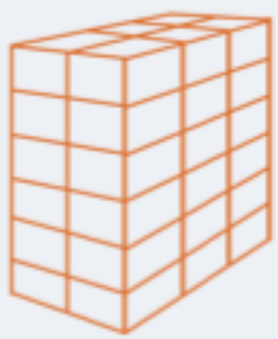


Figure 1.6: Apartment buildings as a stack of SOUs.

NCC fire terminology and requirements

The NCC Section C Part C1 details the fire resistance requirements.

In regard to fire-resisting construction:

> the ‘building class’ in conjunction with the
> building height, expressed in terms of the
> ‘rise in storeys’, is used to determine the
> type of construction’ required (Provision C1.1).

Type of Construction

The NCC defines three types of construction for protecting buildings from fire:

- *Type A* construction – is the most fire-resisting type of construction, limiting the materials that can be used in its construction.
- *Type B* construction – applies similar constraints on the use of timber as Type A, but the FRLs are lower.
- *Type C* construction – is applicable to low-rise buildings. It is the least fire-resisting form of construction and places few fire-related restrictions on the use of structural timber members.

Fire Resistance Levels (FRLs)

The Fire Resistance Level (FRL) is a measure of the period of time that a building delivers satisfactory fire performance. The FRL is expressed as three numbers (time in minutes, e.g. 90/90/90). The numbers represent structural adequacy/integrity/insulation:

- Structural adequacy – time that the system can carry the designated structural loads for the fire limit state
- Integrity – time before flame or hot gases break through the system (wall or ceiling)
- Insulation – time before unacceptable temperatures are reached on the non-fire side of the system.

The NCC defines different FRL requirements within the building. FRLs are required for:

- all fire compartments (in apartments' SOU boundaries)
- any load-bearing elements
- emergency exits such as common corridors and stair wells
- external walls and openings such as windows and doors
- potential fire paths between fire compartments such as risers and penetrations.

FRLs listed for wall or floor systems are always two directional, i.e. the system prevents fire from getting through from both directions. Wall systems are often symmetrical so have the same rating in each direction. However, floor and ceiling systems are not symmetrical. Fire in a fire compartment under the ceiling is more severe than a fire on top of the floor and verification tests always have the fire on the lower (ceiling) side of the system. Floor/ceiling systems are deemed to have the same rating if the fire is on the upper (floor) side of the system.

Resistance to the Incipient Spread of Fire (RISF)

The NCC also has a requirement termed the Resistance to the Incipient Spread of Fire (RISF) which relates to the covering's (e.g. fire-rated plasterboard) ability to insulate voids and timber framed elements so the temperature does not rise to a level that will ignite the timber or spread the fire throughout any concealed spaces. The RISF is expressed in minutes and indicates the time the covering will maintain a temperature below the specified limits. The NCC gives a concession for mass-panel timber construction and therefore a Modified Resistance to the Incipient Spread of Fire (MRISF) value applies. *WoodSolutions Technical Design Guide #37 Mid-rise Timber Buildings* summarises the effective NCC FRL and RISF/MRISF requirements for mid-rise timber buildings.

Achieving FRLs for timber-based systems

NCC DTS Provisions – Specifications

Provision 1.13 of the NCC provides a concession for the use of 'Fire Protected Timber' for certain elements for mid-rise timber buildings, which includes the use of:

- automatic sprinkler suppression system
- fire-protected timber
- cavity barriers
- non-combustible insulation.

The fire-rated wall and ceiling system shown in Figure 1.5 illustrates the use of the DTS method outlined above. Fire protection of the timber members is predominately achieved by encapsulation using fire-resistant linings, e.g. fire-rated plasterboard; thicker linings or multiple layers increase the fire resistance of a system. The NCC DTS solution to meet the RISF for general lightweight framing requires two layers of 13 mm fire-grade plasterboard for walls, and two layers of 16 mm fire-grade plasterboard for ceilings. To meet the MRISF for massive-panel framing one layer of 16 mm fire-grade plasterboard is needed for walls or ceilings. Both of these configurations will meet an FRL of 90/90/90. Other materials can be used but must be demonstrated by testing to reach the required DISF or MRISF. Note: FRLs for different wall and ceiling systems with different product configurations are published in plasterboard manufacturers' design documentation.

Prototype testing to a Standard Fire Test

The Standard Fire Test is described in AS 1530 and involves the placing of a full-scale sample of a wall or ceiling into a furnace. A constant load is applied if a structural resistance level is required. A standard time temperature curve is followed, and the time to failure for structure, integrity and insulation, is recorded to give the FRL. Fire resistance is expressed in 30 minutes intervals e.g. 60/60/60, 90/90/90. RISF values are also determined during the Standard Fire Test.

It is not feasible to conduct a Standard fire test on all configuration of a floor or wall systems. The NCC allows Accredited Testing Laboratories to provide opinions on minor variation to the tested sample, such as, different timber sizes, heights or spans of elements, etc. This form of evidence is the most common method used to substantiate FRLs. Lining manufacturers or timber system providers generally have information to support the FRLs and RISFs of their systems.

Structural Fire Resistance calculation (XX/-/-)

AS 1720.4 includes a method to calculate the capacity of timber members after a given period of time exposure to a fire:

- The density of the timber is used to evaluate a char rate.
- The reduction in cross section after a given period of fire can then be calculated.
- The capacity of the reduced cross section can be evaluated using a duration of load factor appropriate for a short duration load.
- The capacity can be compared with actions calculated for the fire limit state combinations.

This method presently can only be used to determine the structural '*fire resistance level*' of the timber element, it cannot be used to determine an FRL for '*integrity*' or '*insulation*' (Note: AS 1720.4 is presently being revised and will include a provision for FRL's insulation number to be determined. Integrity can only be determined from Standard fire tests). It also does not cover performance for fasteners or glues (connections though can also be protected by ensuring that they are embedded under a depth of timber greater than the depth lost to charring). In order to determine the overall FRL, reference will need to be made to the overall performance of tested systems. The AS 1720.4 method can only be used for timber mass timber columns and beams, not framing protected by fire-rated plasterboard. Also, importantly the method is only suitable for engineered wood products manufactured with phenolic or resorcinol-based glues. It should be noted that glues like polyurethanes, which are used in a great number of EWP's, are excluded from this calculation method due to their behaviour at high temperatures. Designers should seek information from the manufacturer or fabricator as to how their products perform in fire.

1.6.2 Principles of acoustic design and NCC requirements

Part F5 of the NCC is concerned with safeguarding 'occupants from illness or loss of amenity as a result of undue sound being transmitted'. The NCC Performance Requirements for Class 2 and 3 buildings focus on limiting the transmission of both 'airborne' and 'impact'-generated sound via floor and wall building elements (refer WoodSolutions Technical Design Guide #37 Mid-rise Timber Buildings, chapters 2 and 3 for additional detail).

Two effective ways of reducing acoustic impacts in wall and floor/ceiling systems is by 'adding mass', or by 'isolation' of the system elements to reduce the direct transmission of vibration (flanking noise). From a structural design perspective, 'adding mass' also adds dead load, and 'isolation' may impact the form of construction used.

Figure 1.5 illustrates a number of these practical acoustic concepts for an apartment building.

- A double leaf 'discontinuous' wall is used here between SOU's to isolate one SOU from the other. The NCC defines discontinuous wall construction as a wall having a minimum 20 mm cavity between the two separate leaves and there is no mechanical linkage between leaves, except at the periphery, i.e. wall/floor or wall/wall junction. Different wall configurations are discussed further in section 5.1.5.

- In the floor/celling system, resilient supports are used to provide a degree of separation discontinuity, isolating the fire rated ceiling from the floor trusses/joists. *This configuration will typically give over 10 dB of additional airborne and impact performance.*

- With floor/celling systems the next biggest benefit, particularly in resisting impact noise, is achieved in adding 'mass' acoustic elements above the floor. Firstly, an acoustic underlay or mat is used, with the edges turned up at the wall lines. Then above this, mass is added in the form of concrete screed (cheap, but means the use of on-site wet trades), or the use of high-density sheet type products such plywood, fibre cement, etc, (which often can be simply installed off-site when prefabricated floor cassette systems are utilised).See section 4.8 for further information on acoustic toppings for floor systems. Dead load impacts are discussed further in section 2.2.1.

It should be noted that the NCC has no acoustic requirements for external walls. Other authorities such as Local Government may have requirements. Refer to Local Government for more information. For information on the performance of timber framed systems, refer to *WoodSolutions Technical Design Guide #11 Timber frame systems for external noise*.

The NCC DTS provisions set specific requirements for

- 'airborne' sound using the Weighted Sound Reduction Index (R_w) modified by a Spectrum adaption term (C_{tr}), and
- 'impact' sound using the Weighted Normalised Impact Sound Pressure Level ($L_{n,w}$).

Table 1.4: NCC acoustic requirements for walls in apartment buildings.

Space 1	Space 2	$R_w + C_{tr}$ (airborne)	R_w (airborne)	Discontinuous construction
Apartment	Same apartment	No requirement	No requirement	No requirement
Apartment	Different apartment	≥ 50	NA	If wall separates habitable room from a bathroom, toilet, kitchen laundry
Apartment	Plant room, lift shaft or machinery room, public corridor, public lobby	NA	≥ 50	Required in each case

Table 1.5: NCC acoustic requirements for floors in apartment buildings.

Space 1	Space 2	$R_w + C_{tr}$ (airborne)	R_w (airborne)
Apartment	Same apartment	No requirement	No requirement
Apartment	Different apartment	≥50	≤62*
Apartment	Plant room, lift shaft or machinery room, public corridor, public lobby	≥50	≤62*

NOTE:

* The Impact performance of $L_{n,w}$ of not more than 62 was derived from the performance of a bare 200 mm thick concrete slab. The Association of Australasian Acoustical Consultants indicate that the NCC minimum performance values are a bare minimum and should be exceeded to provide an acceptable level of sound separation across floors:

- an $L_{nT,w}$ ≤55 is recommended as the minimum for standard apartments
- an $L_{nT,w}$ ≤50 is recommended as the minimum for luxury residential apartments
- an $L_{nT,w}$ ≤45 is recommended for luxury residential apartments.

Achieving acoustic performance

There are three ways to meet the NCC performance requirements:

1. Deemed-to- Satisfy Provisions – Specification F5.2 – Sound Insulation for Building Elements – solution documented within the NCC.
2. Deemed-to- Satisfy Provisions – Laboratory tested system.
3. Verification Method – field testing.

1. Deemed-to- Satisfy Provisions – Specification F5.2 – Sound Insulation for Building Elements

The NCC contains tables that summarise deemed-to-satisfy building systems with some specific additional requirements. Acceptable forms of construction to meet minimum requirements for wall and floor systems are provided in NCC Specification F5.2, section 2, Table 2.

2. Deemed-to- Satisfy Provisions – Laboratory tested system

This method involves laboratory testing a representative system that is isolated from the test building structure so that only the acoustic performance of the system is measured.

Acoustic laboratory testing can be expensive and is rarely used to develop a system for a single building. Generally, acoustic laboratory testing is undertaken by acoustic system designers and manufacturers of wall and floors systems. Refer to suppliers of these products for further information.

3. Verification Method

This method requires the testing of an acoustic system in-situ. When incorporated into a building, the systems are attached to other structural elements that may cause flanking pathways. To compensate for the inclusion of flanking pathways, the pass criteria for in-situ tests of airborne performance ($D_{nT,w} + C_{tr}$) is 5 dB lower than the requirements for a laboratory tested system specified in Tables 1.3 and 1.4 ($R_w + C_{tr}$). Floor impact noise requirements are the same for the laboratory and the in-situ verification testing. The requirements are:

- Floors
  - Airborne: $D_{nT,w} + C_{tr}$ not less than 45; and
  - Impact: $L_{nT,w}$ not more than 62 (Note: same as laboratory test method requirement)
- Walls
  - Airborne: $D_{nT,w} + C_{tr}$ not less than 45.

Where laboratory testing is used as a means of compliance, field-testing of sample apartments is often used to confirm that the design level of performance has been achieved in the completed building, within the limits of the test scenario.

1.7 Structural Design Standards

The NCC's Section B provides the structural performance requirements required for a building. It lists a number of actions to be considered by a designer to meet the performance requirement.

DTS Solution

To make the task easier, the NCC's DTS provisions list a number of reference Standards that can be used. For design actions, the suite of Standards AS1170 is referenced. Reference Standards for various materials are used to determine structural resistance of various materials and forms of construction.

- *AS/NZS 1170 series – Structural design actions*
- *AS/NZS 1170.0 – General principles*
- *AS/NZS 1170.1 – Part 1: Permanent, imposed and other actions*
- *AS/NZS 1170.2 – Part 2: Wind actions*
- *AS/NZS 1170.3 – Part 3: Snow and ice actions*
- *AS 1170.4 – Part 4: Earthquake actions in Australia.*

For mid-rise timber buildings, the NCC notes two timber design standards: AS1720.1 Design of Timber Structures and AS 1720.5 Nailplate Timber Roof Trusses. The full AS1720 *Timber structures* suite covers:

- *AS 1720.1 – Part 1: Design methods*
- *AS 1720.2 – Part 2: Timber properties*
- *AS 1720.4 – Part 4: Fire resistance for structural adequacy of timber members*
- *AS 1720.5 – Part 5: Nail-plated timber roof trusses.*

Performance solution

For mid-rise timber buildings, and particularly for mass timber solutions, the referenced Standards are not adequate to cover all aspects of a design. Where this occurs, it is also often appropriate for design to use Australian Standards in combination with international design standard information. With CLT design, it is also typical to consider analysis models and methods as set out in Eurocode or Canadian Standard approaches supported by performance data on fasteners from European or American suppliers. In these cases, it is recommended to seek input from CLT manufacturers on how best to achieve this.

This section briefly overviews the different timber design documents and their use with mid-rise timber building design.

1.7.1 AS 1720.1 Timber Structures Part 1: Design methods

AS 1720.1 Timber Structures Part 1: Design methods is the principal Standard used by structural engineers to design timber members and connections in mid-rise timber buildings; this sub-section presents information relevant to some of the key design considerations for a mid-rise timber building using AS 1720.1

***HB 108 Timber Design Handbook* provides background information on behaviour of structural timber and using AS 1720.1 to design or check timber members and connections.**

Deflections

Deflections are calculated using normal elastic deflection formulae. The Modulus of Elasticity (MoE) used is the tabulated average MoE for the product where the consequences of exceeding deflection limits do not lead to damage (e.g. deflection limits for appearance or vibration). However, designers should use the 5%ile MoE where the consequence of exceeding the deflection limit may lead to damage or loss of function (e.g. deflection limits for elements above glass partitions or opening doors or windows. The 5%ile MoE for different timber products is presented at the end of Appendix B in AS 1720.1.

To account for inelastic creep, long term deflections are evaluated using a modification factor (Table 2.4 in AS 1720.1) that is applied to the loads:

- j_2 is used for bending and compression members and for seasoned timber, = 1 for short duration (less than one day) and = 2 for long duration (more than one year) loads.
- J_3 is used for tension members and = 1 for seasoned timber.

j_2 – Duration of load factor for deflections

For deflection, each type of load in a combination of loads is considered separately, and the duration of load for each individual load is the time of a single load event of that type. Examples are given in Table 1.6.

Table 1.6: Duration of load for deflections.

Load type	Example	Duration of a single load event	j_2 (seasoned timber)
Dead loads (permanent actions)	self-weight of structure	years (>1 year)	2
Long-term imposed loads	weight of furniture	years (>1 year)	2
Short term-imposed loads	weight of people	hours (<1 day)	1
Foot fall for vibration analysis	load of a single step	seconds (<1 day)	1
Wind loads	duration of a gust	seconds (< 1 day)	1
Earthquake loads	duration of a movement cycle	seconds (< 1 day)	1

Although AS 1720.1 permits interpolation within Table 2.4 using a logarithmic time scale, it is rarely required.

Once the deflections under each load in a combination have been calculated, the total deflection under the combination is the sum of the individual deflections. For example, to find the total deflection under dead loads (*G*), long term-imposed actions (Q_1), and short term-imposed actions (Q_2), the deflection under 2 x *G*, 2 x Q_1 and 1 x Q_2 are evaluated and added together.

ϕ Capacity factor

The capacity factor is used in the evaluation of the strength of all members and connections. It ensures that structural elements, regardless of the material, have a consistent level of reliability that is meets community expectations.

For timber elements, the capacity factor for member is given in AS 1720.1 in Table 2.1 and for connections in Table 2.2. These tables require designers to classify each element by application category and product type. The application categories of common elements in mid-rise timber framed buildings are summarised in Table 1.7.

Table 1.7: Application category for structural elements in mid-rise timber framed buildings.

Element	Likely tributary area (m^2)	Primary/ secondary	*Category in Table 2.1 or 2.2 AS 1720.1*
Upper storey wall stud	up to 5	Secondary	1
Lower storey wall stud	up to 10	Secondary	1
Floor joist	up to 3	Secondary	1
Floor ledgers	tens	Primary	2 or 3^
Nails in a bracing wall	small*	Secondary	1
Tie-down element for global overturning	hundreds #	Primary	2 or 3^

* Hundreds of nails in a single bracing wall may attract load from tens of square metres. The tributary area of a single nail is small.

Loads from large areas of the building contribute to global overturning forces. Where large elements carry these loads, they are primary elements.

^ Primary elements are classified as category 3 for buildings with a post-disaster function or 2 for other buildings.

Capacity factors for common timber elements in mid-rise buildings include:

- ϕ = 0.9 (Category 1) for MGP10 or MGP12 studs
- ϕ = 0.95 (Category 1) for LVL studs
- ϕ = 0.85 (Category 2) for Glulam transfer beams in apartments (primary members)
- ϕ = 0.95 (Category 1) for LVL floor joists (secondary members).

k_1 – Duration of load factor for strength

Table 2.3 in AS 1720.1 defines duration of load factors for the capacity of members and connections. These factors (k_1) are completely different to those for deflection (j_2).

k_1 is evaluated for a complete load combination. Duration of load for the strength limit state is the total time over the life of the structure for which the element will be loaded at or above the highest load level for the combination (the duration of the shortest load in the combination). Table 1.8 presents some examples from Table G1 in AS 1720.1.

Table 1.8: Duration of load for strength of timber elements (Reproduce Table G1).

Load combination	Load factors	Cumulative Duration	*Duration of load factor k_1*	
			Members	Connections
Permanent action (dead load only)	1.35 G	Years	0.57	0.57
Permanent and short term-imposed actions	1.2 G + 1.5 Q Construction loads Non-trafficable roof loads Floor loads	Days Days Months	0.94 0.94 0.8	0.77 0.77 0.69
Permanent and long term-imposed actions	1.2 G + 1.5 $\psi_l\, Q$	Years	0.57	0.57
Permanent, wind and imposed actions	1.2 G + W_u + $\psi_c\, Q$	Seconds	1.00	1.14
Permanent and wind action reversal	0.9 G + W_u	Seconds	1.00	1.14
Permanent, earthquake and imposed actions	G + E_u +$\psi_c\, Q$	Seconds	1.00	1.14
Fire*	G + $\psi_l\, Q$	Hours	0.97	0.86

* The recommended duration of load for fire in AS1720.1 conflicts with the recommendation in AS1720.4. AS1720.4 duration of load is correct and is a 'normative' requirement unlike AS1702.1 information which is just "informative".

k_4 – Moisture condition factor

k_4 = 1 for seasoned timber used indoors or in sheltered outdoor areas in most parts of Australia. (Refer to Cl. 2.4.2 in AS 1720.1)

k_6 – Temperature factor

k_6 = 1 for timber used in mid-rise timber buildings in most parts of Australia. (Refer to Cl. 2.4.3 in AS 1720.1)

AS 1720.1 defines k_6 as 0.9 for seasoned timber within a zone illustrated in Figure 1.7.

Even in this zone, the temperature inside the building is unlikely to be high enough to reduce k_6 for timber used in walls and floors. However, within this zone, k_6 for timber in the roof space = 0.9.

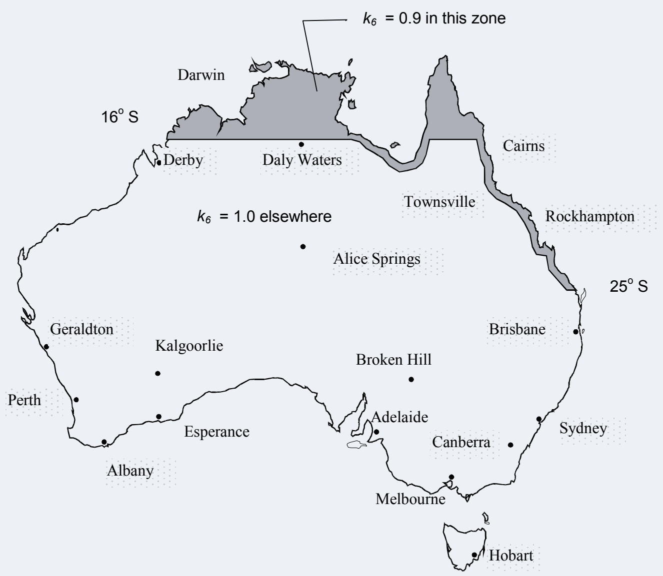


Figure 1.7: k_6 for seasoned timber by location in Australia.

Bending capacity

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.

$$M_d = \varphi k_1 k_4 k_6 k_9 k_{12} f'_b Z \tag{1.1}$$

(Refer to Cl. 3.2.1.1 in AS 1720.1)

k_9 – Strength sharing factor

k_9 = 1 for any engineered wood products such as LVL, I-beams or Glulam members.

k_9 can be evaluated using Cl. 2.4.5 in AS 1720.1 for any sawn timber floor joists or for bending wind actions on sawn timber studs.

k_{12} – Stability factor

For many applications in mid-rise buildings, k_{12} = 1 as:

- floor joists are laterally restrained on the compression edge by flooring; and
- external wall studs subjected to bending wind loads are laterally restrained by linings or battens.

Refer to Clauses 3.3.3 and 3.2.4 in AS 1720.1 for the method of checking the k12 value. Lay is the distance between connectors for flooring or linings.

Compression capacity

$$N_{d,c} = \phi\, k_1\, k_4\, k_6\, k_{12}\, f'_c\, A_c \tag{1.2}$$

(Refer to Cl. 3.3.1.1 in AS 1720.1)

k_{12} – Stability factor

The k_{12} value to be used in calculating the capacity of studs is given in Section 4.3.

(Refer to Cl. 3.3.3 in AS 1720.1)

Combined actions

Studs are subject to combined bending and compression and their capacity to resist both actions must be evaluated, as detailed in Section 5.4.

(Refer to Cl. 3.5.1 in AS 1720.1)

Connection capacity

Common connections used in mid-rise timber buildings include:

- nails
- screws
- bolts
- straps
- framing anchors
- metal angles.

Australian Standards or manufacturers provide the capacities of some connections (see Table 1.9). In some cases, the connection capacity must be calculated using the capacity of all of the fasteners in the connection.

Table 1.9: Common fasteners used in mid-rise timber buildings.

Fastener	Application	Connection Type	*Resource*
Nails	Bracing walls	Type 1 – shear	AS 1720.1, EC5, CSA–O86
	Framing anchor/strap	Type 1 – shear	AS 1720.1, AS 1684
	Nominal connections	Type 1 – shear Type 2 – withdrawal	AS 1684
Screws	Roof or wall battens	Type 2 – withdrawal	AS 1720.1
	CLT	Type 1 – shear Type 2 – withdrawal	Manufacturer
	Cassette-to-cassette	Type 1 – shear Type 2 – withdrawal	Manufacturer
	Reinforcement		Manufacturer or EC5
Bolts	Tie-downs of bracing walls	Type 2 – tension	AS 1720.1
	Shear transfer in bracing walls	Type 1 – shear	AS 1720.1

Connection stiffness will need to be estimated to determine load distribution through stability systems. Stiffness of connections is influenced by the fasteners used. Manufacturers can provide stiffness data for proprietary products. For standard fasteners stiffness can be estimated using AS 1720.1 Appendix C, or Eurocode 5 (EC5) Section 7.1. Stiffness values are an estimate. Engineers should consider sensitivity testing stiffness in order to determine onerous cases for load distribution where effected by stiffness. For example, modelling global stability systems with a stiff core and flexible bracing to determine stability loads attracted by the core, and vice-a-versa for the bracing. In such a case it could be appropriate to consider a range of stiffness of say 0.5 K to 2 K, where K is the component stiffness determined.

1.7.2 AS 1720.3 Timber Structures Part 3: Design criteria for timber-framed residential buildings

AS 1720.3 Design criteria for timber-framed residential buildings sets out the design methods, assumptions and other criteria suitable for the design of timber-framed houses. It provides interpretation of AS 1720.1 for framing. AS 1720.3 cannot be used to design mid-rise timber buildings, but some information from the standard has been used to provide advice in this Design Guide.

1.7.3 AS 1720.4 Timber Structures Part 4: Fire resistance for structural adequacy of timber members

The objective of this Standard is to provide a computational method for determining the fire resistance for structural adequacy of solid, plywood, laminated veneer lumber (LVL), and glued-laminated structural timber members as an alternative to the test method specified in AS 1530.4.

1.7.4 AS 1720.5 Nailplated timber roof trusses

The Standard is applicable to residential, light commercial and non-habitable structure that have a maximum roof pitch of 45 degrees, maximum truss span of 16 m and maximum truss spacing of 1,200 mm. As most timber trussed roofs are designed by supply companies from nailplate provider software, this standard is rarely used and is not discussed in this guide.

1.7.5 AS 1684 series

AS 1684 Residential timber-framed construction specifies requirements for building practice, specification of timber members, bracing and connections for the construction of timber-framed houses. Designers cannot use AS 1684 to design the entire mid-rise timber buildings, however some information from AS 1684, such as bracing and tie-down capacities of timber systems can be referenced as they are not building type specific. Also, a number of building practices contained within this Standard are applicable to mid-rise timber framed building, such as maximum hole size or notches, etc.

1.7.6 Eurocode 5 (EC5)

With care, designers can reference models from sections of EC5 in order to support design decisions made in the engineering design of mid-rise timber buildings. EC5 is similar in character to AS 1720.1. It is the European design standard for timber that references European loading, material and timber production standards in the same way that AS 1720.1 references the AS/NZS 1170 series, and material and timber product standards such as AS/NZS 1748. Some of the structural models and design factors used in EC5 are similar to those used in AS 1720.1, while others are quite different.

The design capacity of timber members in EC5 has a slightly different form to that in AS 1720.1:

EC5	AS 1720.1
$$R_d = k_{mod}\frac{R_k}{\gamma_M}$$ with R_k the capacity of a bending member	$$R_d = \phi k_1 k_4 R_k$$ with $R_k = k_6 k_9 k_{12} f'_b Z$ for bending members

Some of the similarities between EC5 and AS 1720.1 can be used by designers of Australian mid-rise timber buildings:

- the design capacities (R_k) are similar
- k_1 k_4 is similar to k_{mod} (defined in Table 3.1 of EC5)
- ϕ is similar to $1/\gamma_M$ (with γ_M a partial safety factor defined in Table 2.3 of EC5 – values range from 1.2 to 1.3 for strength limit state combinations of dead (permanent), live (imposed), wind and earthquake actions.

Where any ultimate strength is calculated in EC5 and used in an Australian building, the results must be used with appropriate k_1, and ϕ factors in AS 1720.1. The behaviour models in EC5 have been verified using test programs on European softwood species. The models can be used for Australian softwoods but may not apply to hardwoods. (Limited testing of connections with hardwoods has found failure modes that are not included in EC5, so connections with hardwoods that are not included in AS 1720.1 should be checked by testing).

EC5 uses the concept of Service Classes to model moisture conditions in service. Most timber elements in mid-rise buildings are used in sheltered environments with Service Class 1 in EC5, and k_4 =1 for seasoned products in AS 1720.1.

EC5 provides design methods for a number of structural elements that are not addressed in AS 1720.1 and provides design capacities for connections for a larger number of loading configurations than detailed in AS/NZS 1170.1.

Table 1.10 lists clauses in EC5 that contain information relevant to the design of mid-rise timber buildings.

Table 1.10: Potentially useful sections in EC5.

Information	Clause or Section in EC5
Vibration limits for floors	7.3.3
Design of panel-clad bracing walls	9.2.4
Design of horizontal diaphragms	9.2.3
Design of connections	Section 8

1.7.7 Canadian Standards

The Canadian timber design standard, CSA O86 – Engineering Design in Wood, has an excellent section on design of shear walls and horizontal diaphragms. Table 1.11 lists clauses in CSA O86 that contain information relevant to the design of mid-rise timber buildings.

Table 1.11: Potentially useful sections in CSA O86.

Information	Clause in CSA O86
Design of shear walls	11.5.1, 11.4
Design of horizontal diaphragms	11.5.2, 11.4
Deflection of shear walls	11.7.1
Deflection of horizontal diaphragms	11.7.2

If in doubt stick with AS/NZS 1720.1, particularly with regard to ultimate limit state design.

1.7.8 Other useful international resources

There are a number of excellent international timber design publications on the design of mid-rise timber structures including:

- *Mid-rise Wood-Frame Construction Handbook* – FP Innovations (Canada)
- *Technical Guide for the Design and Construction of Tall Wood Buildings in Canada* – FP Innovations (Canada)
- *CLT Handbook'* for assistance with mid-rise timber construction FP Innovations (Canada)
- *Cross-Laminated Timber Structural Design Basic design and engineering principles according to Eurocode* – pro:Holz
- *Cross-Laminated Timber: Structural Principles* – TRADA UK
- *'Shaft Wall Solutions for Wood-Frame Buildings* – WoodWorks
- *Five-Story Wood-Frame Structure over Podium Slab* – WoodWorks
- *Manual for the Design of Timber Building Structures to Eurocode 5* – TRADA

2 Design Loads and Criteria

2.1 Introduction

This section presents key structural design criteria and assumptions for mid-rise timber buildings. More detail on the design approach for specific elements types (walls, floors, etc) is given in later chapters. Where appropriate, *timber-framed* or *mass-timber* construction methods are used to provide clarification. A number of key structural performance requirements are presented that relate to overall building performance.

2.2 Structural Design Loads

Structural design loads are defined in the AS/NZS 1170 series.

2.2.1 Permanent actions/ dead loads

Permanent actions relate to materials installed into the building during construction that remain for the life of the building; they include the self-weight of the building. Timber construction elements have a high strength–to-weight ratio compared with steel or concrete and a lower self-weight than these types of construction.

Mid-rise timber construction is a 'system' of different products that provide the structural, acoustic and fire preformance. Mid-rise timber buildings use significantly heavier linings than timber-framed houses or concrete mid-rise buildings because of requirements for fire or acoustically rated linings and, in some cases, false walls to conceal services. Measures to improve the acoustic or vibrational performance of floors invariably require an increase in the mass of the overall floor system. Accordingly, it is important to accurately estimate the weight of all linings, finishes and additional materials required.

Where specific weights are known, these weights should be used. The density of specific species of timber are given in AS 1720.1 and the weights of many other materials can be estimated from Appendix A of AS/NZS 1170.1.

Table 2.1 can be used as a guide for estimating permanent actions early in the design phase before final decisions have been made for many parts of the building. The weight of acoustic materials typically used on the top of floors has been included in Table 2.1 as extra items. These loads should be added to the basic weight loads of the floor system.

Table 2.1: Permanent actions assumed in the preliminary design of mid-rise timber buildings.

Element	System and materials	Estimated permanent action
Floor	Carpet and underlay, timber-framed floor substrate, joists, fire-rated ceiling on resilient mounts	0.75 kPa
Floor	Ceramic tiles, timber-framed flooring, joists, fire-rated ceiling on resilient mounts	0.9 kPa
Balconies	Ceramic tiles, timber flooring, joists and fire-rated linings	0.85 kPa
Floor	Extra for acoustics – acoustic mats and extra acoustic layers	+ 0.4 kPa
Floor or balconies	Extra for acoustics or dynamics – 50 mm light-weight concrete topping + acoustic mat	+0.9 kPa
Floor	Extra for building services – air conditioning ducting, fire services, plumbing	0.2 kPa
Floor	Extra for false ceilings or floors – extra battens and linings	0.2 kPa
Internal Walls	Double stud fire-rated walls – wall, linings, insulation	0.7 kN/m^2*
Internal Walls	Single stud fire-rated walls	0.5 kN/m^2*
Internal Walls	Single stud non-loadbearing wall, non-fire rated	0.3 kN/m^2*
External walls	Lightweight cladding, single stud, fire-rated	0.7 kN/m^2*
External walls	Heavy cladding, single stud, fire rated	0.5 kN/m^2* + unit weight of the heavy cladding
Roof	Gardens, tanks, building services	by calculation

* Loads evaluated per square metre of the wall and applied to floors as a line load in kN/m.

Permanent actions include: the weight of all building elements, claddings, floor coverings, treatments for fire resistance, services including plumbing, tanks, pumps, the weight of air conditioning (ducts, valves, equipment, coolers), roof fixtures including flooring materials, soil in gardens, aerials, pergolas. Designers might also include the weight of water in tanks as permanent actions (the load factor of 1.2 rather than 1.5 for these actions is quite appropriate as it is impossible to squeeze 1.5 times the volume of water into a tank).

Manufacturers of floor systems, e.g. cross-laminated timber (CLT) and cassettes, can estimate appropriate permanent actions for their products, given an indication of the design span.

2.2.2 Imposed actions/live loads

Imposed actions relate to the occupancy of the building and might include items such as furniture, people and stored materials. These loads relate to materials, equipment and items brought into the building by its users. They are generally out of the control of the designer and sometimes difficult to predict with any precision at the design stage. Some of the loads will remain in place for many years, e.g. furniture; others, e.g. people, will be quite transient.

Table 2.2 can be used as a guide for estimating imposed actions early in the design phase, before final decisions have been made for many parts of the building. AS/NZS 1170.1 lists both distributed and concentrated floor loads. Concentrated loads are rarely critical in mid-rise timber buildings because the floor systems required for acoustic performance distribute the concentrated loads over large enough areas of the floor, so their effect is less than the effect of distributed loads. In this case, it is assumed that the load from internal walls is determined based on the layout required from the apartments, rather than the allowance for future provision of ‘partitions’.

Table 2.2 also includes k_1 for use in the load combination 1.2 G + 1.5 Q. The k_1 factor for the load combination 1.2 G + 1.5 ψ_l Q is 0.57.

Table 2.2: Imposed actions assumed in the preliminary design of mid-rise timber buildings.

(a) Class 2 buildings

Element	Floor use	Estimated distributed imposed action	ψ_l or ψ_c	k_1 for load case with Q
Floor	General, bedrooms	1.5 kPa	0.4	0.8
Floor	Communal corridors	4.0 kPa	0.4	0.8
Floor	Stairs and landings contained in the dwelling	2.0 kPa	0.4	0.8
Floor	Communal stairs and landings	4.0 kPa	0.4	0.8
Balconies	Associated with dwelling	2.0 kPa	0.4	0.8
Roof	Floor type activities associated with dwelling	2.0 kPa	0.4	0.8
Roof	Communal access floor type activities	4.0 kPa	0.4	0.8
Roof	Construction loads only	≥ 0.25 kPa, by calculation only	0	0.94

ψ_l factor for determining quasi-permanent long term values of action (AS/NZS 1170.0)

ψ_c combination of factors for imposed actions (AS/NZS 1170.0)

b) Class 3 buildings

Element	Floor use	Estimated distributed imposed action	ψ_l or ψ_c	k_1 for load case with Q
Floor	General, bedrooms	2.0 kPa	0.4	0.8
Floor	Communal corridors	4.0 kPa	0.4	0.8
Floor	Stairs and landings contained in the dwelling	2.0 kPa	0.4	0.8
Floor	Communal stairs and landings	4.0 kPa	0.4	0.8
Balconies	Associated with dwelling	2.0 kPa	0.4	0.8
Balconies	Communal access	4.0 kPa	0.4	0.8
Roof	Floor type activities associated with dwelling	2.0 kPa	0.4	0.8
Roof	Communal access floor type activities	4.0 kPa	0.4	0.8
Roof	Construction loads only	≥ 0.25 kPa, by calculation only	0	0.94

ψ_l factor for determining quasi-permanent long term values of action (AS/NZS 1170.0)

ψ_c combination of factors for imposed actions (AS/NZS 1170.0)

These loads are usually taken directly from Section 3 in AS/NZS 1170.1.

- Imposed loads on floors are a function of the character of the use of the rooms, which often varies throughout the building. Communal corridors are Category C3 and should be designed for at least 4 kPa, whereas hallways within a unit are designed using Category A2 (General areas).
- Imposed loads on floors can be reduced for elements supporting large areas (tributary area >18 m^2). In timber construction, these elements would be considered as primary elements and include large columns or beams. Even in the lower storeys of the building, studs and floor joists do not generally have sufficient tributary area to give ψ_a <1.
- The design-imposed loads for roofs depend on the function of the roof:
  - roofs that provide only a covering to the building are designed using Table 3.2 in AS/NZS 1170.1
  - roofs that also support activities such as rooftop gardens or recreational areas should be designed for the appropriate floor loads in Table 3.1 in AS/NZS 1170.1.

Combination factors have been taken from Table 4.1 in AS/NZS 1170.0, and k_1 factors from Appendix G in AS 1720.1. However, these tables do not separate corridors, stairs and landings from other floor use. The 4.0 kPa load for communal corridors, stairs and landings, represents crowding in emergency evacuations, and are of very short duration. The long-term imposed loads in these areas are usually zero.

Imposed actions also include construction loads, which may depend on the sequence of construction. Propping loads, loads from scaffolding fixing points, the weight of stacked materials or loads on lifting points should be discussed with the builder so they can be anticipated and calculated at the design stage. (Refer to Section 2.3.3)

2.2.3 Wind actions

Determining wind loads on a mid-rise timber building follows the same process as for a mid-rise building of any construction material. Key points to note:

- Timber mid-rise construction is relatively lightweight and, as such, adequate tying down type elements are needed, particularly in the roof.
- Acoustic requirements may require separated construction, especially for internal walls, which influences assumptions of net internal pressure.
- The wind loading Standard for housing *AS 4055 Wind loads on housing* is not appropriate for mid-rise buildings and AS/NZS 1170.2 should be followed. The Australian Wind Engineering Society's *Wind Loading Handbook* provides background and guidance on the interpretation of AS/NZS 1170.2.

In some parts of Australia, e.g. cyclone-prone regions, wind loads will be the critical lateral loading on the building. The lateral loads on the whole building are determined from the difference in pressure between the windward and leeward faces of the building. They are independent of the internal pressure assumed in design. (Refer to Section 7.2.1 for information on the design of buildings to resist lateral wind loads.)

The lateral design load on stud walls is calculated using internal pressures to find the pressure differential across the walls. Internal pressures also influence net uplift forces on the roof structure. Acoustic requirements for separated construction will lead to a net internal wind load on elements that is independent of the wind pressure on the other side, i.e. not a pressure differential.

Dynamic excitation of mid-rise buildings under wind loads is rarely a problem. However, as timber buildings increase in height, there is growing research in the field of human perception of dynamic wind excitation of mid/high-rise timber buildings (refer to *Dynamic properties of tall Timber Structures under wind-induced Vibration* by Feldmann et al. for more information). Mid-rise timber buildings feature a relatively low mass to excite (compared to more typical forms of construction). Therefore, further investigation will be needed where the building has a low natural frequency or where there are uncertainties in the assumptions made to determine natural frequency. The natural frequency of the building can be:

- estimated using *Equation 2.2* (for earthquake actions) or
- obtained directly from the structural analysis package.

Where the natural frequency is greater than 1 Hz, the C_{dyn} = 1. Only exceptionally narrow buildings would have lower frequencies.

Internal pressures

Internal pressures can be generated if doors and windows are left open or are broken or blown open during the wind event. Designers should assume patterns of openings that maximise the structural demand on each building element. For example, internal pressure can be maximised by assuming that there are openings in the windward wall of the building. Figure 2.1 shows a building with two openings in the windward wall (marked with Xs). These openings will pressurise two units (shown as blue arrows), but the fire and acoustic-rated walls between those units and adjacent units must be effectively sealed, so internal pressure can't be transmitted to the adjacent units.

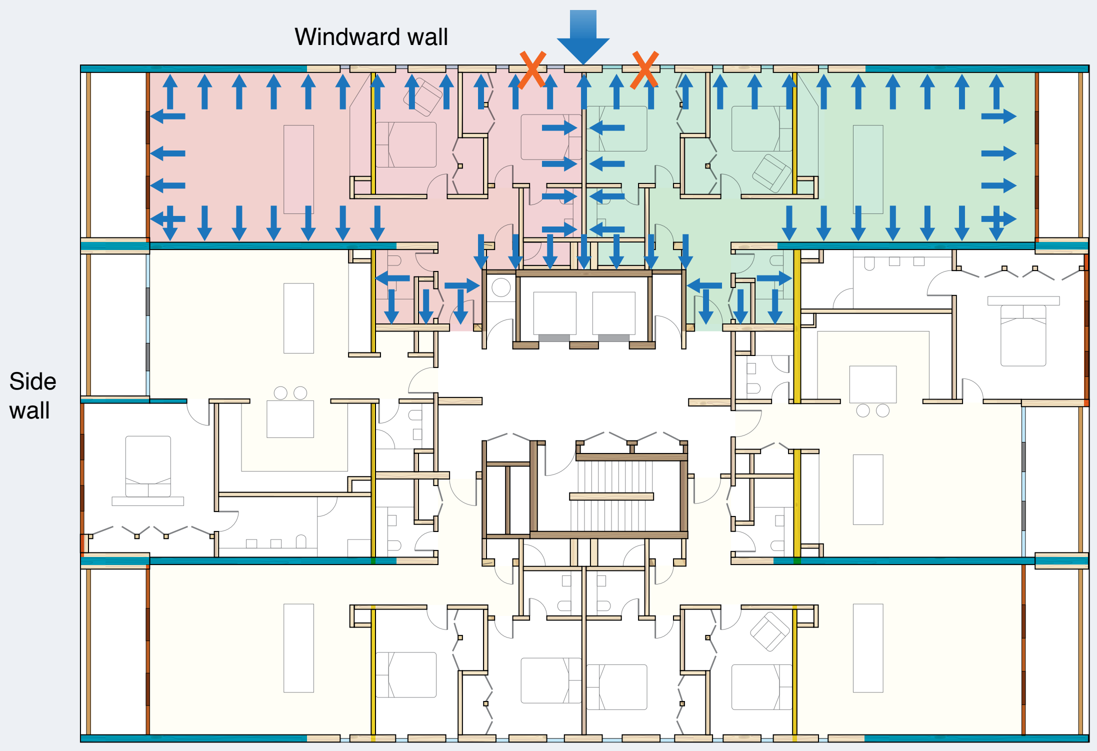


Figure 2.1: Internal pressure with openings in two units on the windward wall.

The high internal pressure is indicated with red shading in Figure 2.1 and will increase the net uplift on the ceiling, which may or may not be separated from the floor above (and the roof if the top storey). The internal pressure will also cause lateral loads on the fire walls between SOUs. These must be considered as bending actions in a combined bending and axial actions check on the studs using Cl. 3.5 in AS 1720.1. (See Section 5.4.5)

Net pressures on individual walls

All buildings that include effectively sealed fire and acoustic walls must consider out-of-plane loads from net pressures between one compartment and another. The resistance of timber-framed walls in out-of-plane loads in mid-rise timber buildings cannot be assumed to exceed the net pressure without a design check.

The design net pressure coefficients across walls (*AS/NZS 1170.2 Cl. 5.3.4*) are:

- external walls = external pressure coefficient plus highest internal pressure of the opposite sign (varies with wind direction)
- fire and acoustic-rated walls (SOU boundary walls) = maximum internal pressure accounting for openings plus 0.2 with a minimum value of 0.4
- other internal walls = 0.3.

Net uplift on roof

The net uplift force on a roof is equal to the external force plus the internal force (unless separated systems).

- External force is a function of the roof shape – refer to Section 3.2.2 and is applied to the whole upper surface of the roof.
- Internal force is an upward pressure on the underside of the ceiling. For balconies, it is calculated from the same pressure on the external wall in that location. For interior spaces, it is calculated from the internal pressures.

2.2.4 Earthquake actions

AS 1170.4 Minimum Design Loads on Structures – Earthquake Loads provides guidance and design procedures to address earthquake forces on buildings. (The other parts of the 1170 series are joint with New Zealand. Each country has separate earthquake action standards to model the inter-plate earthquakes in New Zealand and the intra-plate earthquakes in Australia).

Mid-rise timber buildings up to eight storeys would usually be designed using Earthquake Design Category EDCII depending upon the hazard factor and the site sub-soil classification. However, designers can choose to design any building using EDCIII (dynamic method).

Generally, mid-rise timber buildings will derive most of their lateral resistance from braced wall frames and cores. Table 2.3 indicates the recommended factors.

Table 2.3: Factors for lateral resistance.

Primary lateral resistance	Structural Performance factor S_p	Structural Ductility factor μ
Thin bracing panels (thickness of bracing <4*D*, with *D* = diameter of fastener)	0.77	2
Thick bracing panels (thickness of bracing >4*D* ,with *D* = diameter of fastener)	0.67	3
CLT cores with ductile connections between panels (diameter of connectors *D* <6 mm)	0.67	3
Other	0.77	2

Mid-rise timber buildings are lighter structures than hot-rolled steel or masonry buildings and attract lower earthquake loads. Their lighter weight also makes them more suitable for soft foundations that have the potential to increase earthquake actions.

Unlike wind forces, that are a function of the area of elevation and are often different in each direction, earthquake forces are a function of building mass, so are the same in both directions. Earthquake forces may dictate shear (bracing strength) requirements parallel to the building length. Even if the lateral loads for earthquake design are less than those for wind design, as the centroid of the earthquake loads is higher than the centroid for wind loads, the base moment created by earthquake actions may be higher than the base moment for wind actions. An earthquake load check should be completed for all mid-rise buildings.

Principles of earthquake resistance

The principles of earthquake resistance are very similar to those for robustness checks. These considerations will influence the timber connections through the building.

- There must be a load path for horizontal forces from all parts of the structure to the ground.
- All parts of the structure must be adequately tied together both in the horizontal and vertical plane. Heavy items of plant or machinery (e.g. cooling towers, transformers) must be specifically tied into the structure, and the structure must be designed to carry the lateral loads from these items. Floors should be continuous wherever possible or tied to supporting walls on all edges.

- Walls should be fixed to all floors and roofs (acoustic detailing must work around this).
- Floor, walls, roof beams and columns must be able to resist collapse even though part of the support is missing.

Capacity-based earthquake design is a good philosophy for timber structures although there are some significant differences in the way it is applied to timber structures compared with steel and concrete buildings, as discussed below. Concepts are.

- Select the most ductile components in the building to dissipate earthquake energy in. Design these with enough strength to resist the earthquake forces.
- Design a reserve of strength into the non-ductile components, so that they will not be part of the early failure sequence in the building. The required reserve of strength can be obtained by considering progressive collapse and ensuring that the non-ductile elements are not over-stressed even after a yield of a number of more ductile elements.
- Check that the fixings can accommodate the high deformations expected under earthquake loads.
- Check that vertical members can carry the gravity loads at the maximum building's earthquake deflection (taking into account second order effects).

	Timber buildings	**Steel and concrete buildings**
Typically, most ductile components are	Connections between floors and walls	Beams
Typically, non-ductile components (require a reserve of strength) are	Beams (joists)	Columns, connections between beams and columns
Details for high deformation include	Ductile metal connectors	Compression reinforcement in concrete buildings; high ductility steel in steel buildings.

To ensure ductility in timber connections, it is better to use a large number of small fasteners than a few large fasteners, e.g. 20 x 6 mm screws rather than 5 x M12 bolts. Typically, beams and columns should be designed with a 10 to 20% reserve of strength for the earthquake strength limit state load combination to force any non-elastic behaviour into the connections.

Where buildings have very irregular floor plans or cores are not centrally located, the earthquake response of the building may be influenced by torsional modes. In these cases, a dynamic analysis is recommended.

Mid-rise timber buildings often have a discontinuity above the ground storey where the use classification changes from retail/office to residential; that is the ground storey is often a concrete podium, the level above this then being constructed from timber. Designers must ensure continuity of the lateral load resistance system through this interface. (It the retail area relies on moment frame action to resist earthquake actions, it may have the characteristics of a 'soft storey' and require special details for earthquake design.)

Calculation of earthquake loads

Typical design parameters for mid-rise timber buildings are given in Table 2.4.

		Reference	**Comments**
Importance Level	2	NCC Table B1.2a	Check NCC examples of Importance Level 3 buildings if occupancy levels are likely to be high.
Probability factor kp	1	Table 3.1 AS 1170.4	For importance level 2 buildings, based on 1/500 Annual Exceedance Probability (NCC)
Hazard factor Z	Varies	Cl. 3.2 AS 1170.4	Location dependant
Sub-soil class		Cl 4.2 AS 1170.4	From site soil investigation
Earthquake Design Category	II	Table 2.1 AS 1170.4	Note that the building height for earthquake loads will be a little higher than the effective height of the building in the NCC. AS 1170.4 defines the building height (hn) as the height to the load centroid on the uppermost level (including roof)

All mid-rise timber buildings with an effective height between 12 and 25 m can be designed with EDCII (an equivalent static analysis). In some circumstances, buildings less than 12 m high can be designed using EDCI (simplified lateral force – Cl. 5.3 in AS 1170.4).

Earthquake Design Category II (EDC II)

EDC II is a static design method and involves calculating the total base shear from the total earthquake weight of the building. The earthquake weight (Wi) at each level is given by Equation 2.1. It is determined by summing the weights that would act at that level and includes the weight of the floor, furniture, and walls between storeys. This earthquake mass is assumed to be at the centre of the floor. The exception is the top storey where the mass of the roof and additional items that may be on the roof are included, and the earthquake mass is applied at the centroid of the calculated mass.

Equation 6.2 (6) in AS 1720.4	$W_i = \sum G_i + \sum \psi_E Q_i$	(2.1)

W_i = Earthquake weight at level *i*

G_i = Self-weight (permanent action) at level *i*

ψ_E = Earthquake combination factor = 0.3 for all applications except storage, 0.6 for storage (Section 6.2.2 in AS/NZS 1170.0)

Q_i = Imposed action level *i*

The natural period must be evaluated for the whole building to determine the spectral shape factor (Ch). For EDCII an approximation may be made using Equation 2.2.

Equation 6.2(7) in AS 1720.4	$T_1 = 1.25 k_t (h_n)^{0.75}$	(2.2)

T_1 = Natural period of the building

k_t = 0.05 for timber structures (AS1170.4 Section 6.2.3 – for all other structures)

h_n = Height of the uppermost centre of mass in the building (see Clause 1.5 in AS 1170.4)

The total horizontal equivalent static shear force V acting at ground level is calculated using Equation 2.3.

Equation 6.3(2) in AS 1170.4	$V = \left[k_p Z C_h(T_1) \frac{S_p}{\mu}\right] W_t$	(2.3)

V = Total base shear force on building

k_p = Probability factor (Table 3.1 in AS 1170.4)

Z = Earthquake hazard factor

$C_h(T_1)$ = Spectral shape factor at natural period of structure (soil class and T1) from Table 6.4 in AS 1170.4

S_p = Structural Performance factor from Table 6.5 in AS 1170.4

μ = Ductility factor from Table 6.5 in AS 1170.4

W_t = Earthquake weight of whole building = sum W_i

The structural performance and ductility factors are a function of the type of construction. The structural performance factor is lower if the structure has the ability to share the load between lateral load carrying elements. The ductility factor is higher if the building is able to absorb energy once individual elements have yielded. AS 1170.4 gives four classifications for timber buildings. Three of the classifications share the same earthquake load factors:

- Timber shear walls – include buildings with structurally clad bracing walls where many nails must yield to allow deformation of the building – S_p = 0.67 and μ = 3.
- Other timber systems – include buildings with no structurally clad bracing walls, braced frames and moment resisting frames – S_p = 0.77 and μ = 2.

The structural performance and ductility factors for timber buildings with shear walls are better than those for concrete- or steel-framed buildings without special details for earthquake loads, but not quite as efficient as fully ductile steel or concrete buildings.

The total horizontal earthquake force on the building (*V*) is apportioned between the storeys and gives a distribution similar to that shown in Figure 2.3.

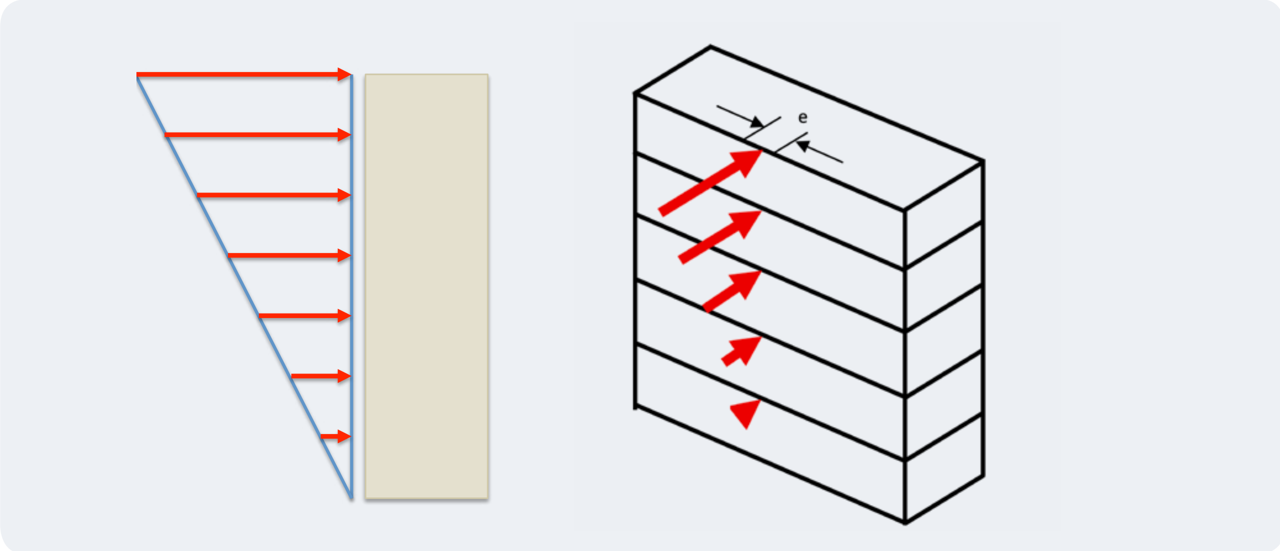


Figure 2.3: Horizontal force distribution with height.

Once the forces have been determined, an elastic structural analysis will give moments and forces in members to be used in the design and indicate the anticipated deflection of the whole building. Calculation of earthquake deflections needs to include $P\Delta$ effects, which will require a second order analysis.

2.2.5 Snow and ice actions

In alpine and sub-alpine areas, snow and ice may cause additional loads on the structure. Snow may accumulate on any roofs (the building roof and balconies) and increase the gravity loads on the structure. Drift snow may apply lateral loads to lower storey walls.

These loads are small compared with other loads in sub-alpine areas but may affect the design of some roof members and balconies in alpine areas. AS/NZS 1170.3 defines alpine and sub-alpine areas in Australia and specifies methods of calculating snow loads. The weight of accumulated ice is significant for power lines and lattice towers, but not for mid-rise buildings.

2.2.6 Soil movement

Loads and moments caused by differential settlement or soil movement can be evaluated by prescribing displacement of selected supports in the structural analysis. Timber buildings are less affected by these movements than more rigid concrete buildings. Because timber creeps under long duration loads, it is likely that small settlement will not over-stress timber members or connections. However, the deflected shape of the building may cause cracking in linings and jamming of doors and windows. Differential vertical movements may be seen between the core and the timber building in the case of a heavyweight concrete core.

2.3 Load combinations

Designers must anticipate a range of different loading scenarios and ensure that the building delivers satisfactory performance for each scenario. The load combinations for each of these scenarios are presented in AS/NZS 1170.0. Limit states design sorts the loading scenarios into groups that have similar characteristics and similar consequences.

- The *Strength limit* state includes rare and extreme load combinations, and the consequences of poor performance are a structural failure with potential for injury or loss of life. The annual probability of exceeding the forces from these combinations is generally greater than 1/500. For timber structures, the different combinations have different durations of loading and each combination must be compared against design capacities that have been separately evaluated for the appropriate duration of load and k_1 factor.
- The *Serviceability limit* state includes more common load combinations, and the consequences of poor performance may include cracking of linings, or excessive deflection or vibration. The annual probability of exceeding the forces from these combinations is generally around 1/20. For timber structures, the different combinations have different durations of loading, and any deflection calculations need to include an allowance for creep, deflections, which is achieved using the j_2 or j_3 factors.
- The *Fire limit state* deals with a single scenario – the building has a well-developed fire and a weakening structure must resist the loads at that time.

The load combinations presented in Section 4 of AS/NZS 1170.0 are also included in the design criteria of specific framing members in AS 1720.3.

2.3.1 Strength limit state

The strength limit state design of mid-rise timber buildings is different to the design of concrete and steel buildings because timber design capacities use a duration of load factor k_1, which differs for some load combinations. The load combinations for the strength limit state are given in Cl. 4.2.2 in AS/NZS 1170.0 and are summarised in Table 2.5 together with the appropriate duration of load factor k1 for each combination.

Table 2.5: Strength limit state load combinations and k_1.

	Load combination	k_1 for members	k_1 for connections
(a)	$1.35G$	0.57	0.57
(b)	$1.2G + 1.5Q$	0.8 0.94	0.69 0.77
(c)	$1.2G + 1.5\psi_l Q$	0.57	0.57
(d)	$1.2G + \psi_c Q + W_u$	1.0	1.14
(e)	$0.9G + W_u$ (uplift)	1.0	1.14
(f)	$G + \psi_E Q + E_u$	1.0	1.14
(g)	$1.2G + \psi_c Q + S_u$ (snow)	0.94 (sub-alpine) 0.8 (alpine)	0.77 (sub-alpine) 0.69 (alpine)

G = Permanent actions; Q = Design imposed action; ψ_l = Long-term factor

Elements need to be checked for a number of different load combinations, and in each case, the design capacity is a function of k_1 (the duration of load factor – See Section 1.7.1 and AS 1720 .1 Table 2.3), so the critical load combination for design is not immediately obvious. For example, the critical load case for studs may be 1.2G + 1.5Q, with a k_1 of 0.8. Wind loads may increase stud loads a little in the combination $1.2G + \psi_c Q + W_u$, but the capacity increases by 20% with the increase in k_1 from 0.8 to 1.0, so the ratio of capacity to design load is higher for this combination than for the critical one.

For CLT it is recommended to use the k_1 factors from AS1720.1 for Australian product or use the appropriate kmod factors from the Eurocodes if designing with European product.

2.3.2 Serviceability limit state

AS/NZS 1170.0 presents some serviceability loads that could be used to evaluate performance at the serviceability limit state. Although the combinations for the strength limit states are mandatory, the combinations for serviceability are optional. The client usually specifies the load combinations and limits that are important to the function of the building. For mid-rise timber buildings, clients may not have enough experience to select the combinations and set the limits.

If structural designers are not given specific guidance by the client, the load combinations and limits summarised in Tables 2.6, 2.7 and 2.8 can be used. These have been taken from AS 1720.3 and are compatible with those in AS/NZS 1170.0 Table C1. Although the load values may be significantly different from those in houses, the loads can be combined in the same way for mid-rise timber-framed buildings. The limits suggested usually give satisfactory performance for residential occupancy.

If the client does not specify serviceability limits, the values in Tables 2.6, 2.7 and 2.8 provide guidance.

Load combinations for serviceability are only considered if there are deformation limits for them.

Floor members

Loads on floor members are usually the same for each typical storey in the building and do not accumulate over the height of the building. Table 2.6 gives serviceability load combinations appropriate for all elements in the floor system under gravity loads. For diaphragm, action see section 7.6.

Table 2.6: Serviceability limit state load combinations for floor members.

	Load combination	*Load*	j_2*		*Typical Limit*
(a)	$G + \psi_l Q$	G $\psi_l Q$	2 2	Long-term serviceability load	Span/300 or 9 mm
(b)	$(1-\psi_l)\ Q$	$(1-\psi_l)\ Q$	1	Transient serviceability load	Span/360
(c)	g_{41}# x 1 kN (applied at mid-span)	0.57	1	Point load for vibration check	2 mm (apartments) 1.5 mm (other buildings)

G = Permanent actions; Q = Design imposed action; ψ_l = Long-term factor

* j_2 is the duration of load factor for creep (inelastic deflection) – see Section 1.7.1, and AS 1720.1 Table 2.4

\# g_{41} is a factor used to evaluate the proportion of out-of-plane load that is not distributed in a grid system, e.g. where a point load is applied immediately above a floor joist or truss, if 30% of the load may be distributed to adjacent joists or trusses by the flooring itself; only 70% is applied to the joist or truss under the point load. In this case, g_{41} = 0.7. It is evaluated using Paragraph E8 in AS 1720.1.

For CLT, it is recommended to follow the j_2 factors set out in clause 2.4.1.2 of AS1720.1 for the appropriate loading duration and moisture contents. For European products, it may be more appropriate to use the kdef factors provided in Eurocode 5, though special consideration should be given to the appropriate load combinations as they differ from Australian Standards.

Typically, serviceability criteria will govern the design of CLT floors, and this may either be deflections or floor vibrations. Most suppliers provide span tables based on commonly used vibration performance criteria, but the assumptions used in these should be understood. When checking the vibration performance of a CLT floor, it is important to understand the following:

- where the panels will likely be split for construction/transport – assumptions about single or double spans should be clearly communicated to the suppliers
- how is the CLT floor supported – supporting the CLT on beams that have their own flexibility will also contribute to the overall floor performance
- what floor finishes are used and their assumed damping characteristics.

Wall members

Loads on load-bearing wall members accumulate over the height of the building and are different for each storey. The serviceability load combinations for wall members can be taken from Table 2.7:

Table 2.7: Serviceability limit state load combinations for wall members.

	Load combination	*Load*	j_2*		*Typical Limit*
(a)	W_s	W_s	1	Wind serviceability load – stud bending	h/150 (Plasterboard) h/400 (brittle linings)
(b)	W_s	W_s	1	Wind serviceability load – inter-storey drift	h/200 (Plasterboard) h/500 (brittle linings)
(c)	$G + \psi_l Q$	G $\psi_l Q$	2 2	Long-term serviceability load	Span/300 (10 mm max)
(c)	$(1-\psi_l)\ Q$	$(1-\psi_l)$ Q	1	Transient serviceability load	Span/250

- Check out-of-plane deflections of studs under wind actions using combination (a).
- Check building sway using combination (b) of Table 2.7. Racking resistance of buildings is given in Section 7.
- Wall plates may have to be checked for deflection under vertical loads in the situation where studs do not align with floor joists or stud below. In this case use combinations (c) and (d) from Table 2.7.
- Lintels may have to be checked for in-plane deformation using combinations (c) and (d) from Table 2.7 and out-of-plane deformation using combination (a).

For CLT walls in tall structures or under heavily loaded walls, the bearing capacity of the floors needs to be checked, as the wood fibre in the floor panel is oriented horizontally and will tend to crush/deform under high loads. This deformation can become a key design issue in taller buildings where the overall building shortening needs to be controlled. Methods to transfer the vertical forces through the floor panel will reduce this crushing. Also, it is important to note that the deformation from crushing can occur on both sides of the floor panel.

Roof members

The serviceability load combinations for roof loads are given in Table 2.8. Where there are building services equipment such as air conditioning units on the roof, these items are included in load combination (a). If the roof is also used as a floor, e.g. rooftop garden or recreational area, then the load combinations in Table 2.6 must also be considered.

Table 2.8: Serviceability limit state load combinations for roof members.

	Load combination	*Load*	j_2*		*Typical Limit*
(a)	$G + \psi_l Q$	G $\psi_l Q$	2 2	Long-term serviceability load	Span/300
(b)	Q	Q	1	Short-tem serviceability live load	Span/250
(c)	W_s	W_s	1	Wind serviceability load	Span/150 (Sheeting) Span /400 (brittle cladding)

Note: The deflection limits for roofs that have a floor function should be the most adverse of roof and floor limits.

22.3.3 Construction performance

Construction loads include propping loads, loads from scaffolding fixing points, the weight of stacked materials or loads on lifting points.

Where construction loads on floors, such as the weight of stacked materials, exceed the imposed actions in AS/NZS 1170.1, a special load case for construction loads must be considered.

Where linings have not been fitted, deflection is often not an issue, but the strength limit state should be checked using the load combination 1.2G + 1.5Q.

Construction loads are relatively short duration and k1 = 0.94 for members and 0.77 for connections.

Construction scenarios and process for incorporating tolerances should be developed with the constructor.

Monitor shortening during construction

- ***may be swamped by tolerances***
- ***evaluate the difference between measured and predicted***
- ***extrapolate to in-service shortening***
- ***correct discrepancies with shims.***

2.3.4 Fire limit state

The load combination for the fire limit state is given in Cl. 4.2.4 in AS/NZS 1170.0 as

$G + \psi_l Q$

where:

G = Permanent actions

ψ_l = Long-term factor

Q = Design imposed action

The fire limit state models the behaviour of the structure near the end of a fire. Once the fire has become established, the occupants will have evacuated, and the only remaining imposed action will be furniture, which is represented by $\psi_l Q$.

The duration of the load for the fire limit state is 5 hours, so k_1 = 0.97 for members and 0.86 for connections.

Plasterboard fire-protected timber-framed walls and floors up to FRL 90/90/90

Before conducting a check of the fire load condition, check to see if the tested fire-rated wall or floors requires a check at all. The NCC requires plasterboard to provide substantial protection to the timber members to meet the ‘fire protected timber’ requirement, resulting in very little char of the studs, less than 12 mm, and very little char on floor joists. As the fire load combination is much lower than the ambient load condition combinations, it is rare for the fire load combination to govern the design.

To support this, Exova Warringtonfire has performed tests in accordance with Standard Fire Test (AS 1530.4) and prepared an assessment on the fire resistance performance of timber-framed walls lined on some Australian manufacturer’s plasterboard. The assessment report is available on the WoodSolutions website (ref RIR 22567A-05). Designers must also consult the relevant lining supplier’s information on how to check the structure at the fire limit state, particular system that use over linings than plasterboard or from manufacturers not included within the assessment.

A similar assessment by Exova Warringtonfire is available for timber-framed floor/ceiling systems, incorporating timber and metal web floor trusses or various engineered joists. The report states that "the depth and spans of primary structural elements may be increased provided they are designed in accordance with AS 1720.5 or AS 1720.1". The assessment report is available on the WoodSolutions website (ref RIR 37600400.1).

Plasterboard fire protected timber mass-timber walls and floors up to FRL 90/90/90

The NCC explicitly prevents AS1720.4 from being used to assign fire resistance levels of fire protected mass timber panels. The reason for this is the insulation and integrity values of the fire resistance levels are outside the scope of the Standard and that the adhesives or fixings method such as used in a dowel or nail laminated panels are outside the scope of AS1720.4. Each individual manufacturer should be approached to determine their specific fire design criteria as it is not sufficient to apply a simple char rate to the panels. Some manufacturers are able to provide design tables to assist with the calculations.

For fire-protected mass timber elements it is also important to engage with the timber supplier regarding their testing to ensure that the plasterboard linings are fixed in accordance with their specification. Penetrations through mass timber elements for pipes, cables, etc, would also be subject to their own fire-test certificates.

2.4 Movement and Serviceability limits – Elements

Serviceability limits include maximum tolerable static deflection and some criteria that model vibration performance of floors. As discussed in Section 2.2.2, the client may provide the designer with the required serviceability limits for the building. If not, Tables 2.6, 2.7 and 2.8 provide some guidance on serviceability limits that are appropriate for timber mid-rise. Specific deflection limits may have to consider the brittleness of the overlays and linings. For example, even relatively small deflections may crack large ceramic tiles on floors and walls, so tighter limits may be required in these cases.

2.4.1 Deflection limits

Refer to Tables 2.6, 2.7 and 2.8 for some indicative deflection limits for elements (floors, walls, roof, etc). Elemental movements can accumulate in the overall building which may, in turn, have a more onerous series of limits or considerations. See section 2.5.

2.4.2 Vibration limits

Dynamics of timber floors can be a critical design consideration given that relatively long spans can be achieved. Vibration limits tend to be subjective and some clients may have tried and tested vibration limits. There are currently no Australian regulated acceptance criteria. European and Canadian standards can provide some guidance; however they have different ways of analysing floor systems for vibration and different acceptability limits. Projects to derive ISO floor vibration acceptability tests are in the early stages of development.

Timber floor elements usually have higher damping than steel or concrete floors and may require a different vibration analysis. The main concern with vibration is usually human discomfort. A full analysis of vibration effects is complex and beyond the scope of this Design Guide, however, the main principles include:

- *excitation* – the dynamic forces that cause some vibration
- *dynamic structural response* – the vibrational behaviour of the structure or element
- *acceptability criteria* – a measure of the tolerance of the building occupants or other structural functions to various levels of vibration.

The two main dynamic sources of excitation for mid-rise timber buildings are footfall and machinery vibrations.

Footfall vibrations

Footfall vibrations affect the floor that is being walked on. Vibration can cause discomfort for other people on the same floor or may produce annoying shaking of furniture.

A natural frequency in the floor (complete with topping and floor covering plus any attached ceiling, etc) of more than 8 Hz is desirable. For better vibration performance in floors without a concrete topping, a limit of 10 Hz can be used. (A concrete topping on a floor with a natural frequency of more than 8 Hz will already give enhanced vibration performance). If dynamic performance is critical, detailed analysis will be required to demonstrate performance of floors.

If the natural frequency limits cannot be achieved or if the dynamic performance is particularly critical, and this can be the case for large span engineered floors, a more rigorous dynamic analysis should be undertaken. Mass-timber (CLT and LVL) manufacturers recommend vibration checks that have been derived to suit the characteristics of their floor panels.

Where the natural frequency of the floor system is greater than 8 Hz, their method of check are recommended and they are summaries as:

- maximum deflection under serviceability short-term imposed actions
- point load at centre span
- Eurocode 5 dynamic vibration check.

Machinery vibrations

Machinery such as lift hoists and air conditioning machinery cause vibration with a much higher frequency, but generally lower amplitude. Isolation of the machinery from the rest of the building can be achieved by:

- separating the parts of the building that support this machinery from the occupied parts
- using special isolating mounts under the machinery.

An extra design case may be required where the building may accommodate rhythmic and synchronised activities, e.g. marching, dancing, aerobic exercise and sports activities, as group activities induce much larger amplitude footfall vibrations.

2.5 Tolerances

There is currently no overall guidance document or national specification for timber fabrication and construction tolerances. It is important that engineers consult early in the design process with fabricators and erectors to gauge suitable tolerances to ensure compatibility with engineering assumptions and the overall building systems. Inappropriate specification of tight tolerances adds cost. Loose tolerances add more cost, make erection more difficult/impossible, add complexity with the overall building systems such as facade, and can lead to incorrect assumptions in engineering analysis.

Mid-rise building generally requires construction to tighter tolerances than low-rise; the height exacerbates any initial out-of-position, and the facade components such as glazing or cladding are generally pre-fabricated with tighter tolerance and higher performance than common low-rise options.

Pre-fabricated timber components can be machined to tight tolerances. For example, cross-laminated timber elements such as CLT or plywood fabricated on a computer controlled router/machine can be fabricated to a tolerance of less than +/- 1 mm. Their cross-laminated construction means that they can be expected to arrive for construction with a similar tolerance (panel length and width).

It is possible for CLT construction overall to be erected with a tolerance of less than +/- 3 mm per storey. The final achievable tolerance depends on building detailing and the experience of the supply chain. Timber-framed construction of prefabricated panels can achieve tolerance as tight as CLT, but is more likely to require a larger tolerance of say +/- 10mm per storey. These figures are highly dependent on the procurement route and fabrication method adopted as well as the team involved. Consult with the fabricators and erectors.

A key consideration of tolerance is at interfaces between materials, systems, or procurement packages such as façade contractors or lifts. For example, construction tolerances of a concrete foundation or podium need to be considered as they will not be as onerous as required for the timber structure above. Best practice is to complete a survey of the constructed concrete elements and prepare to make-good tolerances in advance on starting erection of the timber elements. Options for dealing with tolerances include a high tolerance concrete finish (requires coordination), levelling screed (requires additional depth), or shimming (requires care in engineering design for bearing and shear).

3 Roof Design

The performance requirements for roofs on mid-rise timber buildings include:

- safely resisting all loads (primarily gravity and wind loads, but also earthquake and in some cases, snow)
- shedding rainwater and draining to ground effectively
- weather resistance and protection.

3.1 Structural systems

Roofs on timber mid-rise buildings can either be pitched or flat (to accommodate services, gardens or recreation areas).

Flat roofs are structurally similar to floors and can be constructed using the same range of systems that are available for floors: parallel chord trusses, cassette systems or mass-timber panels. Tie-down of individual panels is required. Flat roofs also need a protective waterproof element sloping at an appropriate angle to allow water shedding and drainage.

Pitched timber roof systems can utilise larger free-spanning trusses. In rare cases, part or all of the roof may be framed on site. For full-spanning roof trusses, tie-down can be designed for the perimeter only.

Consult roof system fabricators early in design process; they can assist with decisions on the best type of system and the tie-downs required.

Roof attachments affect the permanent and imposed loads. Roof systems need to be designed to resist all applied loads, including wind forces on the attachments.

3.2 Loads

Structural design loads are defined in the AS/NZS 1170 series (refer to Section 2.2.)

3.2.1 Permanent and imposed actions

Roofs without floor-type activities

Where the roof is required to only provide a covering for the building, long-term loads are from permanent actions. Imposed actions are only associated with the short-term loads of construction and maintenance .

Some near-flat roofs with membrane linings for water proofing may include a protective layer of gravel or paving that may contribute to the dead load of the structure.

Roofs with floor-type activities

Where the roof is also used for a roof-top garden (see Figure 3.1) or recreational area, the roof must be constructed to resist loads similar to that for floors. Typically, the upper surface will be heavy, e.g. tiles or concrete, and may support garden boxes, soil and extra structures (permanent actions). Appropriate imposed loads are given in Table 3.1 in AS/NZS 1170.1 and summarised in Table 2.2 in this Guide. Where the roof loading includes known imposed loads, such as heavy furniture or outdoor kitchens, the weight of the known items should be added to the general imposed action appropriate for the floor-type activity.

Figure 3.1: Flat roof with roof-top gardens.

Where the roof is flat enough to support floor type activities, particular care is needed to establish effective drainage. In some cases, separate drainage channels underneath the wearing surface may contribute to gravity loads.

Roof attachments

Any additional permanent and imposed actions on lighter weight timber framed roofs need to be anticipated at the design stage. (Gravity loads on the roof may affect the design of all walls in the building.) Additional gravity loads could be caused by:

- safety harness fixings or rails for window cleaners
- roof-top gardens
- pergolas and barbeque structures
- clothes drying areas
- heavy paving
- water-tight layers
- air conditioning and other roof-mounted machinery (Figure 3.2)
- water tanks and hot water systems
- solar panels
- aerials or satellite dishes
- lift equipment.

Roof items are sometimes added to the building later in the design phase; however, it is most design time and cost effective to include loads from these features in the preliminary design. The list above can be used as a check to establish the initial design brief for the roof.

Figure 3.2: Roof mounted equipment and services.

3.2.2. Wind actions

The design wind loads on mid-rise timber buildings are similar to those on other mid-rise buildings. However:

- structural designers familiar with designing low-rise timber buildings need to adjust roof designs to allow for wind loads that are about 20% higher at 25 m than they are at 10 m
- net wind uplift loads are higher on lighter timber roofs than on concrete roofs.

Wind loads are determined using AS/NZS 1170.2. (AS 4055 Wind Loads for Housing cannot be used to design mid-rise timber framed buildings.)

Roof shape

External pressure coefficients are a function of roof shape.

- Flat roofs have a $C_{p,e}$ between –1.3 and –0.9 near the leading (windward) edge and becoming more positive closer to the leeward edge.
- For skillion roofs, use pressure coefficients for flat roofs regardless of the roof slope.
- For all other roofs (e.g. hip or gable), the pressure coefficients can be found directly from Table 5.3 in AS/NZS 1170.2.

Many roof pressure coefficients given in Table 5.3 in AS/NZS 1170.2 and appropriate for mid-rise buildings (h/d can be close to 1) have two entries, a more negative value indicating peak uplift, and a less negative value. Designers need to consider two wind load cases for each wind direction.

- Most negative external pressure combined with openings that give the most positive internal pressure using load combination $0.9G + W_u$ (uplift);
- Most positive external pressure combined with openings that give the most negative internal pressure using load combination $1.2G + \psi_c Q + W_u$.

(Refer to Section 2.2.3 for guidance on internal pressures.)

Parapets

Many mid-rise buildings incorporate parapets, particularly where the roof also has a floor-type function, as the parapets act as balustrades. Parapets can reduce local pressure factors used only for cladding and purlin loads near the outside of roofs, but do not change the pressure coefficients used to calculate uplift loads on trusses or deeper elements in the tie-down load path. The use of parapets means that box gutters must provide the roof drainage system, which can complicate the roof design by forcing a large step-down over or near a support.

Roof attachments

Roof- mounted air conditioning units, aerials, solar panels, hot water systems, etc,. must be fixed to structural elements, not the roof cladding. Where the roof structure design is subcontracted, ensure that the roof designer is aware of any extra elements on the roof that could increase wind loads or cause local moments on the roof structure.

3.3 Design

The design of roofs for mid-rise timber buildings is dictated by permanent, imposed and wind loads.

Roofs with a floor function are designed primarily as a floor and checked for wind uplift as a flat roof. These designs use the techniques outlined in Section 4. Where the roof is flat enough to support floor type activities, particular care is needed to establish effective drainage. In some cases, separate drainage channels underneath the wearing surface are required. There are two approaches to designing fall into flat roofs, irrespective of the structural system used.

- Shape the wall frames in the upper most storey of the building to account for the desired slope within the roof. This involves accurately cutting studs or CLT wall ends to the required slope.
- Build up the top of the wall frame with packing that is cut to the required slope.

Pitched roofs usually use roof trusses, which are often designed and supplied by truss manufacturers.

3.3.1 Truss roof systems

Roof trusses provide an engineered roof frame system designed to carry all the roof loads to selected load bearing walls in the building.

The **spacing of roof trusses** is dictated by the span of the elements above the truss, i.e. trusses that support roofs with floor activities may have spacings of 300 to 450 mm with spans generally around 4 to 8 m. But the spacing of trusses in roofs that do not have a floor function is typically limited by the maximum spans for battens (600 to 900 mm), or purlins (900 to 1200 mm) with spans up to 25 to 30 m.

Roof trusses are designed by truss manufacturers to AS 1720.5, and installed to AS 4440. Truss manufacturers require details of any additional loads, such as the weight of water tanks, solar hot water or photovoltaic systems, air-conditioning and other attachments.

Corner cantilevers – On buildings with large roof overhangs, design support systems to ensure that deflections at the corners are within limits.

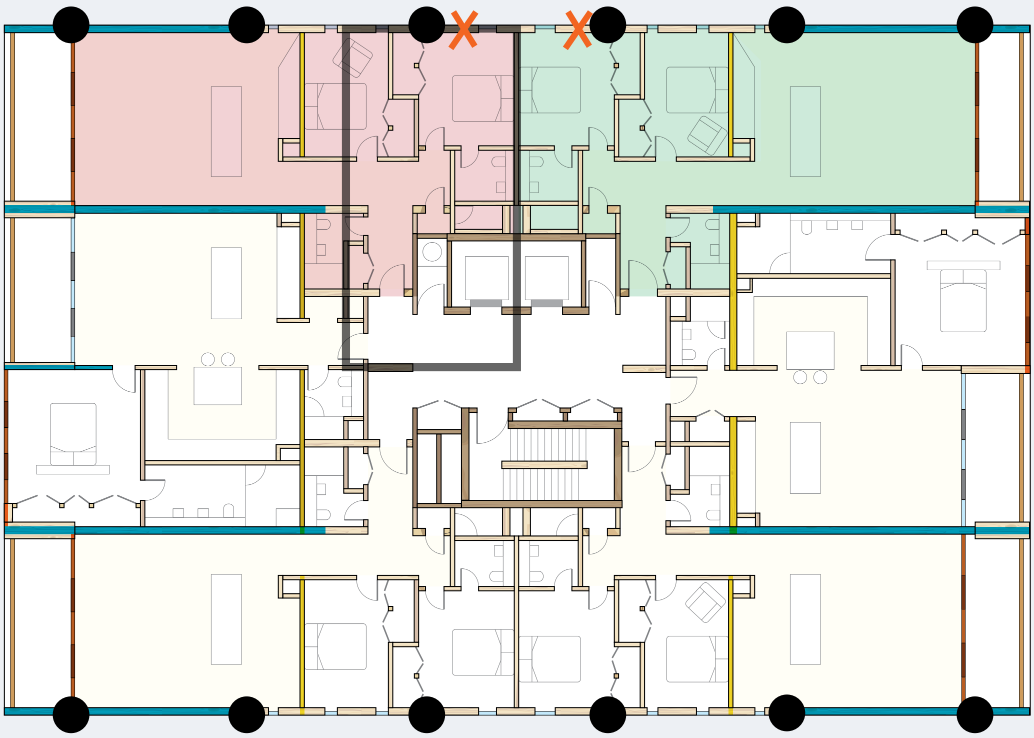
Manufacturers usually design and supply the prefabricated trusses and the connections between the trusses and the top of the wall structure.

3.4 Tie-downs

Each roof panel must be designed to resist the net uplift forces on the complete panel. This can be achieved by:

- anchoring individual floor panels (CLT or cassettes) to supporting walls
- integrating truss tie-downs with tie-downs to resist global overturning (detailed in Section 7.7).

Figure 3.3 shows the upper storey of a building with roof tie-downs highlighted as black circles. Higher internal pressures in two apartments are due to the openings in the windward wall and are shown with red crosses. The two apartments with higher internal pressure are shaded red and green. The tributary area of roof for one tie-down is highlighted with a black rectangle. The forces on the tie-down are calculated using higher internal pressures over the red shaded area (around three-quarters of the tributary area) and lower internal pressures for the remaining quarter of the tributary area.



Net uplift on roof with openings in two units on the windward wall

Figure 3.3: Net uplift on roof with openings in two units on the windward wall.

4 Floor Design

Floors are repeated elements throughout the building. Where the loads and layout on the floors are the same on each storey, the same design can be repeated for each level. Within each storey, different floor spans can be designed but should be rationalised for simplicity and economy. Floor depth, or finished floor level, may vary to accommodate set downs in wet areas or different ceiling heights. In such cases, care is needed in the design of the floor diaphragm action.

Floor systems must satisfy acoustic and fire performance requirements (see Section 1.6), which can influence structural systems and details. Overlay toppings to add mass (such as screeds or panel products) are usually added to lightweight floor systems to improve acoustic performance (see Section 4.8).

Floors have two distinct structural functions. They are designed to:

- carry vertical room traffic loads to their supporting elements, as outlined in Section 4.4 for framed floor systems and Section 4.6 for mass-timber floor systems
- transfer lateral loads (through diaphragm action) caused by wind or earthquake to the bracing walls and systems in the storeys below, as discussed in Chapter 7.

Off-site prefabricated floor systems must also be designed and detailed to suit the proposed transportation, lifting and construction method. The structural system adopted for floor framing may be governed by the loads anticipated during lifting. The strategy should be coordinated with the installation contractor and floor system fabricator.

4.1 Floor systems

The floor system selected for a particular building will be influenced by a range of factors, such as: span, loading, preferred floor-to-floor height, budget, material/system availability, transport, and acoustic and fire-resistance requirements. Table 4.1 provides a summary of the lightweight and mass-panel floor systems and their typical application. These structural systems can be used in combination with various architectural, fire and acoustic details to meet NCC and market performance requirements.

Table 4.1: Floor structural systems commonly used in mid-rise timber construction.

Floor system	Description	Application	
On-site frame floor systems			
Sawn timber floor joists and bearers	Sawn hardwood or softwood members. Lengths up to 6 m.	Small spans such as wet areas and corridors.	
I-beams or LVL joists	I-beams or LVL members generally have higher structural capacities than sawn timber and are available in longer lengths and depths.	General use throughout the building where longer spans than sawn timber are required.	

Table 4.1: Floor structural systems commonly used in mid-rise timber construction (continued).

Floor system	Description	Application	
Floor cassettes *(Also refer to WoodSolutions Technical Design Guide #31 Timber cassette floors)*			
Floor-truss joist cassettes	Utilise floor-truss joists: top and bottom truss flanges fabricated from sawn timber or LVL, and webs either timber or metal.	Use where larger spans required. Open webbed joists can allow services to pass through.	
I-Beam floor joist cassettes	Utilise I-Beam floor joists: top and bottom flanges fabricated from sawn timber or LVL, and webs of timber, steel, plywood or OSB.	Use where larger spans required. Joist webs can have cut-outs to allow services to pass through.	
Rib-slab-floor cassettes	Utilise typically LVL floor joists (ribs) and thick floor-slab floor panels (45-75 mm).	Use for larger spans or if smaller floor depths required.	
Mass-timber floor systems			
Cross-laminated timber (CLT) panels	CLT panels typically up to 12 m long, 3 m wide (transport limitations). Provide good acoustic performance, and are quick to install.	Suitable for two-way slab systems and can be cut and drilled to accommodate services and stacks. Building services generally suspended below.	
Laminated veneer lumber (LVL) panels	Manufactured in billets 1.2 m wide, and up to 75 mm thick. Billets can be glued together to form thicker floor plates (LVL may be cross-banded to assist in reducing panel cupping),	One-way spanning floors. Horizontal distribution of building services to be suspended below	
Glued laminated timber (Glulam) panels	Manufactured by gluing timber boards together to form long length beams, which can be laid on flat to form floors. Glulam floors have similar characteristics to CLT, but do not have cross laminates.	One-way spanning floors; may require details to accommodate shrinkage. Horizontal distribution of building services to be suspended below	

Table 4.1: Floor structural systems commonly used in mid-rise timber construction (continued).

Floor system	Description	Application	
Mass-timber floor systems (continued)			
Nail Laminated Timber (NLT)	NLT manufactured by stacking timber boards on edge and connecting them together with nails, screws or other fasteners. Plywood sheathing and/or concrete top coatings are often added to the upper surface of the panel to improve strength and for finishing.	One-way spanning floors. Building services generally suspended below. Shrinkage movement needs to be accommodated and detailing for acoustic and fire separation needs care.	
Timber-concrete composite (TCC) floors *(Also refer to DG#30)*	TCC composite floor systems use timber joists or mass-timber with a concrete top slab acting as a flange. Shear connectors screwed into the timber, transfer shear between the concrete and the timber elements. Timber element works in tension and the concrete acts in compression.	Larger spanning capacity than traditional floor systems and concrete assists with acoustic and vibrational control	

4.1.1 Framed floor systems

Framed floors are generally constructed on-site and utilise closely spaced lightweight timber floor joists or floor trusses, generally at 450 or 600 mm centres, overlain by flooring/decking, and for deeper floors with intermediate blocking pieces fitted to stabilise the joists or trusses. Serviceability limits, addressing deflection or vibration response, are usually the limiting factors for lightweight timber system floor design, as discussed later in this chapter.

Where floor joists have open webs, or penetrations, building services can easily be run horizontally within the floor depth (see Figure 4.1).

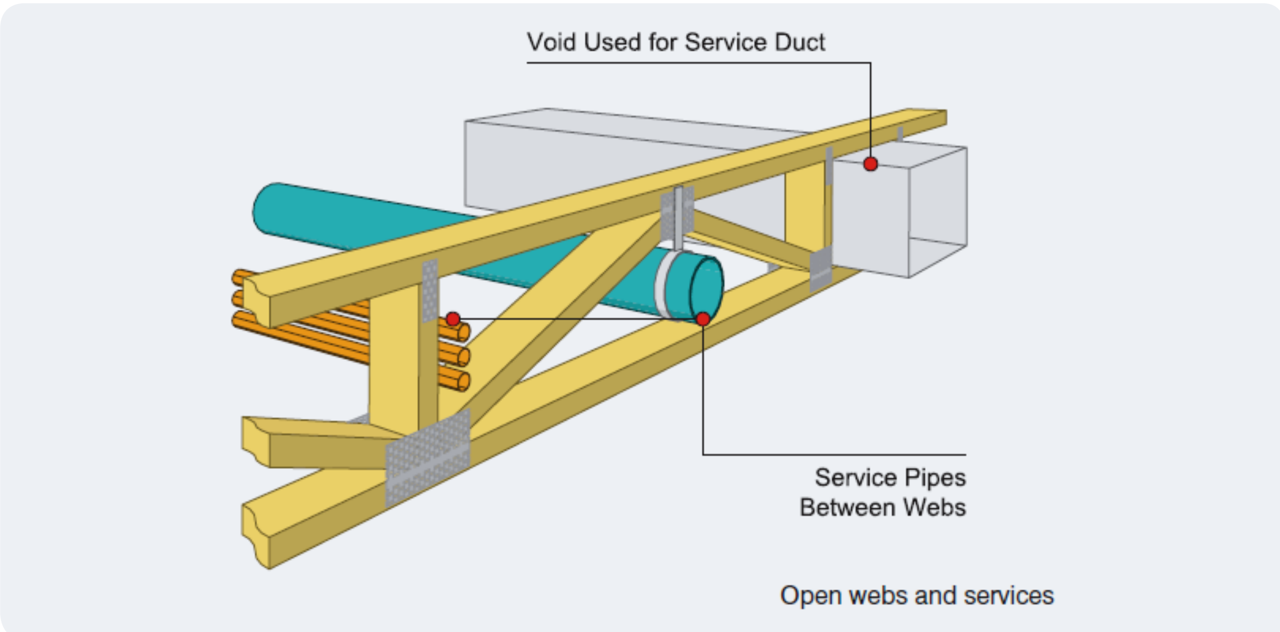


Figure 4.1: Framed floor systems accomodating buidling services within floor depth. (Image: Pryda)

The structural flooring/decking of a floor system must be able to span between the primary spanning elements, be robust enough to resist point loads, and have sufficient in plane performance for diaphragm action. Flooring/decking options include particle board, plywood, OSB, cementitious based floor sheets, e.g. fibre cement, magnesium oxide (MgO), or autoclaved aerated concrete. Some commonly available products are illustrated in Figure 4.2. Options presented here are valid for both site-built and off-site prefabricate cassettes.

Particleboard

OSB or plywood

Fibre cement

INEX

Hebel

Promat

4.1.2 Floor cassettes

A favoured approach for floor construction in mid-rise timber buildings is to utilise timber-framed floor cassette systems that can be manufactured off-site and then transported to site and easily installed into the building. They are valued by builders because they provide a fast, cost-effective way to construct a complete finished floor in a short time and, importantly, they dramatically improve the worksite safety by reducing the risk to workers of falling from heights. (Refer to *WoodSolutions Technical Design Guide #31 Timber Cassette Floors* and consult frame and truss fabricators early.)

Floor cassettes can be prefabricated using a range of different joist options including floor trusses, I-beams or solid timber or LVL joists. Again, using joists with open webs or pre-cut penetrations provides a convenient way to run services horizontally within the depth of the floor, as shown in Figure 4.1.

Floor cassettes will be required to be designed to resist, in addition to normal structural loadings, also transportation, lifting and assembly loadings and impacts. In addition to the flooring and joists members, other important floor cassette considerations therefore also include design of:

- rim-boards – timber boards that cover and support the ends of the cassette joists and also provide transfer of wall forces from the upper to the lower level at bearing points
- connection between cassettes (Section 4.4.3)
- strong-backs – support across the line of joists within the cassette (Section 4.4.4)
- lifting points for installation
- attachments for joining cassettes at their ends.

Figure 4.3: Installation of cassette floor systems.

4.1.3 Mass-timber floors

Mass-timber floors are generally solid wood panels made from engineered timber (CLT, LVL, Glulam or NLT – see Section 1.2). The panels are available in long length and widths, provide good acoustic performance, and are quick to install. They have a relatively shallow structural depth when compared to other floor systems, but horizontal building services distribution (plumbing and air conditioning, etc) has to be suspended below the floor often within false ceilings. (For information on CLT see also *WoodSolutions Technical Design Guide #16 Cross-laminated timber* and *WoodSolutions Technical Design Guide #44 CLT acoustic performance*.)

CLT is available in widths 1.2 to 3.0 m; lengths up to 16 m, but usually a 12 m maximum for transport; and thickness 50 to 350 mm. Consult manufacturers for advice early in the design process.

LVL panels for floor use are available typically in 63 and 75 mm panel thicknesses, 1.2 m widths, and lengths up to 12 m.

4.2 Layout of floor panels

In laying out floor systems, designers can vary the floor span direction and the floor span continuity.

Span direction

While each building will be different, the first step in laying out a floor is to identify which of the walls within the structure might be utilised as loadbearing walls; external walls, walls between apartments and lift and stair shafts will generally be loadbearing. Floor systems are then laid out spanning between these load-bearing walls, generally spanning the shortest distance between load-bearing walls to minimise the floor depth.

- If load-bearing walls are parallel and unidirectional, all floor panels can span in the same direction. If loadbearing walls run in both directions, floor panels may also vary in their span direction.
- Floor spans should be kept as uniform as possible to simplify floor panel design fabrication and construction:
  – If spans are the same, standard length and depth floor panels can be used throughout the building.
  – Standard widths are preferable but may vary depending on layout.
  – Longer spans will require deeper floor panels.
- If cantilever floor systems are used, the layouts should ensure appropriate back-spans.

Figure 4.4 shows an example of selection of load-bearing walls and floor panel layout. All floor panels in this example span top to bottom (example arrows shown).

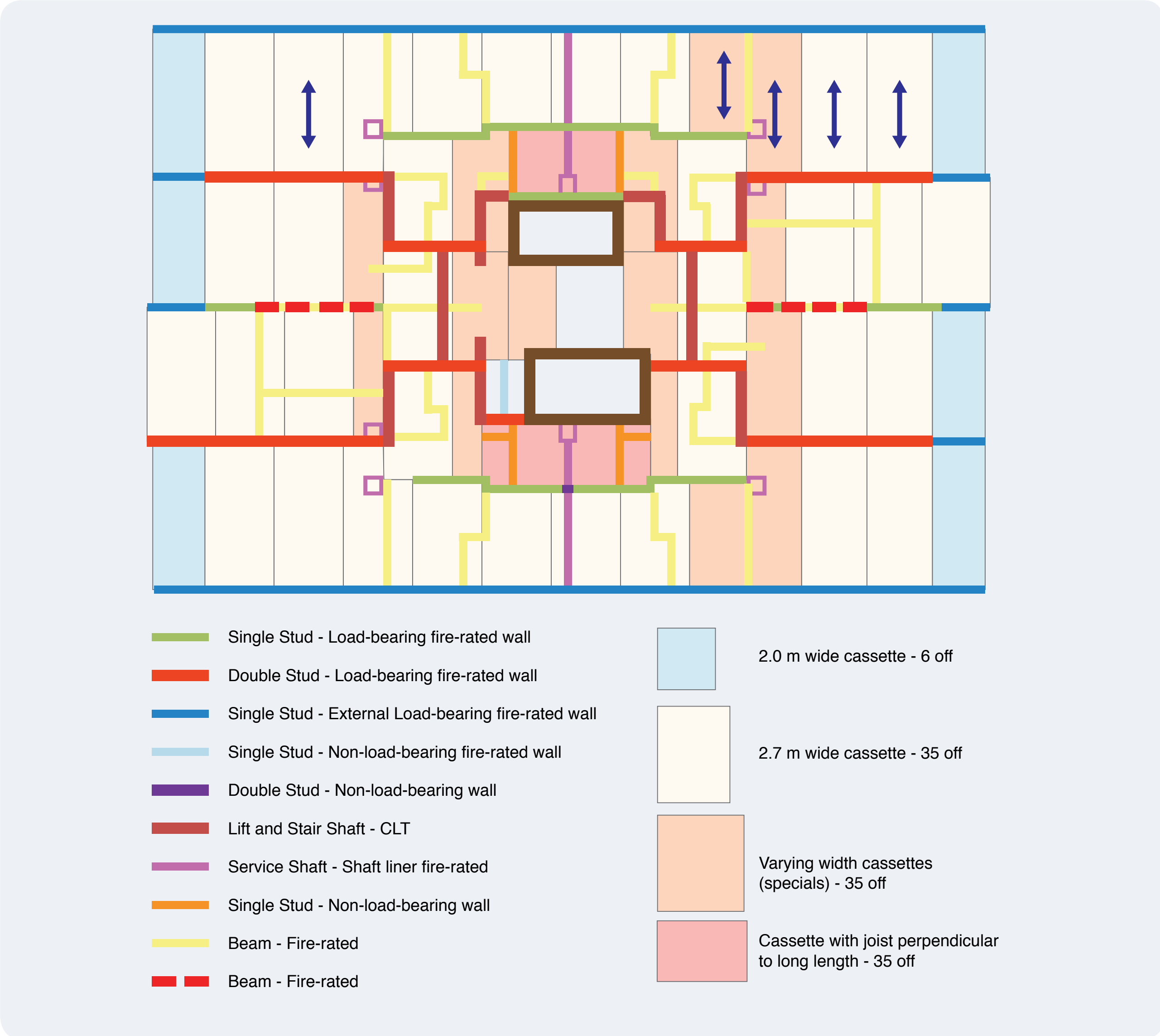


Figure 4.4: Example of load-bearing walls and floor cassette layout.

The floor systems described in Table 4.1 all have a principal spanning direction; they are effectively one-way spanning panels. CLT has two-way spanning capacity but has significantly higher strength and stiffness in the directions parallel to the outer laminate face grain.

Continuous or simple span

In platform frame construction (see Section 5.1.1), floors can be run continuously over some internal load bearing walls. Continuous spanning floor systems provide better options for robustness and earthquake load capacity as well as allowing slightly shallower floor depths.

For both semi-balloon and balloon frame walls (see Section 5.1.2 and 5.1.3), the floor panels are supported on ledgers on the side of walls so floor panels must be simply supported between the load-bearing walls.

Where NCC acoustic requirements require the use of discontinuous walls, and individual floor systems bear on these walls, then a floor infill member is likely to be required to provide continuity for appropriate floor diaphragm action to be maintained (See Figure 4.5 and Section 4.4.3, Figure 4.8).

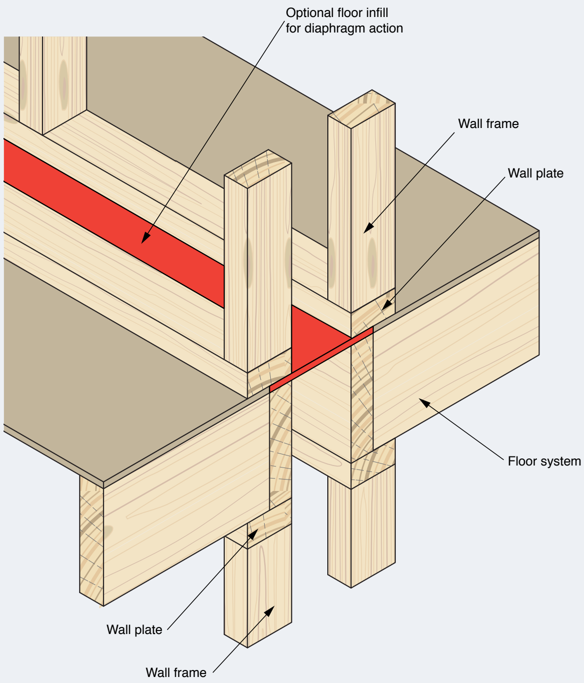


Figure 4.5: Use of a floor infill member between floor systems over a discontinuous wall to maintain floor diaphragm action.

Panel types and sizes

Other floor design considerations include the selected methods of fabrication, delivery and installation of the floor panels, so the following should also be considered.

- Dimensions of floor panels must be selected to permit easy transport, efficient lifting and installation. Panel size is often limited by transportation; typically, around 12 m length and 3 m width can be transported without any travel restrictions.
- Use of larger panels minimises the number of lifts required on-site and the hook-time.
- The weight of panels may be limited by the crane's capacity at its furthest reach.
- Selecting standard widths for panels may simplify fabrication. Fabrication and erection are simplified, if the number of different types and sizes of panels is minimised.

4.3 Loads and Load Cases

Chapter 2 summarises the preliminary Permanent Action design loads (Table 2.1) commonly used with mid-rise timber buildings. The largest permanent load variable affecting the design of floors, and consequently walls, is the use of toppings and overlays above the flooring to assist in acoustic performance, or the use of timber-concrete composites. Section 4.8 presents a range of options available for acoustic toppings. The need for, or type of, acoustic toppings needed to be determined before carrying out a detailed design as the use of toppings affects the sizing and economies of the timber-framed systems.

Chapter 2 also summarises the Imposed Actions in Table 2.2.

The following load cases in AS/NZS 1170.0 Cl 4.2.2 must be considered in the design of floors in mid-rise timber buildings for the strength limit state:

Bending

- $1.35\ G$ — $k_1 = 0.57$
- $1.2\ G + 1.5\ (0.4\ Q)$ — $k_1 = 0.57$
- $1.2\ G + 1.5\ Q$ — $k_1 = 0.8$

Combined bending and in-plane actions

- $G + (\psi e\ Q) + E_u$ — $k_1 = 1.0$
- $1.2\ G + (\psi_c\ Q) + W_u$ — $k_1 = 1.0$

Floor design is often limited by the serviceability limit state (static or dynamic) and the criteria for design and the appropriate serviceability limits are given in Section 2.3 and design methodology presented in Sections 4.4–4.7.

4.4 Preliminary design of lightweight timber-framed floor systems

Where cassette floors are to be used, the final design of floor cassettes will typically be by the manufacturer; however, a preliminary design is required to estimate the floor depth, confirm the system choice (in the context of the broad performance requirements) and establish load paths through the building. AS 1720.1 presents methods for finding the capacity of members in floors for the strength limit state and the deflection of floors for the serviceability deflection limit states. However, in many mid-rise timber buildings the critical case for design is the vibration response. These notes assist with the preliminary sizing of framed floor systems including cassettes.

4.4.1 Structural flooring/decking

Refer to design manuals (EWPAA or manufacturers') for thickness and span of flooring material for the relevant loads. Dead loads should include an allowance for acoustic treatments on top of the flooring and live loads should relate to the classification of the activity or occupancy.

4.4.2 Indicative floor-joist span to depth ratios

For preliminary sizing, the following span-to-depth ratios provide some guidance on system depth with live loads of 2 kPa to 3 kPa. They are all for simply supported configurations for spans up to around 4.8 m. For longer spans, slightly lower span to depth ratios should be used:

- Cassettes using sawn timber, Glulam, LVL – span/depth = 18 to 19
- Cassettes using I-shaped Beams and trusses – span/depth = 15 to 18.

4.4.3 Methods to join floor cassettes

There are a number of ways to join floor cassettes. An effective join provides:

- continuity of the floor plane under vertical loads (deflection of one cassette should not leave a step against an adjacent cassette)
- load transfer within a floor diaphragm - the connection transmits shear and lateral loading from one cassette to its neighbour, in some cases drag straps can assist in transferring load to bracing elements (see Section 7.6)
- load paths for alternate structural mechanisms in the event of accidental damage to structural elements (see Section 8).

The following section discusses the advantages and disadvantages of the different joining methods, though it should be noted that floor cassette manufacturers may have their own preferred methods or specifications.

Abutting cassettes

This method involves locating an additional joist along the edges of the cassette on both sides. The floor sheet extends over one side and is recessed half the joist width on the opposite side to enable the flooring on one cassette to be fixed to the joist on the neighbouring cassette (see Figure 4.6). This method incurs additional costs for double joists at every cassette connection. However, it provides the opportunity to fasten edge joists together directly, giving a solid connection between cassettes. The overhang of the floor sheet is relatively small, which does not interfere with lifting the cassettes into place and minimises damage to floor sheets during installation. Note that the overhang can be vulnerable during handling.

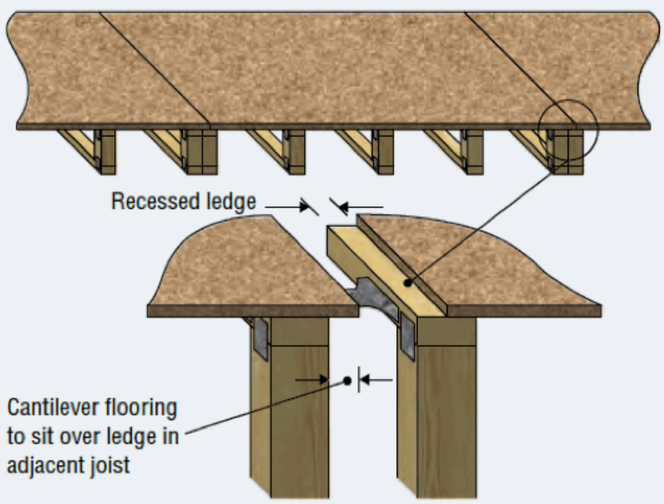


Figure 4.6: Abutting cassettes.

Full cantilevered flooring

This method is similar to the abutting method described above but removes one of the floor joists and as a result has a much larger cantilevered flooring section along one edge, which is usually the width of a standard joist spacing (see Figure 4.7). This reduces the number of joists required in the floor, however extra care is required in transport and handling these cassettes to avoid damaging the overhanging flooring.

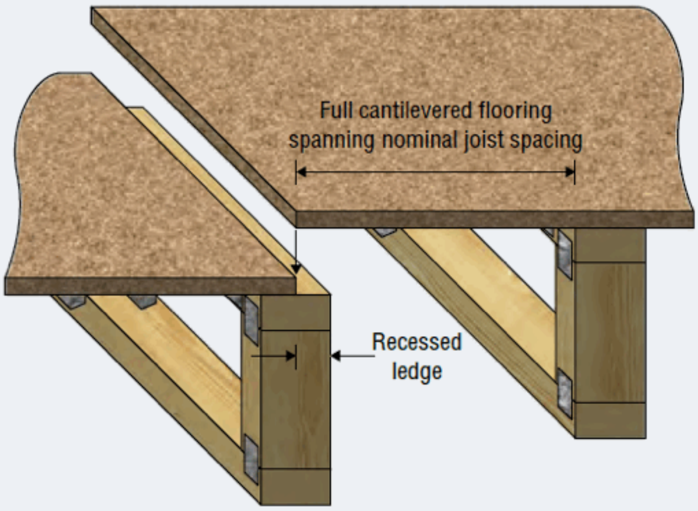


Figure 4.7: Full cantilevered flooring.

Infill flooring

This method of connecting floor cassettes involves installing a narrow floor sheet between the two floor cassettes, after they have been installed (see Figure 4.8).

This system is used:

- at discontinuous wall systems where continuity is needed for diaphragm action transfer
- where cantilevered flooring may be at risk of damage during transport and handling
- when the floor cassette layout is not square or
- if tolerances are required to correct the alignment of the cassettes.

The infill flooring sheet forms a simple span between edge joists in adjacent cassettes. Single span sheeting requires closer supports than the continuous spanning sheeting in the remainder of the cassette, which means that the edge joists are closer together than for full cantilevered flooring. The infill flooring requires on-site measuring and cutting and double the number of connectors, which may slow down the installation of the cassettes.

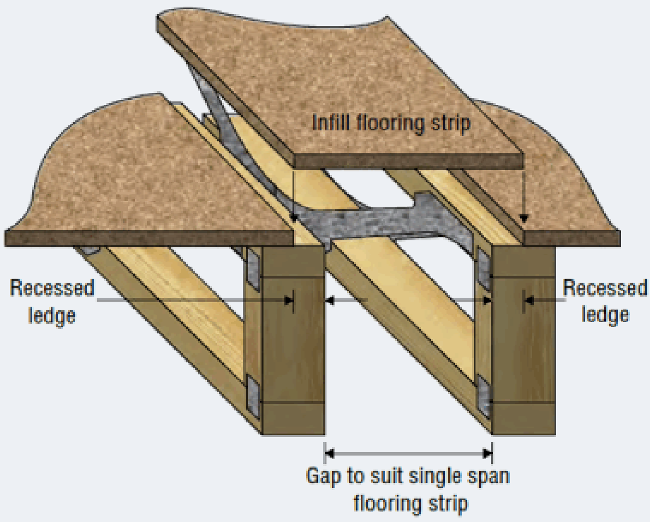


Figure 4.8: Infill flooring cassette.

4.4.4 Strong-backs

Strong-backs are beams that act as a secondary support to primary members, running perpendicular to the trusses in the floor cassette (see Figure 4.9). They act as a replacement for blocking between joists, help keep the floor level, share load between joists, and provide strong lifting points. Strong-backs are usually sawn or engineered timber elements placed within the floor cassette and then bridged on-site.

Figure 4.9: View inside a floor cassette. Note floor trusses, strong-back, edge rim-beam and also insulation. (Image TPC Solutions)

4.4.5 Set downs

Set downs in the floor level are required for wet areas. They can be designed and fabricated by the floor cassette fabricator (see Figure 4.10). Set downs need to be considered with the performance of diaphragm action.are closer together than for full cantilevered flooring. The infill flooring requires on-site measuring and cutting and double the number of connectors, which may slow down the installation of the cassettes.

Figure 4.10: Set down formed in floor truss. (Image: TDA)

4.4.6 Cassette ends

The bearing detail at the end of the cassette must allow for the transfer of vertical loads to the bearing elements. Normally they are designed by the cassette manufacturer, but there will need to be communication about the location and type of the bearings.

The ends of the cassettes must also have sufficient strength to carry and vertical wall loads above between floors, any diaphragm forces within the floor system and provide the connection required by robustness provisions. For vertical wall loading, solid rim-boards or solid blocking between joist members is generally used. In a structure utilising horizontal steel beams blocking may not be necessary (see Figure 4.11).

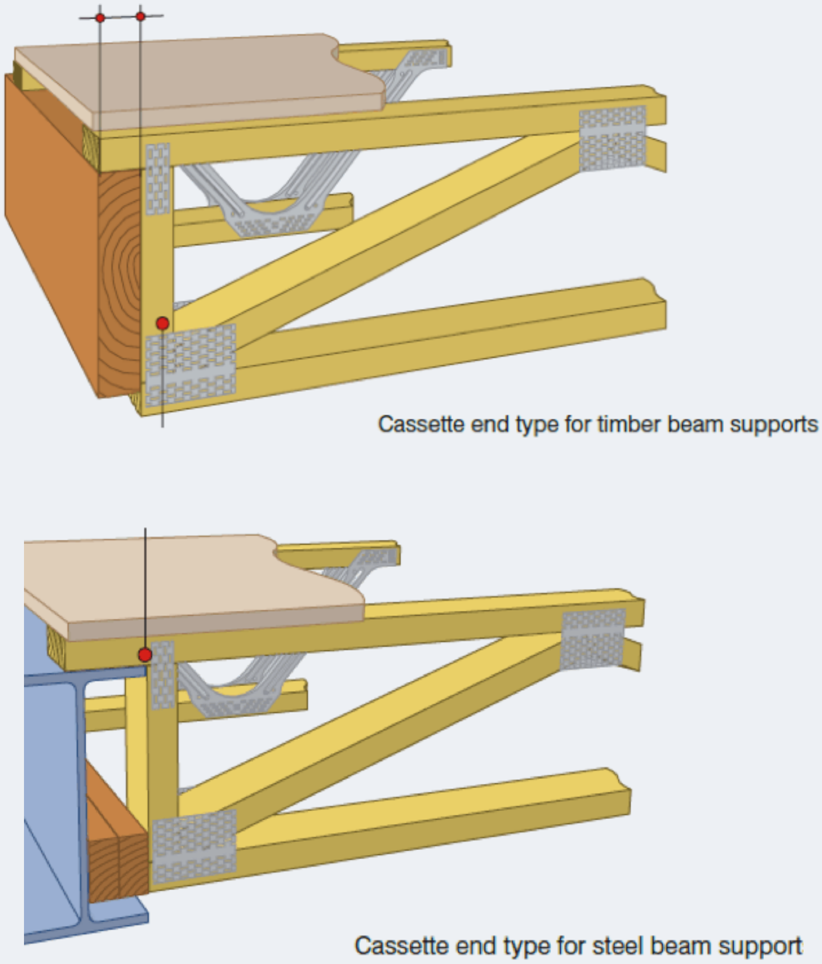


Figure 4.11: Floor cassette bearing details. (Images: Pryda Floor Cassette Guide)

4.5 Detailed design of lightweight timber framed floor systems

Where lightweight prefabricated timber cassettes utilising floor trusses are to be used, the final detailed design of the floor cassette systems will typically be by the frame and truss fabricator. Where this is not the case, the following process is suggested.

4.5.1 Checking performance of a floor system

- Adopt an initial member size based on suggested span to depth ratios.
- *Deflection checks* – use client's acceptance criteria for deflection. If none have been supplied, Table 2.6 suggests parameters to check deflections.
- *Strength checks* – Equation 1.1 (Cl. 3.2.1.1 in AS 1720.1) is used to check the bending capacity of joists.
- *Vibration checks* – The floor vibration response is not directly referred to in Australian Standards, however the three checks indicated in Section 2.3.2 are considered to provide satisfactory performance for typical systems. The three indicative vibration checks are included below.

4.5.2 Checking vibration performance – framed or cassette floors

Analysis for acceptable dynamic performance of floor systems is critical as floor spans increase and performance criteria become more onerous. Timber floor dynamic performance is a developing field. There are many methods for analysis and many forms of acceptance criteria internationally. The three typical methods for dynamic check are presented below. For increased confidence these methods should be cross-referenced.

Maximum deflection under serviceability short-term imposed actions

Table 2.6 load case (b) should be checked against a deflection limit of the minimum of span/360 or 9 mm. This is a deflection check, but it also gives an indication of likely vibration performance.

Point load at centre span

Table 2.6 load case (c) should be checked against a deflection limit of 1.5 mm. This limit is based on Australian practice; the UK limit is C_a (see *Equation 4.3*). The smaller of the two limits gives the best vibrational performance. This load case requires the evaluation of g41 using Cl. E8.2 in AS 1720.1.to calculate the load applied to a single joist. (g41 x 1 kN)

$$g_{41} = 0.883 - 0.34\, log_{10}\left(\frac{n_f h_f}{h_j} + 0.44\right) \qquad (4.1)$$

$0.2 \le g_{41} \le 1.0$ i.e. g_{41} has a minimum value of 0.2 and a maximum of 1.0.

With:

n_f = number of flooring elements crossing the joist

h_f = $\frac{E_f I_f}{S^3}$

h_j = $\frac{E_j I_j}{L^3}$

E_f = Modulus of elasticity of flooring in the direction at right angles to the floor joists

If = Second moment of inertia of a single flooring element

S = Spacing of floor joists

E_j = Modulus of elasticity of joists

I_j = Second moment of inertia of a single joist

L = Span of floor joists

For trusses and I-beams, $E_j\, I_j$ is the flexural stiffness of the element from the manufacturer's information.

Eurocode 5 dynamic vibration check

EC 5 provides some models for checking the vibration performance of floors. Some are very close to the two checks outlined above. However, there is a dynamic check that involves some acceptance parameters defined in national standards.

The following steps are recommended:

- Use Equation 4.2 to estimate the natural frequency of the floor.

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{EI_j}{m}} \qquad (4.2)$$

With:

L = span of the floor joists in m

EI_j = $\frac{E_j I_j}{L^3}$ in Nm^2/m

E_j = Effective modulus of elasticity of joists

I_j = Second moment of inertia of a single joist

S = Spacing of floor joists

m = mass per unit area of the completed floor (does not include partition or furniture weight) in kg/m^2.

Check that the natural frequency is greater than 8 Hz. If it is not, the floor should be stiffened to improve vibrational response. (Note that the mass required for acoustic performance can work against good vibrational performance, so it is better to balance these two conflicting requirements by changing the stiffness of the floor rather than the mass.)

- Estimate the acceptance criteria for timber floors. Equations 4.3 and 4.4 have been taken from the UK annex to EC5.

C_a = 1.8 mm for L < 4 m (EC5 uses '*a*' for this parameter) = $\frac{8.27}{L^{1.1}}$ mm for L > 4 m (4.3)

$0.2 \leq g_{41} \leq 1.0$ i.e. g_{41} has a minimum value of 0.2 and a maximum of 1.0.

With:

L = Span of floor joists in m

C_b = 180 – 60*a* for C_a < 1 mm (EC5 uses '*b*' for this parameter) = 160 – 40*a* for C_a < 1 mm (4.4)

With:

C_a = Acceptance criteria from Equation 3.3 in mm

Estimate the acceptance criteria for timber floors. Equations 4.3 and 4.4 have been taken from the UK annex to EC5.

$$n_{40} = \left(\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{B}{L}\right)^4 \frac{(EI)_j}{(EI)_f}\right)^{0.25} \quad (4.5)$$

With:

f_1 = natural frequency of the floor (first mode)

B = width of the floor panel in m

L = span of the floor joists in m

EI_j = $\frac{E_j I_j}{S}$ in Nm²/m (flexural stiffness of joists)

E_j = Effective modulus of elasticity of joists

I_j = Second moment of inertia of a single joist in m^4

S = Spacing of floor joists

EI_f = $\frac{E_f I_f}{1}$ in Nm²/m (flexural stiffness of flooring); this value can be increased by the flexural stiffness of any strongbacks used in the cassettes

E_f = Effective modulus of elasticity of flooring

I_f = Second moment of inertia of one metre width of flooring in m^4/m

- Calculate the unit impulse velocity response using Equation 4.6.

$$v = \frac{4(0.4+0.6n_{40})}{mBL+200} \quad (4.6)$$

With:

n_{40} = number of vibration modes with natural frequency < 40 Hz

m = mass per unit area of the completed floor (does not include partition or furniture weight) in kg/m²

B = width of the floor panel in m

L = span of the floor joists in m

- Check that $v < C_v$ the acceptance criteria given in Equation 4.7.

$$C_v = C_b^{(f_1\zeta - 1)} \qquad (4.7)$$

With:

C_b = acceptance criteria given in Equation 4.4

f_1 = natural frequency of the floor (first mode)

ζ = damping coefficient (use 0.02, i.e. 2% damping)

In general, if the dynamic performance of the floor fails any of the criteria, improved performance can be achieved by increasing the flexural stiffness of the floor joists.

4.6 Preliminary design of mass-timber floors for vertical loads

4.6.1 Indicative span to depth ratios

For preliminary sizing, the following span-to-depth ratios provide some guidance for different mass timber system floor depths:

- CLT floors with
  - 3-layer panels – span/depth = 26 to 28
  - 5-layer panels – span/depth = 25 to 27
  - 7-layer panels – span/depth = 22 to 23
- Mass LVL floor panel – 28 to 35
- LVL closed cassette system – 25 to 30
- LVL open cassette system – 20 to 25
- Glulam panels – 25 to 28
- NLT panels – 20 to 25.

4.6.2 Cross-Laminated Timber (CLT)

Due to the cross-laminating composition of CLT it has a principal spanning direction parallel to the outer laminates face grain, and effectively also a secondary spanning direction at right angles to this due to the cross-laminates (see Figure 4.12). As a floor, CLT is generally used as either simply-supported or continuous spanning plates in the principal spanning direction.

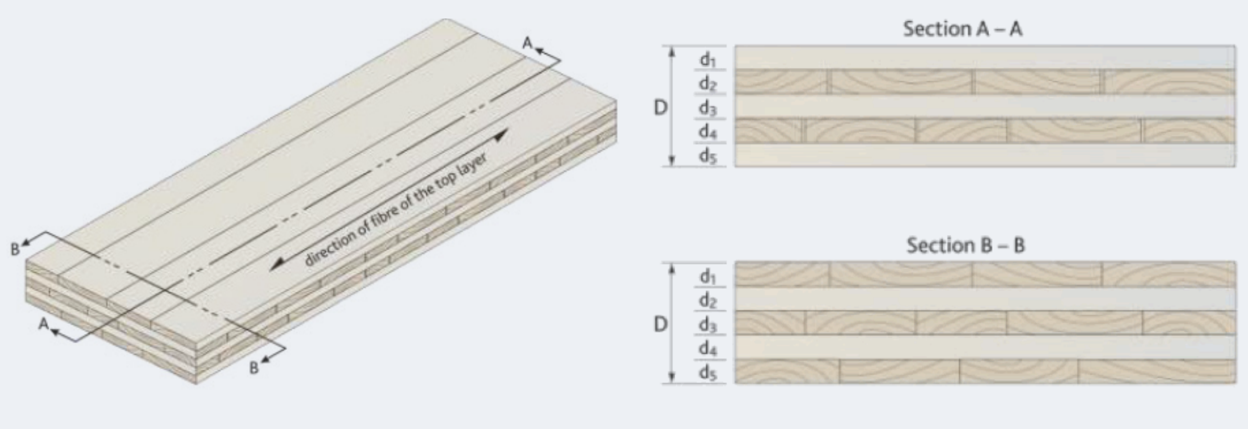


Figure 4.12: Two-way spanning capacity of CLT.

Manufacturers provide information for their products, including:

- material structural properties
- structural design data for floor systems including strength, deformation and vibration
- acoustic properties of the panels and system
- fire resistance test results.

Designers should utilise an early supplier involvement (ESI) approach and consult with manufacturers in the preliminary and detailed design phase to ensure selection of the most cost-effective panel lengths and widths. Panel sizes may be limited by manufacturing constraints or transportation limits. Cut-outs for shafts and services can be included in the fabricated panels.

AS 1720.1 does not contain any direct information specific to the design of CLT panels, however, it can be utilised along with design resources available through CLT manufacturers. Also refer to international resources such as the Canadian FP (Forest Product) Innovations' CLT Handbook or those listed in Section 1.7.8.

In terms of design approaches:

- Transformed sections are used to calculate an effective stiffness (EI) and moment capacity. Material properties are reduced for layers in which the grain direction is perpendicular to the flexural stresses (typically, E90 = 1/30 E0). A number of different transformed section analysis methods are available; the Shear Analogy Method is accepted as the best to model both strength and serviceability under out-of-plane loads.
- Floor design incorporates out-of-plane loading on a plate for CLT. Flexural strength may be limited by rolling shear in the layers where the grain direction is perpendicular to the flexural stresses. Shear capacity should always be checked using the shear strength for rolling shear (typically 0.4 times the normal shear strength).

4.6.3 Laminated Veneer Lumber (LVL)

LVL manufacturers provide information on the specific properties and design approaches for their particular products. AS 1720.1 Section 8 includes provisions for the design of LVL members. They are normally designed as a one-way spanning floor plates; however, performance for both strength and serviceability may be underestimated as there may be some two-way action in the panels.

LVL can be cross-banded with some veneers in the build-up perpendicular to the majority. Cross-banding increases dimensional stability and perpendicular–to-grain tensile capacity, while slightly reducing parallel to grain properties. Perpendicular veneers introduce a layer of lower strength and stiffness in the panel resistance to longitudinal shear. Standard LVL panels do not have problems with rolling shear and can be modelled as plate elements. However, the cross bands in cross-banded LVL introduce rolling shear and a transformed section analysis is required.

Manufacturers provide assistance with detailed design of LVL plate floor systems, including the design of connections between the plates and with other structural elements.

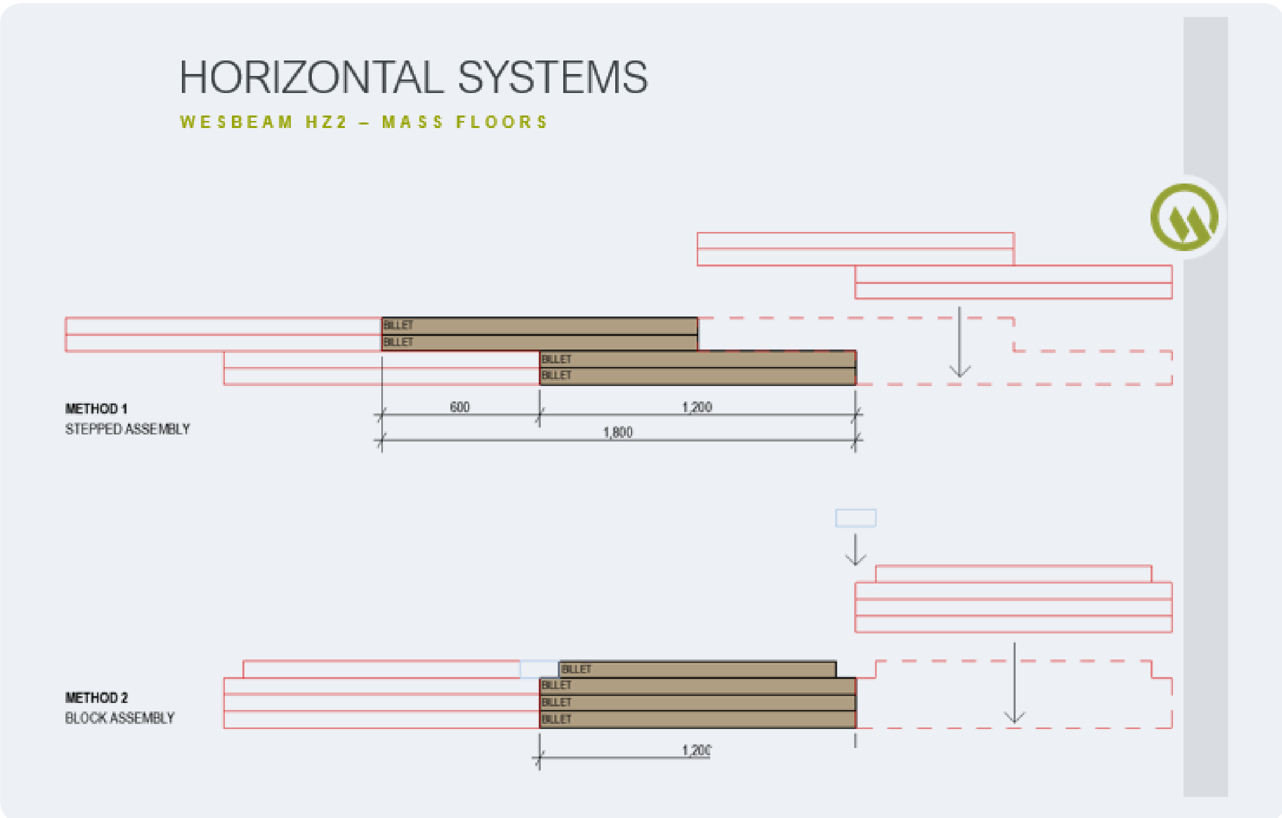


Figure 4.13: Possible joining arrangements for LVL mass floor panels. (Image: Wesbeam)

4.6.4 Nail Laminated Timber (NLT)

NLT floor panels can be manufactured from lower structural graded timber that, due to the mass nature-redundancy impacts in the system, can still provide appropriate structural capacity. There are two main NLT configurations:

- Flat NLT, made from boards of the same nominal size and has flat upper and lower surfaces.
- Staggered NLT, made from boards of two different widths that are alternated in the layup. An uneven upper surface gives a better bond with a topping. An exposed uneven lower surface gives less reverberation in the room below.

NLT has not often been used for floors in Australia but has been used as floor panels in North America and Europe for many years, particularly under concrete toppings where the use of different size boards can leave an uneven surface that bonds better with the concrete.

There are few commercial manufacturers of NLT, but NLT panels can be fabricated using sawn timber either on-site or in prefabrication facilities. Design information for NLT is available from www.thinkwood.com/products-and-systems/nail-laminated-timber-nlt-guide. NLT can be modelled as bending members with ncom > 10 in the evaluation of k9. (See Section 1.7.1)

4.6.5 Methods to join mass-timber floor panels

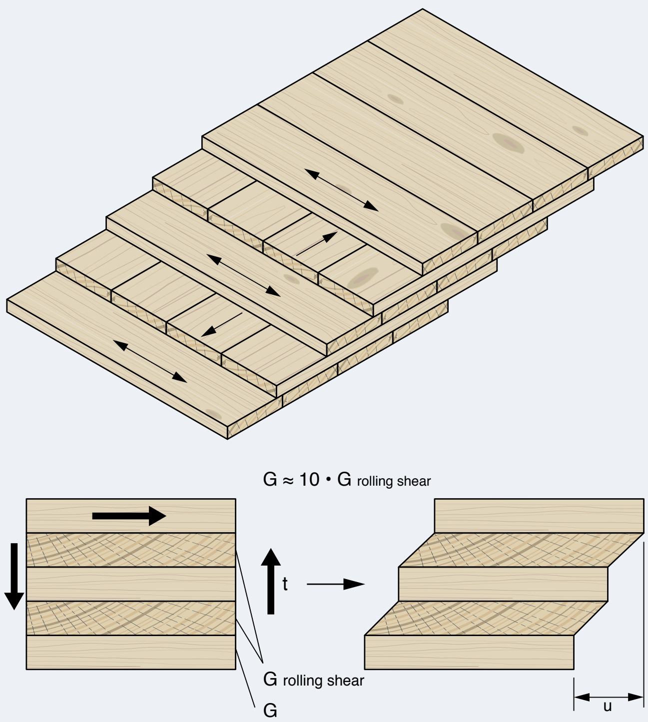
There are a number of ways to join floor mass-timber floor panels. An effective join provides:

- Continuity of the floor plane under vertical loads. Deflections of one panel should not leave a step against an adjacent panel.
- Load transfer within a floor diaphragm. The connection transmits shear from one panel to its neighbour. In some cases, drag straps can assist in transferring load to bracing elements (see Section 7).
- Load paths for alternate structural mechanisms in the event of accidental damage to structural elements (see Section 8).

See Section 7.6.2 for examples of connections between mass-timber floor panels.

4.7 Detailed design of CLT floor panels

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 the design of CLT. Rolling shear can contribute significantly to a panel's deflection under bending due to the shear deformation of the transverse layers.



Rolling shear deformation of a 5-layer CLT panel

Figure 4.14: Rolling shear deformation of a 5-layer CLT panel.

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).

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.

4.7.1 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 have integrated a CLT module into their RFEM software which provides the structural analysis for CLT.

Another important document has been released by FPInnovations in Canada, called the CLT Handbook (Karacabeyli & Douglas 2013). It provides comprehensive documentation of the manufacturing, design and construction of CLT.

This guide considers four methods for calculating the strength and serviceability properties for timber: CLTdesigner, gamma, composite k and shear analogy. 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). In such situations, the designer can 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).

A summary of these methods as well as the material and capacity factors from AS1720.1 (2010) is given in Table 4.2.

Table 4.2 Summary of available methods for determining design of CLT.

Method	Design Process	Properties calculated
AS1720.1 (Standard 2010)	Factors for bending Factors for shear Factors for bearing	AS1720.1 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}

4.7.2 Strength design for CLT

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.

Strength approach using AS1720.1

Bending check using AS1720.1

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 AS1720.1 does not provide guidance for CLT.

$$M_d = \varphi k_1 k_4 k_6 k_9 k_{12} f_b' Z_{CLT} \tag{4.8}$$

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. Research to conduct what the sharing factor should be for CLT. In accordance with AS 1720.1 CL2.4.5.3 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 CLTdesigner Method

Shear check using 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 3.6.2.2 is recommended to calculate the shear plane area.

$$V_d = \varphi k_1 k_4 k_6 f_s' A_s \tag{4.9}$$

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 CLTdesigner Approach

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/mm2 and 0.7 N/mm^2 respectively. 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.

Bearing check using AS 1720.1

To calculate the bearing strength the area of applied load, A_p, and the characteristic bearing strength of the timber, f_p' is required. The strength of bearing calculated as:

$$N_{d,p} = \varphi k_1 k_4 k_6 k_7 f_p' A_{pi} \tag{4.10}$$

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 Approach

f_p' For CLT is given as 2.85 N/mm^2 (Thiel, 2014).

The other modification factors are the same as those for bending strength.

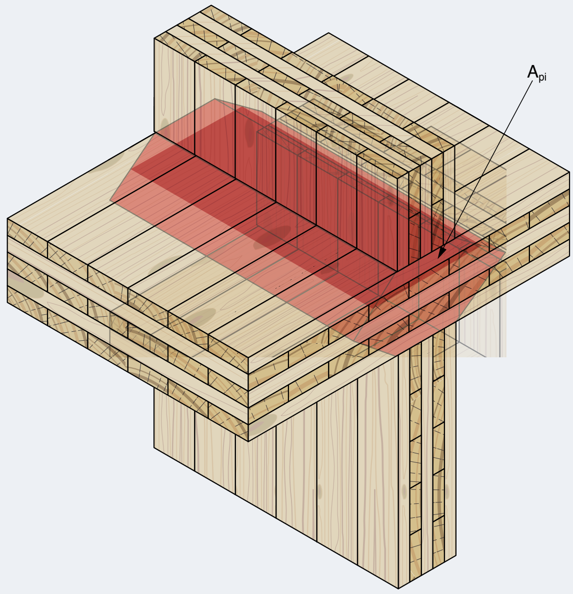


Figure 4.15: Bearing area of the floor for a CLT wall/floor transition.

Strength approach using CLTdesigner

Bending strength check using CLTdesigner

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 orientated such 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 4.16 and the bending stiffness, K_{CLT} is calculated using Equation 4.11.

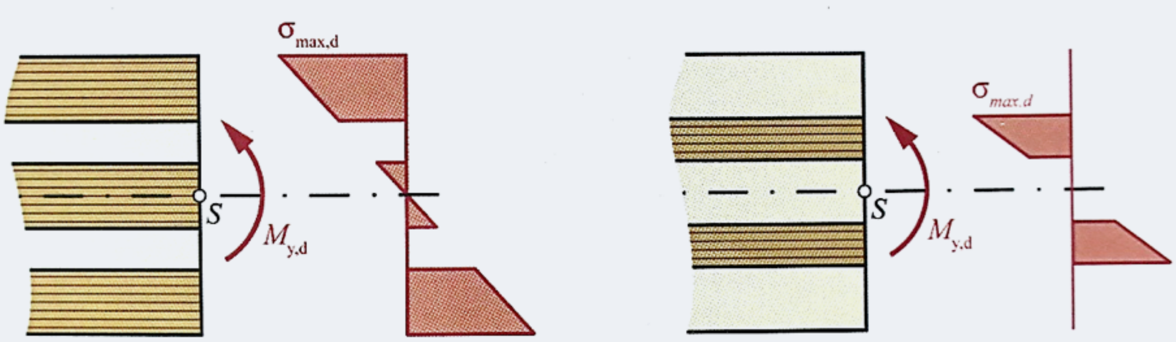


Figure 4.16: The normal stress distribution of a CLT panel for the bending moment of a floor panel for longitudinal bending (left) and transverse bending (right). (Thiel 2013)

$$K_{CLT} = \sum(E_i I_i) + \sum E_i A_i z_i^2 \tag{4.11}$$

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 ith 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 4.12.

$$\sigma_{max,d} = \frac{M_{y,d}}{K_{CLT}} \frac{t_{tot}}{2} E_1 \quad (4.12)$$

t_{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 2nd based 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.

$$f_{m,CLT,k} = k_{m,CLT} f_{t,0,1,k}^{0.8} \quad (4.13)$$

$k_{m,CLT}$ is equal to 3 for timber with a tensile strength CV of 25% and 3.5 for a CV of%

$f_{t,0,1,k}$ is the characteristic tensile strength of the timber

Rearranging Equations 4.13 and 4.14, we find an expression for the design bending moment My,d:

$$M_{y,d} = \frac{2K_{CLT}}{t_{tot}E_1} f_{m,CLT,k} \quad (4.14)$$

Equation 4.14 is in the form of Euler Bernoulli's beam theory, $M_{y,d} = f Z_{CLT}$, where Z_{CLT} is the section modulus for cross laminated timber calculated using Equation 4.15. 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 AS1720.1.

$$Z_{CLT} = \frac{2K_{CLT}}{t_{tot}E_1} \quad (4.15)$$

Shear strength check using CLTdesigner

CLTdesigner uses the classical procedure for unidirectional layered cross sections to calculate the shear stress distribution given by Equation 4.16. The assumption that $E_{90} = 0$ means that there is no shear stress increase in the cross layers as shown in Figure 4.17.

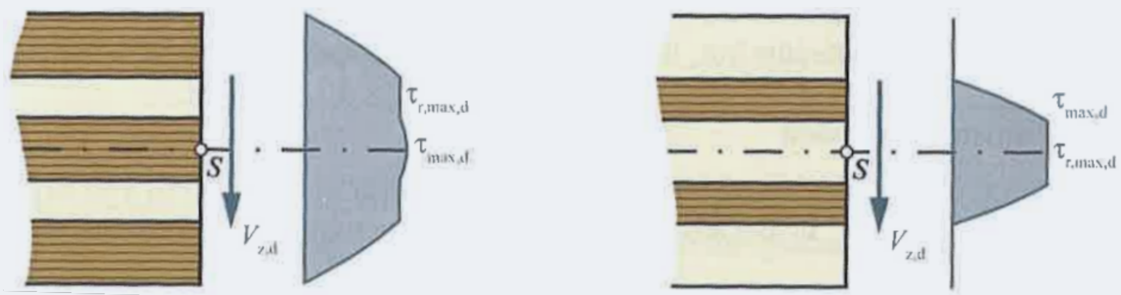


Figure 4.17: The shear stress distribution in a CLT cross section for shear caused by longitudinal bending (left) and transverse bending (right). (Thiel 2013)

$$\tau = \frac{V_Z \int_A E(z).z.dA}{K_{CLT}.b(z_0)} \quad (4.16)$$

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 4.17.

$$\frac{\tau_{max,d}}{f_{v,CLT,d}} \leq 1.0 \text{ and } \frac{\tau_{r,max,d}}{f_{r,CLT,d}} \leq 1.0 \quad (4.17)$$

$$V_{mid} = f_{v,CLT,d}\frac{K_{CLT}}{\left(E_1t_1z_1+\frac{E_3t_3^2}{8}\right)} \quad (4.18)$$

$$V_{rolling} = f_{r,CLT,d}\frac{K_{CLT}}{(E_1t_1z_1)} \quad (4.19)$$

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 4.18 and 4.19 the effective area for shear strength at the mid-point of the section and shear at the transfer layers are given by:

$$A_{eff,mid} = \frac{K_{CLT}}{\left(E_1t_1z_1+\frac{E_3t_3^2}{8}\right)} \quad (4.20)$$

$$A_{eff,rolling} = \frac{K_{CLT}}{(E_1t_1z_1)} \quad (4.21)$$

K_{CLT} is the bending stiffness

E_1,t_1 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,t_3 are the elastic modulus and thickness of the middle layer of a 5-layer CLT panel.

Bearing strength using CLTdesigner

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. The appropriate multiplying factors to account for the location of bearing, k_7, are shown in Table 4.3.

Table 4.3: 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

Strength approach using gamma method (Europe)

Bending strength using gamma method

The gamma method has been developed from mechanically jointed beam theory and is detailed in Eurocode 5. It is also detailed in the CLT Handbook by FP Innovations. 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 (γ_i) which is calculated using Equation 4.23. The effective stiffness can then be determined using Equation 4.22.

$$EI_{eff} = \sum_{i=1}^{n}\left(E_iI_i + \gamma_iE_iA_iz_i^2\right) \quad (4.22)$$

$$\gamma_i = \frac{1}{1+\pi^2\frac{E_iA_i}{l^2}\cdot\frac{t_1'}{G_Rb}} \quad (4.23)$$

$$\gamma_i = \frac{1}{1+\pi^2 \frac{E_i A_i}{l^2} \cdot \frac{t_1'}{G_R b}} \tag{4.23}$$

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 4.24 and can be used to determine the bending moment capacity of CLT.

$$Z_\gamma = Z_{CLT} = \frac{(EI)_{eff}}{E_1(\gamma_1 z_1 + 0.5 t_1)} \tag{4.24}$$

z_1 is the distance from the centroid of the cross section to the centroid of the outer layer.

Shear strength using gamma method

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:

$$A_{eff,mid} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1 z_1 + E_1' A_1' z_1' + \gamma_2 E_2 \frac{A_2 t_2}{2\ 4}\right)} \tag{4.25}$$

$$A_{eff,rolling} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1\left(z_1 - \frac{t_2}{2}\right) + E_1' A_1'\left(z_1' - \frac{t_2}{2}\right)\right)} \tag{4.26}$$

Strength approach using composite theory

Bending strength using composite theory

Composite theory was developed for calculating the bending strength of plywood. From composite theory, the design bending moment is calculated using Equation 4.27. Where 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 4.28. Any shear deformation is not considered using the k-method.

$$M_{y,d} = \varphi k_1 f_{b,eff} S_{gross} \tag{4.27}$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right)\left(\frac{a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3}{a_m^3}\right)\right] \tag{4.28}$$

The value for a_m is shown in Figure 4.18. 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 $E_{90} = E_0/30$.

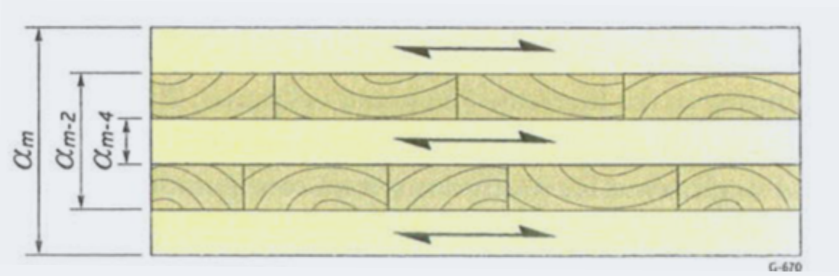


Figure 4.18: Cross section values for calculation of value k using composite theory.

The section modulus for bending strength for composite theory is given by:

$$Z_k = Z_{CLT} = k_1 S_{gross} \tag{4.29}$$

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.

Strength approach using the shear analogy method

Section properties using 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 4.30 calculates the true bending stiffness, where the values for BA and BB for the two virtual beams, are given in Equations 4.31 and 4.32, respectively.

$$EI_{eff} = B_A + B_B \tag{4.30}$$

$$B_A = \sum_{i=1}^{n} E_i I_i \tag{4.31}$$

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2 \tag{4.32}$$

The shear stiffness of the beam is considered for this method and is calculated using:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{t_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i b_i}\right) + \left(\frac{t_n}{2G_n b}\right)\right]} \tag{4.33}$$

Where:

$$a = t_{total} - \frac{t_1}{2} - \frac{t_n}{2} \tag{4.34}$$

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 presented by the CLT Handbook by FPinnovations presents a simplified method to calculate the bending moment capacity where the section modulus is given by the following:

$$Z_{simp} = Z_{CLT} = \frac{(EI)_{eff}}{0.5 E_1 t_{tot}} \tag{4.35}$$

4.7.3 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 $EI_{eff.}$ The shear analogy method calculates the deflection due to shear slip.

Gamma method

The gamma method is straight forward to implement after the EI_{eff} has been calculated using Equation 4.22. 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:

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} \tag{4.36}$$

Composite method

The composite method is straight forward to implement with the composition factor, k_1 calculated using Equation 4.37. 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:

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1E_0I_{gross}} \tag{4.37}$$

Shear analogy method

The maximum deflection in the middle of a uniformly loaded CLT slab using the shear analogy method is given by Equation 4.38. 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.

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} \tag{4.38}$$

k is a shear coefficient factor equal to 1.2 according to Timoshenko.

An example of the shear analogy method can be found in Appendix 2.

Long-term deflection

The factors for creep 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.

Estimates for the load duration factor varies, the j2 can be assumed to be 2 as per AS1720.1.

4.7.4 Vibration

The design of CLT floors for vibration performance is dependent on three aspects of design: (i) floor loads that cause the vibration response; (ii) response of the structure defined by the modal properties; and (iii) vibration perception/experience by the user measured using acceptability criteria, as shown in Figure 4.19.

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 4.19, 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.

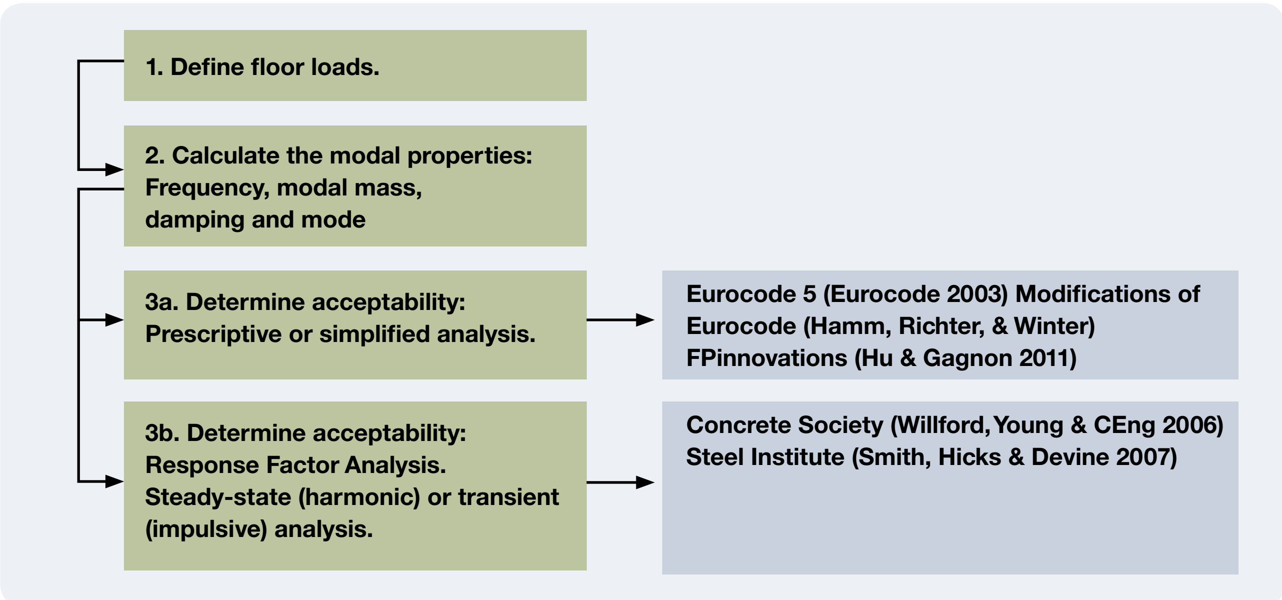


Figure 4.19: Summary of procedure to determine vibration performance of a floor.

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 and 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 (2.5 Hz x 4) are considered resonant response floors while floors over 10 Hz are considered to have a transient response.

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 for one-way spanning CLT

The closed form solution for the natural frequency of a simply supported beam is given by Equation 4.39. The natural frequency is proportional to the ratio of the stiffness of the structure to the modal mass.

$$f_j = \frac{j^2\pi^2}{2\pi l^2}\sqrt{\frac{EI}{m}}$$

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 4.40. Values for K are given in Table 4.4.

$$f_j = \frac{K}{2\pi l^2}\sqrt{\frac{EI}{m}}$$

Table 4.4: 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 4.4 and has some sort of partial restraint, the beam can be represented as a symmetrically elastically supported beam shown in Figure 4.20. Advanced computations are required to calculate the K values for this type of beam.

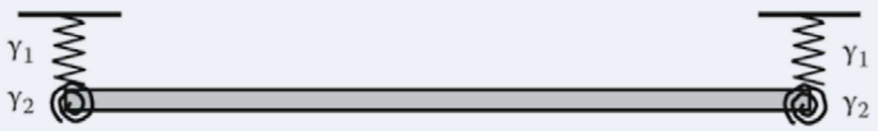


Figure 4.20: Beam with symmetrically elastically restrained ends (Wang & Wang 2013).

Frequency for two-way spanning CLT

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 at University of Technology Sydney (UTS) 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. 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 4.41. In Equation 4.41, D is the flexural rigidity defined in Equation 4.42 and the dimensions of the plate are shown in Figure 4.21.

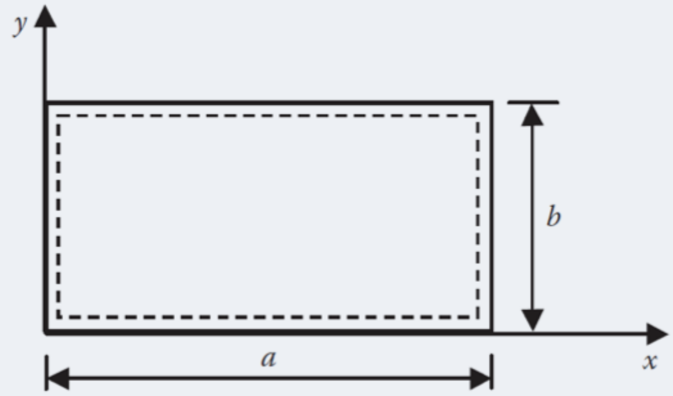


Figure 4.21: Coordinates and dimensions of two-way spanning plate.

$$f_{m,n,iso} = \frac{\pi}{2}\sqrt{\frac{D}{\rho t}}\left[\left(\frac{m}{a}\right)^2 + \left(\frac{n}{b}\right)^2\right]$$

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

$$D = \frac{Eh^3}{12(1-\upsilon^2)}$$

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 4.43 includes the orthotropic effect of CLT. Equation 4.44 gives the flexural and torsional rigidity.

$$f_{n,n,ortho} = \frac{\pi}{2a^2}\sqrt{\frac{m^4 D_x + 2Hm^2n^2\left(\frac{a^2}{b^2}\right) + n^4 D_y\left(\frac{a^4}{b^4}\right)}{m}}$$

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

$$D_x = D_y = H = \frac{Et^3}{12(1-\upsilon^2)}$$

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.

Modal mass and modal 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 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, there should be a unity normalised measurement 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 4.45.

$$M_n = \frac{KE_n}{2\pi^2 f_n^2} \tag{4.45}$$

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 conducted at UTS found one-way spanning, simply supported CLT had a damping ratio for the first mode of 0.5–1.5%. Subsequent modes did not vary significantly.

The damping is affected by the configuration, material, support conditions and the loading type. The occupant of a floor can change the damping characteristics. On-site damping has been reported to be higher than in laboratory experiments. A study 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.

4.7.5 Simplified vibration 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 4.5.

Table 4.5: Comparison of available analytical models for determining vibration performance.

Vibration Performance Method	Stiffness (Unit Displacement)		Floor Natural Frequency		Floor Velocity		Acceleration (floors under 8 Hz only)	
	Load kN	Limit (mm)	Load Case	Limit (Hz)	Velocity	Limit	Frequency Range (Hz)	Limit m/s²
Eurocode 5	1	≤1	G_{TOT}	≥8	Eq 4.50	Eq 4.49		
Hamm et al	2	≤0.5	G_{TOT}	≥8			4.5 - 8	≤0.05
Mohr	1	≤1	G_{TOT}+0.3Q	≥8	Eq 4.54	Eq 4.54	3.4 - 8	≤0.1
CLT Handbook	1	*	GTOT	*				

* According to the 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) require 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 research 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 4.46, 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 4.4 can be used.

$$f_1 = \frac{k_m}{2\pi l^2}\sqrt{\frac{(EI)_l}{m}} \geq 8\ Hz \tag{4.46}$$

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 4.47) 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) (Figure 4.22). 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 4.48 (Mohr, 1999).

$$w_{EC5} = \frac{1}{48}\frac{Fl^3}{EI_b} \leq a\ mm/kN \tag{4.47}$$

$$b_{eff} = \frac{l}{1.1}\sqrt[4]{\frac{EI_t}{EI_l}} \tag{4.48}$$

The velocity (v) due to an impulse of 1Ns is then calculated using Equation 4.49 and limited by Equation 4.50. Only the number of first order modes with natural frequencies up to 40 Hz is considered and calculated using Equation 4.51. A value for damping, ζ = 1%, is provided by the code.

$$v \leq b^{(f_1\zeta-1)}\ m/(Ns^2) \tag{4.49}$$

$$v = \frac{4(0.4+0.6n_{40})}{mbl+200} \tag{4.50}$$

$$n_{40} = \left\{\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{b}{l}\right)^4 \frac{(EI)_l}{(EI)_b}\right\}^{0.25} \tag{4.51}$$

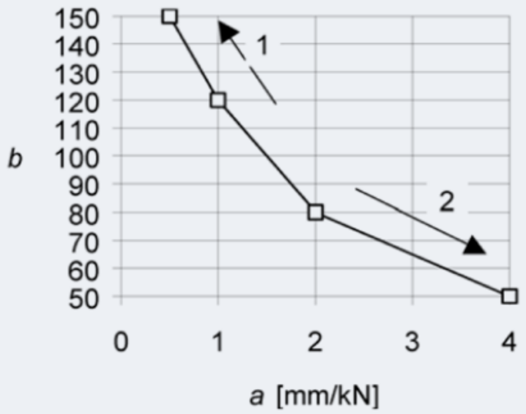


Figure 4.22: Interaction between the limit values of a, and b; directions 1 and 2 correspond to better and worse behaviour respectively (EC5).

Modifications on Eurocode 5

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, 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. A flow chart that outlines the design procedure is shown in Figure 4.23.

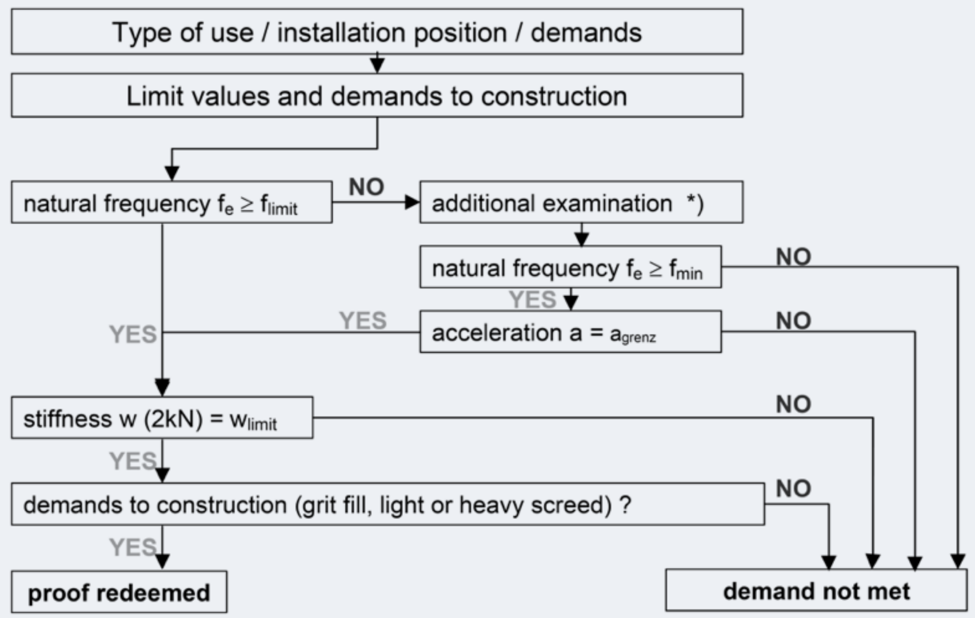


Figure 4.23: Flow chart for the design and construction of timber floors, the additional examination only applies for heavy floors with wide spans, or timber concrete composite systems (Hamm et al. 2010).

The frequency is calculated using the same method as Eurocode 5 considering only the static mass of the floor. The stiffness criterion is 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 rather than 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 3.53 and is limited to 0.05 m/s^2. In Equation 4.53, 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 F_F are given in Table 4.6. The generalised mass, M_{gen}, is equivalent to half the effective area contributing to vibration performance (Equation 4.53) 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.

$$a \approx 0.4\frac{P_0\alpha_i(f_1)}{M_{gen}}\frac{1}{\sqrt{\left[\left(\frac{f_1}{f_F}\right)^2-1\right]^2+\left(2D\frac{f_1}{F_F}\right)^2}} \leq 0.05\ m/s^2 \qquad (4.52)$$

$$M_{gen} = m\ \frac{l}{2} b_{eff} \qquad (4.53)$$

Table 4.6: Fourier coefficient, dependent on the fundamental frequency of the floor (Mohr, 1999).

Fundamental Frequency Hz	Fourier coefficient	Forcing frequency F_F Hz
$3.4 < f_1 \leq 4.6$	0.2	f_1
$4.6 < f_1 \leq 5.1$	0.2	f_1
$5.1 < f_1 \leq 6.9$	0.06	f_1
$f_1 > 6.9$	0.06	6.9

See Appendix 2 for a detailed worked example using this approach.

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, which 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 4.54. A damping value of 1% is assigned to floors without any additional boarding for sound isolation as outlined by Mohr (1999).

$$v_{MOHR} = \frac{0.6}{m_f^{0.5} EI_l^{0.25} EI_t^{0.25}} < v_{lim,MOHR} = 6 \times 100^{(f\zeta - 1)} \qquad (4.54)$$

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/s2.

CLT Handbook

The 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 4.55 provides an inequality based on the fundamental frequency and the effective stiffness of the floor under a unit load.

$$\frac{f}{\Delta^{0.7}} \geq 13 \qquad (4.55)$$

The deflection is calculated considering a 1 m-wide CLT panel and the frequency is calculated considering static mass only.

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4.8 Acoustic toppings for floor systems

Extra layers of acoustic mats and overlay topping products will be required to achieve the appropriate acoustic performance. This is particularly important for floor impact sound in apartment buildings where higher levels of performance than minimum NCC values are often required by the market (see Section 1.6.2).

Decisions about the type of topping should be made by consultation between the acoustic designer, builder, design engineer and client. Floor system build ups should be checked by the acoustics engineer to ensure appropriate performance levels are achieved but for general structural engineering calculation an added mass of around 40 kg/m^2 should be initially assumed as this will significantly improve acoustic the acoustic performance of a timber system (some overlay option commonly available are shown in Table 4.7).

Table 4.7: Overlay topping options to achieve a 40 kg/m^2 system mass target.

Product	Density (kg/m^3)	Unit Thickness	Unit Mass (kg/m^2)	# of Layers in System	System Mass (kg/m^2)	System Thickness (mm)
Cement Screed Mix (w/ particle board for lightweight)	2000	40	80	1	80.0	40
Sand (dry) installed between battens with particle board top	1600	25	40	1	40.0	25
Promat System Panel	1100	18	19.8	2	39.6	36
JH Compressed Fibre Cement (CFC) Structural Flooring	1920	15	28.8	2	57.6	30
Particleboard	746	19	14.2	3	42.5	57
Knauff Brio Board	1100	18	19.8	2	39.6	36
Supaboard	1200	18	21.6	2	43.2	36
BGC Durafloor	1300	19	24.7	2	49.4	38
CSR Cemintel Compressed Sheet+	1625	18	29.3	2	58.6	36
Fiberock	950	16	15.2	3	45.6	48

Wet screed type toppings may need to be thicker than requirements for acoustic performance to prevent toppings from breaking up under service loads or curling as they cure, so this needs to be factored into the floor dead load design.

Most toppings will also require the use of a resilient acoustic mat between the timber flooring and the topping. The resilient mat is often made from rubber, is around 10 mm thick and has a density of around 1,000 kg/m^3.

4.9 Floor-celling system build-ups for fire and acoustic performance

Table 4.8 illustrates a number of floor-celling system build-ups. The different floor systems achieve impact ($L_{n,w}$ 50 to 55) and airborne ($R_w + C_{tr} > 50$) sound requirements 'recommended for apartments' by the Association of Australasian Acoustic Consultants, and meet Type A construction, satisfying the NCC fire requirements for timber mid-rise residential apartments. An indication of total system depth is also provided.

Table 4.8: Floor-celling system build-ups for fire and acoustic performance in apartment use.

	Timber framed	Mass Timber	Concrete	Timber Framed Floor	Mass Timber	Concrete
						
Floor finish	Hardwood floor	Hardwood floor	Hardwood floor	Carpet	Carpet	Carpet
Floor overlay topping	Mass (screed or sheets) of 40 kg/m^2 + 10 mm rubber mat	Mass (screed or sheets) of 40 kg/m2 + 10 mm rubber mat	3.0 mm Dunlop Thermacoustic underlay	Foam underlay	Foam underlay	Foam underlay
Structure	190 mm joists at 450 mm centres, 22 mm particleboard floor sheet	140 mm CLT	150 mm concrete	190 mm joists at 450 centres, 22 mm particleboard floor sheet	140 mm CLT	150 mm concrete
Ceiling	Resilient mounts channels (cavity 40 mm), 75 mm insulation (14 kg/m^3), 2 x 16 mm fire grade plasterboard	16 mm fire grade plasterboard, resilient mounted furring channel ceiling grid (cavity 67 mm), 75 mm insulation (14 kg/m^3) and 13 mm standard grade plasterboard	Resilient mounted furring channel ceiling grid (cavity 67 mm), 75 mm insulation (14 kg/m^3) and 13 mm standard grade plasterboard	Resilient mounts channels (cavity 40 mm), 75 mm insulation (14 kg/m^3), 2 x 16 mm fire grade plasterboard	16 mm fire grade plasterboard direct fixed to CLT, resilient mounted furring channel ceiling grid (cavity 40 mm), 50 mm insulation (14 kg/m^3), 13 mm fire grade plasterboard	Adjustable clip furring channel ceiling grid (cavity 40 mm), 50 mm insulation (11 kg/m^3) and 13 mm standard grade plasterboard
$R_w + C_{tr}$ (airborne)	51*	53#		51*	50#	51*
$L_{n,w}$ (impact)	55*	53#	50 to 55+	to 50*	25 to 30#	45 to 50*
Fire	Type A construction	Type A construction	Type A construction	Type A construction	Type A construction	Type A construction
Total depth	300 – 400 mm		250 – 350 mm	225 - 325 mm	284	209

* CSR Redbook, #XLAM Acoustic predictor, + Dunlop underlay

4.10 Balcony Floors

Correctly designing and specifying balconies is extremely important with timber-framed buildings. A key consideration is that balconies protrude from, or are attached to, the external facade of the building, whose major function is to enclose the building and provide controls against fire, air movement, thermal performance and water and vapour intrusion. Good balcony design must limit the breaching of or impact on this external skin.

Structural design of the floor members is straight forward, similar to normal floor design though with different loadings, but structural engineers also need to understand the importance of proper balcony detailing to ensure their structural members are best protected. The aim with balconies should be to provide a solution to maximise longevity and minimise ongoing risk. It needs to be acknowledged that despite the best design and construction intentions and efforts, all balconies will leak at some point over their life and materials and construction approaches need to allow for this.

4.10.1 Balcony types

Balconies with timber framed buildings can be:

1. Built utilising a self-supporting structure.

2. Cantilevered from the timber building structure.

3. Inset into exterior wall niches.

Figure 4.24: Different balcony configurations for mid-rise timber buildings.

Balconies utilising self-supported structures

If a self-supporting structural post system is used, then it is recommended that posts be used to support all four corners. The system can then be supported independently of the timber building and any long-term vertical movement of the building will not affect the balcony or its fall.

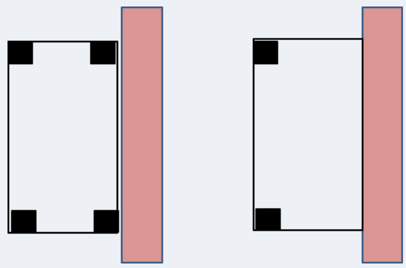


Figure 4.25: Posts supporting balconies.

If posts at the wall are not used, and the balcony fixes to the timber building, then the exterior posts need to be designed to accommodate the expected accumulated wood shrinkage in the building and ensure the slope remains adequate to provide drainage. If steel exterior posts are used, the posts should be designed to be progressively shorter, with the longest post between the grade level and the second storey and the shorter post in the top storeys.

The UK document *Differential Movement in Platform Timber Frames*[1] *(section 2.5)* provides the following advice for balconies.

The requirement for movement joints will be determined by the support arrangement for the balcony. Balconies are often independently supported but tied back to the floor zone of the frame to provide lateral stability. Inevitably these lateral fixings pass though the cladding zone and will move down with the frame. Movement joints are required below the ties to allow this to occur without unduly loading the connection. It should also be noted that the detailing of the connection itself should allow for differential movement / rotation. This often requires a pinned joint or slotted hole arrangement. Without such consideration, excessive joint loading may lead to damage.

Where balconies are, both restrained laterally and vertically supported on the building side there will be some rotation of the balcony floor as the building shortens. This effect can be offset by pre-setting the balconies to allow for the anticipated shrinkage. It is not unusual to "split the difference" and initially set the balcony out of level to allow for 50% of the calculated frame shrinkage. This allows for the possibility that the frame will not shrink as much as predicted whilst minimising the in-built fall. For independently supported balconies the difference in level between the threshold and balcony level will vary as the movement occurs. Flashings and other associated details should take account of this.

Avoiding Problems

1. Ensure that connection design allows any required movement.
2. Ensure joints are constructed to the correct size.
3. Ensure any designed in falls are constructed accurately.

[1] UK - Differential Movement in Platform Timber Frame www.pinewood-structures.co.uk/download/10/Differential Movement.

Cantilevered balconies

Cantilevered balconies provide a challenge to detail because they penetrate and breach the plane of the exterior wall facade assembly. This affects the full range of façade performance functions: fire protection, water and air movement exclusion and thermal and acoustic performance. Long term structural timber durability protection is also a major consideration, as by their cantilevered nature any longer-term maintenance and rectification actions, particularly member replacement if needed will be difficult and expensive to undertake.

The US and Canadian CLT Manuals advise that “cantilevered CLT balconies are not recommended because of the risk of water intrusion”. The FP Innovations *Mid-rise Wood Frame Construction Handbook* also does not recommend cantilevered balconies for lightweight framed members “due to the difficulty in maintaining the continuity of the weather and air barrier”.

A cantilevered balcony can theoretically be achieved structurally either through a continuous structural member that spans through the external wall of the building, or by a ‘bolt on approach’ at the external wall face to an appropriate mounting system.

For cantilevered balconies which utilise the internal floor structure spanning continuously through the external wall of the building, a number of other issues exist in regard to water ingress, fire and air control. It is difficult to satisfy all criteria without compromising on the façade function.

1. The deflected shape of the floor system, which can affect the direction of the fall
2. Providing a set down to minimise risk of ponding.
3. Providing a rigid connection that can transfer the bending moment between the exterior and interior floor system.

Balconies inserted into exterior wall niches

If a self-supporting structural post system is used, then it is recommended that posts be used to support all four corners. The system can then be supported independently of the timber building and any long-term vertical movement of the building will not affect the balcony or its fall.

4.10.2 Suggested approaches and design considerations

The key considerations and solutions for structural, durability and movement for balcony construction are outlined in Table 4.9.

Table 4.9: Key considerations for structural, durability and movement for balcony construction.

Durability Design Issue	**Recommendations**
Water intrusion into timber, leading to decay and fungal attack	✔ Provide robust waterproof membranes as the primary defence against water intrusion (possibly use two separate membranes). ✔ Membranes should preferably be laid over appropriately thick fibre-cement sheet and lapped up all vertical wall surfaces. ✔ Timber used externally should be treated to H3 or Class 1 durablity. ✔ Install drip moulds to avoid water dripping through structure.
Ponding at the exterior balcony – internal water ingress	✔ Provide set-down at the exterior face of the façade, so water line is below internal building.
Insulation can degrade	✔ Moisture resistant insulation to be specified.
Flow of moisture internally	✔ A minimum 2% fall should be designed to allow the flow of water away from the building.
Drainage system and/or balustrade needs to penetrate protective layers to floor structure	✔ Waterproofing membranes need to be appropriately detailed and installed at any penetration point.
Movement Issue	**Recommendations**
Vertical movement of support structure	✔ Connections should allow for differential movement.
Vertical movement of balcony support column	✔ A height adjustable column can be specified. ✔ Connections must allow for differential movement.
Dimensional changes of CLT (if specified) due to moisture fluctuations	✔ Do not apply protective membrane directly to the timber panels, utilise wet area FC sheeting and installation specification and apply waterproofing barrier above this.
Structural Design Issue	**Recommendations**
Deflection of support structure	✔ Note deflections for cantilevered balconies can be upwards at outer edge if internal support spans are heavily loaded, additional balcony fall should be allowed for.
Screws penetrating timber, creating a path for the flow of water	✔ Fix screws horizontally to structure, rather than vertically, to minimise risk.
External balcony support can penetrate floor structure (self-supporting balconies only)	✔ Design floor to sit on top of support to avoid penetration of balcony support column. ✔ If penetration necessary, allow for air gap for the flow of water.
Moment connections to support structure (cantilevered balconies only)	✔ Connection detail to allow the push-pull action, without compromising on façade integrity.

Some of the criteria in Table 4.9 may seem to contradictory. For example, a set-down is required to reduce the risk of ponding, but then the connection detailing required to transfer bending moment in the case of a cantilevered balcony, becomes much more difficult. For balcony design and specification, it is sometimes difficult to satisfy all the criteria, so a ***hierarchy of importance may need to be considered*** when discussing and negotiating between design issues. The following is suggested.

1. Protection of structural wood from moisture

Moisture poses the biggest threat to timber balconies; protection against long-term moisture issues should be considered the priority consideration for designers. Key considerations need to include: moisture barrier specification/installation/and future maintenance, adequate flashings, use of treated wood products, strategically placed air-gaps to allow water sheading and airflow, potential condensation issues.

2. Proper installation of waterproof membrane

Proper specification, positioning and installation of appropriate waterproof membranes is the first fundamental consideration. Balconies are somewhat analogous to bathroom wet areas and specifications to adequately protect timber can be similar.

- It is recommended that FC sheeting be utilised above the timber elements with the waterproofing/tanking applied to this FC surface. This is recommended as external timber will be exposed to cyclical moisture fluctuations causing expansion and contraction which may cause gaps between panels/boards on the face to open/close causing fatigue or cracking in the waterproof membrane. The FC sheet will assist in preventing this.
- It is recommended that waterproof membranes need also to be continuously run up vertical wall surfaces for an appropriate height to protect against water ingress behind the barrier.

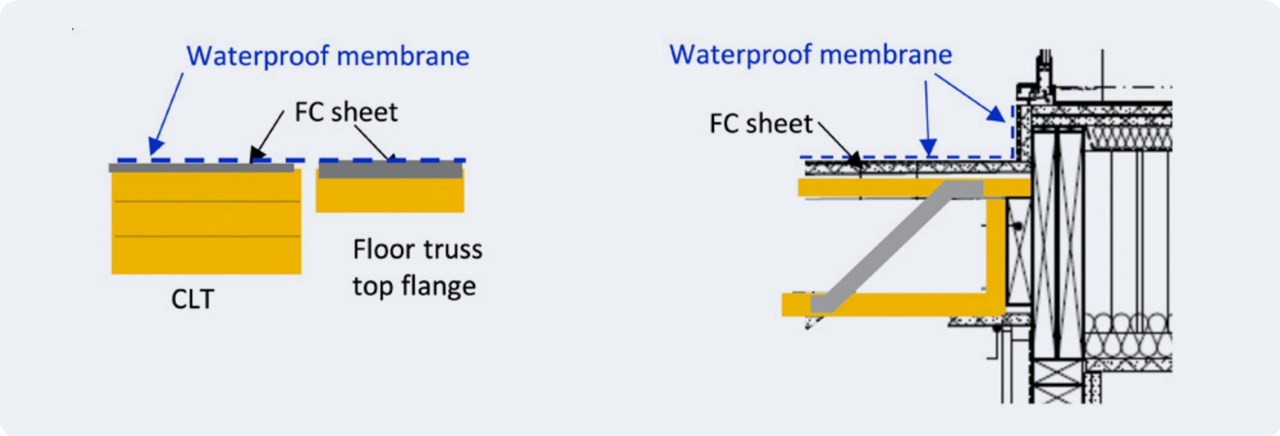


Figure 4.26: Installation of waterproofing membranes.

3. Use of treated wood

Any structural timber used in external balconies should be either treated to an H3 treatment level or be of Class 1 natural durability (this is also recommended for any appearance timbers used).

4. Structural support

Where possible balconies should be designed to be structurally supported outside the building envelope on:

- a self-supporting structure (beams and columns)
- supporting ledgers fixed to the wall structure (could be discrete steel angles, see sketch to right), or
- using a bolt-on-cantilever system.

Continuous cantilevered floor systems (internal to external over the external wall) are not recommended. Structural support members need to be either treated wood or galvanised steel.

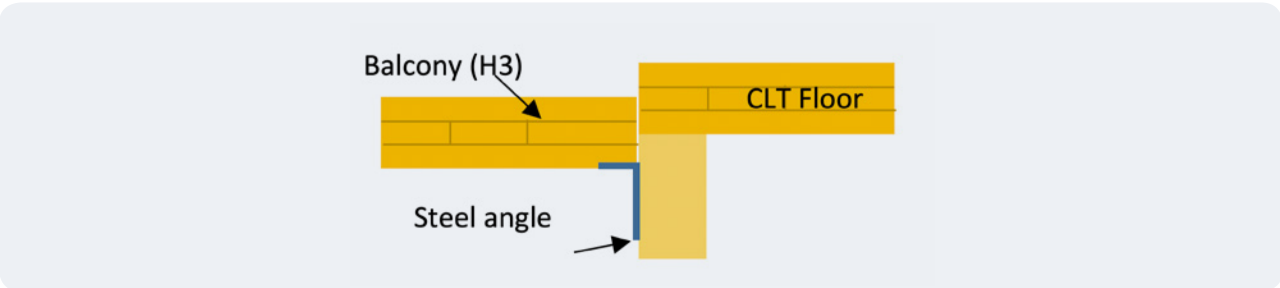


Figure 4.27: Installation of waterproofing membranes.

5. Providing appropriate slope for water drainage

A sloping surface of at least 2 degrees is recommended to allow water to drain to an appropriate collection point. The sloping surface can be achieved by:

- laying the floor panel or joists at an angle
- utilising a graded screed (minimum of 20 mm is required at the thinnest edge); or
- utilising a FC board (with membrane above) on graded battens (timber treated to H3).

It is recommended that slopes are directed away from the building wall towards the balcony edge.

6. Providing a set-down to protect against internal flooding

It is recommended that the balcony top surface be set down from the internal floor surface to prevent against unwanted flooding ingress.

7. Maintaining the integrity/continuity of the external cladding skin

It is a fundamental design tenet that wherever possible the integrity and continuity of the protective external façade be maintained to enclose the building and provide controls against fire, air movement, thermal performance, water and vapour intrusion, and noise (acoustics). Balcony design needs to closely consider how to minimise the interruption to these external wall control layers.

5 Wall Design

This section focuses on wall-type structural elements that carry vertical loads between levels and resist out-of-plane loads from wind forces (i.e. walls that do not form part of the lateral stability system). See Section 7 for information on resistance of in-plane forces in the lateral stability system).

Walls for mid-rise timber buildings can be constructed using either prefabricated lightweight timber framing or mass-panel timber. Considerations for walls in mid-rise buildings include:

- Although it is possible to design mid-rise buildings up to eight storeys using lightweight framed walls, it is likely to be structurally and economically more effective to use lightweight timber in the upper storeys and mass-panel timber for the lower storeys. Mass panels could be used for all levels if one consistent material type was desirable to simplify the supply and construction process.
- Building shortening is an important consideration for structural engineers in wall design for mid-rise timber buildings.
- Using the same wall layout on all storeys minimises transfer structures required. Continuity of walls, allowing vertical loads to travel directly down through the building to ground, is highly preferred.
- Internal wall layouts will influence the floor system approach utilised (floor spans, direction of spans).
- Openings in walls will lead to concentrated vertical forces on either side so minimise the size of any openings where possible.
- In addition to meeting structural performance, selected wall systems will also have to demonstrate compliance with acoustic and fire performance requirements.
- Off-site prefabrication allows for simpler preinstallation of some fire and acoustic system products ; walls can be partially or fully enclosed, saving on-site construction times (see Figure 5.1).

a) Lightweight partially enclosed

b) Lightweight fully enclosed

c) Mass panel CLT wall

Figure 5.1: Prefabricated lightweight and mass panel wall systems.

5.1 Lightweight Timber-framed Wall Systems

Lightweight timber-framed walls can be constructed using three framing systems:

- platform framing
- balloon framing
- semi-balloon framing.

5.1.1 Platform Framing

Platform framing is the most common form of construction in Australia. In this configuration, the floor systems bear directly on top of the top plate of the supporting wall in the storey below. The next storey wall is then placed directly above the floor system (see Figure 5.2). Wall frames are independent for each storey and individual studs are shorter than those used in semi-balloon and balloon framing systems.

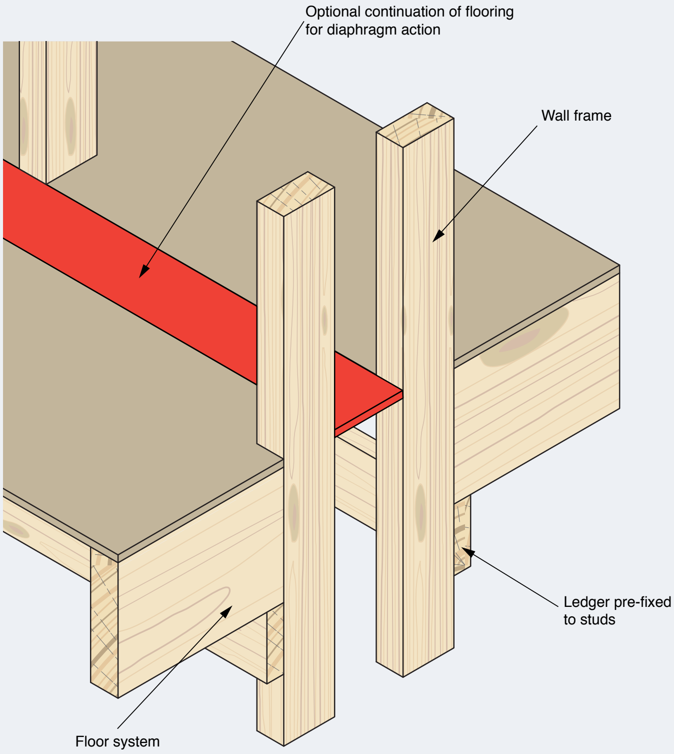


Figure 5.2: Platform floor frame with discontinuous double stud wall.

This form of construction allows simple installation of floor systems, particularly when prefabricated timber floor cassettes are used. This configuration also allows the floor to be designed and constructed continuously over the walls, so more structurally effective multiple span floor systems are possible. This method of framing is very cost effective as it uses commonly available stud lengths and prefabricated wall systems are easy to transport and erect.

Platform framing can be used without special detailing for timber-framed buildings up to three storeys or for the top three storeys of a taller mid-rise building with mass panel walls below, but requires appropriate detailing to assist in limiting axial shortening of the building if used within buildings more than three storeys high. The key issues include potential shrinkage in floor system members (use only seasoned timber products) and compression perpendicular-to-grain crushing in top and bottom plates or floor system rim-beams (if an issue, use higher strength LVL or hardwood products to reduce perp-to-grain crushing).

With platform framing, it is generally desirable to vertically align studs in walls, either side of a floor, with the position of the floor joists (see Figure 5.5(a)). This will minimise bending actions in wall plates so that their function is a packer rather than a bending element. With prefabricated cassette floor systems, a solid rim-board is generally utilised (also sometimes solid blocking between floor joists) and these solid between-floor members mean that stud alignment is less of an issue.

5.1.2 Balloon Framing

Balloon framing was developed in North America in the early 19th century when long lengths of timber were readily available for construction of two-storey houses. It replaced heavy post and beam systems.

The studs in this type of framing system are more than one storey high and the floor is attached to the side of the studs using a ledger plate (see Figures 5.3 and 5.5(c)). Balloon framing is rarely used in the construction of mid-rise timber buildings as it does not lend itself to the easy-installation of prefabricated floor cassette systems.

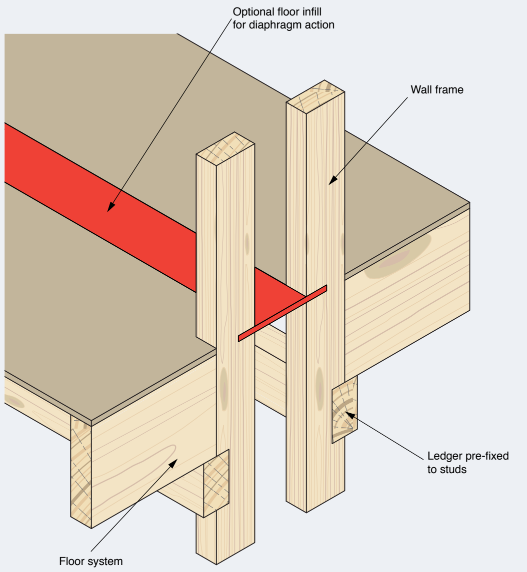


Figure 5.2: Platform floor frame with discontinuous double stud wall.

This form of construction allows simple installation of floor systems, particularly when prefabricated timber floor cassettes are used. This configuration also allows the floor to be designed and constructed continuously over the walls, so more structurally effective multiple span floor systems are possible. This method of framing is very cost effective as it uses commonly available stud lengths and prefabricated wall systems are easy to transport and erect.

5.1.3 Semi-balloon Framing

Semi-balloon framing is similar to balloon framing; however, the stud walls are only single storey. This form of framing is often used for multi-storey projects in North America. The wall panels consist of studs and wall plates similar to platform framing, with the floors supported from the sides of the studs on ledger plates (see Figure 5.4). The wall panels are higher than platform-framed walls (by the depth of the floor system) and stud lengths are longer.

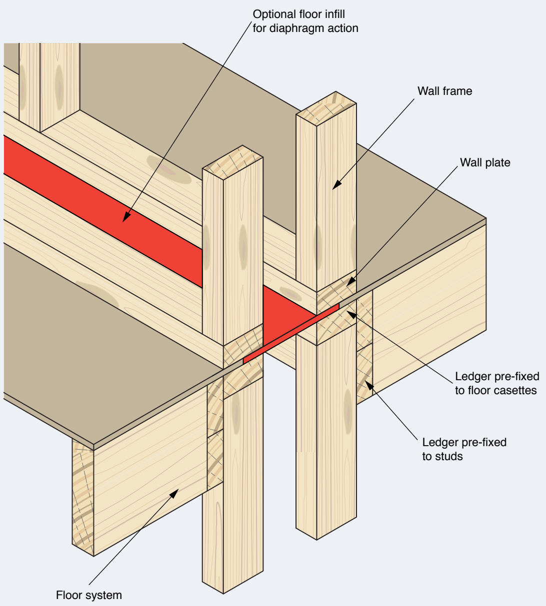


Figure 5.4: Semi-balloon framing.

The wall framing of one storey is stacked directly on the wall framing of the storey below. This configuration takes the actual floor system out of the load path and, in theory, helps reduce potential axial shortening due to potential shrinkage or perpendicular-to-grain compression issues in the floor system.

Semi-balloon framing systems require longer studs and shear walls than platform framing, and the floors also place eccentric loads onto the studs, which needs to be designed for. Additional blocks and more connections may also be needed compared to that required for platform framing.

With semi-balloon framing, it is critical to align the studs in walls above, with the studs in the wall below (see Figure 5.5(b)).

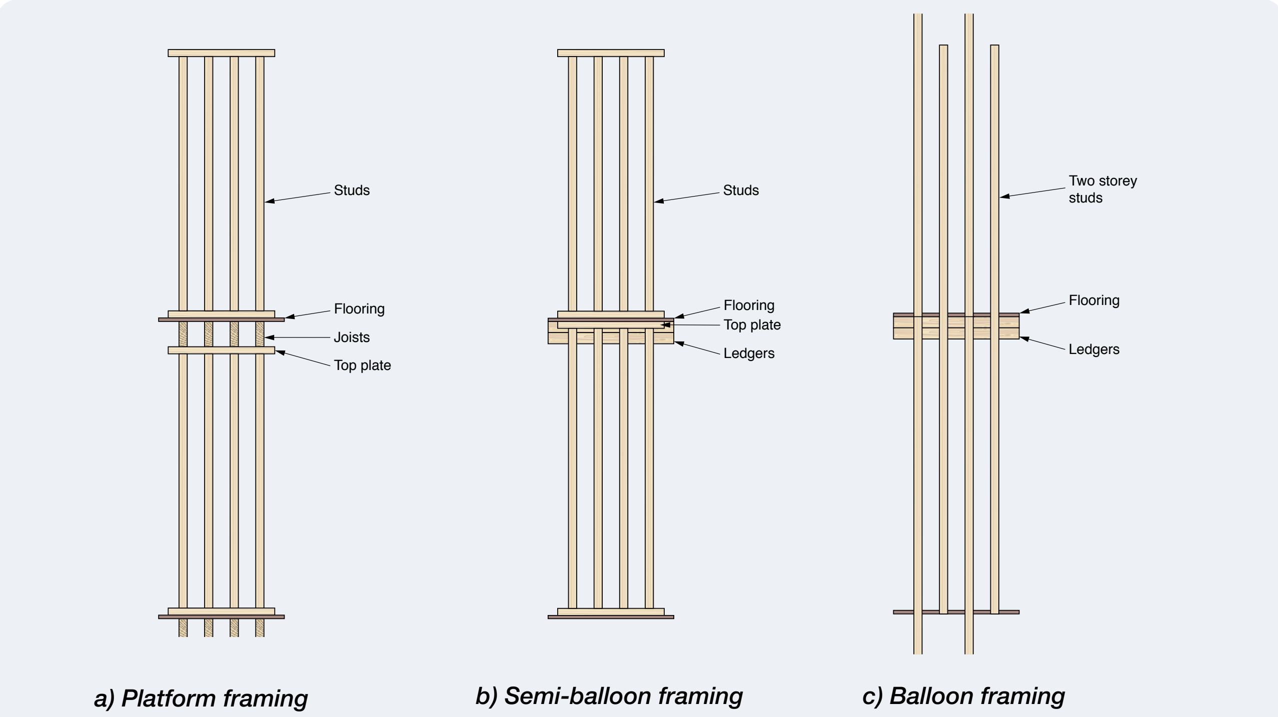


Figure 5.5: Alignment of studs and joists for most effective vertical load transfer.

5.1.4 Structural Implications of Different Framing Systems

The different framing systems have different impacts from a structural design perspective.

With platform framing, loads are in a consistent vertical path, but horizontal members loaded perpendicular-to-grain need to be considered to reduce localised crushing and overall building shortening. The best approaches are to:

- use horizontal members of higher density and perpendicular-to-grain strength (ie LVL or hardwood), and/or
- effectively reduce the axial load stress in the stud elements by increasing the stud area, for example using more studs (i.e. double studs, triple studs, etc).

Engineers also need to be careful in in assuming that using a 'deeper stud' is a better option than using double or triple studs, as thicker walls have a negative price impact for the developer or realtor. While the actual cost of the material might be lower, deeper studs take up more floor net lettable or saleable space, so the effective loss of sales opportunity in the project might be significantly higher than the additional cost of materials.

With both balloon and semi-balloon framing, each floor load is applied on the edge of the stud through a ledger. This introduces a bending moment in the studs:

- The bending moment applied at each floor level is the floor load times half the depth of the stud (depending on ledger detail).
- As the studs cannot carry a bending moment through an end joint, the moments do not accumulate through the height of the building but are resolved with horizontal reactions at each floor level.
- The moment at mid-height of a stud is roughly half the moment caused by the eccentricity; this value should be used in a combined bending and compression check.

Table 5.1 summarises the structural implications of selecting different wall framing systems.

Table 5.1: Summary of structural implications of selecting different wall framing systems.

	Platform framing	Semi-balloon framing	Balloon framing
Floor joists	In vertical load path	Fastened to the side of the joists – not in vertical load path	Fastened to the side of the joists – not in vertical load path
Eccentricity of floor loads	Centrally applied	Eccentricity = 1/2 stud depth	Eccentricity = 1/2 stud depth
Wall plates	In vertical load path	In vertical load path	Not in vertical load path
Stud alignment	Best to have aligned with joists and with studs in floor below	Best to have aligned with studs in floor below	Can only be aligned with studs in floor below
Effect on building shortening	Maximum depth of timber loaded perpendicular to grain – design to minimise building shortening	Intermediate depth of timber loaded perpendicular to grain – intermediate effect on building shortening	Minimum depth of timber loaded perpendicular to grain – minimum effect on building shortening

5.1.5 Wall Configurations for Acoustic Performance

In mid-rise timber buildings acoustic separation is required between apartments, or between apartments and common areas, so the type of wall system will be influenced by the acoustic performance required.

Wall configurations typically used in mid-rise timber framed buildings include:

- double stud walls
- staggered stud walls
- single stud walls with resilient support
- single stud walls (for walls without acoustic requirements).

Decisions on the wall type should be made before structural design begins.

Double leaf stud walls

Double leaf stud walls utilise a pair of single stud walls parallel to each other that are lined on the outer sides, usually with fire-rated plasterboard (see Figure 5.6). The separation of the lined walls significantly reduces sound transmission, making them the most efficient system to meet acoustic performance requirements.

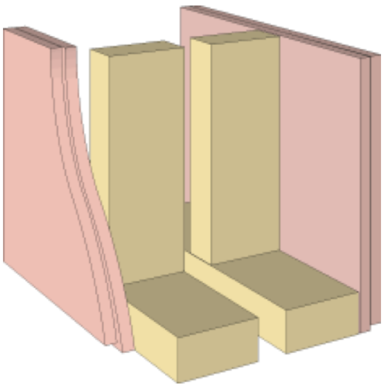


Figure 5.6: Double leaf stud wall frame.

Double leaf stud walls are the only framing option considered to be discontinuous and are required under the NCC for walls in bathrooms, toilets, laundries or kitchens that separate habitable rooms (except kitchen); and plant rooms that separate SOUs. The NCC requires a minimum of 20 mm of separation. In a double leaf stud arrangement, the wall plates are also separated.

Staggered stud walls

Staggered stud walls utilise wall top and bottom plates that are wider than the actual studs. Studs are then staggered to either side of these plates within the same wall frame (see Figure 5.7). Wall linings are fixed effectively every second stud in the wall, so noise vibrations are not transferred directly through the wall; the only direct path is through the top and bottom plates. Non-combustible acoustic insulation can also be threaded through the gaps between the studs and the lining on the opposite side to improve acoustic performance.

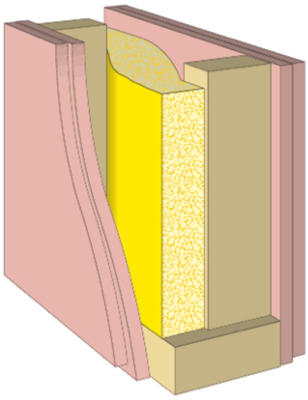


Figure 5.7: Staggered stud wall frame.

Because of the common plates, staggered stud walls are not as acoustically efficient as a discontinuous wall, but they can be used under the NCC acoustic requirements in certain locations and the narrower wall depth reduces the loss in usable floor area.

Single stud walls with resilient supports

The acoustic performance of single stud walls can be enhanced by supporting the linings on channels that are fixed to the stud with acoustic separators. The most common types of acoustic separators are: rubberised resilient mounts (Figure 5.8(a)) or a channel fixed to the stud on only one leg of the channel (Figure 5.8(b)). The rubberised mounts are the most commonly used. They also hold the channel further from the stud so that the chances of placing lining screws across the channel into studs (accidentally creating a sound bridge) are minimised. Like staggered studs, single studs with resilient mounts can achieve the minimum NCC acoustic levels but for a narrower wall depth than discontinuous construction, but the resilient mounts also add to the construction costs. Where resiliently mounted, the wall lining cannot be considered to contribute to the structural performance of the wall.

Figure 5.8 (a): Resilient mounted channel. Figure 5.8(b): Resilient channel.

Single stud walls

Single stud walls are traditional wall frames where linings are directly attached to both sides of the studs and have no acoustic separation. Single stud walls are generally used within an apartment of single occupancy where acoustic performance is not required, e.g. for load bearing fire-rated walls, exterior walls or non-fire rated partition walls.

5.1.6 Fire resistance of walls

Many of the walls within a mid-rise timber building are required to have a specific Fire Resistance Level (FRL) or must comply with the fire-protected timber requirements in the NCC outlined in Section 1.5.2.

Fire-grade linings are used to provide the fire resistance as well as the fire-protected timber requirements. Suitable fire-rated linings include plasterboard, fibre cement, magnesium oxide (MGO), calcium silicate or other cementitious boards. Manufacturers generally publish both FRLs and the appropriate fire resistance details based on their tests of complete wall systems to AS 1530.4 – Methods for fire test. For example, to meet a 90/90/90 FRL two layers of appropriately detailed 13 mm fire-rated plasterboard on each face can be considered acceptable. See *WoodSolutions Technical Design Guide #37 Mid-rise Timber Buildings.*

5.2 Mass-Timber Walls

Mass-timber wall panels are solid timber elements that generally have higher capacity than open timber-framed walls. The design philosophy is to evaluate the performance of the complete panel.

The connections between mass-timber panels and the floor systems must have the capacity to resist in-plane loads (see Section 7), transfer shear from out-of-plane loading and any tensions in the wall. The vertical compression loads are usually transferred by timber-to-timber bearing.

Mass-timber wall panels have the potential to also act as deep bending members (in-plane bending) for secondary load paths used in robustness calculations. Some of the connections between the wall panels and floors, or between the wall panels themselves, may need to be strengthened to achieve robustness requirements.

5.2.1 CLT Walls

CLT wall panels are generally used for the load-bearing walls of a building in conjunction with other systems for the non-load bearing walls, such as lightweight framing.

Design of walls for vertical loads involves calculating the area of the panels where the grain is aligned vertically and determining a slenderness coefficient for the whole panel. These parameters can be used to calculate the axial load capacity of a whole panel. Therefore, the most effective use of CLT wall panels is to orient the face layers so that the grain is vertical.

The design of CLT walls for out-of-plane wind loads involves treating the panel as a wide beam spanning vertically between the floors. Rolling shear (see Section 4.7) in the horizontal layers will affect the capacity of a wall panel to resist out-of-plane wind loads.

The detailed design of CLT wall panels for both axial forces and out-of-plane bending can be achieved using a number of behaviour models. CLT manufacturers will advise on the ones that work best with the materials and manufacturing processes they use.

CLT wall panels can be manufactured with cut outs for doors and windows, and recesses for wiring and other services. Manufacturers can provide reports on the fire and acoustic performance of CLT panels as part of overall systems. The details used to achieve the fire and acoustic ratings may affect the structural behaviour models used in design. It is possible to visually express CLT wall panels in the building finishes using a Performance Solution under the NCC but, in general, encapsulation with fire-rated plasterboard is used to achieve the required fire rating.

5.2.2 LVL or Glulam Wall Panels

LVL and Glulam wall panels are manufactured with the grain in all layers oriented vertically, which means that the axial loads are carried by the whole cross section and the slenderness of the panel is easier to calculate. There are no horizontal fibres in the panel to resist out-of-plane bending, so there is no need to model rolling shear for deflection or strength. Use AS 1720.1 to design these elements under axial load and/or out-of-plane bending.

The analysis of LVL and Glulam wall panels is more straightforward than that for CLT as all the layers are parallel. However, they must be detailed to accommodate the potential for swelling or shrinkage if there may be moisture content changes in the panels either during construction or in service.

5.2.3 Nail-laminated Wall Panels

Nail-laminated wall panels perform in a similar way to Glulam panels but may have less rigidity in transferring loads as a deep beam. The flexibility of the connections between each of the laminates means that shrinkage movements are more likely to be distributed throughout the wall than concentrated at the ends. Swelling movements must still be absorbed in joints at the edges of the panels. NLT panels may have gaps through tolerance and shrinkage that need to be considered in the overall acoustic and fire performance.

Nail-laminated wall panels can be designed using AS 1720.1 as nail-laminated studs involving very large numbers of elements.

5.2.4 Non-loadbearing Partitions

It is likely that there will be non-load bearing partitions in the building. The construction of the partitions is likely to be similar to that for a timber-framed wall; studs, plasterboard, etc. Although non-loadbearing for gravity loads, the partitions must feature adequate head and base restraint to resist lateral loads such as internal wind pressures. The head detail of the non-loadbearing partitions should be detailed to allow movement of the structure above without forming a vertical load path.

5.3 Loads and Load Cases for Wall Design

The following load cases in AS/NZS 1170.0 Cl 4.2.2 must be considered in the design of walls in mid-rise timber buildings:

Axial compression

- 1.35 G — $k_1 = 0.57$
- 1.2 G + 1.5 (0.4 Q) — $k_1 = 0.57$
- 1.2 G + 1.5 Q — $k_1 = 0.8$
- 1.2 G + 1.5 Q (non-floor) roof* — $k_1 = 0.94$ (top floor only)
- 1.2 G + (0.4 Q) + W_u — $k_1 = 1.0$
- G + (0.4 Q) + E_u — $k_1 = 1.0$

Bending

- W_u — $k_1 = 1.0$
- E_u — $k_1 = 1.0$

* A (non-floor) roof is a roof that does not also have a floor function, so the imposed action Q is based on Table 3.2 in AS/NZS 1170.1.

For European products, it may be more appropriate to use the k_{def} factors provided in Eurocode 5, though special consideration should be given to the appropriate load combinations as they differ from Australian Standards.

Jamb studs and studs carrying concentrated loads

Lintels cause axial forces on jamb studs, and anchorage of the door or window cause out-of-plane loads on jamb studs. The axial compression capacity of studs carrying concentrated loads from floors above should be checked.

5.4 Timber-framed Wall Member Design

Wall members include studs and, in most cases, top and bottom plates, and noggings. Some walls also include bracing elements. Considerations for timber-framed wall design include:

- Align studs throughout the building.
- Minimise depth of studs, i.e. thickness of wall to increase internal room size saleability.
- Higher capacity studs required for lower storeys can be provided by nail laminating and/or using higher grade material, e.g. LVL or hardwood.
- Load bearing walls must be fire rated.
- Non-load bearing walls can be fire-rated if part of a SOU boundary, or non-fire rated if internal to an SOU. They should be detailed to support lateral loads, but allow vertical head movement.
- DTS fire and sound-rated walls for 90/90/90 FRL – double stud, 2 x 13 mm fire-rated plasterboard, 20 mm cavity, cavity barriers, non-combustible acoustic insulation.
- Compare framing options – platform vs semi-balloon framing and prefabricated panels vs prefabricated frames.
- Assess options to minimise building shortening.
- Design of studs:
  - Capacity factor for Application Category 1: sawn timber = 0.9; LVL = 0.95.
  - Stability factor k_{12} – major axis usually governs; $g_{13} = 0.85$ for studs and plates of the same grade.
  - Noggings improve capacity of unclad walls under construction and fire loads.
  - Evaluate combined compression and bending for all walls with differential pressure.
- Stud walls to transfer slab:
  - Level the concrete, bearing under timber < bearing capacity of concrete.
  - See Section 7 for bracing wall design – lateral and uplift load transfer uses concrete anchors – chemical anchors are required at edges of slab.

Stud design

The following nomenclature is used in this document, consistent with Section 3.3 in AS 1720.1:

b = breadth or smaller cross-sectional dimension of the stud

d = depth or larger cross-sectional dimension of the stud (wide face).

Studs carry compression from gravity forces from the storeys above. External and some internal studs also carry lateral loads from wind forces. They are generally designed as compression members and checked for their performance under combined bending and compression actions. The design compression capacity is given in AS 1720.1 as Equation 5.1.

$$N_{d,c} = \emptyset\ k_1 k_4 k_6 k_{12} f'_c\ A_c \quad (5.1)$$

with terms defined in AS 1720.1 Cl 3.3.1.1.

Refer to Section 1.5 AS1720.1 for background on ϕ, k_1, k_4, k_6.

Studs are generally considered to be secondary elements in mid-rise construction, as:

- the tributary area for a stud, even in lower storeys, is usually less than 10 m2
- there are many parallel elements carrying loads; and
- studs are closely spaced (particularly in lower storeys).

The capacity factor ϕ used for stud design = 0.9 for sawn timber and 0.95 for LVL (Category 1 in Table 2.1 AS 1720.1).

Compression members have the potential to buckle in two directions – major axis buckling and minor axis buckling. For studs, major axis buckling is out-of-the-plane of the wall and minor axis buckling is in-the-plane of the wall. Noggings and/or wall linings in the plane of the wall restrain studs against minor axis buckling as shown in Figure 5.8(b), but it is not possible to restrain studs against major axis buckling. (Appendix E7.3.2 in AS 1720.1 enables calculation of the force that each restraint must resist. For noggings, it is typically 0.075 N* and for fixings to linings, around one-quarter¼ of that force. These are well within the capacity of nominal direct fixings for linings and the nominal fixings of noggings.)

The potential for buckling under axial compression affects slenderness calculations and therefore the value of k_{12} calculated for use in Equation 5.1. As the values for k_{12} can vary considerably according to the assumptions made, the use of safe, but realistic parameters can lead to cost-effective design of wall members.

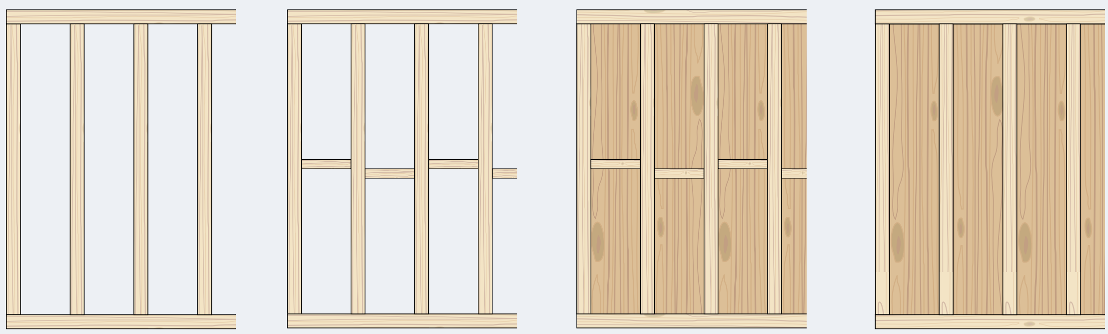
Top and bottom plate design

Top and bottom plates can be loaded in both bending and bearing in platform and semi-balloon framing, outlined in Section 5.1. Where studs and floor joists align, there is no bending in the plates and the only action on top and bottom plates is bearing. The design bearing capacity for top and bottom plates loaded perpendicular to grain and studs loaded parallel to grain is given in Section 3.2.6 of AS 1720.1.

Refer to Section 1.7 for detail on the parameters ϕ, k_1, k_4, k_6.

Noggings

Noggings are horizontal members fitted between studs in a wall frame (see Figures 5.9(b) and (c)). In some situations, noggings provide lateral restraint for studs. They can be made from non-structural timber.



(a) Without noggings ***(b) With noggings*** ***(c) With lining*** ***(d) Lining and no noggings***

Figure 5.9: Use of noggings in lightweight timber-framed walls.

In the case of mid-rise buildings, it is assumed that the plasterboard on structural walls will be fire-rated with associated detailing and fasteners. The fire-rated plasterboard is assumed to be intact throughout the service life of the building. In such a case, compression studs of typical sizes may be assumed restrained in the min or axis at max. 600 mm centres (fastener spacing) by the wall lining using panel bracing walls or fire-rated walls. For bending, torsional restraint to studs of typical sizes and wall height is provided at the ends of the studs and at noggings; and the tension edge of the stud is assumed to be restrained by the wall sheet using common fixings for fire-rated plasterboard. For deep studs (thick walls) the assumptions of buckling restraint provided by a single side of sheathing should be investigated. If in doubt, use noggings. Care is required in design of studs without noggings to ensure that structural performance under all load cases, including accidental, is adequately addressed.

Timber-framing for houses that complies with AS 1684, must include noggings at 1350 mm centre. However, designers may choose whether or not to use noggings in mid-rise buildings based on the following considerations which may affect the design of studs.

- ***Fire performance:*** Load-bearing walls in mid-rise timber buildings must be fire-resistant and evidence to support their FRL is required. Many of the systems tested for fire resistance were wall systems that used noggings at 1350 mm centres. Therefore, if relying on the fire test certificates for these walls, they will also need to include the noggings to comply with the configuration used in the assessment. Alternatively, a designer could check the capacity for minor axis buckling under fire load conditions with and without noggings but with no lining on the fire side. This will determine whether it is possible to achieve the structure FRL without the noggings.
- ***Staggered stud walls:*** It is not possible to install noggings in staggered stud wall systems. For these walls, the studs must be designed without noggings.
- ***Single stud walls with resilient channels:*** The fixings on resilient support channels used to improve acoustic performance on single stud walls allow some flexibility in the connection between the vertical studs and the horizontal channels. They cannot be regarded as a substitute for noggings.
- ***Pre-fabrication:*** Installing noggings during prefabrication of wall frames may slow the manufacturing process. Wall frames may come to the site with sheathing already installed.
- ***Construction loads:*** Linings on walls are generally fixed sometime after the frames are loaded by floors above. For example, where wall linings are installed on site, the linings may be pre-loaded onto the floors before the next floor is installed to make use of the easy crane access. The studs have to resist construction loads before they are fully restrained by the lining. Noggings may not be necessary if construction loads are very small. However, without linings in place, minor axis buckling may limit the capacity during construction (see Example 5.1). Solutions include installing noggings, back-propping floors that are used to store construction materials, or installing sheet bracing systems (plywood or OSB) during prefabrication.
- ***Maintenance***: Wall frames with noggings can more effectively resist out-of-plane bending forces if wall linings need to be removed in service for any reason.

The advantages of using noggings include:

- Back propping is often not required under construction before linings are fixed to the frame. (Check reduced capacity of studs restrained by noggings but not by linings – Example 5.1)
- Improved structural capacity for the fire limit state as noggings laterally restrain studs if the fire-rated plasterboard separates from the frame.

5.4.1 Minor Axis Buckling of Studs

The capacity of a stud limited by minor axis buckling is a function of the lateral restraint offered by noggings and/or wall lining. Figure 5.10 shows the buckled shape of studs (a) without noggings, (b) with noggings and (c) with lining. Once the lining is installed, any restraint offered by the noggings is exceeded by the restraint from the lining. Therefore, the restraint of the studs in Figure 5.3(c) is unchanged with or without noggings.

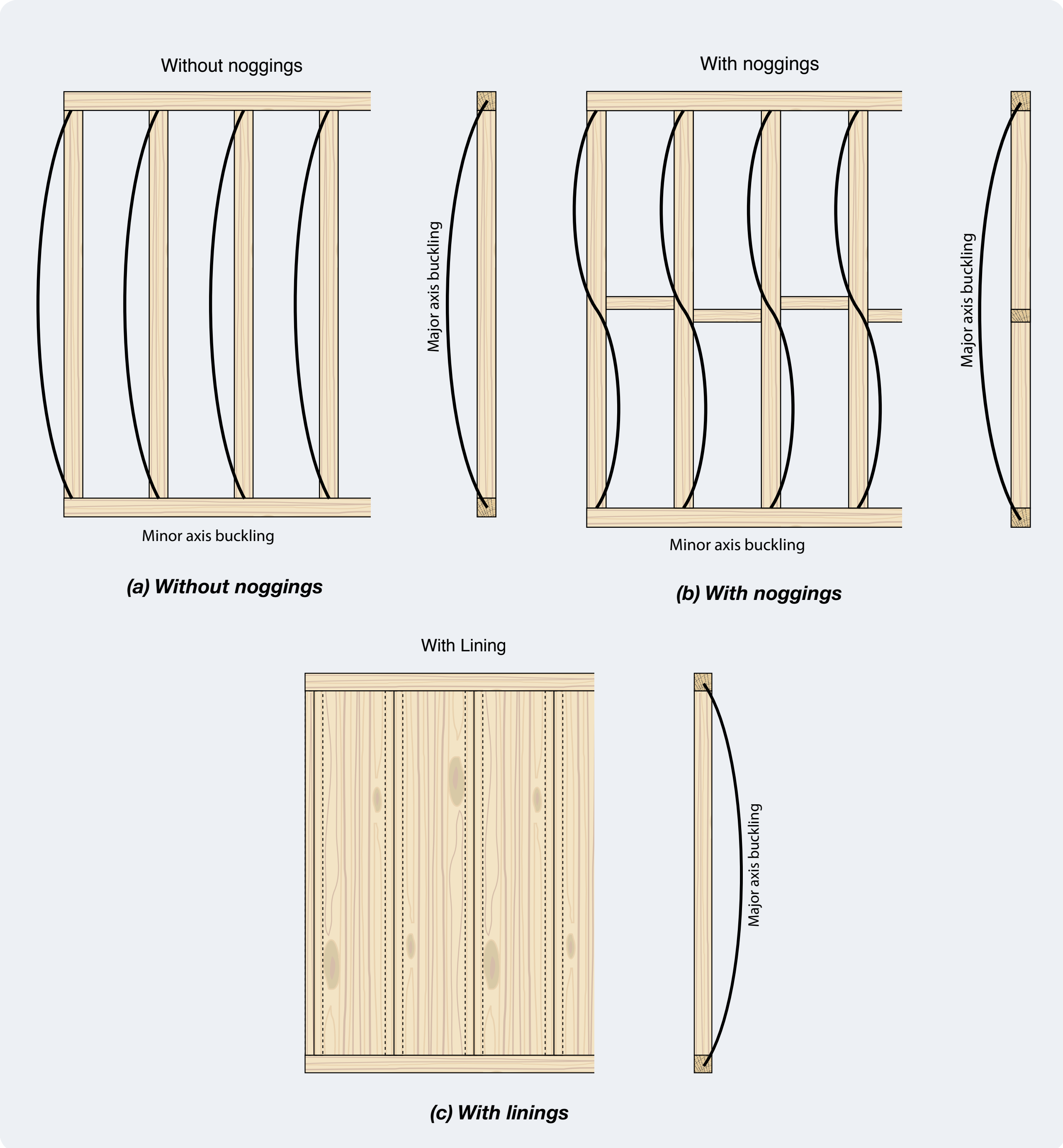


Figure 5.10: Buckling of studs in lightweight timber wall frames.

In Figure 5.10(a), the studs are buckling over their full length, and the minor axis slenderness is given by Equation 5.2. In this case, the g_{13} value for studs in AS 1720.1 Table 3.2 (i.e. 0.9) is appropriate.

$$S_4 = \frac{g_{13}L}{b} \tag{5.2}$$

with terms defined in AS 1720.1 Cl. 3.3.2.2.

In Figure 4.8(b), the studs are buckling over the height between the noggings, and the minor axis slenderness is given by Equation 5.3. For the case shown with one set of noggings, L_{ay} = L/2, but with two sets of noggings, L_{ay} = L/3.

$$S_4 = \frac{L_{ay}}{b} \tag{5.3}$$

In Figure 5.10(c), the studs are fully restrained by the lining. This will be the case if the spacing of the lining fixings along a stud is less than 360 mm for 35 mm studs and 460 mm for 45 mm studs. For full restraint, the slenderness is calculated using Equation 5.4.

$$S_4 = \frac{3.5d}{b} \tag{5.4}$$

with terms defined in AS 1720.1 Cl. 3.3.2.2.

Where the lining fixings are further apart than the values given above, the slenderness is calculated with L_{ay} as the distance between lining fixings.

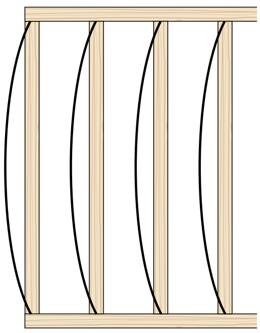
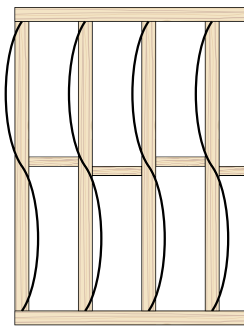
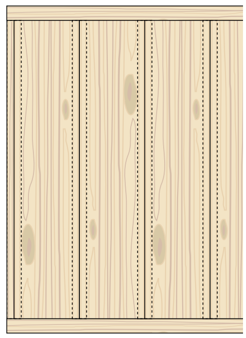
In a completed building, under all permanent and imposed actions, the restraint offered by the wall lining will be available and minor axis buckling of studs will not usually limit design. However, during construction, unless bracing panels have been installed on the frames before the next floor is installed, gravity loads must be resisted before the lining is fixed to the wall frame, and the studs must be designed using the restraint shown in either Figure 5.10(a) or Figure 5.10(b).

The restraints make a substantial difference to the compression capacity limited by potential minor axis buckling, as shown in the example in Table 5.2.

Example 5.1: Minor axis buckling of studs

For 90x45 mm MGP10 studs with a length of 2700 mm, the following capacities were evaluated for k_1 =0.8 (appropriate for loads of several months duration).

Table 5.2: Comparison of buckling with and without noggins.

	Construction with no noggings	**Construction with noggings**	**Finished clad wall**
			 Lining fixed at 600 mm along studs
ϕ	0.9	0.9	0.9
k_1	0.8	0.8	0.8
k_4	1	1	1
k_6	1	1	1
ρ_c	0.96	0.96	0.96
g_{13}	0.9		
S_4	$\frac{g_{13}L}{b} = \frac{0.9\times2700}{45} = 54$	$\frac{L_{ay}}{b} = \frac{1350}{45} = 30$	$\frac{L_{ay}}{b} = \frac{600}{45} = 13.3$ $\frac{3.5d}{b} = \frac{3.5\times90}{45} = 7$ (Use 7)
k_{12}*	0.074	0.241	1.0
$N_{d,cy}$	**3.9 kN #**	**12.7 kN #**	**52.5 kN**

* k_{12} evaluated using Clause 3.3.3 and ρ_c from Table 3.3 in AS 1720.1

\# Limiting capacities, see Example 5.2 (Table 5.3)

Example 5.1 shows that the capacity of studs without noggings in the wall frame limited by potential minor axis buckling is significantly less under construction loads before the lining is installed. After linings are installed, the stud capacity is limited by major axis buckling.

5.4.2 Major Axis Buckling of Studs

As no lateral restraint against major axis buckling is possible for studs, the slenderness of the stud is given by Equation 5.5, which requires designers to select an appropriate g_{13} (effective length factor) value.

$$S_3 = \frac{g_{13} \times L}{d} \tag{5.5}$$

with terms defined in AS 1720.1 Cl. 3.3.2.2.

Table 3.2 in AS 1720.1 gives a range of g13 values from 0.7 to 1.0 that may be appropriate for compression members in braced frames. Studs are part of braced structures, so g_{13} will be less than or equal to 1.

The experiments that produced the values in AS 1720.1 Table 3.2 don't exactly match the conditions of studs in major axis buckling in mid-rise platform, semi-balloon or balloon frames:

- Flat ends – describes timber elements on flat, unyielding plates. Top and bottom timber plates can yield in bearing and in any timber framing do not provide the fixity to give g_{13} =0.7.
- Studs in light framing – refers to studs in domestic construction under minor axis buckling. Studs in major axis buckling have a larger restoring moment once one edge lifts off the restraint. In mid-rise construction, the rigidity of the frames is higher due to the higher vertical and horizontal loads that must be resisted, so are likely to have a lower value than the 0.9 tabulated.

There is only one entry for studs in AS 1720.1 Table 3.2, but recent research on stud behaviour in framing has been incorporated in AS 1720.3, where g_{13} for studs is presented a function of the length of the studs.

AS 1720.3 also indicates that the appropriate g_{13} value to be used in design of 3 m long studs in major axis buckling = 0.85. This value is greater than 0.7 and less than 0.9, so is compatible with the critique of values given in AS 1720.1 above. Recent research on bearing in wall plates confirms that g_{13} = 0.85 where the wall plates are the same or higher density material as the studs. However, if the plates are of a lower density material, it allows more rotation at the ends of the stud under buckling, and g_{13} = 0.9 is more appropriate. Continuing research may produce updated values for use in design.

Current recommendations:

g_{13} = 0.85 for major axis buckling of studs where top and bottom plates are an equal to or better grade than the studs.

g_{13} = 0.9 for major axis buckling of studs where top and bottom plates are a lower grade than the studs.

g_{13} = 0.9 for minor axis buckling over the full length of all studs.

Typically, g_{13} = 0.85 increases the capacity of studs by between 10% and 15% compared with the use of g_{13} = 0.9.

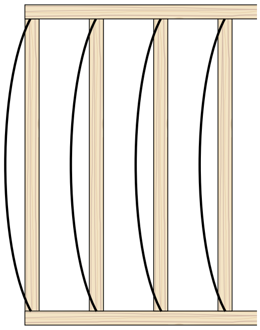
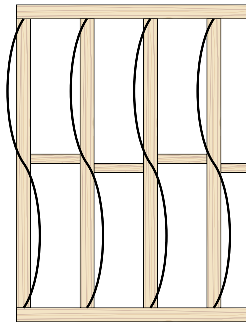
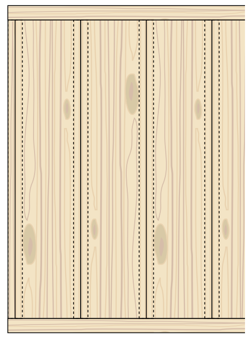
Example 5.2: Major axis buckling of studs

Example 5.1 will be checked for major axis buckling; 90x45 MGP10 studs 2700 mm long, with MGP10 top and bottom plates.

ϕ	= 0.9
k_1	= 0.8
k_4	= 1
k_6	= 1
g_{13}	= 0.85
S_3	$= \frac{g_{13} L}{d} = \frac{0.85 \times 2700}{90}$ = 25.5
ρ_c	= 0.96
k_{12}*	= 0.334
$N_{d,cx}$	**= 17.5 kN**

The compression capacity in major axis buckling is the same for studs irrespective of minor axis restraint (i.e. the same major axis capacity applies to studs with or without noggings, and studs with lining) as shown in Table 5.3.

Table 5.3: Comparison of capacity with and without noggins.

	Construction with no noggings	Construction with noggings	Finished clad wall
			 Lining fixed at 300 mm along studs
$N_{d,cy}$	3.9 kN	12.7 kN	52.5 kN
$N_{d,cx}$	17.5 kN	17.5 kN	17.5 kN
$N_{d,c}$	3.9 kN	12.7 kN	17.5 kN

The limiting design capacity is the minimum of $N_{d,cx}$ and $N_{d,cy}$. This shows that the two construction scenarios were limited by minor axis buckling, while the completed wall scenario is limited by major axis buckling. While noggings do not contribute to the capacity of the completed wall, they triple the capacity of studs under construction loads. Fire-rated plasterboard can separate from the studs during a fire. The normal fire resistance certificate applies if noggings are specified in wall frames. However, the minor axis capacity of studs must be calculated if noggings are omitted to verify that the wall can resist its fire loads without lining.

The discrepancy between the major and minor axis capacities and hence between final capacity and construction capacity increases if d/b for the studs is higher.

5.4.3 Buckling of Nail-Laminated Studs

Previous studies on light timber framing indicate that for compression capacity, nail-laminated studs have the same slenderness as a single stud for both minor axis and major axis buckling. This means that k_{12} is the same for single, double, triple and quadruple studs. Therefore, increasing the number of studs nailed together increases the capacity only by increasing the compression area, i.e. the capacity of two identical studs nailed together is double the capacity of a single stud. This model of laminated stud behaviour is valid for closely spaced compression members in any framed building including mid-rise construction.

Nails at maximum 600 mm centres should be used for all nail-laminated studs.

5.4.4 Choice of Stud Size and Arrangement

Selecting the right grade or stud size for the structural demand in the building is the key to cost-effective design of mid-rise timber framed buildings. The following factors should be considered:

- wall thickness
- acoustic separation requirements
- availability of products, sizes and grades
- relative cost of products, sizes and grades
- fabrication costs and prefabrication requirements.

Collaboration between the structural designer, fabricator, architect, and acoustic and fire consultants in the early stages of design will assist in developing an optimal system for selecting stud sizes. The optimal solution may be different for each project depending on the relative importance of the considerations listed above.

Some designers will opt for similar grades and stud sizes for the highest three storeys with stronger studs in the lower floors; others may choose to optimise timber volumes at each storey. Options for satisfying acoustic requirements are presented in Section 5.1.5. The following sections discuss the issues in more detail.

Wall thickness

Designers aim to minimise the thickness of walls to maximise available internal space in the building.

This can be challenging, particularly in the lower storeys of six storey mid-rise timber-framed buildings where vertical loads are higher. Better axial capacity can be achieved by increasing the depth of studs compared with increasing the breadth of the studs. However, deeper studs increase the overall wall thickness (and decrease the useful floor area), particularly in double stud walls that are used to separate SOUs to improve acoustic performance.

It is necessary to balance the lower volume of timber required for the stud walls using deeper studs against the value of the rentable space lost in the wider walls. Example 5.3 compares the capacity of two different studs.

Example 5.3: Capacity of studs with different cross-sections

Table 5.4 summarises some calculations for the design capacity of two different studs:

- 120 x 35 MGP10 stud and
- 90 x 45 MGP10.

Table 5.4: Comparison of stud sizes.

	120 x 35 MGP10	90 x 45 MGP10
ϕ	0.9	0.9
k_1	0.8	0.8
k_4	1	1
k_6	1	1
f'_c	18 MPa	18 MPa
A_c	120 x 35 = 4200 mm^2	90 x 45 = 4050 mm^2
$g_{13,x}$	0.85	0.85
L_{ay} (mm)	600	600
ρ_c	0.96	0.96
S_4	$\frac{L_{ay}}{b} = \frac{600}{35} = 19.1$	$\frac{L_{ay}}{b} = \frac{600}{45} = 25.5$
$k_{12,y}$	For $10 \leq \rho_c \leq 20$ k_{12}=1.5-0.05 ρ_c S = 1.5 – 0.05 x 0.96 x 17.1 = 0.677	For $10 \leq \rho_c \leq 20$ k_{12}=1.5-0.05 ρ_c S = 1.5 – 0.05 x 0.96 x 13.3 = 0.860
S_3	$\frac{g_{13}L}{d} = \frac{0.85 \times 2700}{120} = 19.1$	$\frac{g_{13}L}{d} = \frac{0.85 \times 2700}{90} = 19.1$
$k_{12,x}$	For $10 \leq \rho_c \leq 20$ k_{12}=1.5-0.05 ρ_c S = 1.5 – 0.05 x 0.96 x 19.1 = 0.582	$\rho_c \geq 20$ $k_{12} = \frac{200}{(\rho_c S)^2}$ $k_{12} = \frac{200}{(0.96 \times 25.5)^2}$ = 0.334
$N_{d,c}$	31.7 kN	17.5 kN

ρ_c from AS1720.1 Table 3.3

f'_c from AS1720.1 Table H3.1

In this example, the 120 x 35 MGP10 stud has around 1.8 times the capacity of a 90 x 45 stud even though the cross-sectional area of the 120 x 35 was only 1.04 times the area of the 90 x 45.

It may be economical to use deeper studs where there is no acoustic requirement, such as for load-bearing walls in the interior of the apartment or for external walls where single stud walls may be appropriate. Where impact sound resistance is not an issue (such as corridor walls), staggered stud walls may be used to limit the effect of using deeper studs in the lower storeys compared to double stud walls in the same location.

Costs

Costs are influenced by material cost, fabrication cost and construction cost as well as many other factors. The following section discusses optimising stud choice in the context of understanding the impact on material cost. The engineer must set this in the context of the other cost considerations, albeit qualitatively.

Figure 5.11 shows that as the stress grade increases, the capacity of the stud also increases. It is a non-linear relationship as different grades will have different ρ_c factors and may also have different ϕ factors.

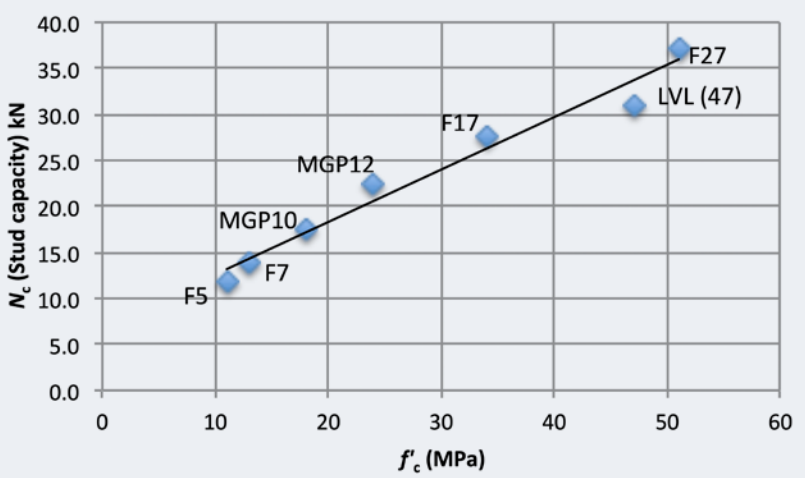


Figure 5.11: Stud capacity by grade.

The cost of higher-grade material per m3 is also generally higher. The unit costs of different materials change from time-to-time and may also vary with location. Therefore, there can be no 'rule' for determining the most cost-effective materials for studs. The following process is appropriate for deciding on a strategy for stud selection.

- Evaluate the capacity of a single size stud in a number of different commonly available grades.
- Divide the capacity of each stud by the cost ratio for the grade.
- The stud with the highest capacity/cost is the most cost effective.

Example 5.4 – Cost-effectiveness of studs by grade

The cost-effectiveness of MGP10, MGP12, LVL, F17 hardwood and F27 hardwood studs is compared in Table 5.5, based on 2.7 m long studs with a 90x45 cross section, and with lining fixings at 600 mm along each stud.

Table 5.5: Comparison of stud costs.

Grade	MGP10	MGP12	LVL	F17	F27
f'_c (MPa)	18	24	47	34	51
ρ_c	0.96	0.98	1.20	1.08	1.14
ϕ	0.9	0.9	0.95	0.95	0.95
k_1	0.8	0.8	0.8	0.8	0.8
k_4	1	1	1	1	1
k_6	1	1	1	1	1
S_4	13.3	13.3	13.3	13.3	13.3
$k_{12,y}$	0.860	0.847	0.700	0.780	0.740
$N_{c,y}$ (kN)	45.1	59.3	101.2	81.6	116.2
$g_{13,x}$	0.85	0.85	0.85	0.85	0.85
S_3	25.5	25.5	25.5	25.5	25.5
$k_{12,x}$	0.334	0.320	0.213	0.264	0.237
$N_{c,x}$ (kN)	17.5	22.4	30.9	27.6	37.2
N_c (kN)	**17.5**	**22.4**	**30.9**	**27.6**	**37.2**
*Grade cost ratio**	1	1.3	2.3	3.0	4.0
Capacity/cost (kN)	**17.5**	**17.2**	**13.4**	**9.2**	**9.3**

* Illustrative grade cost ratio only. Values will vary at different times and in different places.

Table 5.5 shows the calculation of the stud capacity, based on the different grade parameters shown shaded at the top of the table. Variations in the material factor (ρ_c) lead to changes in the k_{12} factors for each grade.

The Grade cost ratio is the cost per metre of the grade divided by the cost per metre for MGP10. The values used in the table may vary with location and will change with time as well. They need to be revised for each job and reflect the prices in the building location. The most cost-effective studs are those with higher values for capacity/cost for a given depth.

Table 5.5 shows that there is little difference between MGP10 and MGP12, with a small reduction to LVL and a further reduction to F27. Doubling studs does not significantly change the capacity/cost ratio as a doubled stud produces double the capacity for double the material cost.

Spacing - 300, 450 or 600 mm centres

It is possible to vary the capacity per metre of a wall by changing the stud spacing. The maximum spacing of 600 mm is based on lining requirements. By reducing the stud spacings, the capacity is increased. Halving the spacing doubles both the compression capacity of the wall and the cost of the studs.

Optimise the stud spacings at the lower storey first then align studs above each other for subsequent storeys, avoiding the need for wall plates to behave as load transfer beams for higher storeys. Alternatively, wall plates can be designed to transfer loads between misaligned studs. Care is needed in calculations for building shortening which will need to include greater perpendicular to grain depth of the wall plates, plus a component for long-term bending deflection of the wall plates.

It is also necessary to consider stud breadth (*b*). For example, MGP10 90 x 45 @ 600 centres (19.8 kN/m) has similar capacity to 90 x 35@ 450 crs (20.5 kN/m) and either can be used for a particular storey; if the stud spacing for the storey below is 600 mm, use MGP10 90 x 45, or if the stud spacing in the storey below is 450 mm, use MGP10 90 x 35.

Lengths required for framing

Semi-balloon framing requires longer studs than platform framing; for a typical floor–to-floor height of 3.1 m, semi-balloon framing requires 10-15% more timber than platform framing. However, the length of floor joists required is reduced.

Timber can be supplied in a number of lengths up to 6.0 m. If 3.1 m stud lengths are required, carefully consider timber orders to minimise cutting waste.

5.4.5 Combined bending and compression in studs

The following loads can induce out-of-plane bending actions in the wall (major axis bending in the studs).

- Wind actions on external walls – inward loads on windward walls and outward loads on side and leeward walls. These load cases have a k_1 = 1.0.
- Eccentric loads in balloon or semi-balloon framing from gravity loads from floor ledgers applied at the inside edge of the studs as indicated in Table 5.1. These actions induce a moment in the stud equal to the gravity load in the floor times half the depth of the stud. These load cases have the same k_1 value as the stud under axial compression for the same load case.

The combination of bending and compression in Clause 3.5 of AS 1720.1 requires that both Equations 5.6 and 5.7 are satisfied.

Minor axis combination

All capacities evaluated with k_1 =1 for wind actions or the relevant k_1 for gravity loads in semi-balloon framing.

ULS bending moment

ULS axial force

$$\left(\frac{M^*{}_x}{M_{d,x}}\right)^2 + \left(\frac{N^*{}_c}{N_{d,cy}}\right) \leq 1 \qquad (5.6)$$

Design bending capacity

Design axial compression capacity with k_{12} calculated using only S_4

$M_d = \emptyset\, k_1 k_4 k_6 k_9 k_{12} f'_b\, Z$

$N_{d,cy} = \emptyset\, k_1 k_4 k_6 k_{12} f'_c\, A_c$

with $S_4 = \frac{L_{ay}}{b}$ or $S_4 = \frac{g_{13}L}{b}$ or $S_4 = \frac{3.5d}{b}$ for minor axis buckling of studs g_{13} = 0.9

Major axis combination

All capacities evaluated with k_1 =1 for wind actions or the relevant k1 for gravity loads in semi-balloon framing.

$$\underbrace{\left(\frac{\overbrace{M^*{}_x}^{\text{ULS bending moment}}}{\underbrace{M_{d,x}}_{\text{Design bending capacity}}}\right)} + \left(\frac{\overbrace{N^*{}_c}^{\text{ULS axial force}}}{\underbrace{N_{d,cx}}_{\text{Design axial compression capacity with } k_{12} \text{ calculated using only } S_3}}\right) \leq 1 \qquad (5.7)$$

$$M_d = \emptyset\, k_1 k_4 k_6 k_9 k_{12} f'_b\, Z \qquad N_{d,cx} = \emptyset\, k_1 k_4 k_6 k_{12} f'_c\, A_c$$

with $S_3 = \dfrac{g_{13}L}{d}$ for major axis buckling of studs, g_{13} is given in Section 5.4.2.

5.4.6 Stud holes and notching

The bracing methods detailed in this guide do not require notching of studs. Wall studs should not be notched in mid-rise construction unless justified by calculation.

The acceptance of any holes in studs in mid-rise construction should be considered by engineers on a case-by-case basis with understanding of stud grade, loads, redundancy, and importance. The following guidance applies to most typical situations, but holes in heavily or unusually loaded studs should be justified by calculation. Holes containing bolts or screws in compression elements can typically be ignored. Holes should only be permitted in the wide face of the stud and should conform with the requirements of Table 5.6 unless justified by calculation.

Table 5.6: Limits on size and extent of holes in studs

Description	Limits
Hole size	Not exceeding one quarter of the stud depth up to a maximum of 30mm
Hole spacing	At least four hole diameters centre to centre
Hole location	At least 150mm from top or bottom of the stud and not located more than one quarter of the stud height from the end
Hole location	On the stud centreline and at least four hole diameters from any knots or natural holes

5.4.7 Wall Plate Detailed Design

Bearing

Perpendicular to grain crushing is a strength check, but as the wood crushes it still transmits bearing loads and rarely causes any collapse. As crushing progresses, the structure deforms, so bearing perpendicular to grain is pragmatically a serviceability issue rather than a strength issue. It still must be evaluated as a strength check to ensure conformance with AS 1720.1

Two bearing capacities must be evaluated using Section 3.2.6 in AS 1720.1:

- bearing in the stud – parallel to grain; and
- bearing in the top and bottom plates – perpendicular to grain.

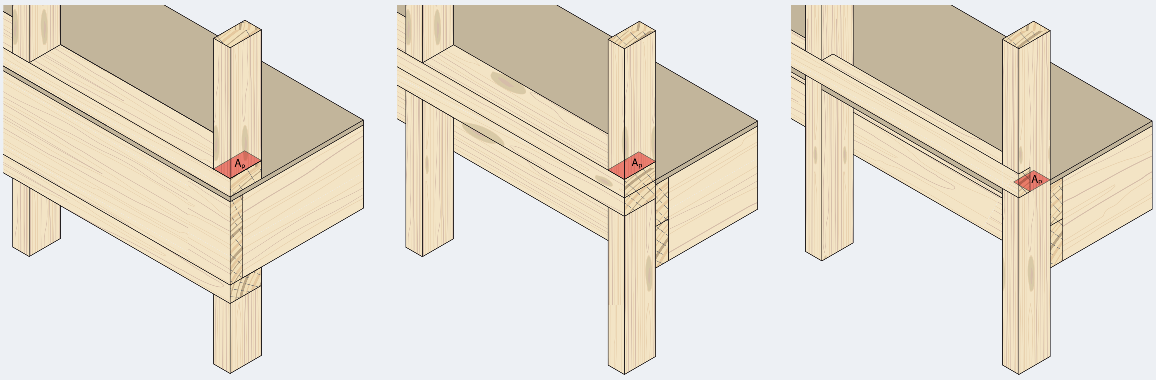
Bearing in the stud

The design bearing capacity for studs loaded parallel to grain is given in Cl. 3.2.6.2 in AS 1720.1 (Equation 5.8).

$$N_{d,\ell} = \emptyset\ k_1 k_4 k_6 f'_\ell\ A_\ell \tag{5.8}$$

f'_ℓ is the bearing strength parallel to grain given in Table H2.2 in AS 1720.1 for most materials. (Specific values for MGP grades are presented in Table H3.1 and LVL properties are available from the manufacturer). Bearing strength parallel to grain is generally around three times bearing strength perpendicular to grain.

A_ℓ is the cross-section of the stud at the bearing surface. In most cases for platform and semi-balloon framing, this is the full cross-sectional area of the stud, as shown in Figure 5.12(a) and (b). Where ribbon nogging is used in balloon framing, the bearing area may be reduced by the notch as shown in Figure 5.12(c).



(a) Platform framing ***(b) Semi-balloon framing*** ***(c) Balloon framing***

Figure 5.12: Bearing in studs.

Bearing in top and bottom plates

The design bearing capacity for top and bottom plates loaded perpendicular to grain is given in Cl. 3.2.6.3 in AS 1720.1 as Equation 5.9.

$$N_{d,p} = \emptyset\ k_1 k_4 k_6 k_7 f'_p\ A_p \tag{5.9}$$

k_7 is the length of bearing factor defined in Section 2.4.4 of AS 1720.1. It is only applicable to bearing perpendicular to grain and has the value 1.0 unless both:

- the bearing length is less than 150 mm
- the bearing is more than 75 mm from the end of the member.

The k_7 value in Table 2.6 in AS 1720.1 is greater than 1 where the conditions above are satisfied. Table 5.7 presents the k_7 values for different numbers of nail-laminated studs with element breadths of 35 mm or 45 mm. (These values have been interpolated from Table 2.6 in AS 1720.1.)

Table 5.7: k_7 for plates by common stud sizes.

Number of nail-laminated studs	b of stud (mm)	Length of bearing (mm)	k_7
1	35	35	1.32
2	35	70	1.16
3	35	105	1.12
4	35	140	1.04
1	45	45	1.24
2	45	90	1.135
3	45	135	1.06
4	45	180	1

The critical studs for design purposes are usually internal studs in a wall. Figure 5.13 shows that k_7 = 1 for Stud 1 as the bearing is less than 75 mm from the end of the bottom plate, and k_7 > 1 for Studs 2, 3 and 4 as the bearing is more than 75 mm from the end of the bottom plate. However, Stud 1 has half the tributary area and loads of Studs 2, 3 and 4. (Studs either side of openings – jamb studs – are usually doubled. Their tributary area is higher than conventional studs as they pick up extra load from the lintel over the opening.)

The bearing length is the b of the stud; for example, if the studs have b = 45 mm, Table 4.6 indicates that for single studs, k_7 = 1.24 for the studs labelled as Studs 2, 3 and 4 in Figure 5.13. These studs carry 2 x the load of Stud 1, but their capacity is only 1.24 x the capacity of Stud 1. They are the limiting studs in the wall.

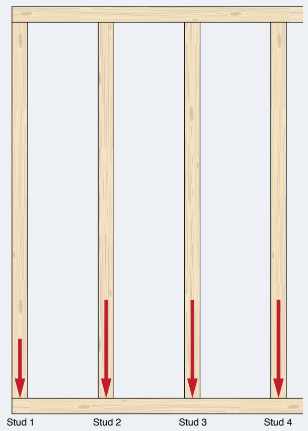
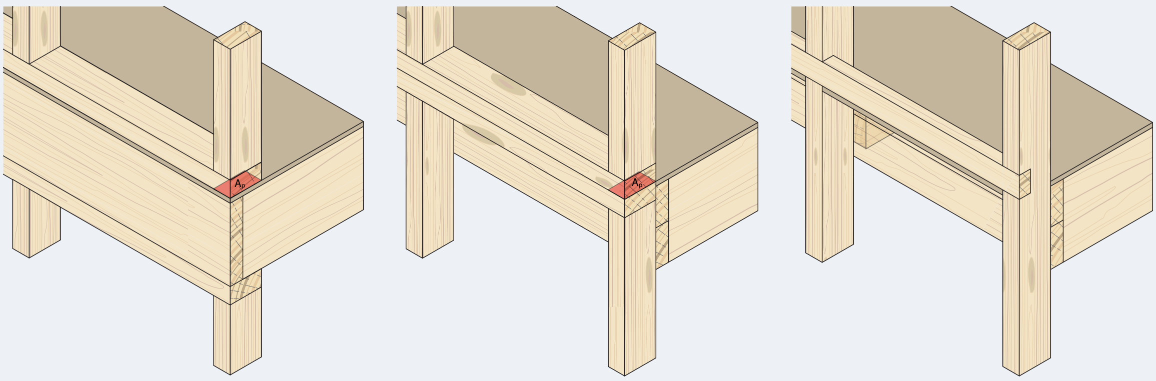


Figure 5.13: Loads in end and internal studs.

Where there are lintels or other concentrated loads on the end studs, these studs may be more critical than the internal studs in the same wall.
f'_p is the bearing strength perpendicular to grain given in Table H2.2 in AS 1720.1 for most materials. (Specific values for MGP grades are presented in Table H3.1 and LVL properties are available from the manufacturer.)
A_p is the cross-section of the stud bearing on the surface of the top or bottom plate. In most cases for platform and semi-balloon framing, this is the full cross-sectional area of the stud as shown in Figure 5.14(a) and (b). It is not appropriate in balloon framing as the plates are not on the gravity load path as shown in Figure 5.14(c).



(a) Platform framing ***(b) Semi-balloon framing*** ***(c) Balloon framing***

Figure 5.14: Bearing in plates.

For platform framing and semi-balloon framing, the bearing area at the end of the stud (A_ℓ) is the same as the bearing area on the top or bottom plate (A_p). However, bearing strength parallel to grain (f'_ℓ) is around three times bearing strength perpendicular to grain (f'_p), so the limit to bearing capacity is bearing perpendicular to grain in the plates.

Balloon framing only has wall plates in contact with the concrete at the bottom of the building and immediately under the roof. Both of these plates must be checked for bearing perpendicular to grain. Elsewhere within a balloon framed wall, there is direct end bearing from stud to stud as shown in Figure 5.14(c) and parallel to grain bearing is checked.

Reinforcing plates against perpendicular to grain bearing

The studs for the lower two storeys of a six-storey mid-rise timber framed building could be sized to prevent perpendicular to grain failure in wall plates, i.e. studs will be thicker than required for compression to prevent crushing perpendicular to grain in wall plates. However, this approach could be less cost effective for a large project.

A better alternative may be to use stronger or reinforced wall plates to prevent wall plate behaviour from dictating the sizing of studs:

- wall plates with higher perpendicular to grain compressive strength such as hardwoods with density more than 800 kg/m^3; or
- engineered timber with some layers that have parallel to grain timber in the load path, i.e. CLT wall plates.

More information is available in an FWPA report (TDA, 2018) on the influence of perpendicular to grain compression and creep in four to eight storey lightweight timber-framed buildings.

5.5 Mass-timber Wall Design

CLT walls have high vertical load capacity compared to more traditional timber structures and this has enabled the construction of otherwise not possible tall timber buildings. In most cases the CLT wall should be designed so that the outer layers are vertical and the grain of most of the cross-section is running in the direction of the applied load. The cross-layers running perpendicular to the load are ignored in the axial design checks for the wall, although they can be useful to form lintel or header panels above doors and windows.

Section properties can again be calculated through several methods and the design checks are covered in detail in the FP Innovations guide. The capacity of the walls is governed by their slenderness which can be calculated through the equations for composite cross-sections in AS1720.1, E4.4. Slender walls will naturally be more sensitive to issues like eccentric loading and construction imperfections and a suitable allowance for these should be made in the design.

Mass timber wall sizes may typically increase the further down a building as the loads increase. Typically, they need to be checked on the basis of both ambient and fire load cases to determine the appropriate sizes.

It is not necessary (or typically economic) to specify every wall in a building as a mass timber wall and designers should identify the main walls for gravity or lateral loads and designate as mass timber while other walls may be framed to reduce costs. Mass timber buildings are much more economic when walls line up between floors as transfer structures are often difficult to achieve within timber floors. Mass timber walls also tend to work best when they are in consistent lines on plan too as this will allow walls to be made of single panels with openings cut in them, reducing fabrication, fixings and construction time.

Dimensional tolerances of mass timber panels typically run to ± 2 mm. Eccentricity of vertical loads on walls should be considered, particularly for walls along external elevations where they are only loaded on one side. Allowing for a vertical out-of-plumb of H/500 is recommended, with a minimum eccentricity of vertical load of T/15 for application of vertical load. The resulting induced moment needs to be considered as a combined effect with the axial load.

CLT panels used as wall systems are subjected to three types of loading:

1. Vertical in-plane loading from the gravity loads.
2. Lateral in-plane loading coming from wind and earthquake loads.
3. Lateral out-of-plane loading that comes from wind loads.

5.5.1 CLT wall panels under axial In-plane-loads and out-of-plane loads

CLT walls under axial in-plane loads in combination with out-of-plane wind loads can be designed using different approaches.

Design of CLT walls – combined bending and compression (AS1720.1)

The stability checks of CLT walls are conducted with reference to what reported in section 3.5.1 of AS 1720.1. The following expressions need to be satisfied:

$$\left(\frac{\sigma_b^*}{f'_{b,d}}\right)^2 + \frac{\sigma_c^*}{f'_{c,d}} \leq 1 \qquad (5.10)$$

$$\frac{\sigma_b^*}{f'_{b,d}} + \frac{\sigma_c^*}{f'_{c,d}} \leq 1 \qquad (4.25)$$

A detailed design example can be found in Appendix 2 of this guide.

Design of CLT walls – mechanically jointed columns theory using EC5

The method for the mechanically jointed columns theory can be found in Annex C of EC5. Using this approach outlined, the effective moment of inertia is calculated as:

$$I_{eff} = \frac{(EI)_{eff}}{E_{mean}}$$

$(EI)_{eff}$ effective stiffness can be calculated using methods outlined in 4.7.2

E_{mean} is the modulus of elasticity of the boards that are parallel to the axial load

The slenderness ratio is typically limited to 150, as specified by CLT manufacturers.

$$\lambda_{eff=l} \cdot \sqrt{\frac{A_{tot}}{I_{eff}}}$$

A_{tot} is the total cross-sectional area of the panel

l is the height (the buckling length) of the wall element

Design of CLT walls – CSA O86 approach combined with mechanically connected beams theory

FPInnovations Chapter 4 outlines an approach that combines mechanically jointed beams theory with Canadian Timber Design Standards CSA O86-09 for the design of CLT walls. In this method, the layers oriented in the direction of load are considered. Using CSA O86-09 the slenderness ratio can be calculated as:

$$C_{c=} \frac{H}{d} = \frac{H}{2\sqrt{3} \cdot r_{eff}}$$

$$r_{eff} = \sqrt{\frac{I_{eff}}{A_{eff}}}$$

C_c is the slenderness ratio for rectangular CLT walls

I_{eff} can be calculated using methods proposed in Section 3.6

A_{eff} can be calculated as:

$$A_{eff} = b \cdot h_{eff} = b \cdot \sum_i h_i \tag{5.15}$$

b is normally taken as 1000 mm

h_i is the thickness of wall panels that are parallel to the axial load

The design procedure for determining the buckling strength can continue as specified in Clause 5.5.6 of CSA O86-09, substituting the cross-section area A with A_{eff} and the total thickness d with the effective thickness b_{eff} .

Using the same substitutions, the compressive resistance of CLT walls with combined out of plane loadings can be calculated using Section 5.5.10 of CSA O86-09.

The P-Δ effects can be accounted for by:

$$\left(\frac{P_f}{P_r}\right)^2 + \frac{M_{f,P-\Delta}}{M_r} \leq 1 \tag{5.16}$$

P_f is the factored compressive axial load

M_r is the factored bending moment resistance

$M_{f,P\text{-}\Delta}$, the factored bending moment that includes p-delta is calculated by:

$$M_{f,P-\Delta} = M_f + \frac{P_f \cdot (P_f + e_0 + \Delta_0)}{1 - \frac{P_f}{P_E}} \tag{5.17}$$

Where:

c_0 panel deflection due to axial load eccentricity. Eccentricity should be taken as d/6, where d is the panel thickness

Δ_0 initial wall imperfections in the mid of the panel usually taken as H/500, where H is the wall height

Δ^f deflection due to out-of-plane loading (bending)

P_E Euler buckling load in the plane of the bending moment using I_{eff} and E_{05} boards parallel to the axial load.

The Euler buckling load P_E, is given as:

$$P_{E.v} = \frac{P_E}{1 + \frac{k \cdot P_E}{(GA)_{eff}}} \tag{5.18}$$

K_c is the effective length factor and L is the wall height.

Shear deformations play a role in determining the design properties of the CLT panels, which can be accounted for using this approach. The basic buckling formula that accounts for shear deformations, the axial load capacity can be calculated as.

$$P_{E.v} = \frac{P_E}{1 + \frac{k \cdot P_E}{(GA)_{eff}}} \tag{5.19}$$

k is the shear deformation form factor, use 1.2

5.5.2 Design approaches for CLT elements used as beams and lintels

CLT elements can act as deep beams or lintels, which is explored in the following section.

Calculating In-plane bending strength

The bending stress for a CLT panel can be calculated as:

$$\sigma = M \cdot y \cdot \frac{(E_{mean})}{(EI)_{eff}} \tag{5.20}$$

y is the location of maximum stress, (in the middle of the panel typically, H/2)

Using the CSA O86 design approach:

$$\sigma_{max} \leq \phi \cdot F_b \tag{5.21}$$

F_b is the bending strength of the panel

The factored bending moment resistance can then be expressed as:

$$M_r = \phi \cdot F_b \cdot \frac{(EI)_{eff}}{E_{mean}} \cdot \frac{1}{0.5H} \tag{5.22}$$

E_{mean} is the modulus of elasticity of the longitudinal layers in tension

And I_{eff} can be calculated as:

$$I_{eff} = \frac{h_{eff} \cdot H^3}{12} = \frac{H^3}{12} \cdot \sum_i h_i \tag{5.23}$$

This method assumes a composite action between effective longitudinal boards. A more conservative approach would be to sum the individual moments of inertia for individual boards.

In-plane loads don't have the same shear deformations, therefore the second moment of inertia is much greater than out-of-plane loading.

Composite Theory-k Method

Using the CSA-086 design approach, the maximum bending stress can be calculated as:

$$\sigma_{max} = \frac{M}{S} \quad (5.24)$$

$$\sigma_{max} \leq \phi \cdot F_{b,eff} \quad (5.25)$$

$$M_r = \phi \cdot F_{b,eff} \cdot S_{gross} \quad (5.26)$$

where S_{gross} can be calculated as:

$$S_{gross} = \frac{h_{tot} \cdot H^2}{6} \quad (5.27)$$

h_{tot} is the total thickness of the CLT panel

H is the total beam depth

This approach assumes composite action between all the longitudinal boards, and therefore isn't conservative.

5.6 Exterior Walls

Exterior walls are generally load-bearing walls and, in most cases, they also incorporate the building façade. This means in addition to meeting the wall structural element (lightweight or mass-panel) and member fire protection requirements, the external wall system needs to also meet the façade performance requirements of non-combustible claddings, insulation and thermal performance, airtightness, weather protection and water and vapour intrusion.

Whilest it generally does not affect structural design, apart from material loading consideration, it is important that structural engineers appreciate the product system components required within the external wall of a mid-rise timber building.

The external wall layout generally includes the following components – from the inside out:

- internal fire rated plasterboard
- structural timber wall system (lightweight framed wall or mass-panel wall)
- external fire rated plasterboard – moisture resistant
- vapour permeable moisture resistant membrane
- non-combustible insulation (at this point for mass-panel walls or inside the wall cavity for lightweight walls)
- batten system to provide vented gap
- external non-combustible cladding fitted to batten system.

The extremal wall elements described above are illustrated in Figure 5.15.

Figure 5.15: System components in an external lightweight timber-framed wall.

6 Vertical Movement Design

6.1 Building shortening

All multi-storey buildings shorten as the load is applied. This shortening must be accommodated through the construction process both in the structure and the finishes. Higher strains (more shortening) occur in elements with higher stresses and smaller cross-sectional areas. Even in concrete buildings, columns tend to shorten more than cores and are designed to accommodate this shortening as the building is built.

Checking the vertical movement is a critical design step for structural engineers in the selection of the structural system and facade details. For timber-framed and mass-timber mid-rise construction, building shortening is caused by a combination of:

Shrinkage of the timber - perpendicular to grain - and parallel to grain	Shrinkage perpendicular to grain is about 40x greater than parallel to grain per unit height. Limit timber used perpendicular to grain in the vertical load path.
Crushing, deformation and creep of the timber - parallel to grain and - perpendicular to grain	Timber perpendicular to grain is up to 30 x more flexible than parallel to grain (depending on the presences of features, knots, etc). Long-term creep is twice short-term elastic movement. Limit timber used perpendicular to grain in the vertical load path.
Closure of joints.	Production tolerances of elements affect dimensions perpendicular to the grain. These are typically -0/+2 or 3 mm for a section. Fabrication tolerances typically affect the length of members parallel to grain and are in the order or 0.5 mm per cut. Limit timber used perpendicular to grain in the vertical load path.

Vertical movement of the building can be reduced through careful material specification and detailing of the structural components and it can be estimated using Equation 6.1.

$$\delta_{Total} = \delta_{s,\ell} + \delta_{s,p} + \delta_{c,\ell} + \delta_{c,p} + \delta_j \qquad (6.1)$$

where:

δ_{Total} = Total estimated shortening of the building

$\delta_{s,l}$ = Total shrinkage parallel to grain (Refer Section 6.1.1)

$\delta_{s,p}$ = Total shrinkage perpendicular to grain (Refer Section 6.1.1)

$\delta_{c,l}$ = Total compression parallel to grain (Refer Section 6.1.2)

$\delta_{c,p}$ = Total compression perpendicular to grain (Refer Section 6.1.3)

δ_j = Total closure of joints (Refer Section 6.1.4)

Axial shortening of a building is a long-term problem. Therefore, all loads considered in the estimation of shortening are long-term gravity loads. It is a serviceability issue and uses a load combination $G + \psi_l Q$.

Because there is a large variability in many of the parameters, the measured shortening may differ significantly from the estimate (US practice allows about 6 mm shortening per storey for platform framing up to four storeys).

The magnitude of building shortening is heavily determined by the wall framing selected and how the wall framing interacts with the floor framing system adopted. The most common form of assembly, platform construction (see Section 5.1.1), uses wall panels measuring a storey in height, and the floor system runs continuous over, or terminates, one top of them. This can cause compression perpendicular-to-grain in horizontally placed floor system members. A significant proportion of the overall shortening of taller buildings could come from the floor and this needs to be controlled through appropriate detailing during the structural design process. This compression process happens continually over time as the building creeps and j2 factors should be applied to the deformation to account for long term behaviour.

In addition, if acoustic separation mats are used between vertical load bearing elements, the crushing of the mats or pads should be also included into the total shortening of the building. Refer to suppliers of these products for crushing rates for particular products. The following sections provide guidance to estimate the axial shortening for specific cases.

Tables 6.1 and 6.2 provide an illustration of typical shortenings for mid-rise timber buildings.

Table 6.1: Example of typical shortening (in millimetres) in timber-framed building (see Appendix 1 for calculation). Six storeys of timber frame above a concrete podium (Level 1).

	Roof	Level 6	Level 5	Level 4	Level 3	Level 2	Total
Elastic dead	0.22	0.66	1.01	1.30	1.73	2.11	**7.03**
Elastic live	0.18	0.35	0.48	0.58	0.75	0.90	**3.24**
Creep	0.40	1.01	1.49	1.88	2.48	3.01	**10.27**
Shrinkage	1.50	2.21	2.39	2.21	2.39	2.39	**13.11**
Gaps	0.7	0.7	0.7	0.7	0.7	0.7	**4.20**
Total	**3.00**	**4.93**	**6.07**	**6.67**	**8.05**	**9.11**	**37.85**

Estimate 6 mm per storey. The majority of which will need to be accommodated in the facade and finishes. Given the light-weight nature of the timber construction, even the elastic movement will be significantly influenced by cladding and fit out.

Table 6.2: Example of shortening (in millimetres) in mass-timber building. 6 storeys of CLT above a concrete podium (Level 1).

	Roof	Level 6	Level 5	Level 4	Level 3	Level 2	Total
Elastic dead	0.05	0.10	0.16	0.21	0.26	0.31	**1.09**
Elastic live	0.00	0.02	0.03	0.05	0.06	0.07	**0.24**
Creep	0.06	0.12	0.19	0.25	0.32	0.39	**1.33**
Shrinkage	2.41	2.41	2.41	2.41	2.41	2.41	**14.46**
Gaps	0.70	0.70	0.70	0.70	0.70	0.70	**4.2**
Total	**3.22**	**3.35**	**3.49**	**3.62**	**3.75**	**3.88**	**21.31**

Estimate 3.5 mm per storey

6.1.1 Shrinkage of timber in service (moisture content)

Timber within a completed building is in a dry environment, and if the building is continually air-conditioned it may have a long-term moisture content (mc) of around 7% to 12%. It is likely, however, that the seasoned timber elements will have been delivered to site with a moisture content of 12% to 15%. If it has rained during construction, the mc may have increased to around 16%. For CLT, manufacturers also need a certain level of moisture for the glue to cure and this is typically in the range of 12% ± 2%. Actual moisture levels in service once a building is clad and air-conditioned could be around 8%, representing an approximate 4% change in moisture content.

Moisture changes in the timber will cause the building to shorten, both over the height of the wall panels and across the floor panels (if it is built using platform construction). Due to the anisotropic nature of timber the shrinkage across the floor will be much greater as it is across the grain and this will be responsible for most of the building shortening. On a 10-storey building the shortening could be in the region of 40-50 mm overall due to creep and shrinkage.

Shrinkage is greater perpendicular to grain. Because shrinkage is independent of the load on the timber, there is no need to separate elements by storey; their dimensions can be summed over the height of the building.

Estimating shrinkage parallel to grain

Shrinkage parallel to grain in walls (see Figure 6.1(a)) contributes to reduction in the height of the building though less than shrinkage perpendicular to grain as shrinkage parallel to grain is typically 1/40 of the shrinkage perp to grain.

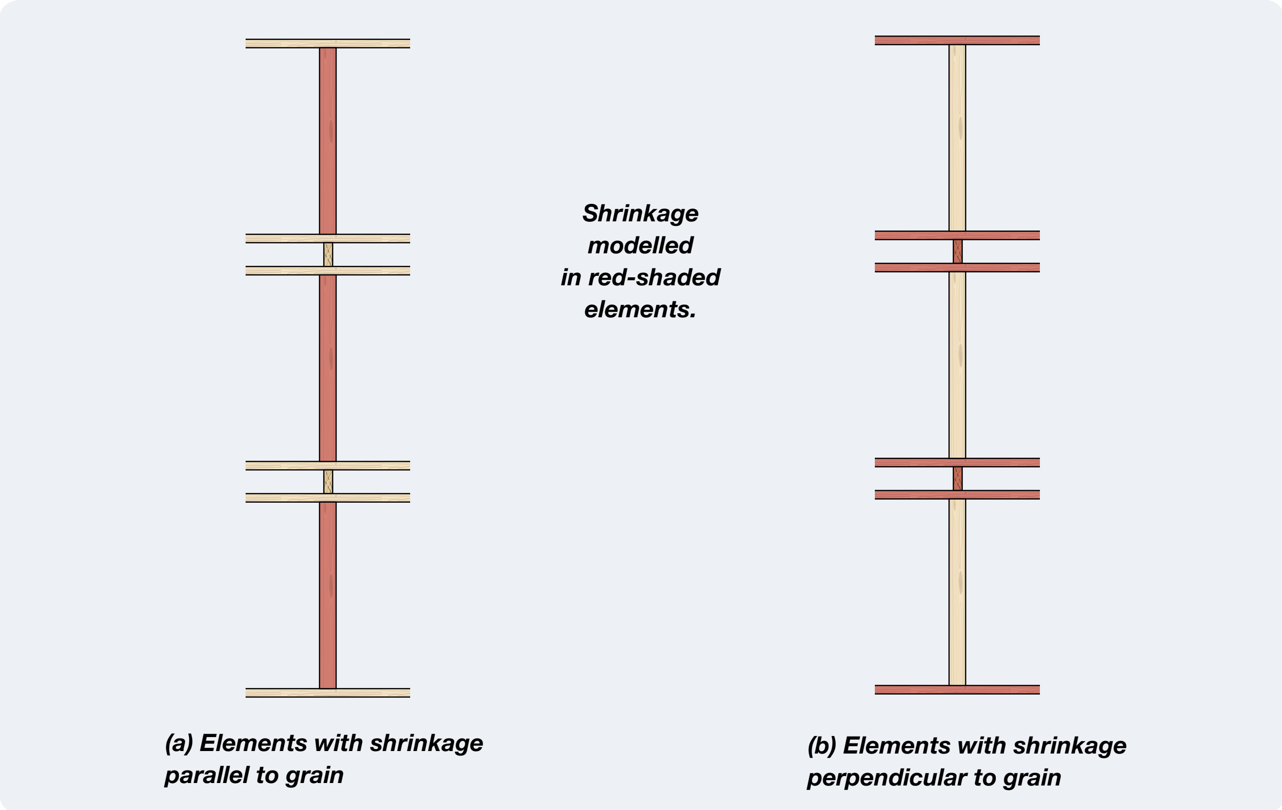


Figure 6.1: Shrinkage leading to shortening of building.

Shrinkage rates used are based on Unit Tangential Movement, which is the percentage tangential movement for any change in moisture content between 3% to fibre saturation point of the timber species. The Unit Tangential Movement is listed in column 14 in Table 1 of AS 1720.2 (and column 9 in Table G1 in AS 1684 Parts 2 and 3). The total estimated building shortening due to shrinkage parallel to grain is given by Equation 6.2.

$$\delta_{s,\ell} = U_\ell(\Delta mc)L \tag{6.2}$$

where:

$\delta_{s,\ell}$ = Total shrinkage parallel to grain (mm)

U_ℓ = Unit movement parallel to grain ~1/40 of Unit Tangential Movement of the species (mm/mm/ % change in *mc*)

Δmc = In-service change in moisture content (% change in *mc*)

L = Total length of wall studs in the height of the building (mm)

As an example of shrinkage rate calculation, radiata pine has a Unit Tangential Movement of 0.27% mm/mm/ % change in *mc*.

$U\ell$=1/40×1× 0.0027= 0.0000675 mm/mm/ % change in *mc*.

Estimating shrinkage perpendicular to grain

Shrinkage perpendicular to grain in wall plates, rim beams and floor joists (refer to Figure 6.1 (b)) also contributes to reduction in the height of the building. The shrinkage rate perpendicular to grain used in this check is Unit Tangential Movement, which is listed in column 14 of Table 1 of AS 1720.2 (and column 9 in Table G1 in AS1684 Parts 2 and 3). The total estimated building shortening due to shrinkage perpendicular to grain is given by Equation 6.3.

$$\delta_{s,p} = U_p(\Delta mc)d_p \tag{6.3}$$

where:

$\delta_{s,p}$ = Total shrinkage perpendicular to grain (mm)

U_p = Unit Tangential Movement of the species (mm/mm/ % change in *mc*)

Δmc = In-service change in moisture content (% change in *mc*)

d_p = Total depth of timber in plates and joists in the height of the building (mm)

As an example of shrinkage rate calculation, radiata pine has a Unit Tangential Movement of 0.27%. U_p=0.0027 mm/mm/ % change in *mc*.

6.1.2 Parallel to Grain Compression Deformation and Creep

The compression load in walls differs from floor to floor, hence the axial shortening of walls will also be different from floor to floor. The estimate of axial shortening of the studs under load must be evaluated at each floor level and summed over the height of the building as shown in Equation 6.4.

$$\delta_{c,\ell} = \sum\nolimits_{floors} \frac{j_2 N_{c,i} L_i}{E_i A_i} \tag{6.4}$$

where:

$\delta_{c,\ell}$ = Total compression shortening parallel to grain (mm)

j_2 = Duration of load factor for long-term loads = 2

$N_{c,i}$ = Long-term compression forces in a stud in storey *i* (kN)

L_i = Length of a stud in storey *i* (mm)

E_i = Modulus of elasticity (MoE) of studs in storey *i* (MPa)

A_i = Cross-sectional area of studs in storey *i* (mm^2)

If vertical members in floor trusses, or stiffeners in I-joists, are part of the gravity load path, the shortening of these elements can be calculated for each storey and added to the deformation parallel to grain.

For long duration loads, the load in each stud ($N_{c,i}$) is given by $G + 0.4Q$ and j_2 =2. Production typically targets a MoE 5 to 10% higher than the design value.

It is conservative to assume that on average studs will have an E of 1.05 x the design MoE.

6.1.3 Perpendicular to grain deformation and creep of beams and wall plates

The compression load in studs differs from floor to floor, and these loads are transferred to the wall plates and floor joists or trusses that resist the loads by compression perpendicular to grain.

The axial shortening of the building caused by deformation and creep of elements loaded perpendicular to grain will be different from floor to floor. Axial shortening of these elements must be evaluated at each floor level for any elements in the vertical load path and summed over the height of the building using Equation 6.5.

$$\delta_{c,p} = \sum\nolimits_{floors} \frac{j_2 N_{c,i} d_{2,i}}{E_{p,i} A_{p,i}} \tag{6.5}$$

where:

$\delta_{c,p}$ = Total compression shortening perpendicular to grain (mm)

j2 = Duration of load factor for long-term loads = 2

$N_{c,i}$ = Long-term compression forces in a stud in storey *i* (kN)

$d_{2,i}$ = Height of elements perpendicular to grain in storey *i* (mm)

$E_{p,i}$ = Modulus of elasticity perpendicular to grain in storey *i* (MPa)

$A_{p,i}$ = Loaded cross-sectional area of elements perpendicular to grain in storey *i* (mm^2)

The deformation perpendicular to grain is caused by stresses throughout the depth of the loaded section. Because of lateral dispersion of the load, the area changes throughout the depth as shown by the red shading in Figures 6.2 and 6.3. An average cross-sectional area is used and is shaded in these figures.

For long duration loads, the load in each stud ($N_{c,i}$) is given by $G + 0.4Q$ and j_2 =2, and this load is transferred to the elements perpendicular to grain.

The MoE perpendicular to grain ($E_{p,i}$) is j_r x MoE parallel to grain given in design properties.

Typically, for clear wood, j_r = 1/30. However, if knots are present in the loaded area, the MoE perpendicular to grain can be close to the MoE parallel to grain. Current research is investigating the value of j_r that applies for different Australian species of timber and engineered wood products. Until the results are released, it is recommended that designers use j_r = 1/25, which allows for the presence of knots in some of the elements in the height of the building.

The dimensions of the loaded cross-sectional area vary through the thickness of the elements as the bearing stresses are distributed. Examples are shown in Figures 6.2 and 6.3. A realistic bearing area in these elements is larger than the cross-section of the stud. Different expressions are used for each element in the vertical load path:

- Platform framing involves top and bottom plates and floor joists.
- Semi-balloon framing involves only top and bottom plates. (Balloon framing has top and bottom plates that are not in the vertical load path, leaving no elements loaded perpendicular to the grain.)

Solid floor joists and beams (for example)

Where solid timber joists or beams are parallel to the wall plates (as with the use of rim-beams or blocking), there are no stress concentrations under studs, and the compression of the joists or beams is very small and can be ignored. However, where solid timber joists or beams are perpendicular to the wall plates as shown in Figure 6.2 (a), there are high bearing stresses under the studs and the deformation needs to be calculated using Equation 6.5 with the dimensions shown in Figure 6.2 (a), and $A_{p,i}$ as determined using Equation 6.6. Where the wall is at the end of a joist, Figure 6.2 (a) applies and a quarter of the beam or plate depth can be used for the calculation of $A_{p,i}$.

If the wall bears within the length of the joist or beam, then Figure 6.2 (b) applies, but the calculation for the wall at the end of the joist or beam gives a conservative estimate.

$$A_{p,i} = max\left(b_2 + \frac{d_3}{2}, k_7 b_2\right) \times b_3 \quad (6.6)$$

where:

k_7 = Length of bearing factor defined in Table 2.6 in AS 1720.1 for length of bearing = b_2

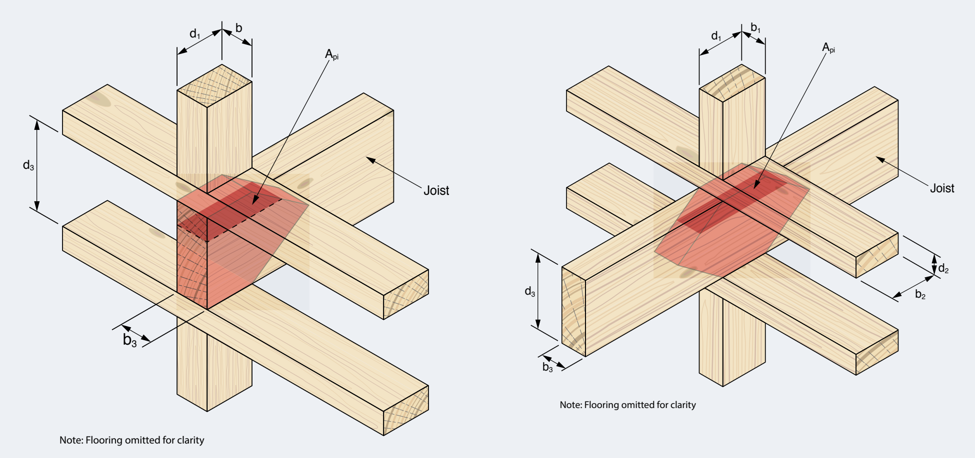


Figure 6.2a: Bearing perpendicular to grain in solid floor joists (left image: load at edge, right image: load at centre).

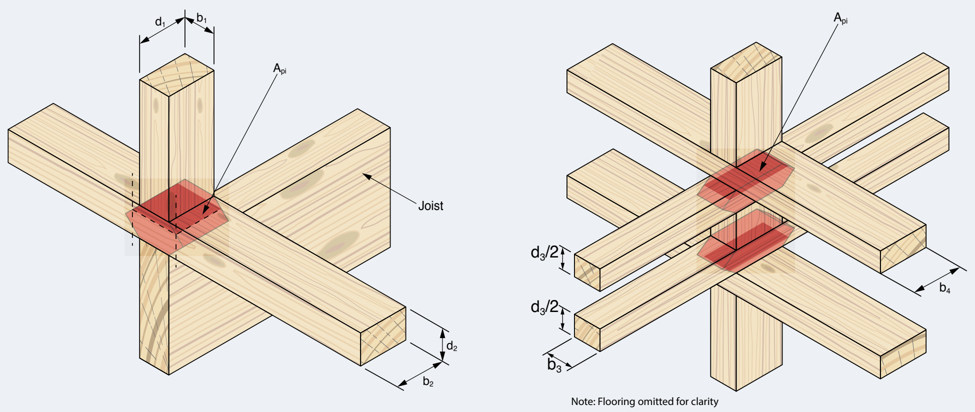


Figure 6.2b: Bearing in top plate (left) and top and bottom floor truss (right).

Examples of bearing area calculations for different scenarios can be found in Timber Design Handbook: in accordance with the Australian Limit State Timber Design Code AS 1720.1-2010: timber structures, Part 1: Geoffrey N. Boughton, Keith I. Crews.

Mass-timber

Where the vertical load path passes through a mass timber floor, the floor panel is loaded perpendicular to grain under the wall plate. A similar technique to that illustrated for floor joists in Figure 6.3 is used to estimate $A_{p,i}$. Again, there will be different values for walls above the edge of a panel and those over the interior of a panel. Various techniques exist for stiffening the load path through the perpendicular to grain CLT floor panel such as stiffening with screws or inserting solid plugs parallel to grain. These bespoke responses and should be developed in detail on a case-by-case basis with fabricators/manufacturers. A detailed calculation for CLT shortening can be found in Appendix 2

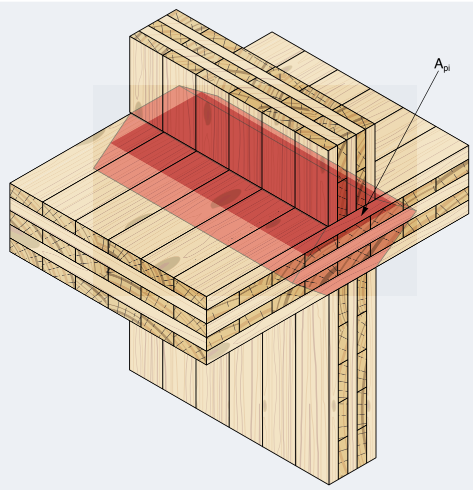


Figure 6.3: Bearing perpendicular to grain in CLT.

6.1.4 Settlement or embedment of joints

The large number of joints in the vertical load path due to fabrication of the structural elements will "bed-in" and close up as the structure is assembled and the vertical loads on the elements increases (Figure 6.4) Illustrates the location of these fabrication joints).

For example, saw cuts on the ends of studs that are not quite orthogonal, or grit in the joint at fabrication can cause slight gaps during construction that close as vertical loads are applied.

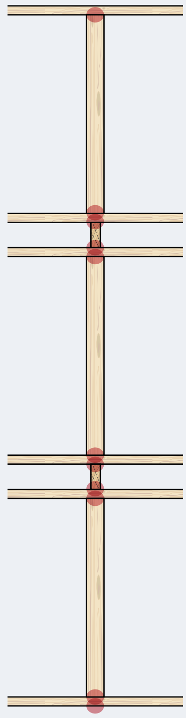


Figure 6.4: Closure of joints in the vertical load path.

The contribution of joint closure to building shortening can be estimated using Equation 6.7, allowing δ_{gap} = 0.2 to 0.5 mm per joint in the vertical load path:

- prefabrication in tightly controlled factory conditions generally gives smaller gaps e.g. 0.2 mm
- on-site construction may give larger gaps, e.g. 0.5 mm.

$$\delta_j = n_{joints}\delta_{gap} \quad (6.7)$$

where:

δ_j = Total shortening due to joint closure

n_{joints} = Number of joints in the vertical load path

δ_{gap} = Estimated gap per joint

6.1.5 Dimensional tolerances

All timber members have dimensional tolerances. For the seasoned products that are likely to be used in mid-rise timber buildings, the tolerances on cross-sectional dimensions for seasoned timber are typically (–0, +2-3 mm). Therefore, top and bottom wall plates may be thicker than the nominal dimension and floor joists may be slightly deeper than the nominal dimension when first installed.

It is difficult to estimate the effect of dimensional tolerances, but for seasoned timber products, they may counteract the shortening caused by the factors discussed above and quantified in Equation 2.4. One method of incorporating dimensional tolerances into design calculations is to offset the accumulation of dimensional tolerances against the accumulation of settlement of joints and use $\delta_j = 0$.

In construction, the effect of dimensional tolerance of timber and joint closure is taken into account by making adjustments to the floor level. A common building practice is to review the accumulation of timber tolerance at a number of stages during construction and adjust the wall height on the next floor to maintain the designed building height.

6.2 Support Structure Movement

Differential movement occurs when different parts of the building deform or settle different amounts. If shrinkage, creep or foundation movement are causing the differential movement, it may increase with time and only become obvious sometime after the completion of the building. For example, concrete lift or stair cores settle soon after installation, but the timber frame settles over a longer period.

Heavyweight concrete cores may also lead to larger differential vertical foundation movement relative to the lightweight timber structure of the frame either because the heavyweight core settles more or because it settles less if it has a distinct foundation solution. (Dead loads are a higher percentage of the total load for concrete compared with timber frame).

The result of differential movement is that lines or surfaces that are meant to be level are no longer straight. This may cause cracking in brittle linings or floor coverings, or slopes in floors. Figure 6.6 shows damage to plumbing as the riser did not settle, but the vertical movement of the timber frames has dragged the horizontal waste pipe downwards. Other effects of differential settlement may include:

- misalignment of floors at lift or stair cores
- misalignment of flashings (effects visual and weathertightness)
- damage to facades or external building linings.

Care to avoid the consequences of differential movement is particularly important at the interface between different materials, for example:

- timber frames and concrete cores
- timber frames and steel posts or portals
- timber frames and masonry veneer.

Figure 6.6: Plumbing broken due to axial shortening of the timber frame. (Image is taken from Accommodating Shrinkage in Multi-Storey Wood-Frame Structures Wood Product Council)

Designers of mid-rise timber buildings should take particular care when designing transfer and supporting structures to ensure that the long-term incremental deflections are able to be accommodated. The higher in-plane stiffness of a CLT panel, when compared to a timber-frame wall could mean that a concrete slab below could end up deflecting more than the CLT wall and leaving a gap between the timber and concrete. This could cause issues with load concentrations in the walls or fire/acoustic issues with the gapping.

Typically designing supporting structures for transfer structure performance and deflections of span/500 or a maximum of 10 mm would be suggested a limit to start design and to be refined.

Differential movement can be accommodated in mid-rise timber buildings with appropriate details for structure, finishes, building services, etc.

6.2.1 Suggested differential limits

Defining target limits for movements and tolerances should be done on a project-by-project basis by the design team. However, it is recommended that the anticipated vertical differential movement across a storey (after installation of linings and fixings) is limited to approximately 6 mm in adjacent bays. The total building shortening may be a larger value than this but restricting the differential settlement in adjacent bays to a maximum of 6 mm typically allows the successful installation of linings and services without introducing unreasonable costs for additional specialist connections and details.
A limit should be included in the standard notes on structural drawings, specifications and architectural specification so that building consultants and trades are aware of the expected differential movement per storey. Where installations may be susceptible to differential movement a separate calculation of the expected movement, may be required and should be noted on drawings for that installation.

6.2.2 Potential differential movement problems and solutions for timber-framed buildings

Table 6.3 summarises potential differential movement issues in mid-rise timber-framed buildings and provides some suggested solutions.

Table 6.3: Examples of differential movement issues.

Element	Issue	Suggested solution
Structural steel supports	Where heavily loaded beams are supported by steel columns, there may be differential movement with the surrounding timber structure. This movement may lead to cracking of linings.	Instead of steel columns, use timber elements, e.g. LVL, GLT or multiple timber studs (see Figure 6.7) to support the beam.
Propped balconies	Where balconies are supported on steel, concrete or masonry columns, differential movement between timber structure and the balcony columns may cause the balcony floor to slope towards the building (see Figure 6.9).	Design balconies with an exaggerated slope away from the building to accommodate differential movement and provide adequate drainage away from the building.
Lift cores	Differential movement between the timber floor and concrete, masonry or steel frame cores may cause misalignment of the floor level at the lift or stair doors.	Use articulated links in the floor surrounding the core as detailed in Section 7.4.4.
Services	Problems may develop at junctions in rigid pipes and ducts for plumbing, air conditioning or gas supply where timber frames settle in service.	Incorporate expansion joints or links in the services, and allow clearance between services and the timber frame. Limit axial shortening to no more than 6 mm per storey.
Linings	Linings may buckle and crack as the timber frame behind them shortens.	Allow a gap at the bottom of linings to accommodate differential movement. The gap can be hidden behind skirting boards.
External walls location where most differential movement occurs		
Cladding	The cladding is exposed to significant variations in temperature compared with the timber frame, which causes expansion and contraction of the cladding.	Detail joints in façade elements to accommodate both differential movement and expansion and contraction of the cladding.
Flashings	Standard flashings do not allow enough overlap to accommodate the axial shortening or differential movement that may occur in a timber-framed building.	Allow generous overlap to accommodate the total differential movement (see Figure 6.9).
Brick veneer	Bricks expand over time (brick growth). This combined with the shortening of the timber frame accentuates differential movement. It can lead to problems with flashings, windows and penetrations.	Use low expansion bricks. Allow rotation of brick ties as the timber framing shortens and the bricks expand. Install brick ties so that they slope down away from the building. Support the brickwork at each floor onto the timber frame structure itself – note that this will increase the load on the external wall studs. An alternative is to use faux bricks tiles that are fixed directly to the timber framing.
Penetrations into facades	Windows and services in external walls need to accommodate some axial shortening of the wall in which they are fixed. If not, the load will be transferred through the window frame or the services and lead to jamming or breakage. Details at the top of walls are particularly susceptible to damage from differential movement.	Windows need clearance at the top and bottom (see Figure 6.10) Eaves and Verges require extra clearance above the cladding (see Figure 6.11) Service penetrations should have extra clearance on the underside of the service (see Figure 6.12).

Figure 6.7: Timber studs supporting steel beam.

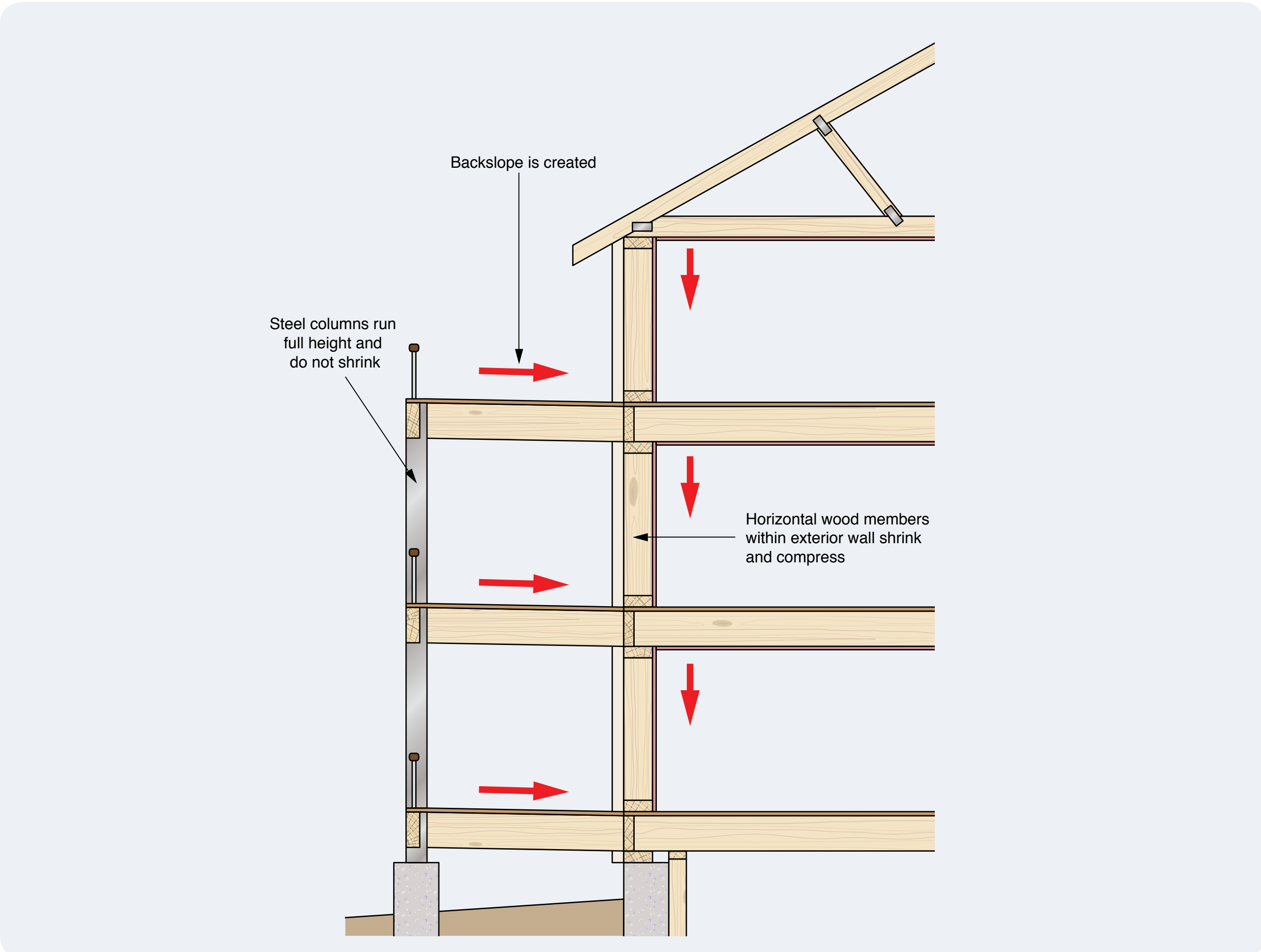


Figure 6.8: Independently supported balcony.

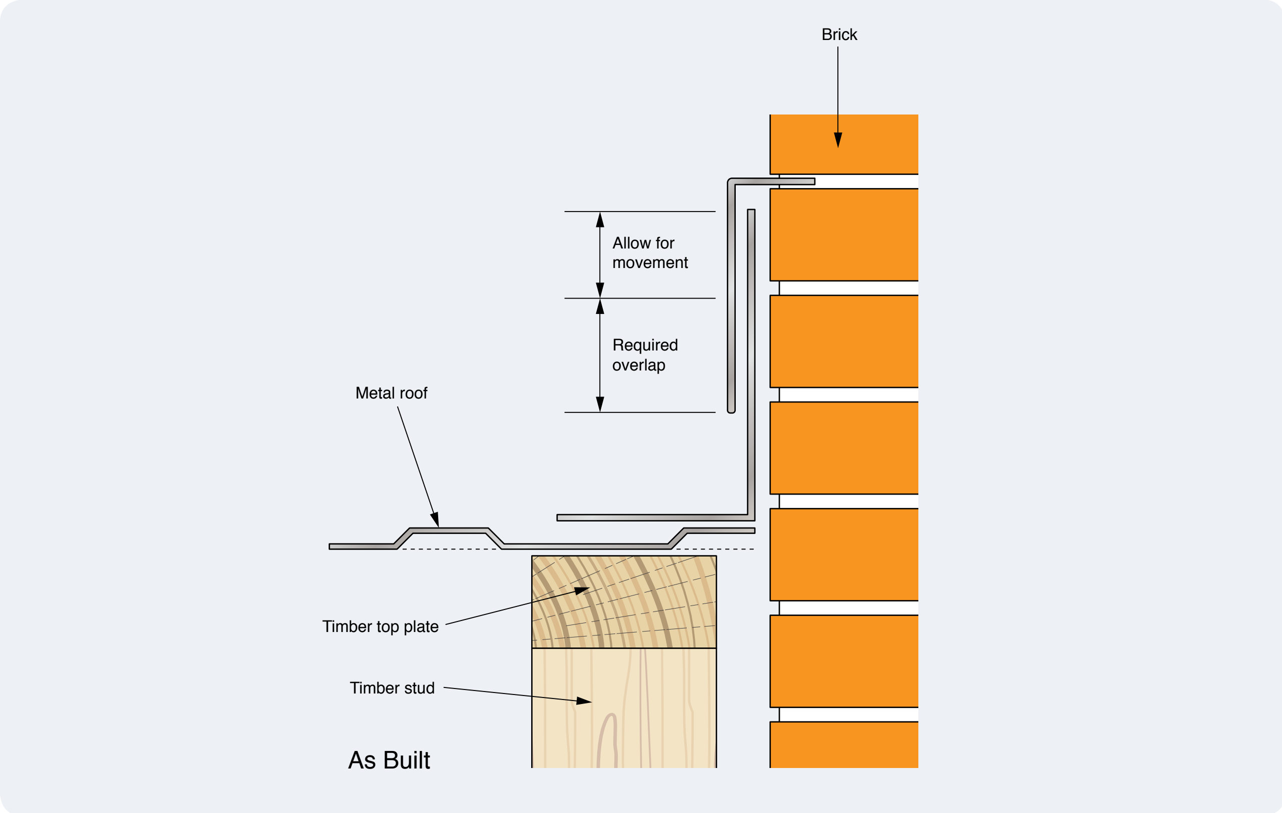


Figure 6.9: Flashing design for greater over lap to accommodate axial shortening.

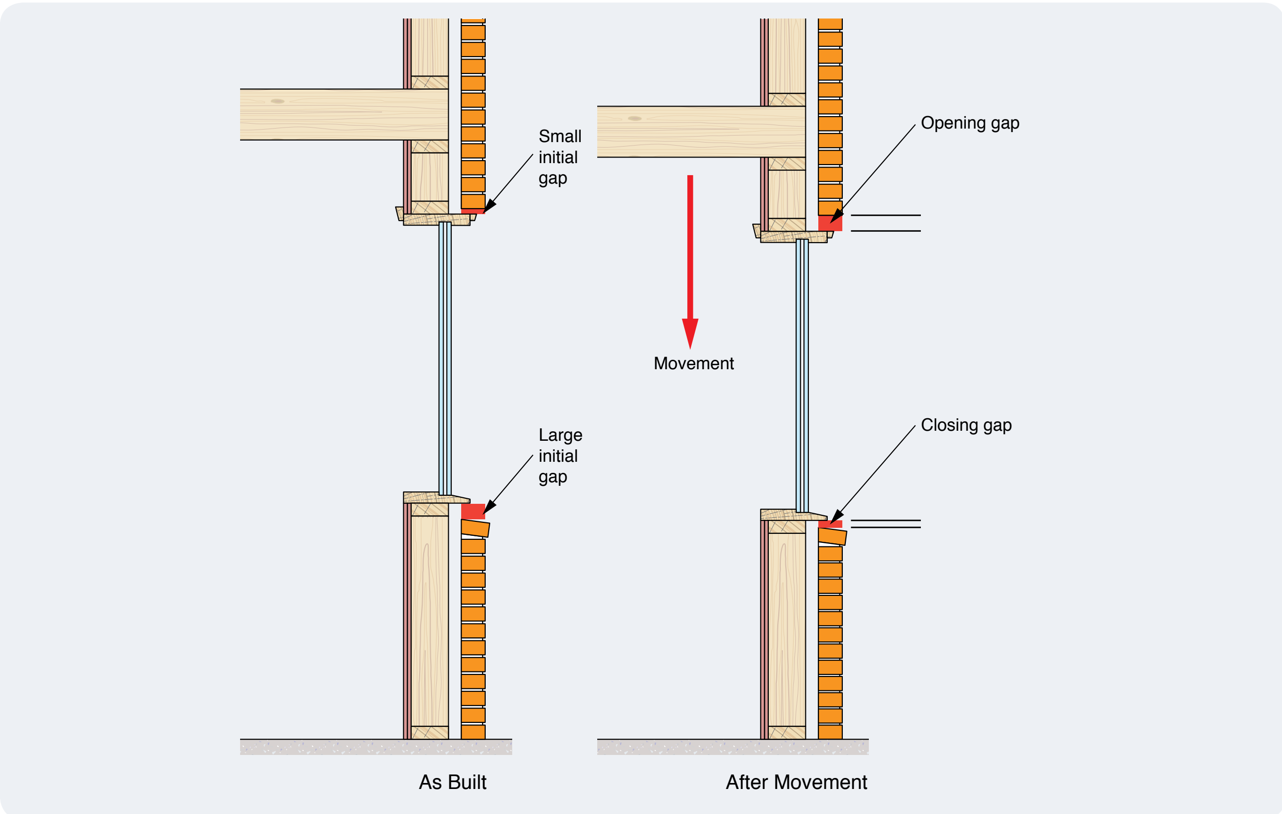


Figure 6.10: Extra clearance required around windows.

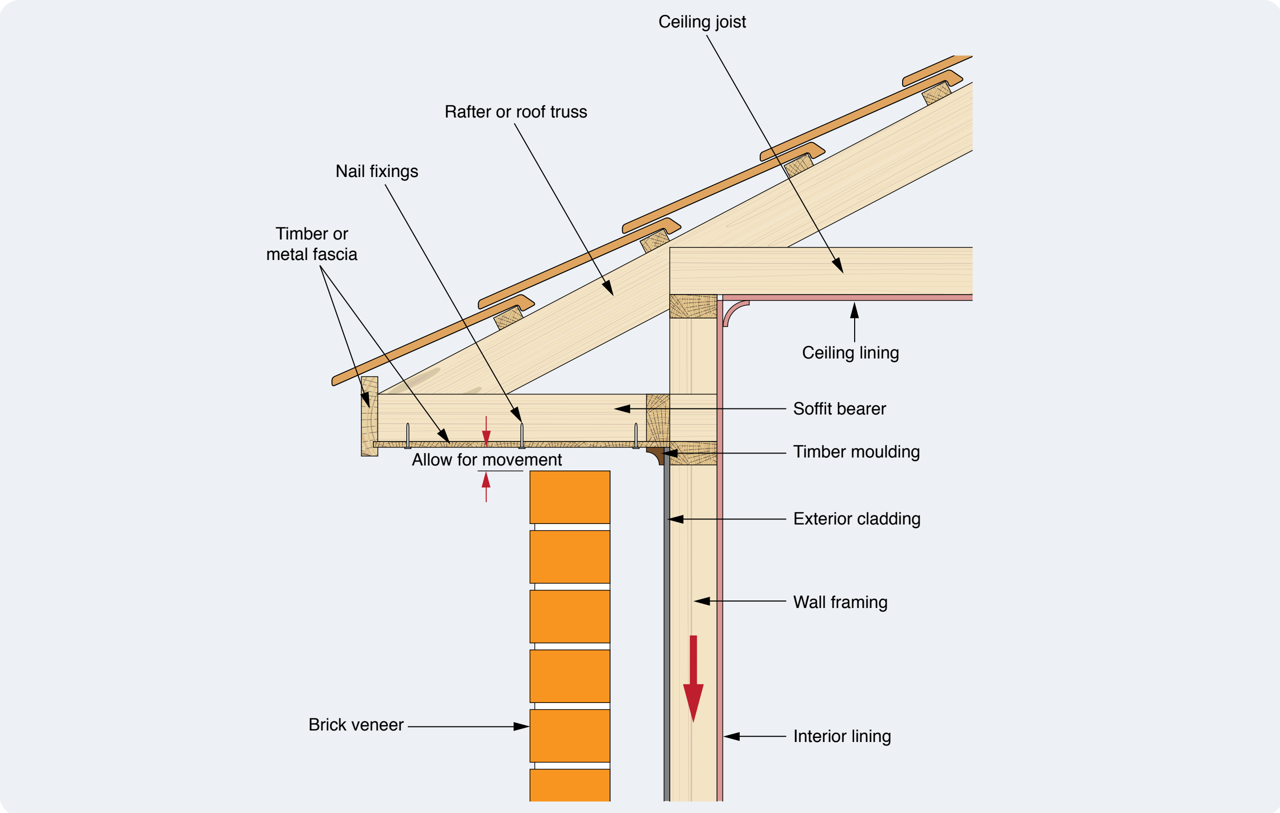


Figure 6.11: Gap to brick veneer under eaves.

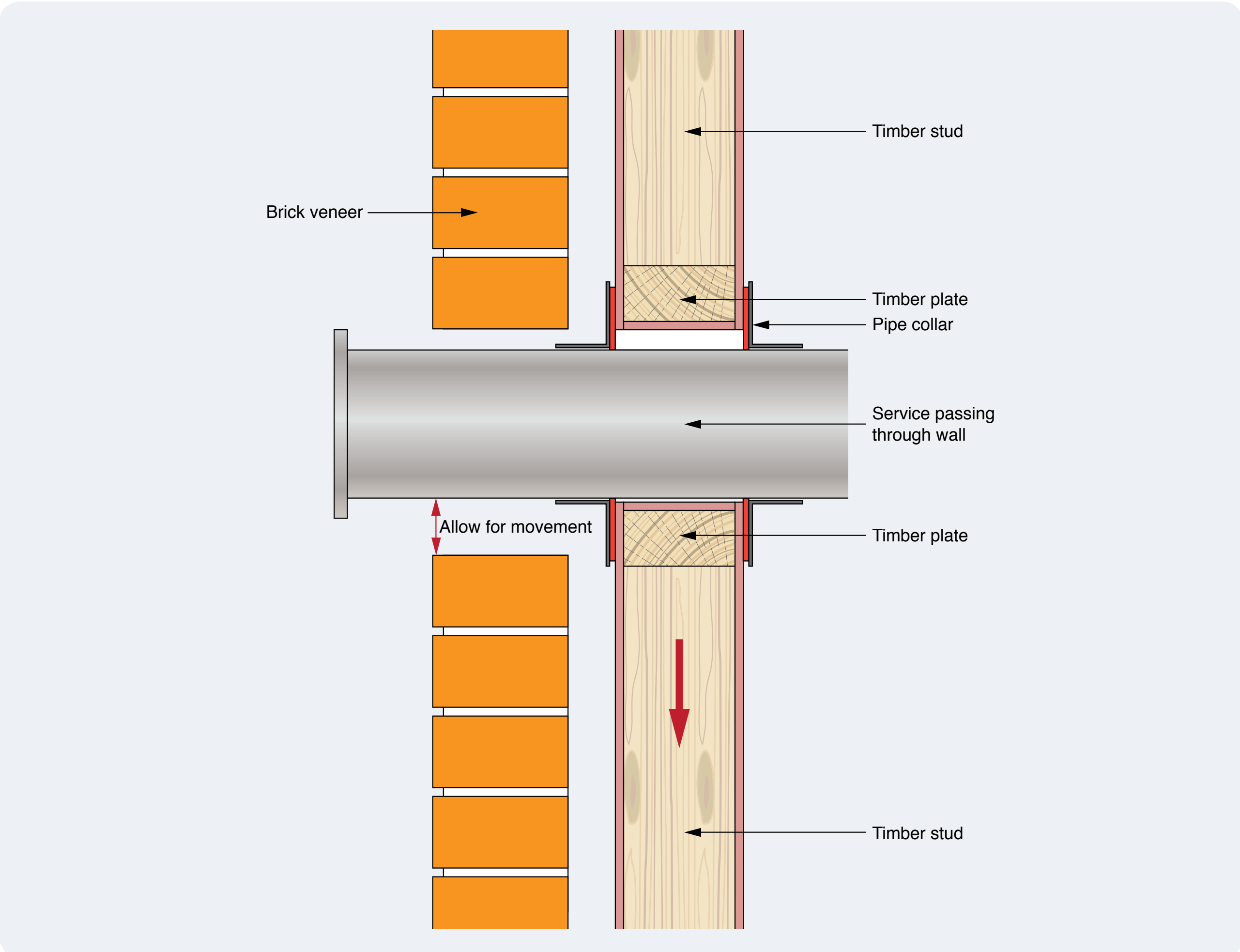


Figure 6.12: Extra clearance under service penetrations.

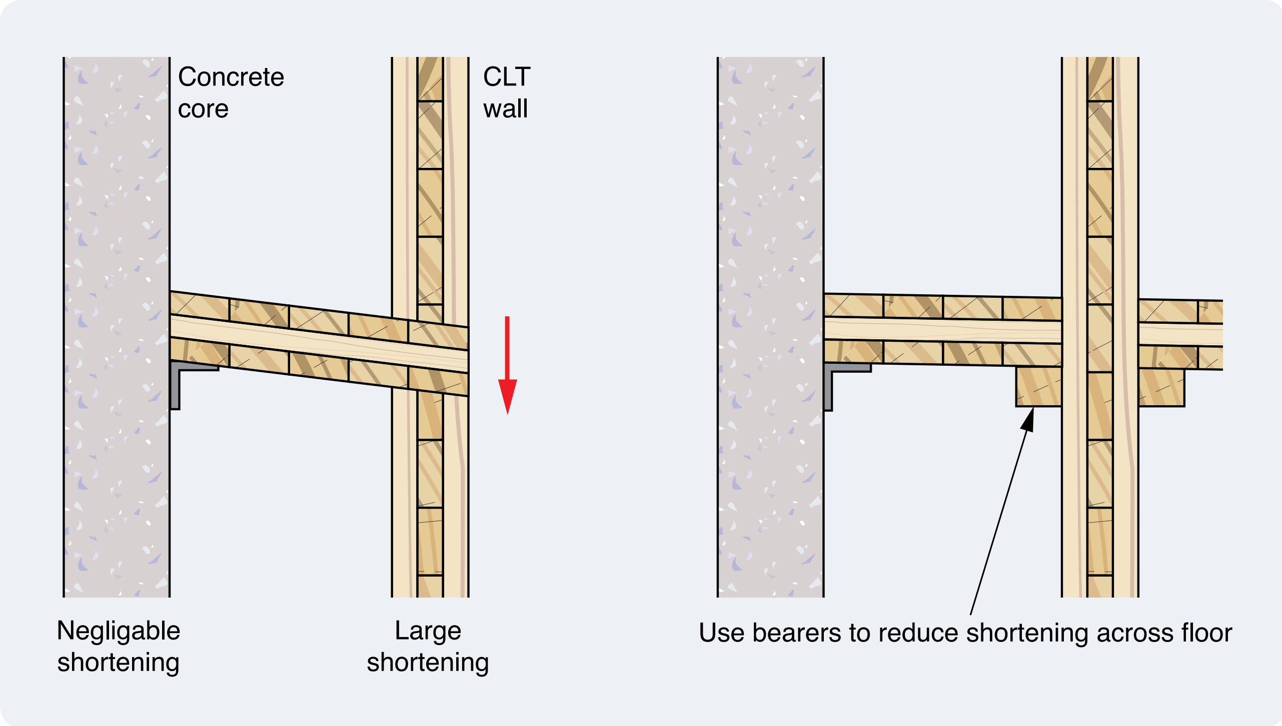


Figure 6.13: Use of bearers across the floor to reduce shortening.

7 Lateral Load Resistance Design

Mid-rise timber buildings resist lateral loads through a stability system typically comprising cores and/or bracing walls in combination. Material choice, detailing and form of the stability elements vary between buildings. The distribution of load into the components of the stability system depends on the relative stiffness of the contributing elements. Timber buildings are relatively lightweight structures so engineers must consider the need for specific tie-down elements within the stability system. The design approach for typical stability systems and combinations of stability elements is discussed in this section.

To evaluate the structural resistance to lateral loads on mid-rise timber buildings:

- determine design and layout of all walls
- calculate lateral load on building from wind and earthquake actions
- determine layout of contributing cores, bracing and shear walls
- calculate lateral load resistance of cores
- evaluate capacity required for bracing, shear walls or moment frames working with the cores
- design cores, bracing and shear walls
- evaluate diaphragm capacity to distribute load between stability elements
- consider global overturning.

7.1 Stability Systems

Lateral loads on mid-rise buildings are resisted by a three-dimensional system of elements that carry the lateral loads from any part of the building to the ground.

- Elements such as external walls and windows carry load to the floor systems by out-of-plane bending.
- Horizontal diaphragms transmit the load to the vertical stability elements.
- Vertical stability elements such as cores, bracing walls and frames transmit the lateral loads from one level to the one below and down to the ground.
- Stability systems must transfer shear forces to the ground and resist the global over-turning. Timber mid-rise buildings are relatively lightweight and specific tension connections to the foundations are typically needed. Steel rods near the outside of the building bracing can resist global overturning tying to foundation level.
- It is essential to align bracing elements vertically through the building for an efficient structural response. Forces generated within bracing elements which do not align floor to floor will lead to complex structural behaviours and complex, expensive detailing.

Figure 7.1 illustrates one principle of providing lateral stability in a mid-rise building.

- Bracing walls and diaphragms in an individual unit are represented by the single box on the left.
- The stacked bracing walls in units above each other transfer lateral load down through the building to the ground (centre diagram). Lateral loads in bracing elements are larger in walls of the lower storeys. If the stacked bracing walls and connections are stiff enough, they control building sway under lateral loads.
- Bracing walls combine with cores to resist lateral loads on the building with load shared based on stiffness and position.
- One approach is to share the bracing loads between all units (as in a stacked prefabricated volumetric system for example), or to focus bracing into a number of bays, highlighted with a dashed line on Figure 7.1 for example.

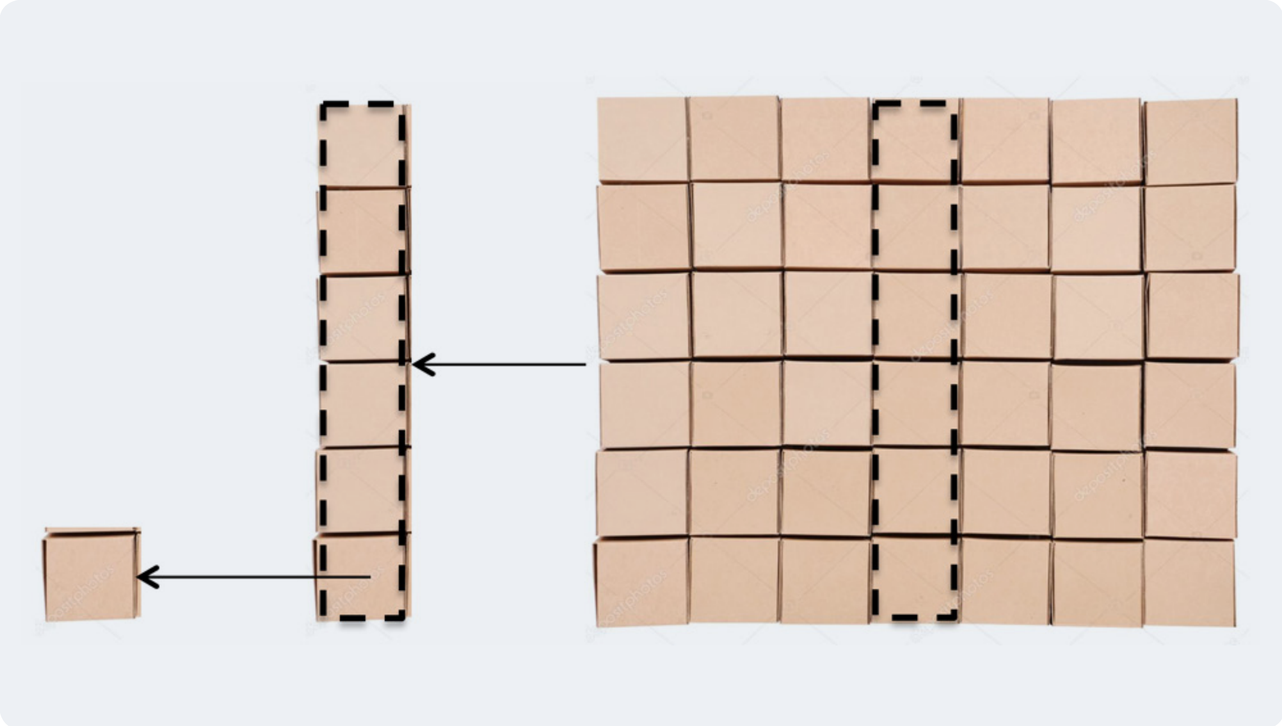


Figure 7.1: Models of stacked rigid units in a building.

To achieve effective lateral load resistance, the centre of stiffness of the overall stability systems should be near symmetrical to minimise torsional effects under resistance of lateral loads. Options for the arrangement of vertical bracing elements in mid-rise buildings that resist lateral loads are illustrated in Figure 7.2 and 7.3. They are, in order of stiffness in resisting lateral loads:

- cores
- bracing walls
- moment frames.

Most stability systems will feature a combination of systems.

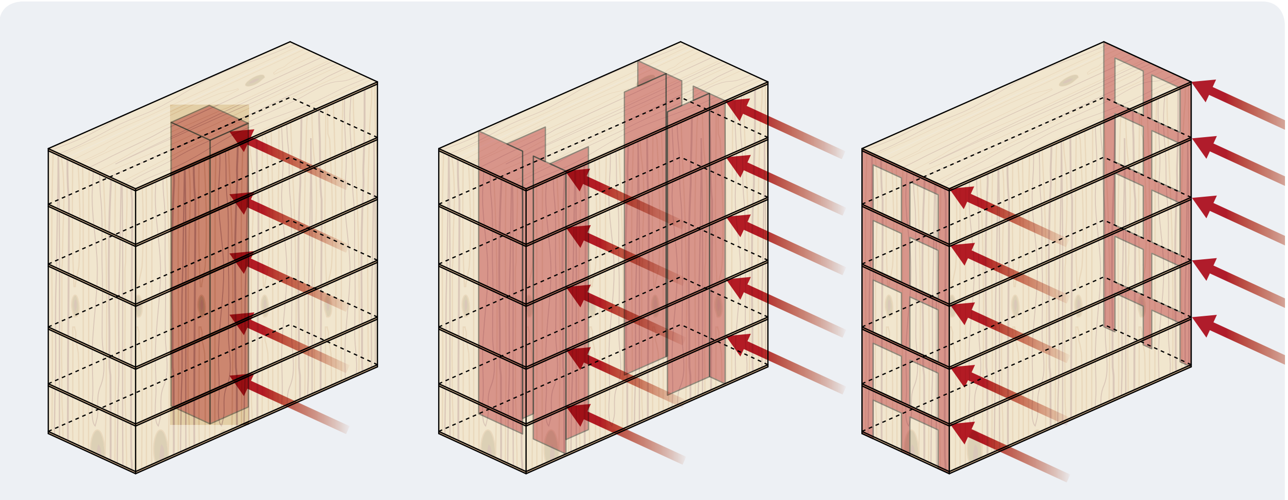


Figure 7.2: Vertical bracing elements.

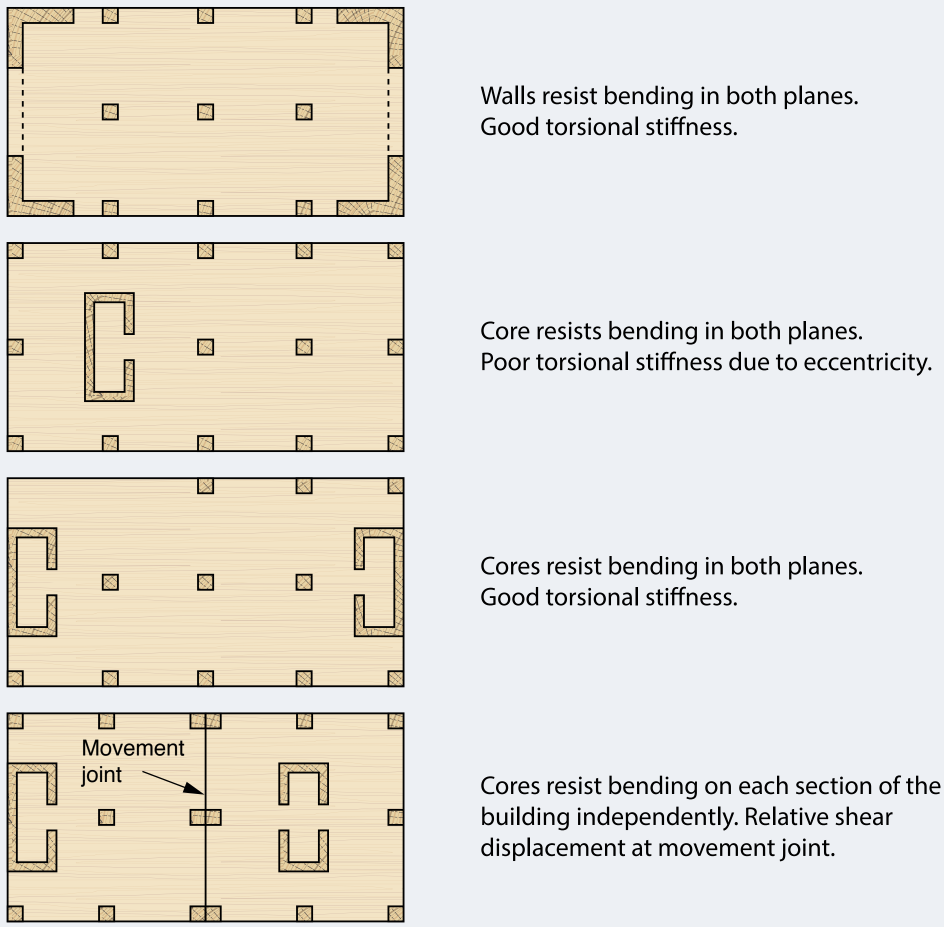


Figure 7.3: Plan view of different core and bracing wall layouts and their lateral response.

7.1.1 Cores

Cores accommodate stairs, lifts, and mechanical equipment and building services distribution, but may also be used as major structural elements in resisting lateral loads. Cores of mid-rise timber structures may be timber or other materials. Timber core behaviour is a function of the capacity of the connections between the vertical plates in the core which influences the extent to which the core section may be considered as composite. The centre of resistance of a core structure is the shear centre, which might be outside the core itself. The location of the shear centre of the core should be taken into account when designing the preliminary bracing system. Depending on the floor span directions the core may or may not carry a significant portion of the gravity loads of the building.

Key factors to be considered in core requirements, layout and position of core in the overall building layout are:

- Consult lift manufacturers early in design process. Lift tolerance, movement and fire requirements can be onerous.
- Cores have a major role in lateral load resistance of mid-rise timber buildings. Position cores symmetrically (or target symmetrically with overall stability system).
- Consider effective wall height as restraint to walls may not be present at each floor.
- Timber cores –
  - shear connections at corners affect the capacity, stiffness and the position of shear centre
  - fire resistance level (FRL) and resistance to incipient spread of flame (RISF) are required for the core, as well as non-combustible door sills
  - with framed cores, walls are designed as bracing walls, calculate axial shortening
  - with mass-timber cores, design as separated shear walls or vertical cantilevers, possible reinforcement to panels at openings using long screws, axial shortening minimal.
- Concrete cores –
  - may require specifically designed connections with timber elements to accommodate vertical movement of timber
  - allow for more generous construction tolerances in concrete compared with timber.

7.1.2 Bracing walls

The structural requirements for bracing wall layout must coordinate with the architectural layout requirements. The typical floor spans adopted in timber mid-rise suit typical apartment planning. The walls around each apartment satisfy multiple performance requirements, including vertical load-bearing, fire and acoustic. Utilising these walls as shear walls as part of the stability system is straightforward. Bracing walls must be full-height between the top of the floor below and the underside of the floor above so they can transfer lateral loads from the diaphragm above to the top of other walls below. For this reason, partition walls within an apartment or around services areas of a building that sit within the fire compartment are generally not sufficiently connected to the floor diaphragm above and consequently cannot be readily used as part of the stability system.

- ***Where possible, use external walls as bracing elements as they provide the most efficient bracing stiffness against torsional loads.***
- ***Many load-bearing walls could also be bracing walls.***
- ***Bracing wall arrangement should be developed to be evenly distributed through the building plan. The most effective bracing walls will be wall panels without significant openings (windows and doors). As openings become larger so the structural behaviour of the wall tends towards a moment frame for stability which is far more flexible.***
- ***Bracing wall length should coordinate with the constraints of the system adopted such as sheathing panel size or CLT panel sizes.***
- ***Higher bracing capacity is required in lower storeys (see Figures 7.5 and 7.6).***
- ***An efficient design will consider floor and wall capacity for carrying vertical and lateral loads simultaneously. But not all bracing walls need to be loadbearing; it depends on floor span direction.***

Where the floor plan for each storey is identical, the bracing walls function as stacked bracing walls to transfer shear from one level to another. If there is a difference in wall layout between floors, transfer beams will be required to distribute vertical and lateral loads between the load bearing walls. Horizontal diaphragms can be used to redistribute bracing forces at levels with changes in the floor plan, but care is required in resolving vertical forces generated at transfers. The influence on bracing stiffness of transfer structures needs to be checked on a case-by-case basis. Although discontinuities at walls between SOUs have better acoustic performance, many building designs need continuous floors to provide diaphragm action.

7.1.3 Moment frames

In some cases, large openings in walls leave few long runs of uninterrupted wall that can be used for bracing walls. This makes it difficult to achieve bracing capacity on those walls. Moment frames can both span the openings as a lintel and provide bracing capacity. The moment frames are effectively portal frames with moment-carrying connections at the knees that give them rigidity under lateral load. Moment frames are typically less stiff than more common forms of bracing such as shear walls. They include:

- LVL or glulam portals
- plywood box systems
- moment carrying trusses (Figure 7.4)
- steel portals.

Figure 7.4: Moment-carrying frames (photograph is from USA).

7.1.4 Diaphragms

All conventional floor systems are designed principally for out-of-plane loads, but they also have the capacity to transmit in-plane loads. Where connections between panels are adequate, the whole floor panel can function as a continuous deep beam with supports at the vertical bracing elements from the storey below. Specific details at connections between floor panels might be required to achieve diaphragm strength, stiffness and connection with vertical bracing elements.

The requirements for acoustic separation can conflict with the requirements for diaphragm action. It is common to require upgrade to acoustic wall build-ups in order to allow continuous floors for diaphragm action. Openings and changes in floor level affect the performance of the diaphragm.

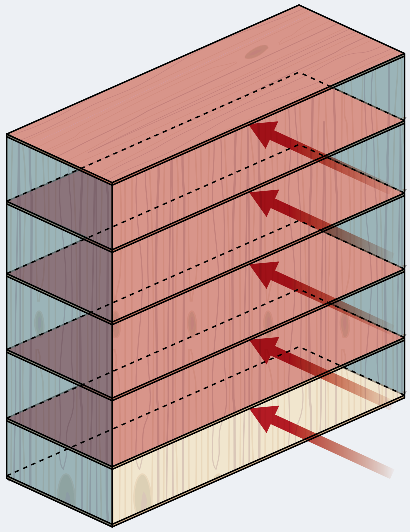


__Figure 7.5: Horizontal diaphragms transferring loads to end bracing.__

7.2 Overall building loads

Designers evaluate lateral loads to determine:

- global forces on the whole building
- loads on each single storey to evaluate the out-of-plane performance of external walls and windows, and of horizontal diaphragms
- accumulated lateral loads down the building for the design of the bracing system (cores and walls).

Mid-rise timber-framed buildings are usually more sensitive to lateral wind loads compared with concrete or masonry mid-rise buildings as they are lighter and more flexible, but less sensitive to earthquake loads for the same reasons. Lateral loads from wind and earthquake occur separately and the same bracing systems can be used to resist both actions.

7.2.1 Wind

The net lateral loads on the whole building are determined from the difference in pressure between the windward face and the leeward face of the building using AS/NZS 1170.2 (AS 4055 and AS 1684 cannot be used to determine bracing forces). Lateral wind forces on the whole building are independent of the internal pressure assumed in design. Area reduction factors cannot be used on either windward or leeward walls.

Figure 7.6(a) shows the tributary area (shaded blue) for pressures on the windward and leeward faces that contribute to lateral loads on the edge of a floor diaphragm. These loads are transferred to the top of all vertical bracing elements below the floor diaphragm.

Lateral wind loads at each horizontal diaphragm will be similar, as wind pressures do not vary much between storeys in buildings up to 25 m high. However, wind loads accumulate down the building so that bracing elements in the lowest storey carry the largest lateral loads. This principle is illustrated in Figure 7.6(b).

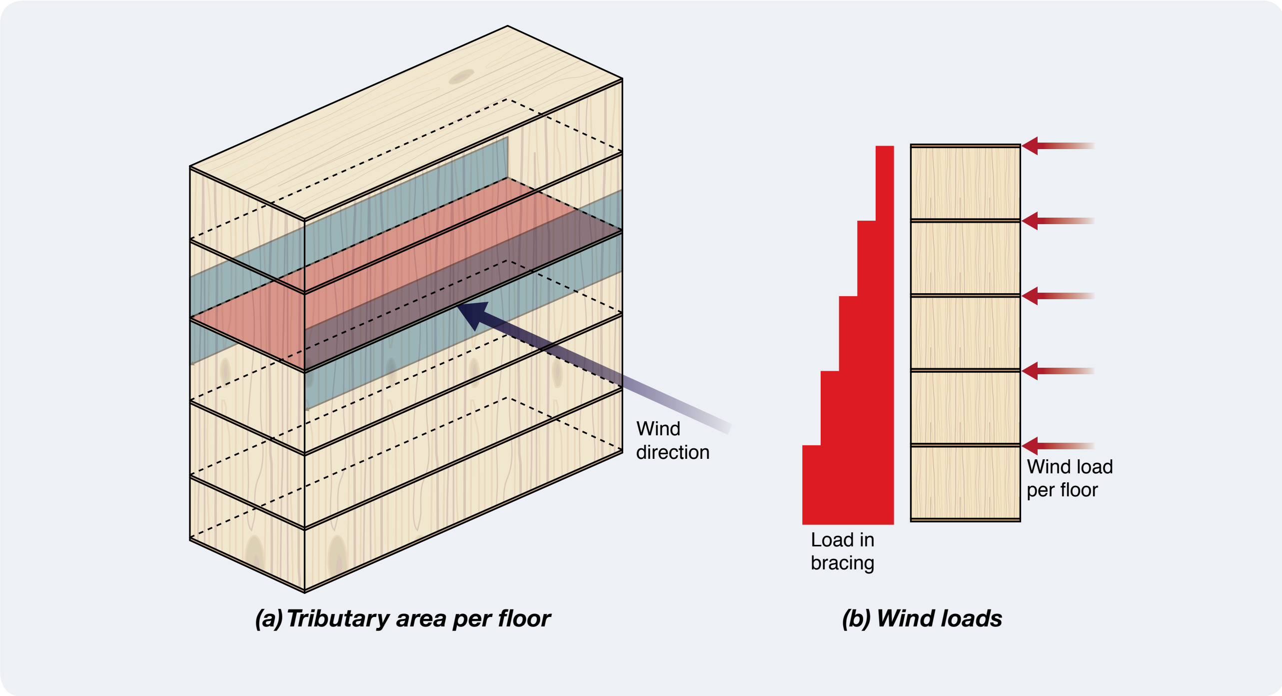


Figure 7.6: Lateral wind loads.

7.2.2 Earthquake

Earthquake loads can be evaluated using AS 1170.4. The earthquake loads are also applied at each floor level but model the weight of all items at that storey as shown in Figure 7.7(a). The horizontal earthquake loads per floor are higher for the upper floors than the lower floors. The distribution of shear forces through the structure is different to that for wind forces, as shown in Figure 7.7(b). The bracing elements on the lower floors still have to carry the highest bracing forces.

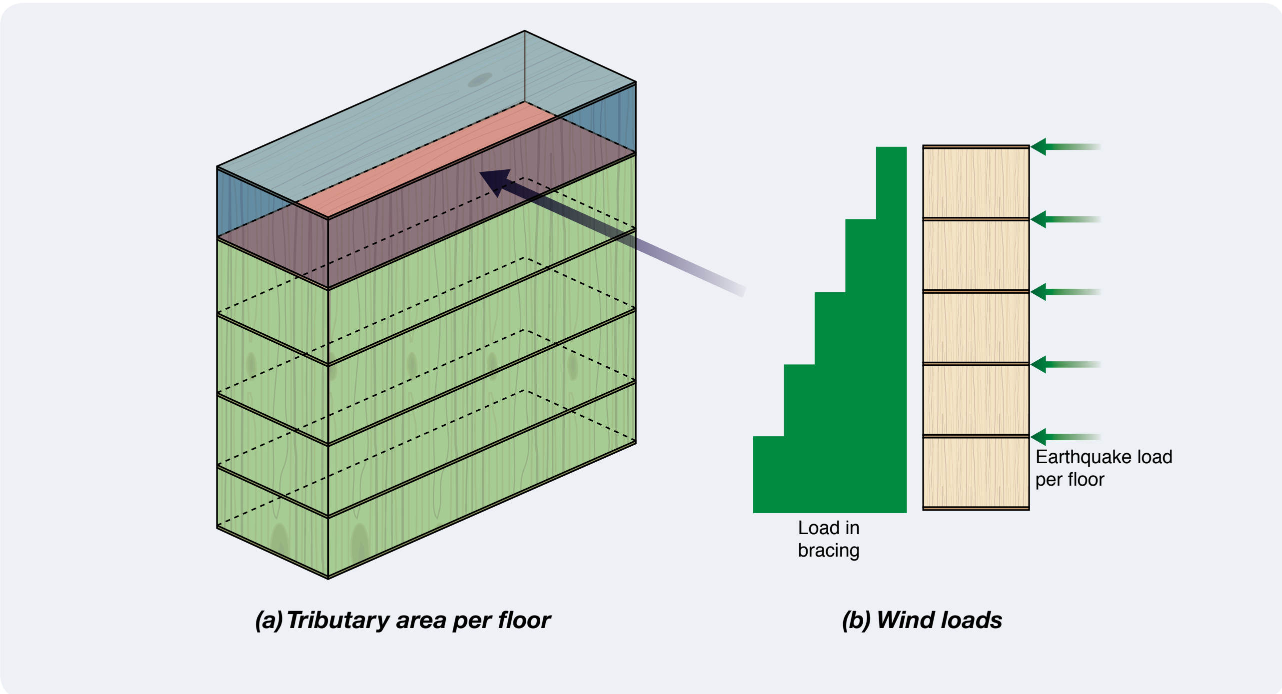


Figure 7.7: Lateral earthquake loads.

	Wind (kN per floor)	Earthquake (kN per floor)
Roof	69	196
Floor 7	207	392
Floor 6	346	555
Floor 5	484	686
Floor 4	622	783
Floor 3	761	849
Floor 2	899	881
Floor 1	968	881

7.2.3 Temporary bracing

Depending on construction sequence and stability system, temporary bracing may be needed. This should be considered on a case-by-case basis.

7.3 Load Distribution into Stability System

Lateral loads onto the building are transferred through the floor diaphragms into the vertical bracing elements. In most typical cases, the diaphragms are assumed to be stiff. Load distribution into the vertical bracing elements is proportioned according to relative the stiffness of the bracing elements.

For simple, symmetrical bracing arrangements with uniform bracing stiffness and diaphragm stiffness, load distribution can be estimated by hand as a first step. Refer to Appendix 1 and 2 for an example of this process. For most cases all vertical and horizontal elements that carry lateral loads should be incorporated into a structural computer model of the building. This allows the natural frequency of the building to be determined and horizontal wind and earthquake forces to be allocated for each element in lateral resistance system.

Methods for determining stiffness of systems are approximate and for timber structures the system stiffness is highly dependent on fastener/connection stiffness, whether timber-framed or mass-timber. In establishing load distribution through the bracing systems engineers are encouraged to conduct sensitivity-testing of the stiffness assumptions for the various components in order to determine maximum likely loads in the contributing elements. For example, consider a stiff core and flexible bracing walls to determined lateral core loads, and vice-a-versa to determine the lateral bracing wall loads. A reasonable range to sensitivity test stiffnesses determined to standards might be to half and double the estimated stiffnesses (0.5 x K and 2 x K).

The primary source of stiffness information for bracing walls or cores is test data on systems constructed to typical practices. For mass-timber systems the stiffness data on connections is typically from proprietary fastener suppliers. Refer to Appendix 2 for an example of the calculation process. For timber-framed construction, a number of methods exist for estimating the stiffness of a bracing wall combining assumptions on fastener stiffness, panel stiffness, edge member stiffness, and tie down stiffness. A detailed approach can be found in CSA O86 11.7.1.2, and A.11.7.1, which is described in Appendix 1. Figure 7.8 presents the close comparison of methods; AS 1684 (tabulated), AS 1720 (simple model), EC 5 (simple model), and CSA O86 (lower and upper bound). The comparison in Figure 7.8 considers fastener diameter, spacing and sheathing to match the AS 1684 standard practices in order to develop a relative comparison. Note that these methods for determining stiffness are approximate and are sensitive to the input parameters.

- AS 1684 (tabulated values) – considers the bracing capacity as presented in AS 1684 Part 2 or 3 Table 8.18 and assumes the capacity reported is governed by a limit of sway deflection of height/333. Note AS 1684 Part 2 or 3 Table 8.18 assumes a height of 2.7 m. For heights greater than 2.7 m refer to capacity multiple in Table 8.19 for other wall heights.
- AS 1720 (simple model) – considers bracing sway deflection due to three actions: horizontal shear of the fasteners in the wall plates, the tie-down of the fasteners to the bottom plate, and the panel shear stiffness of the sheathing. Fastener stiffness is taken from AS 1720.1 Appendix C. Tie down is assumed to be relatively stiff.
- EC 5 (simple model) – is as the AS 1720 (simple model) but with EC 5 connection stiffnesses from EC 5 section 7.1.
- CSA O86 – this is the most detailed approach considering stiffness of studs, sheathing, fasteners, and tie downs. This is the preferred method in the absence of test data. This method is followed in the worked example in Appendix 1.

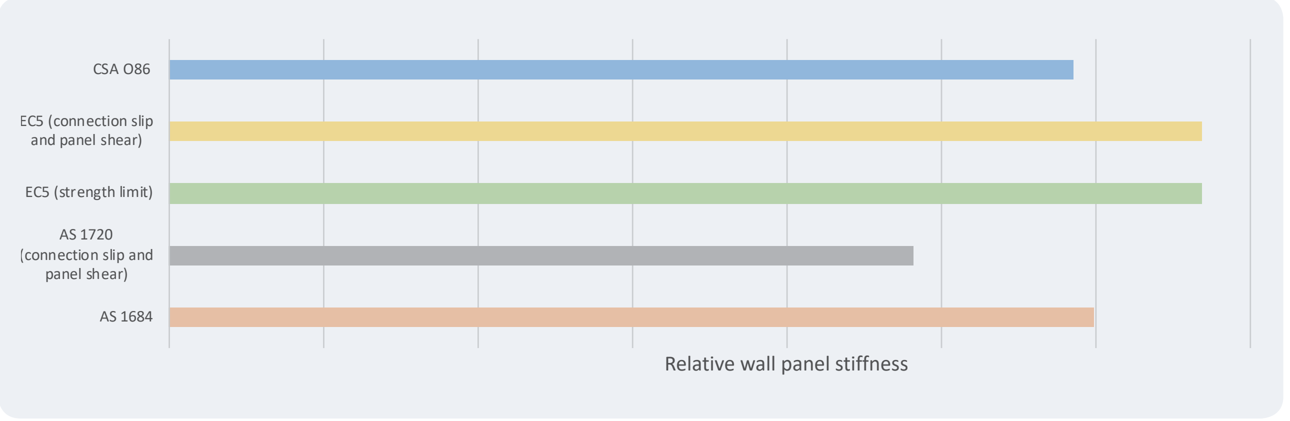


Figure 7.8: Comparison of approx. panel stiffness by various calculation methods (by the authors).

The assumptions used in modelling the structural behaviour of the core, whether timber-framed or mass-timber, depends on the stiffness of the core which is significantly influenced by the core connections. Cores can be modelled as a compound section (often C-shaped because of significant openings) if there are strong and stiff connections along the vertical lines between adjacent core wall panels. These connections must be designed for the shear between the panels. Where the connections are not rigid and strong enough to transmit the shears from plate to plate, the structural model should be of discontinuous plates with the most effective components contributing to stability by bending about their major axis as highlighted in Figure 7.9(b). The widest wall shown in Figure 7.9(b) will be significantly stiffer than the other walls of the core and will be the only effective element. Figure 7.9 also illustrates the location of the shear centre of the composite core in these open sections. The location of the shear centre can be found from the computer modelling of the core.

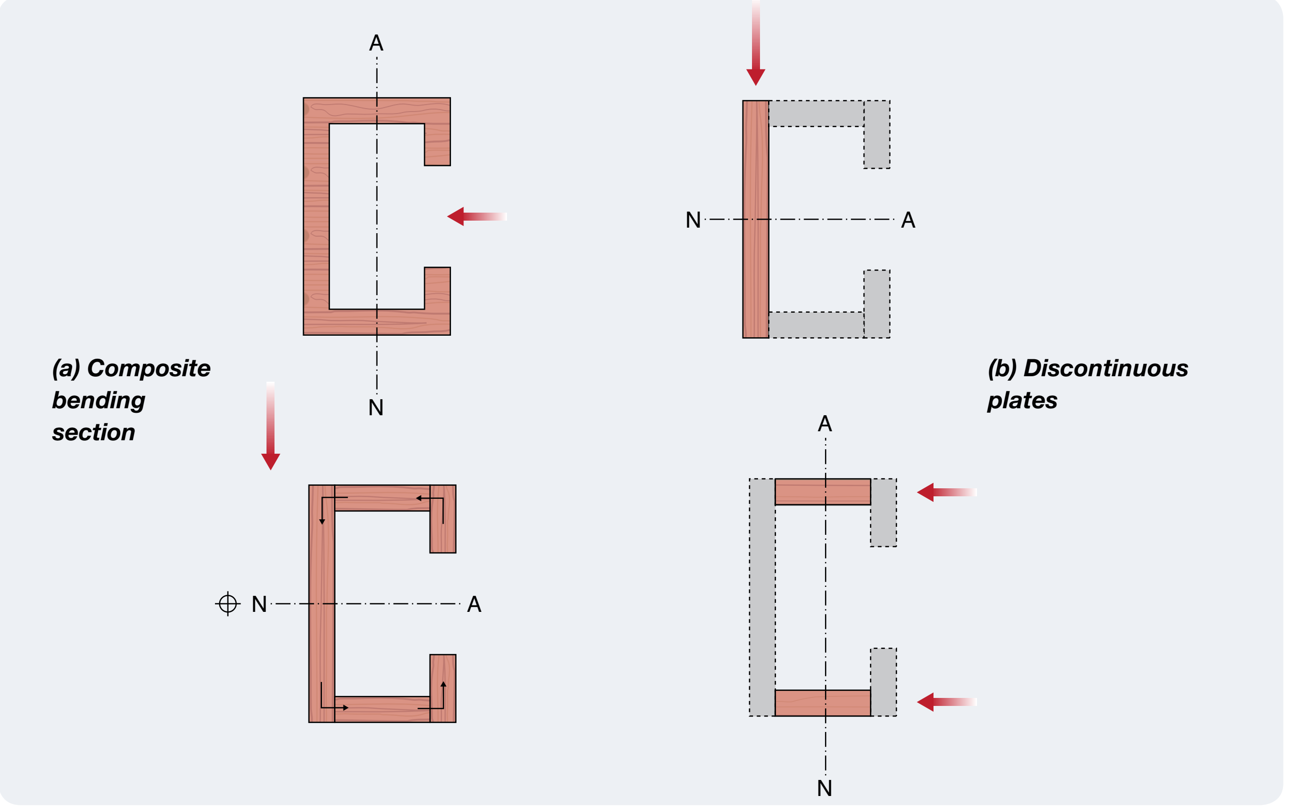


Figure 7.9: Plate connections in composite bending sections and discontinuous plate model of cores.

It is usual practice to model the core in a structural analysis package unless it is appropriate to make an assumption of very simple behaviour of the core, such as case (b) in Figure 7.9. In order to make computer analysis assumptions of the core, its stiffness needs to be estimated and modelled, including both the panels and their connections junctions.

Designers should discuss the type of lift core and design requirements with a number of lift manufacturers and installers early in the design process. Lifts often require specific shaft tolerances or locations for attachments. All lift cores have fire requirements for the sill and lift door surrounds (some lift manufacturers may not yet have a system compatible with timber cores). A number of buildings in Australia have timber lift cores, so there is precedent for fitting lifts into timber buildings.

Table 7.1 provides an estimate of the relative stiffness of different forms of core construction compared with elastic analysis of concrete cores. (Under wind actions, behaviour is elastic with relative flexural stiffness = 1.0, but under earthquake actions, there is ductility and the relative flexural stiffness = 0.7). Where the core has a low relative stiffness, most of the lateral load must be transmitted to the ground by other systems, e.g. bracing walls.

Table 7.1: Relative flexural stiffness of cores.

Core	Relative flexural stiffness
Concrete*	0.7 to 1.0
CLT composite	0.3 to 0.5
CLT separate panels**	0.06 to 0.2
LVL composite	0.4 to 0.6
LVL separate panels**	0.1 to 0.3
NLT panels	Case-by-case
Framed and braced	0.01 to 0.1

* Stiffness of 1.0 represents assumptions of a cracked core.
** Separate panel analysis models each panel as a separate bending element as illustrated in Figure 7.9(b).

7.3.1 3-D modelling

Structural analysis packages enable designers to develop a three-dimensional model of the building to follow load paths from gravity and horizontal loads. The level of complexity in the structural model varies with software packages. Some packages focus on analysis for load distribution and movements, while others also enable the 'design' of elements and systems.
As with all structural software, care is needed in understanding the processes adopted in the software. Engineers should interrogate the software through a series of test cases to develop an understanding as to capability or limits of the software. Worked examples presented in the appendices of this guide use hand calculations to present calculations on an example building for completeness and to develop understanding.

- Walls function as vertical plates with stiffness assumptions.
- Floor systems can be modelled as one-way slabs. The in-plane behaviour of the floor system can be modelled as a truss for flexible diaphragms and as rigid elements for mass-timber systems.
- Connections between elements should generally be modelled as pins with the following exceptions:
  - slotted connections to accommodate vertical differential movements should be modelled as rollers
  - Moment-carrying frames should be modelled with rigid connections at the knee joints.

Worked examples presented in the appendices of this guide use hand calculations to present calculations on an example building.

7.4 Core Design

Cores in concrete structures are usually structurally designed as vertical tubes fixed at the base and free at the top, i.e. a vertical cantilever. In mid-rise timber buildings, these assumptions apply to typical concrete cores, and to mass-timber cores where appropriate details have been used between panels. The connection between adjacent vertical panels of braced timber-framed cores may have relatively flexible shear stiffness which should be considered in the stiffness of the core model. The choice of construction type and material choice will depend on the architectural requirements, the structural requirements, procurement and construction type.

7.4.1 Concrete and masonry cores

Although concrete and masonry cores are very stiff and provide good lateral stability in mid-rise timber buildings, they may accentuate differential axial shortening as the concrete walls shorten under long-term gravity loads significantly less than timber framing over the life of the building. (Refer to section 6 and appendix A.)

Concrete cores are a commonly used lateral load resistance mechanism for mid-rise timber buildings. Table 7.1 shows that concrete cores have high bracing stiffness compared with all of the timber core options. This is primarily due to its higher Modulus of Elasticity (MoE) as a mass element compared with timber, which means that concrete cores attract a higher proportion of the total lateral load than either mass-timber or timber-framed cores.

The structural design of concrete cores in mid-rise timber buildings is the same as for concrete or masonry mid-rise buildings. However, more complex detailing is required for the interface between the concrete core and the remainder of the timber structure.

- **Connections** between timber floors and concrete cores **must be able to accommodate differential vertical movement** (see Section 7.4.4).
- **Larger tolerances in connections** (than usually considered in timber construction) are required to accommodate the larger acceptable dimensional tolerances in concrete construction.
- **Connections must also have the capacity to transfer the higher lateral forces between the floors** and the high stiffness concrete core associated with diaphragm action.

Figure 7.10: An example of a concrete core in mid-rise timber-framed apartments.

7.4.2 Timber-framed cores

Figure 7.11 shows an example of a timber-framed core. Timber-framed cores rely on structural cladding for resistance to lateral loads and are designed as stacked bracing walls, with relatively flexible shear stiffness compared to a CLT panel. The lateral load capacity of each core panel (frame plus cladding) is evaluated using the same techniques as other bracing walls in the building. They are generally designed as independent shear panels rather than a composite vertical cantilever bending section because of their relatively flexible panel shear stiffness.

Under gravity loads, the axial shortening of framed cores use the same axial shortening calculations as the rest of the building. Fire-rated linings in cores should have contraction joints to accommodate this shortening.

There is limited differential shortening between a timber-framed core and the rest of the timber-framed structure, so relatively simple details can be used to connect the cores to the other framed structural elements in the building.

Framed core walls tend to attract a lower percentage of the lateral load compared with CLT and concrete cores when shared between a range of stability systems through the building.

Figure 7.11: Timber-framed core. (Photograph: WoodWorks USA)

For cores against the edge of the building, one core wall is also an external wall. For this wall, there may be no floors to provide lateral support, and there is potential for buckling or bending of the wall over the full height of the core. Designers must use wind beams to span the width of the wall and carry out-of-plane loads from the ends of the studs to the rest of the core (caution needed with walls between lift cores). For cores located completely inside the building, all studs are laterally restrained at each floor and any out-of-plane loading causes bending over the height of just one storey. There is no potential for buckling over the full height of the core.

The detail on the requirements for fire performance of timber-framed cores is given in *WoodSolutions Timber Design Guide #37 Mid-rise Timber Buildings.*

7.4.3 Mass-timber cores

Mass-timber cores have been used in a number of Australian mid-rise timber buildings. Mass-timber cores can be effectively used in mid-rise structures that are mass-timber or predominantly timber-framed or post and beam.

The lift door openings decrease the stiffness of mass-timber cores to lateral forces and increase their potential for warping in the same way as they do in concrete cores. Core design is often under the assumption of separated walls (Figure 7.9 (b)).

Detailing of lifts in mass-timber buildings should be considered at an early stage, particularly around fire rating, dimensional tolerance, dimensional stability and lift equipment loads on the timber core. Guidance can be found in *WoodSolutions Timber Design Guide #37 Mid-rise Timber Buildings* for potential detailing solutions including allowing for a separate steel stud wall around the lift door and concrete sill panels to allow pre-existing fire-testing certificates to be used. For further, more refined and specific details it is usually necessary to engage directly with the mass-timber and lift suppliers.

Acoustics is a key consideration in mass-timber lift cores due to the relatively lightweight nature compared to traditional concrete or blockwork cores. Vibration from the lift can be transferred into the structure and cause noise issues elsewhere in the building. It is also an acoustic requirement in the NCC that core walls are discontinuous construction from neighbouring habitable rooms. This could be mitigated against by designing the lifts to be separated from any living areas but if this is not possible the structural load path from the lift core to the rest of the structure needs to be broken in some way. This could be done in one of two ways:

1. Design the lift with a two-skin core. The lift connects to the inner core and it is allowed to move somewhat independently of the outer skin that supports the floors. This has the advantage of a very definite break in structure, but it does require additional space, additional costs and causes some detailing challenges around fire and differential vertical movement.
2. Allow for resilient mounts within the timber structure and connect the lift rails, etc, to the isolated structure. This can be challenging as it is often difficult to fully isolate sections of the structure without compromising the overall building performance. This should only be considered in buildings with clad walls and independent stud work to minimise acoustic transfer. This approach requires consultation with the project acoustic engineer.

Differential vertical movement between lifts cores and adjacent structure can be an issue for taller buildings. Detailing around door openings, lift thresholds and connections into other parts of the building structure need to be considered.

The dimensional and construction tolerances for mass-timber cores are similar to those for timber-framed walls, so connections between the core and floors need only accommodate standard timber tolerances.

Simple details can be used to connect timber floors to mass-timber cores.

Designers of lift cores in mass-timber buildings need to consider whether to approach the design as a composite section or a discontinuous series of walls.

- Discontinuous walls – Walls are treated as individual unconnected panels and loads are apportioned between them in relation to their stiffness. In many cases, lift walls are relatively lightly loaded in terms of dead load and with less pre-compression, they rely more heavily on strapping and tie-downs. This can sometimes tend to make them less stiff than walls supporting more floor loading as the floor-wall joints will tend to move more under lateral loading. In these cases, there are often not significant forces on the connections between the lift wall panels (see Figure 7.9(b)).
- Composite section – Designing the whole lift as a composite box section is likely to be necessary for post and beam mass-timber buildings to increase the stiffness of the building. It has some advantages in terms of lateral stiffness of the building, but it needs to be specifically designed. Two areas that will require detailed considering will be headers and wall panel connections. Headers over the top of a lift opening will attract a significant load in these composite cores, and the connections will require careful design. Connections between perpendicular wall panels will also require a specific design check and the stiffness of the connection needs to be high to ensure they function effectively as a web/flange element of a whole composite box. Angle brackets and perpendicular screws are unlikely to be stiff enough and inclined fully threaded screws may be a better alternative. In order to achieve a composite section for the core, careful detailing and specification of fasteners, construction processes, and details are needed. There will need to be significant consultation with mass-timber producers and fastener manufacturers early in the design phase to ensure valid assumptions are made (see Figure 7.9(a)).

If the connections of the core can resist stability forces with sufficient stiffness, they can be modelled using a transformed section. If not, then they must be modelled as discontinuous, separated plates with relatively flexible connections. Mass-timber cores tend to comprise mass-timber panels which are stiff compared to the relatively flexible connections.

- For fully composite bending sections, some of the panels are effectively tension and compression flanges, and the others carry in-plane bending as webs in the composite section (see Figure 7.9(a)).
- A core might be semi-composite under serviceability loads. Load distribution within the core elements can be determined considering connection stiffness in an appropriate analysis model.
- All discontinuous mass-timber panels (see Figure 7.9(b)) are modelled as bending elements about their major axis. The relative stiffness of the elements about their major axis means the stiffer elements will attract a higher portion of the stability loads.

WoodSolutions Technical Design Guide #37 Mid-rise Timber Buildings provides guidance on determining fire resistance of mass-timber cores.

CLT cores

Figure 7.12 shows an example of a mass-timber core in a mid-rise timber building. Shrinkage both vertically or horizontally is minimal in CLT cores, but special details may be required to accommodate shrinkage in thickness of the CLT panels. Designers have the option of using different thickness panels for each side of the core.

The face grain of the CLT panels in the core will be oriented vertically over the full height of the building, therefore axial shortening due to deflection under axial loads, creep and shrinkage will be less than the axial shortening in timber-framing that incorporates elements with horizontal grain.

Designers should allow for differential axial shortening between frames and CLT cores.

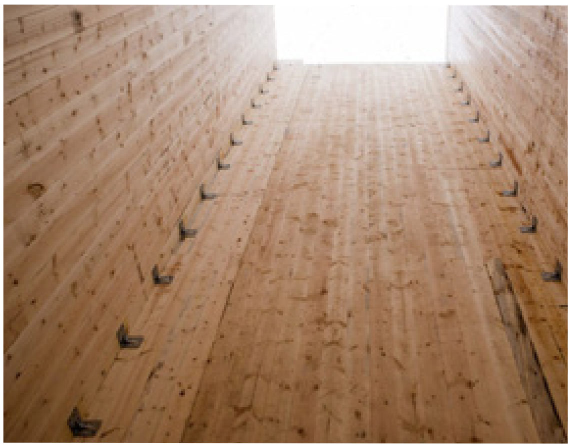
 

Figure 7.12: Mass timber core. (Photograph: Lend Lease)

The in-plane stiffness of CLT panels is relatively high. In-plane loading of CLT does not produce rolling shear that can compromise the shear strength of CLT when loaded out-of-plane. Strong and stiff connections between the panels enable the whole core to be modelled as a composite bending section. If the connections highlighted in Figure 7.9(a) are not designed to stiffly transmit the full shear flow between the plates, then the CLT core must be modelled as disconnected plates with only the significant long walls carrying stability loads.

A number of CLT handbooks are available to assist with the design of CLT (e.g. CLT Handbook by FP Innovations, Canada), and CLT manufacturers also provide guidance on design to determine the capacity of their products.

As each CLT manufacturer uses different timber grades, bonding methods and thickness of each lamination, design properties will vary from manufacturer to manufacturer. Obtain structural design properties from the manufacturer likely to supply the core material for input into the structural behaviour model.

Nail-laminated timber cores

Figure 7.13 shows an example of a nail laminated timber (NLT) core. Nail-laminated panels use timber elements oriented with the grain in a vertical direction. Each piece is effective in transmitting vertical loads, but shear on the panels (either direct shear or bending shear) causes shear between the laminated pieces, which is only resisted by the nails.

NLT cores shrink minimally in the vertical direction, but shrinkage can occur perpendicular to grain in the width and thickness of the core walls which may affect finishes and fire protection.

The shrinkage across the width of the panels is distributed between the nail-laminated boards, and usually causes a small gap at the interface between the boards. It is difficult to estimate the extent of shrinkage of the panel that may occur in this direction, but restraint from rim beams often minimises shrinkage of the whole panel. Nail-laminated timber cores have less resistance to shear forces than CLT cores, and shrinkage may cause an additional decrease in shear capacity as shear span on fasteners increases with the gap. Most designers rely on structural cladding such as plywood or OSB for the shear resistance of NLT cores. The design of clad NLT panels to resist lateral loads is, therefore typically the same as the design of framed bracing panels.

Figure 7.13: Nail laminated timber core. (Photograph: Jane Arnolda, Structerre)

The axial stresses in NLT cores are low due to the large cross-section of timber loaded parallel to grain by gravity forces. The axial shortening of an NLT core is small compared with the axial shortening of the wall frames in the rest of the building, so the connections between the NLT core and floors must accommodate differential axial shortening.

7.4.4 Core interfaces

The structural connections between the core and the remainder of the timber structure must be able to:

- accommodate differential vertical movement as discussed in Section 4
- accommodate dimensional tolerances in core construction and the building frame
- transfer the lateral forces between the floors and the core under building lateral loads
- transfer axial loads between floor and core associated with tying forces
- support the floors under gravity loads.

These connections must be suitable for installation on site.

Acoustic and fire requirements can be satisfied by separating the core from other walls and floors in the building. However, the final design must be a compromise between the need to effectively transfer lateral loads between each floor diaphragm and the core and the need to isolate other elements for acoustic performance.

Articulation with floors

An articulated connection between the frame and core can transfer horizontal loads between the floor diaphragm and core. Where the shortening behaviour of the core is similar to that of other walls in the building (framed cores), the articulation is less critical. However, in other cases, axial shortening of the frame can be accommodated by rotation of the articulated link from a pre-set position (see Figure 7.14(a)).

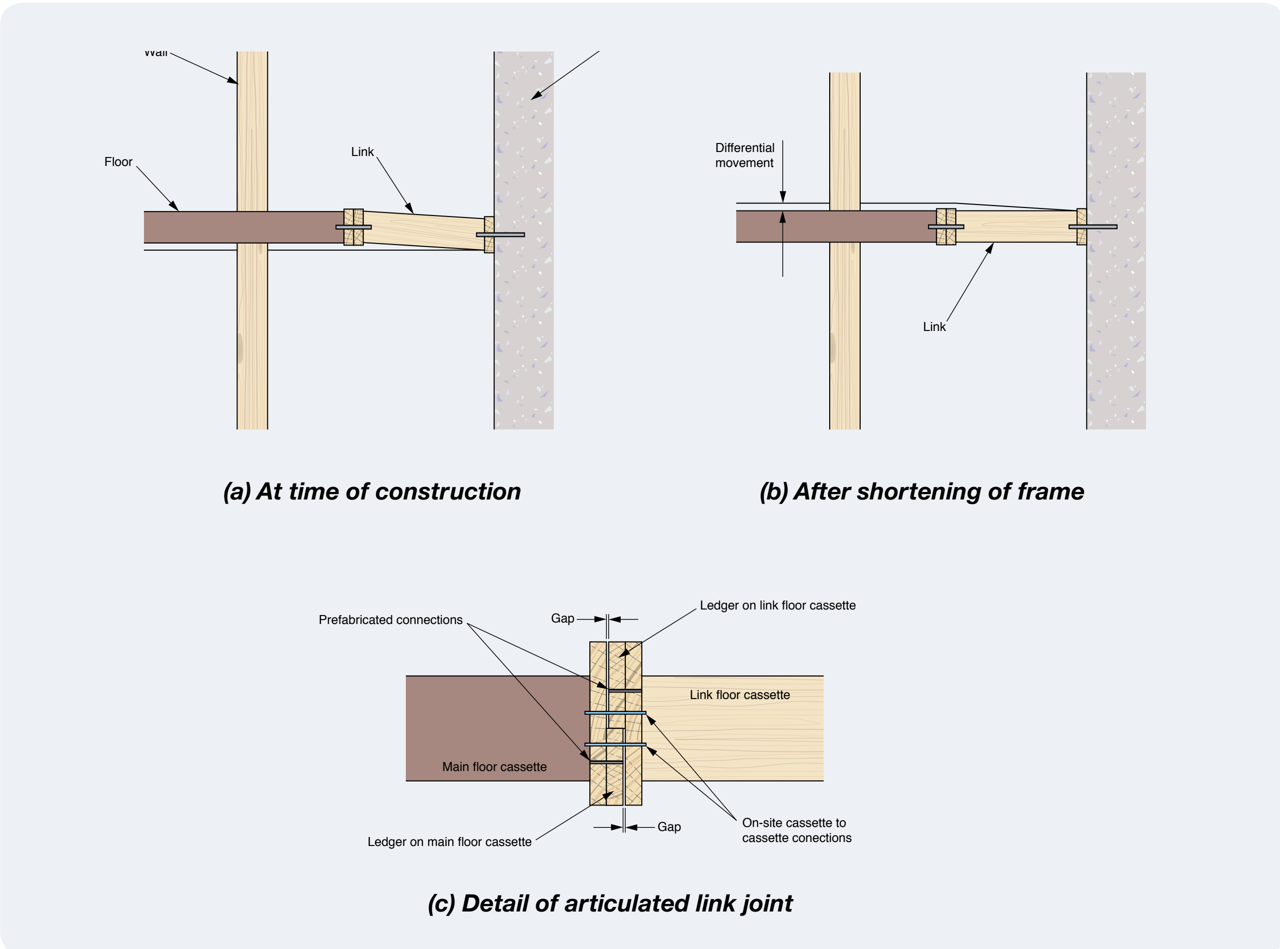


Figure 7.14: Articulation of connection between floors and core.

Articulated links between the core and floor system can accommodate differential shortening without causing a difference in height between the floors and lift sills. However, the change in floor slope over a small section may be noticed. Flooring needs to be flexible at the articulation points (e.g. tiles that are continuous over these points are likely to crack over time).

Sliding connections

Shortening of the frame could also be accommodated using sliding connections (Figure 7.15) between the floor system and core. These connections can be detailed to transfer horizontal forces between steel plates but still allow relative vertical movement. The floor position can be adjusted as the building settles so that there is no step between the floor and the lift core sill.

Figure 7.15: Example of sliding connection between floors and core.

7.5 Braced Wall Design

The choice of wall construction type is influenced by multiple parameters, including structural, cost, acoustic, fire, procurement and construction. Within a given building there may be a mix between mass-timber and timber-framed walls if suited to the project procurement route and contractor. For example, the structure might use CLT core full height and CLT shear walls on the lower floors, with braced timber-framed shear walls on the upper floors.

The structural model of the building determines the loads in each bracing element. These loads can be used to select appropriate bracing systems based on stiffness.

At each storey, the sum of the bracing capacity of all elements must exceed the lateral forces transmitted across that storey.

7.5.1 Timber-framed bracing walls

Stiffness requirements are likely to govern the bracing design requirements of timber-framed bracing walls. Sheathed bracing systems are typically designed for stiffness performance and cross-checked against strength performance.

Stiffness of timber-framed bracing walls

The stiffness of timber-framed bracing walls can be modelled in two different ways:

- Some software programmes are able to model the interaction of the framing, sheathing, and the connectors between the bracing panels. These programmes aim to give a sufficiently accurate representation of the internal response of the bracing wall. These software packages tend to be specific to timber-framed solutions.
- Alternatively, an elastic truss analogy can be used (see as shown in Figure 7.16). The diagonal element models the shear stiffness of the wall;, comprising studs, wall-plates, sheathing and fasteners. The stiffness of connections between bracing walls will need to be considered separately.

To model the wall as a truss:

- Estimate the lateral load per mm of drift for representative bracing walls (see Appendix 1 for detail).
- When conducting analysis using the truss analogy, the panel bracing stiffness can then be modelled using an equivalent diagonal bracing set that has the same shear stiffness to model the bracing wall using Equation 7.1. This assumes linear elastic stiffness responses under serviceability loads.

Equation 7.1 gives the axial stiffness (EA) of a single diagonal with equivalent stiffness to the bracing wall given a load per mm deformation of P/δ N/mm for a bracing wall.

$$EA = \frac{P}{\delta}\sqrt{h^2 + L^2} \tag{7.1}$$

Where:

E = Modulus of Elasticity of the diagonal member

A = Cross sectional area of the diagonal member

L = length of wall panel

h = height of wall

P = applied lateral load at the top of the wall

δ = deflection of the top of the wall under the lateral load P.

Figure 7.16 illustrates the truss analogy model. The model uses a diagonal member to carry the lateral load through the bracing wall and produce similar elastic behaviour under lateral loads as the original bracing wall.

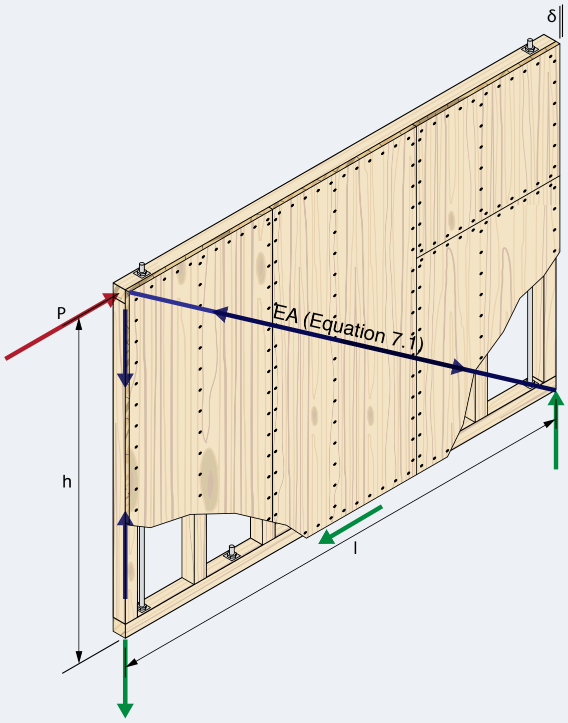


Figure 7.16: Bracing wall model using a diagonal element.

Capacity of timber-framed bracing walls

Bracing system manufacturers publish the capacities of their tested products and provide technical guidance for their design and installation. Published capacities may either be strength or stiffness governed. Frame and truss manufacturers, and manufacturers of plywood and other structural sheeting products can also provide information on the bracing capacities of their products.

EC5 Section 9.2.4 (and CSA O86) provide methods for calculating bracing capacity of timber-framed walls clad with timber panel linings. Detailed explanation of stiffness design is presented in Appendix 1.

Metal strap cross braces

A common method of bracing timber-framed walls is to use galvanised metal straps, with tensioners to take up slack to achieve a 'tension' brace. Galvanised tensioned crossed metal straps (typically 30 x 0.8 mm) positioned at an angle of between 45° and 60° to the top plates (Figure 7.17) has a capacity of 3 kN/m length of wall for JD4 framing; or 2.5 kN/m length of wall for JD5 framing, for a wall height of 2.7 m. The performance is based on limiting lateral deflection at the top of the wall under lateral load. This performance information is available from strap manufacturers. Similar information is presented in AS1684.2 and 3.

The metal straps must be fixed to each stud they cross using a 30 x 2.8 mm galvanised flathead nail; and fixed to the top and bottom plates with 4 x 30 x 2.8 mm galvanised flathead nails.

Top and bottom plates must be fixed to the studs at each end of the cross-bracing strap (shown in Figure 7.17 as red dots) using either:

- 30 x 0.8 mm galvanised metal straps looped over the plates and fixed to the studs with 4 x 30 x 2.8 mm galvanised flathead nails; or
- 2 x framing anchors with 4 nails per leg.

Top and bottom plates must be fixed to the floor, the floor above or roof framing. The capacity of the fixing is to be the same as the capacity of the bracing, in this case 3.0 kN/m. Refer to manufacturers or AS 1684 Part 2 or 3 Table 8.22 for typical fixings.

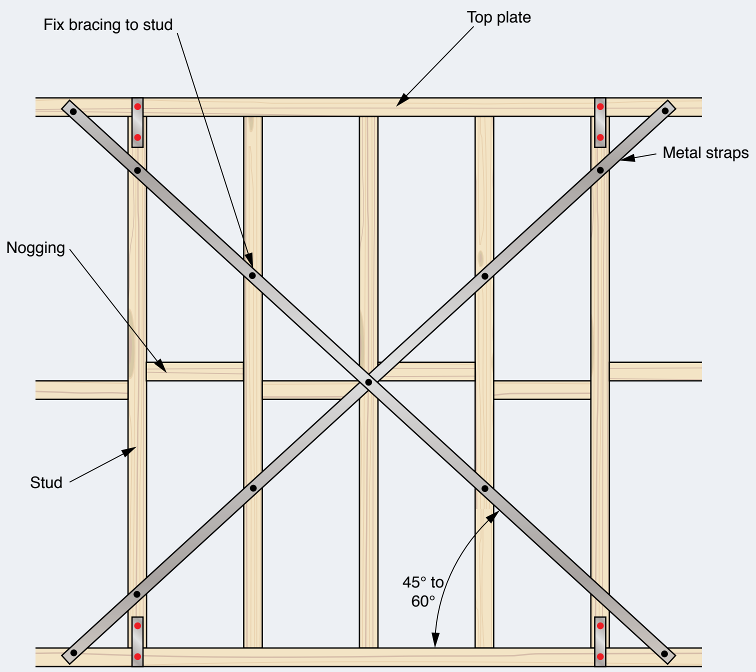


Figure 7.17: Metal strap cross braces.

Plywood, oriented strand board (OSB) or hardboard

Structural sheathing on a timber-framed wall can be used to provide bracing. Sheathing is typically provided by a panel-based engineered wood product such as plywood, OSB or hardboard. For example, 7 mm thick F11 plywood with tie-down rods, i.e AS 1684 Table 8.18(i) configuration, has a capacity of 8.7 kN/m length of wall with JD4 framing; or 7.3 kN/m length of wall for JD5 framing (Figure 7.19) for a wall height of 2.7 m. OSB systems are not included within AS 1684 so suppliers of this products should be contacted.

- Plywood is nailed to the wall frame using 30 x 2.8 mm galvanised flat head nails or equivalent.
- Maximum spacing of nails on top and bottom plates = 50 mm.
- Maximum spacing of nails on studs = 100 mm.
- Horizontal butt joints between plywood must be fixed at noggings with nails in each panel @ 50 mm centres.
- Top and bottom plates must be fixed to the floor, the floor above or roof framing. The capacity of the fixing is to be the same as the capacity of the bracing. Refer to manufacturers or AS 1684 Part 2 or 3 Table 8.22 for suitable fixings.

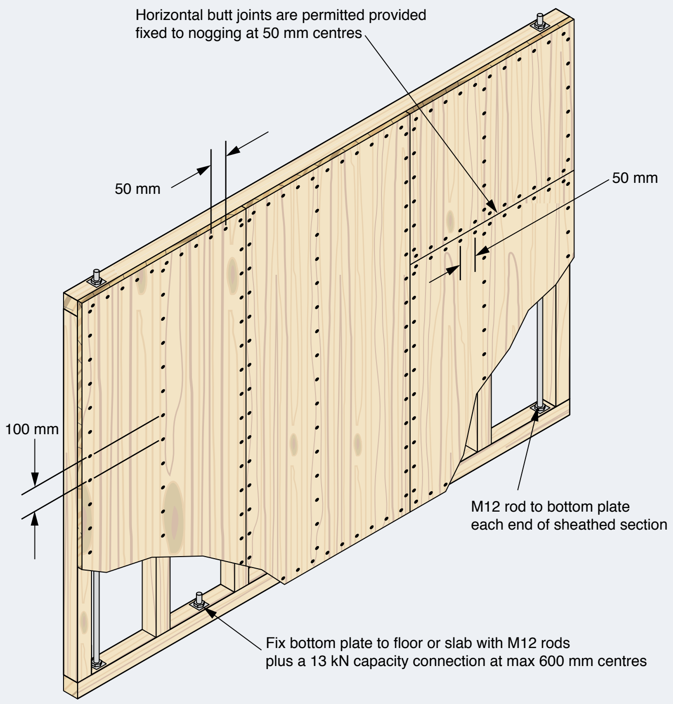


Figure 7.18: Plywood or OSB bracing.

Thicker panel-based engineered wood product may slightly increase the bracing capacity. However, the performance of panel bracing is affected more by the size of nails and nail spacings than the thickness of the panel. It is not practical to reduce the nail spacings or increase the nail diameter as they are likely to split the top and bottom plates. However, some plywood or OSB manufacturers may have test results that justify design capacities for other combinations of connectors, bracing panels, frames and tie-downs. Further design information can be found in EC5 section 9.2.4 or consult with fabricators.

Wall height

Thc apacity of bracing units in AS1684 is generally based on an assumed 2.7 m high wall. For higher walls alternative arrangements of sheathing may be required as wall height may exceed panel size. Detailed design checks are required but as a first pass (depending on stiffness model used), where walls are higher than 2.7 m the system capacity should be reduced in accordance with Table 7.2.

Table 7.2: Bracing wall capacity/height multiplier.

Wall height (mm)	Multiplier
3000	0.9
3300	0.8
3600	0.75
3900	0.7
4200	0.64

Proprietary products

Alternatives for providing bracing can be prefabricated nail-plate trusses, I-joists or other prefabricated elements (see Figure 7.19). For design guidance on such systems refer to manufacturers.

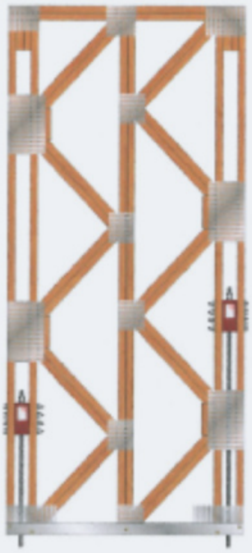


Figure 7.19: Example of a proprietary short-wall bracing system.

7.5.2 CLT shear walls

Stiffness of CLT shear walls

The stiffness of stability systems incorporating CLT shear walls tend to be governed by the relative flexibility of the connections between CLT panels. The overall stiffness of a CLT shear wall is calculated taking into account the contribution of the following components:

- CLT panel (shear translation and bending)
- shear connections – angle brackets (ka)
- hold-down or tie-down (kh).

A detailed computation for the stiffness of a single CLT panel can be found in Appendix 2.

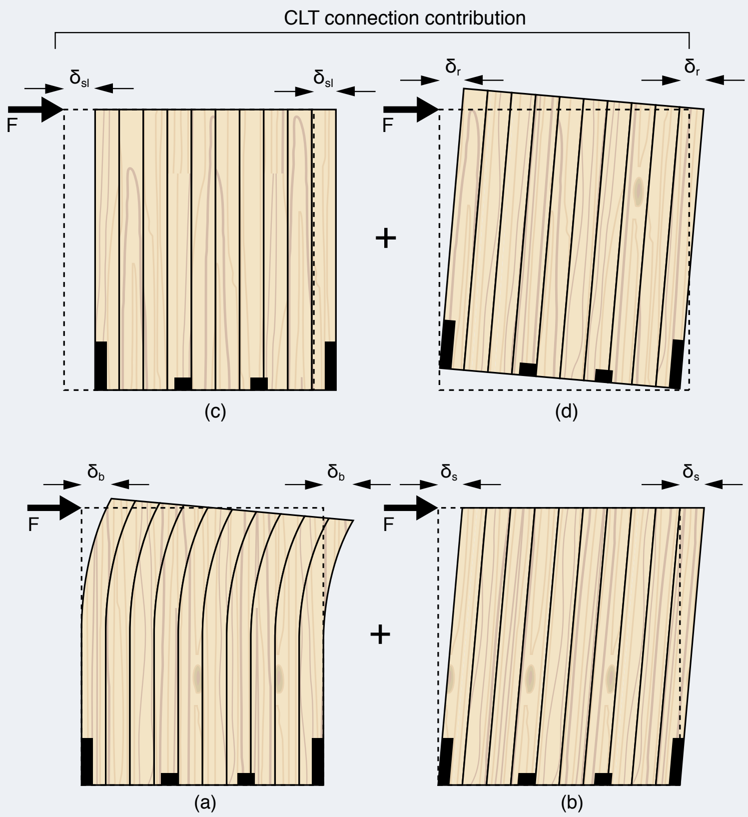


Figure 7.20: Mechanical model for CLT walls overall stiffness.

Capacity of CLT shear walls

The applied action induces in-plane bending moments and shear forces within the bracing wall and adjacent connections. The in-plane bending and shear strength of a CLT panel is significantly greater than that of the connections, which typically govern the design. Refer to Appendix 2 for a CLT shear wall design.

Capacity of moment frames

Moment- carrying frames are portal frames that carry lateral loads around large openings. They can be designed using portal frame or truss design software. They tend to have lower stiffness and capacity than bracing walls and are generally less cost effective; they are usually only used when other options are not available.

Steel portal frames

Conventional steel portal frames can be designed to span large openings and be incorporated into the timber frame walls. However, although timber wall frames in the rest of the building will shorten in service, the steel frame will not. Special details are required to accommodate differential vertical movement between the timber and the steel and give an effective load path for transfer of lateral loads between timber floor diaphragms and the steel portal frame.

Timber portal frames

Plywood box beam frames, LVL portal frames, and glulam portal frames can all be designed and fabricated and incorporated into timber frame walls. Design software is available from industry groups and manufacturers.

Timber trusses

It is possible to use prefabricated trussed portal frames (see as illustrated in Figure 7.4). They are the lightest of the moment frame options and can be prefabricated. However, they are not yet commonly used in Australia. Refer to nail plate providers for further information.

7.5.4 Edge connection of bracing walls

Each bracing system has an accompanying requirement to fix to the floor or roof frame at both the top and bottom of the panel to transfer shear and tie-down. Refer to the system provider or AS 1684 Part 2 or 3 for guidance on estimating capacity. Refer to Appendix 1 and 2 for detail.

Bracing walls develop tension at one end, and compression at the other, to equilibrate the shear force transferred between the levels in the building. The tension or compression can be calculated using Equation 7.2.

$$T = V\frac{h}{L} \tag{7.2}$$

Where:

T = tension or compression force at the end of the wall

V = shear force = lateral bracing force transmitted by the wall

h = height of the bracing wall

L = length of the bracing wall

Bracing walls carry axial compression from the levels above as well as the lateral forces from wind and earthquake actions. If bracing walls are not part of the overall load bearing system, i.e. in non-loadbearing walls, then there will be limited dead load from above to help resist uplift/tie-down.

A tie-down bolt at each end of the wall can address local overturning of the wall and shear transfer to the diaphragm. Typically, these bolts on individual walls extend from the upper surface of the bottom plate through the floor to the lower surface of the top plate on the storey below. If the length of the bolt required is too long, a large threaded rod can be substituted. Tie-down connection floor-to-floor has lower strength than a full-height threaded rod but can provide a stiffer response for the shear wall. Global tie-down of the whole building is addressed in Section 7.7. Refer to Appendix 1 and 2 for worked example details.

The shear walls requiring anchoring the foundation or structure below. This interface is usually between concrete and timber. The relative tolerance of the concrete construction is loose compared to the timber typically. Solutions can be cast in or post-fixed but should be developed to suit construction methods and safe site work practices.

Although the shear transfer is accommodated at each bracing wall, it is recommended that a global check of shear transfer from floor to wall and wall to floor is carried out, particularly if lateral loads are earthquake govern.

7.6 Horizontal Floor Diaphragms

Horizontal diaphragms need to be checked for three effects:

- shear capacity between discrete panels in the diaphragm
- bending strength of the diaphragm acting as a deep beam
- lateral deflection of the diaphragm between bracing walls.

Where the floor plan is similar on every storey, the horizontal diaphragm is only loaded by the lateral forces on the edge of the floor itself; either from wind pressure on the windward or leeward face of the building, or from earthquake actions on the mass supported by the floor.

A floor diaphragm panel can be analysed as a continuous deep beam to give the shear force and bending moment diagram. The bending moment can be checked at maximum positive and negative moment points, and the shear force checked at joints between the discrete panels that make up the floor diaphragm.

Differential movement at diaphragm interfaces

Building shortening can affect connections that are designed to carry lateral loads between horizontal and vertical members. Building shortening could lead to differential movement between floors and stiffer vertical elements such as cores.

The connections between diaphragms and cores need to be detailed to accommodate the differential vertical movement, but transmit the horizontal and axial forces required.

7.6.1 Design of timber-framed floor diaphragms

Stiffness of timber-framed floor diaphragms

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':

- Characteristic wind pressure less than 1.5 kPa.
- Span:depth is less than 2:1 in any wind direction
- Span less than 12 m between stability walls
- Sheathing panel is minimum 15 mm thickness for plywood or OSB, and 18 mm for particleboard.
- Panel-to-panel junctions are nailed to a common joist, rafter or batten.
- Minimum fasteners to be 3.1 mm dia. ringed shank machine driven nails or 4 mm dia. wood screws, length 2.5 times panel thickness, at max 150 mm cts on panel edges.
- Maximum fastener centres within the panel to be 300 mm.

The approach is valid for the design of timber-framed diaphragms to AS 1720.1 within typical construction practices. If there is any doubt as to the validity of the assumptions or the above assumptions are not met, then diaphragm stiffness should be modelled. Caution is needed in accommodated steps and level changes in the structural decking through the diaphragm.

If there is any doubt about diaphragm stiffness, framed floor systems including cassettes should be modelled as flexible.

Flexible floor diaphragms can be modelled using a truss analogy (see Figure 7.21). However, equivalency is not as straightforward as for bracing walls, as the stiffness of the diaphragm is a function of the in-plane flexural and in-plane shear behaviour of the floor system components. In this case, it is assumed the structural decking is sufficiently restrained to avoid buckling. Openings are treated as reduced depth, Vierendeel bays or notches in the truss model.

The behaviour of a single diaphragm panel can be modelled by the stiffness of the rim beams and their connections (acting as tension and compression chords of a horizontal truss), the shear stiffness of the structural decking, and the fastener stiffness along the boundaries of the floor panels. The truss diagonals represent the shear behaviour of the diaphragm using the model shown in Figure 7.22. Once stiffness of diaphragm elements is estimated, then an equivalent truss model can be established with elements cross-sections defined using an axial stiffness (EA) value to simulate the diaphragm stiffness.

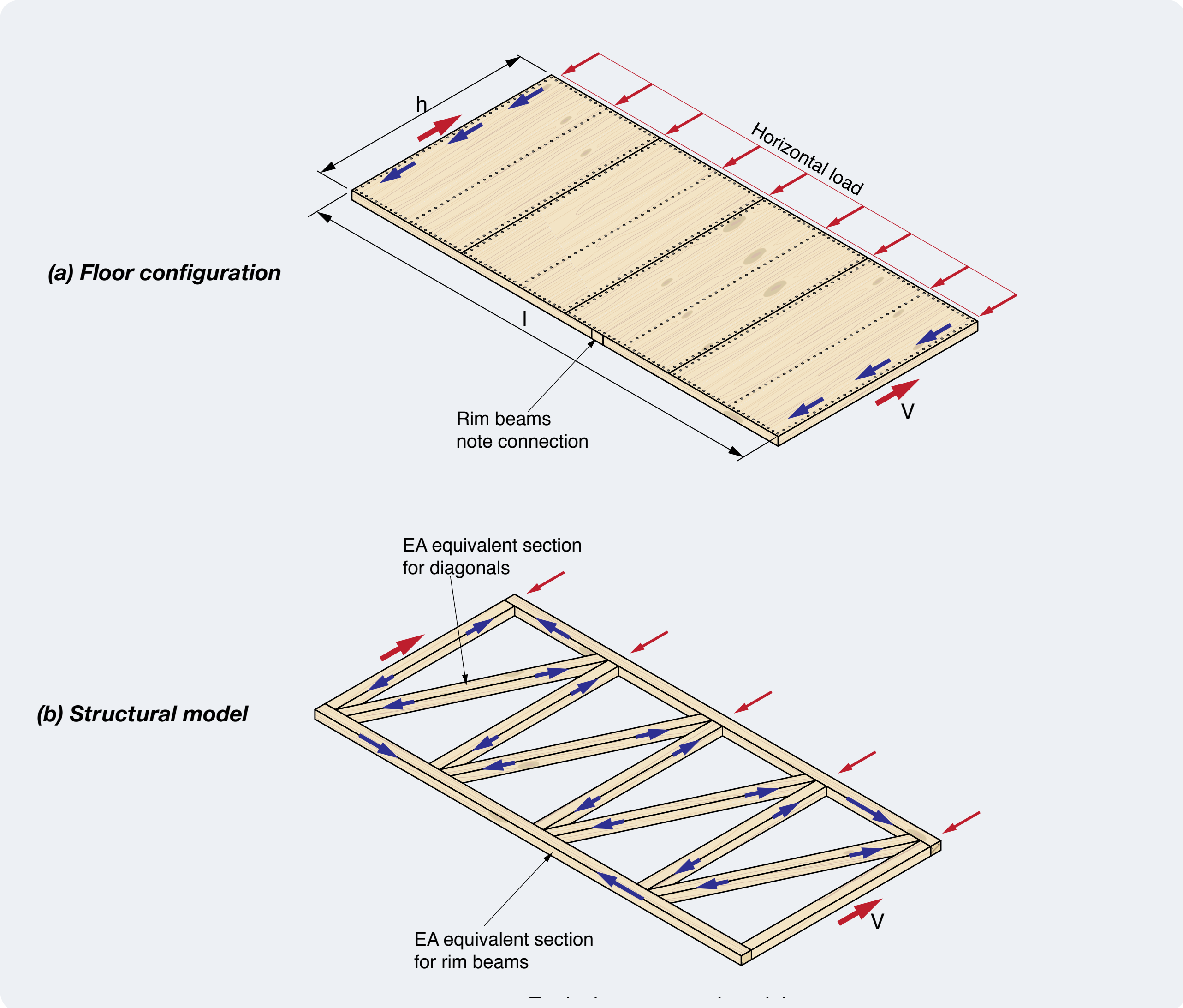


Figure 7.21: Floor diaphragm model using diagonal elements.

In order to estimate the stiffness of the diaphragm truss model, the stiffness of the equivalent *EA* member components needs to be estimated. The following steps provide a method to estimate stiffness. It is likely that in most cases the stiffness (flexibility) of the connections will govern:

- *EA* stiffness to determine equivalent truss section of 'chord' elements is determined by the actual axial stiffness of the actual rim beams, plus an allowance for the stiffness of connections through the axial load path. The stiffest response will be provided by a continuous rim beam. A direct end-to-end connection of the rim beam will be the stiffest connected path. Connection stiffness can be estimated using fastener manufacturers data, AS 1720.1 Appendix C, or EC5 Section 7.1. Stiffness can be summed using the theory of springs in series.
- *EA* stiffness for 'shear' will need to consider the stiffness of the fasteners securing structural decking elements, the panel stiffness of the decking, as well as the estimated stiffness of junctions between floor cassettes, if appropriate. Connection stiffness can be estimated using fastener manufacturers data, AS 1720.1 Appendix C, or EC5 Section 7.1. Panel stiffness can be estimated from manufacturer's data of using AS 1720.1 5.5.8. Diaphragm shear stiffness can be summed using the theory of springs in series. The stiffness values can then be resolved to the appropriate angle for the diagonals in the truss model to determine required stiffness at the diagonal angle. An equivalent EA for the truss model can then be determined.

This method is very sensitive to estimations of fastener stiffness, which are difficult to predict. Therefore, it is important to test the sensitivity of the system to changing assumptions of fastener stiffness (say 0.5 x K, 1.0 x K, and 2.0 x K). A further sense check could be provided by working through the method suggested in section 11.7.2 of *CSA O86-14 Engineering Design in Wood*, the Canadian timber design standard. *WoodSolutions Timber Design Guide #35 Floor Diaphragms in Timber Buildings provides further information.*

Capacity of timber-framed floor diaphragms

EC5 9.2.3 provides guidance on the detailed design of floor diaphragms. A detailed explanation can be found in the Manual for the Design of Timber Building Structures to Eurocode 5 section 5.8 published by the Institution of Structural Engineers. The method follows the assumption of the diaphragm acting as a deep beam and checks in turn the members and connections contributing to the 'chords' (flanges) and web. Refer also to *WoodSolutions Timber Design Guide #35 Floor Diaphragms in Timber Buildings.*

Shear capacity

The shear capacity of a diaphragm is typically governed by the shear strength of the connections between individual floor segments. For cassette floor systems, this is the shear capacity of the connections between adjacent cassettes or panels. AS 1720.1 Appendix I provides guidance for buckling of plywood diaphragms for situations requiring specific checks, particularly where floor blocking is limited. Connections between cassettes can be reinforced by steel plates or plywood flitches running the length of the connection and securely screwed to each floor panel. Shear capacity of connections can be determined using AS1720.1 section 4 and fastener manufacturers' data. Stiffness of cassette connections can be approximated by assuming the fasteners are the flexible component of the system.

Moment capacity of cassette floor diaphragms

The moment capacity of a diaphragm is calculated using the tension and compression strength of the rim beams and connections on either side of the diaphragm. Rim beams at each end of the cassette floors act as flanges or 'chords'.

WoodSolutions Technical Design Guide #35 Floor Diaphragms in Timber Buildings, EC5 and the Canadian Standard O86 (Clause 11.5.6) provide a method to calculate the bending capacity based on the truss analogy of framed floors with the rim beams acting as tension and compression chords (Equation 7.3). Rim beam connections also need to be considered, which may include some contribution from the fixing of the structural decking.

$$M_r = P_r D_{TC} \tag{7.3}$$

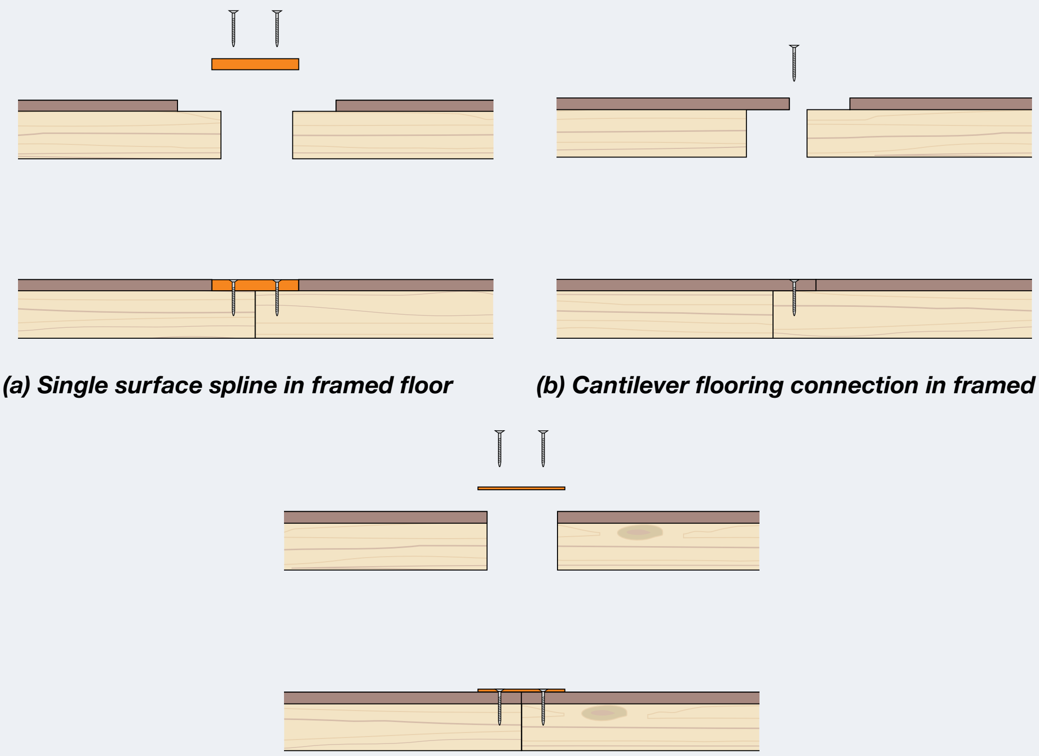
Where:

P_r = factored axial tension and compression capacity of diaphragm chords

D_{TC} = distance between centre lines of the tension chord and the compression chord

Connections between adjacent cassettes

Figure 7.22 illustrates some methods of connecting adjacent cassettes. The shear capacity of connections between adjacent floor units can be calculated and compared with the panel shear calculated by treating the diaphragm as a deep beam.



(a) Single surface spline in framed floor

(b) Cantilever flooring connection in framed floor

(c) Metal cover strip in framed timber floors

Figure 7.22: Mechanical model for CLT walls overall stiffness.

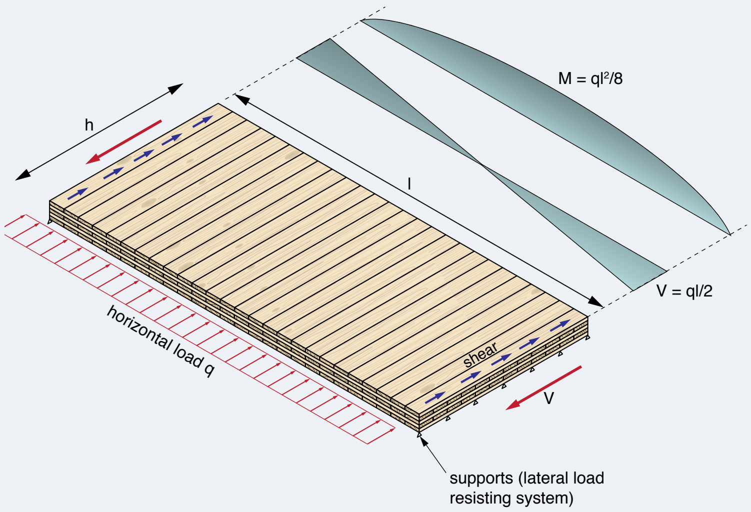
7.6.2 CLT diaphragms

CLT (and other mass timber) panels will tend to behave relatively stiffly when subjected to dynamic in-plane loads (long-term deflections of CLT used as deep beams are a more complex matter that is outside the scope of this document; more information can be sought from manufacturers). Typically, diaphragms for a residential apartment building could be assumed as stiff/rigid for the purposes of structural analysis due to the frequent and regular shear walls across the entire footprint. For post and beam structures with more isolated stability structures then the diaphragm behaviour needs to be considered in more detail. Using the floor plates as part of a strut and tie model between beams is most frequently used to justify connections. Strapping plates may be required to transfer the tension loads across joints, but shear loads can be accommodated in typical panel-panel or panel-beam connections.

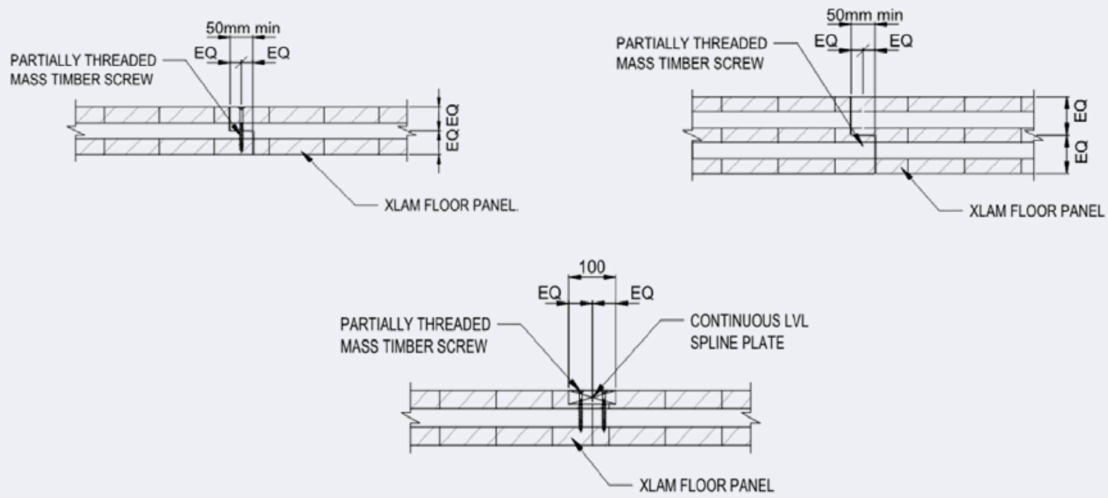
7.6.3 Capacity of mass-timber floor diaphragms

The load for diaphragms is in plane, and therefore the calculation routines to determine the capacity is like that of section 7.5.2. Typically, the capacity of the diaphragm is governed by the connections between adjacent panels and shear walls.

The bending capacity of a mass-timber panel can also assist as in-plane bending will induce tension or compression in all of the layers that are parallel to the loaded faces of the building.



(a) illustrates diaphragm actions within the CLT panel



(b) Methods of connecting adjacent panels in mass-timber panels (source XLam).

Figure 7.23: Mass timber floor diaphragms.

7.6.4 Continuity of floor diaphragms

Acoustic performance at acoustic separation lines or sole occupancy unit (SOU) walls is maximised by having no sound bridges across the cavity between the two leaves of the wall. However, diaphragm action is improved by continuing the flooring across the cavity.

Floor diaphragms continuous through wall cavities

The typical solution is to continue the floor diaphragm through all SOU walls and increase the effectiveness of the sound isolation elsewhere in the wall to compensate for the sound bridge formed by the continuous diaphragm. This is usually the strategy used for mass-timber floor systems.

Floor diaphragms and sound isolating walls that incorporate a cavity

A compromise solution can be achieved by:

- maintaining the sound isolation in walls adjacent to critical spaces such as bedrooms, kitchens and living rooms
- bridging the cavity with the floor diaphragm under front doors, balconies, adjoining bathrooms and laundries.

This compromise allows strong bridges between adjacent segments of the floor diaphragm that could be designed to develop in-plane bending action in the diaphragm for the whole floor and transfer loads between diaphragm segments. Figure 7.24 illustrates points in a floor plan at which these connections can be made:

- The blue walls are SOU boundary walls that are usually sound isolation walls.
- The ellipses show areas in which the cavities could be bridged without compromising the sound performance of the SOU walls.
- Each floor diaphragm segment has at least four points at which it can be connected to other segments in the building contributing to a diaphragm that can transfer load across the entire floor plan.

A solution of this form adds complexity to construction, detailing and structural design and is not commonly adopted.

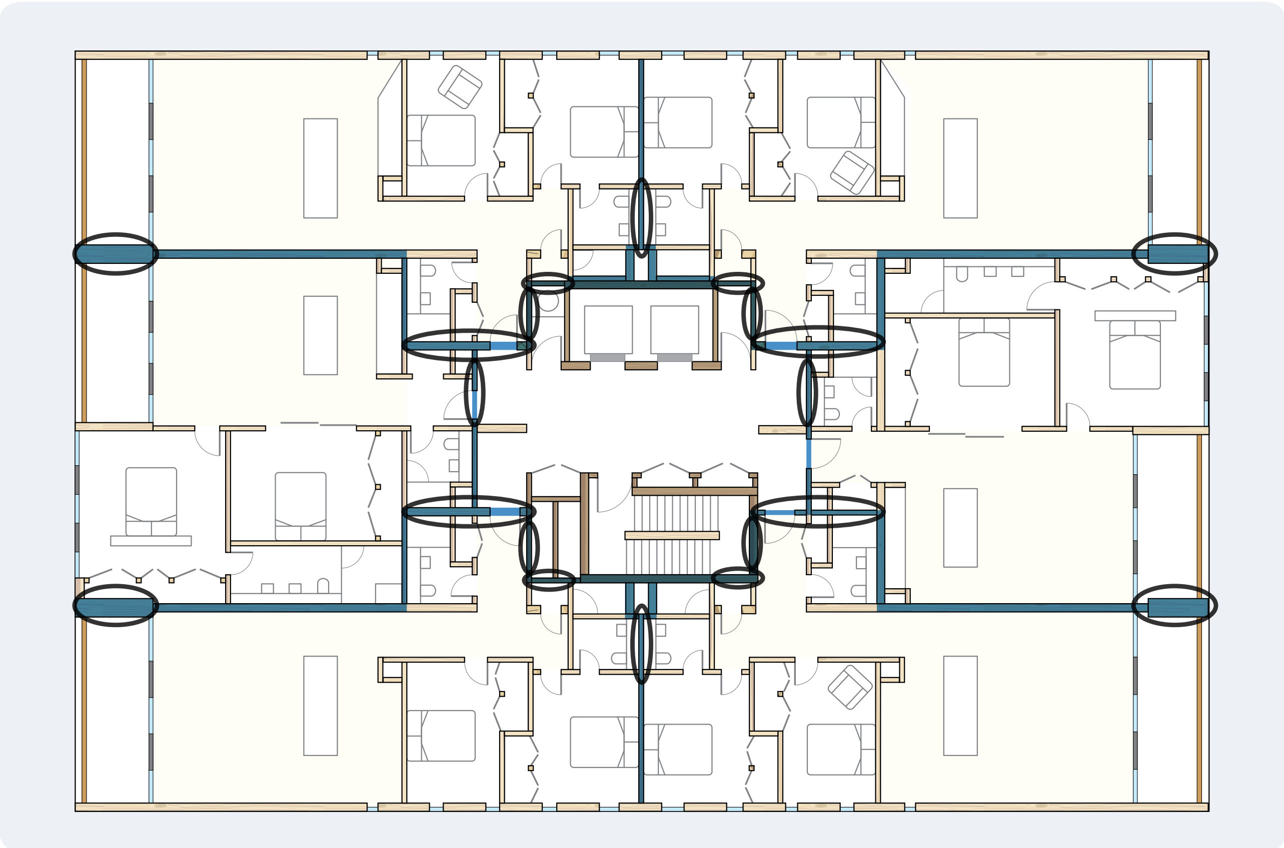


Figure 7.24: Compromise connections between floor diaphragm segments.

Voids and re-entrant corners

Voids in floors reduce the length available for connections between adjacent panels. Connections must be either stronger or more closely spaced to increase the shear capacity per metre.

Re-entrant corners on beams concentrate stresses. Re-entrant corners on horizontal diaphragms behave in a similar way to notches in conventional timber beams. It is possible to reinforce re-entrant corners. Designers should consider appropriate methods. One method adopted in Canada is to use drag straps to minimise stress concentrations in the diaphragm. These straps are designed to carry 120% of the diaphragm flexural stresses in the floor at that point. (CSA O86 Cl. 11.8.6). Drag straps (also known as drag strips, drag struts and collectors) are metal strips that are fixed across the top of diaphragms at shear walls to carry forces from the diaphragm into the area next to the shear wall where they can be transferred into the shear wall. Figure 7.25 shows a drag strap running the width of the building adjacent to a wall that has parts of it as a bracing wall. The drag strap transfers force from the diaphragm to the parts of the wall that are the bracing walls. Analysis and design of such reinforcement should be considered in the diaphragm truss analogy model.

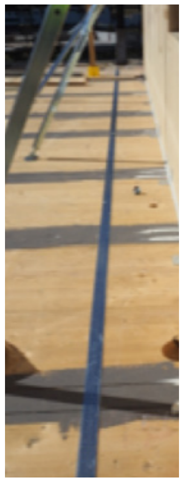


Figure 7.25: Drag straps.

7.7 Resistance to global overturning

The relationship between local tie-down of shear walls and global tie-down of the building is complex. Local tie-downs are required at the end of each bracing wall to prevent local rotation of the bracing wall as it transfers shear from one level to the one below. The bracing walls ensure that each of the individual bays of bracing floor to floor (the 'boxes' in Figure 7.1) remain rigid and shear is transferred from one level to another. However, under lateral loads, the whole building (the assembly of boxes on the right of Figure 7.1) could still rotate as a complete rigid body if there is not sufficient tie-down around the edges of the whole building to resist global overturning.

Consider the shear wall arrangement in Figure 7.26. Shear walls orientated in Y work together to resist lateral loads in Y. The connecting floor is stiff in plane, but flexible out of the plane. So, movement compatibility of the shear walls is ensured in Y. The floors span in Y so the shear walls are non-loadbearing. The walls share load based on stiffness but do not, as shown, work compositely so global overturning is resisted by local overturning of each shear wall contributing.

Figure 7.26: Example shear wall arrangement.

The honeycomb nature of the internal non-bracing walls (not highlighted in the diagram above as part of the stability system) will significantly contribute to the connection between the shear walls and therefore apparent composite action to contribute to global overturning. Given the building layout presented, it is anticipated that the central section highlighted orange in Figure 7.27 will tend to work compositely under lateral loads and will therefore contribute to resistance of global overturning over the full building cross-section. The grey areas either side of the orange central portion are considered as too flexible to contribute globally but will still contribute locally through load sharing into the shear walls through the diaphragm. More detail can be found in the worked example presented in Appendix 1.

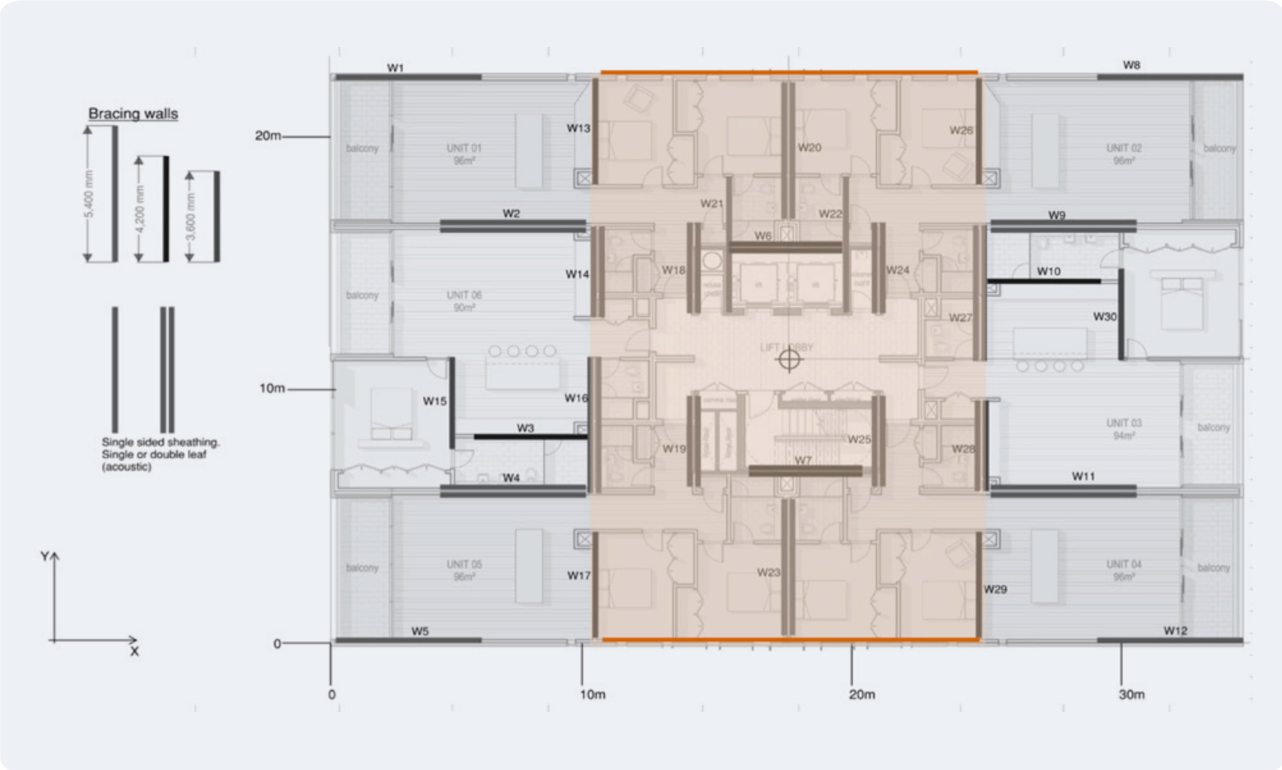


Figure 7.27: Activated bracing zone.

7.7.1 Deflection of stacked shear walls

As floor plans are often similar on each level of the building, shear walls can function as a 'stack' of bracing elements.

- Where the shear walls run the full width of the building (see Figure 7.28(a)), the stacked walls can function as a vertical cantilever. Tie-down rods at each end of the wall can be continuous over the full height of the building. These rods function as both bracing wall tie-downs and global overturning tie downs. Normal bending deflection expressions can be used to estimate the deflected shape of the building.
- Where the shear walls incorporate openings and discontinuities (such as central corridors), the shear walls are coupled walls (but non-composite) and behave in a different manner, as shown in Figure 7.28(b). Each bracing panel rotates relative to the wall frame behind it and creates a moment that resists the lateral force. The shear deflection of each frame gives the inter-storey drift, which can be compared with appropriate limits (say h/400 for example).

Figure 7.28(b) shows that, at the central corridor, the wall on one side has a net upward force and the wall on the other side has a net downward force. The vertical forces on either side of the corridor cause local floor rotations. If the floor system between has moment continuity then some composite action can be achieved. Significant moment continuity is required to achieve full composite action to transmit the longitudinal shear between the walls. The tie-downs on the face of the building function as tie-downs for both local and global overturning if composite action is achieved, while the tie-downs at the central corridor are tie-downs for local overturning only. The diagram in 7.28(b) broadly presents an assumption of stiff bracing walls with stiff vertical bearing and tie-down, but with relatively flexible shear connection between floors. This is as would be expected in typical timber-framed construction.

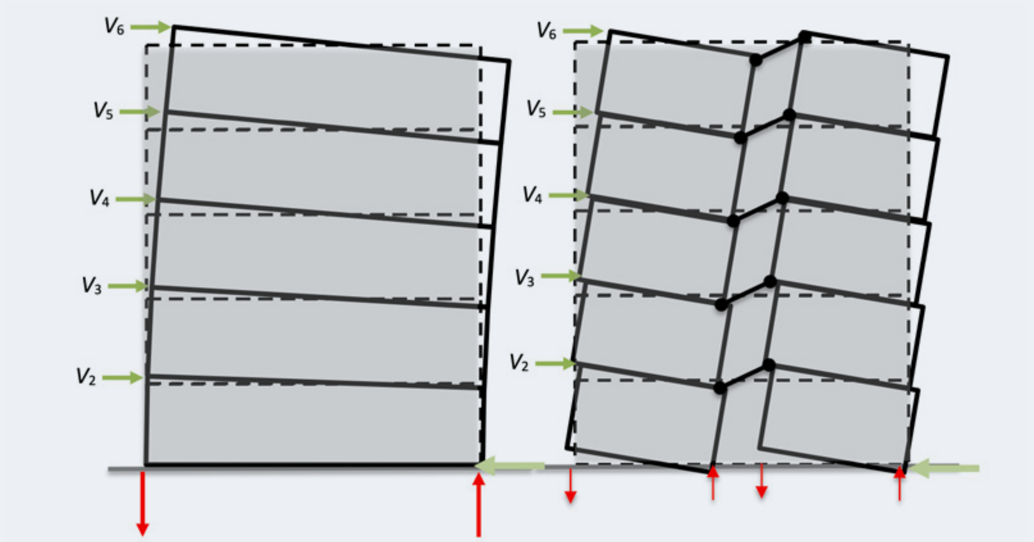


Figure 7.28: Exaggerated lateral displacements in stacked shear walls.

7.7.2 Global overturning moments

Global overturning moments are evaluated on the complete building considering the whole building as a rigid box. A global overturning moment produces:

- an upward vertical reaction along one edge of the building that increases compression in walls along that edge
- a downward vertical reaction along one edge of the building that decreases compression in walls along that edge and may introduce tensions along that edge.

When checking the compression capacity of walls, where loads are increased by the potential for global overturning, the loads are short duration (either earthquake or wind) and $k_1 = 1$ for these load combinations.

- ***The tie-down force required to resist global overturning is calculated by considering total loads on the entire building.***
- ***Wide, thin buildings are more susceptible to overturning effects – tie-down forces are higher across the small building dimension.***
- ***Distribute tie-downs evenly around the perimeter of the building where load distribution through the façade can activate it.***
- ***Roof tie-downs can double as bracing tie-down if they are next to bracing elements.***
- ***Tie-downs contribute to the robustness of the building.***

The tie down force is calculated by moment equilibrium using Equation 7.4 with the following assumptions.

- Potential tipping point of building is at ground line close to the side of the building in compression.
- Total overturning moment is calculated by summing the moment from all lateral forces with the lever arm as the height of the force above ground line. See Figure 7.29.
- The compression force (*C*) is aligned with the inner most external wall on the compression side of the building.
- The tension force (*T*) is aligned with the centroid of the tie-down elements (straps or rods).

$$T = C = \frac{M}{d'} \tag{7.4}$$

Where:

T	= Total tension force in tie-down elements along one side of the building
C	= Total compression force in frames along one side of the building
M	= Total overturning moment
d'	= The distance between Tension and Compression forces

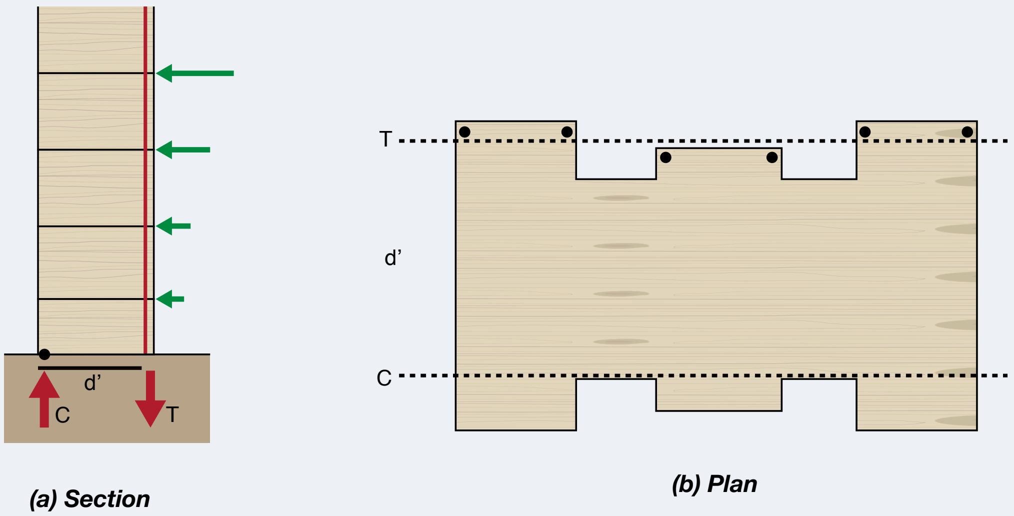


Figure 7.29: Global overturning forces.

Calculation Example
Building depth = 12 m; Net lateral force = 40 kN/m @ 18 m height.
Overturning moment = 40 x 18 = 720 kNm/m
Tension force = 720/12 = 60 kN/m
Typical permanent load = 1.5 kPa per storey
For 7 storeys = 7 x 1.5 x 12/2 = 63 kN/m
Net tie-down required = 60 – 0.9 x 63 = 3.3 kN/m.

7.7.3 Tie-downs to resist global overturning in framed wall construction

The tie-downs for global overturning are typically continuous rods.

Continuous tie-down rods in timber-framed walls

Threaded rods (often M20 or larger) that are securely anchored to the footings and run the full height of the building are often used. Each storey is connected at the top of the wall via a nut and bearing plate. As multiple storey threaded rods are impractical the threaded rod is usually joined at every storey above the bearing plate as shown in Figure 7.30.

Figure 7.30: Continuous threaded rod tie-down with coupler.

Design of tie-down rods

Under normal circumstances, there is no tension in the tie-down rods. However lateral forces on the building will cause an overturning moment and hence tie-down forces. Tie-down bars will elongate under these tension forces. The extension of the rod can be found using Hooke's Law (Equation 7.5).

$$\delta = \frac{Th}{AE} \tag{7.5}$$

Where:

δ = extension of the tie-down rod under load
T = tension force in a single tie-down element
h = height of rod (in many cases, the height of the building)
A = cross sectional area of steel steel in a single tie-down element
E = MoE of steel in the tie-down element.

The rod elongation is usually less than about 5.0 mm per storey. Also, the shortening of the building may reduce the effectiveness of the rod as shown in Figure 7.31 For this reason, shrinkage compensators are often used to ensure that the nut and bearing plate bears on the timber even after shortening of the building.

Figure 7.31: Result of shortening on tie-down connection to timber frame. (Image: SST)

To design the tie-down rod:

- ***calculate extension of the tie-down rod – typically 5 mm (Equation 7.10);***
- ***estimate shortening of the building – typically around 30 mm (Section 2.4);***
- ***estimate crushing of the timber under the bearing plate – typically <1 mm.***

Crushing of the timber can be estimated using either:

- a conservative estimate of 1 mm, or
- the following steps that are based on preliminary test data from Australian research projects.

1. Calculate the load transferred from the tie-down rod to the steel bearing plate under the nut.
2. Use the steel bearing plate size to calculate the bearing pressure on the timber under the steel plate.
3. Calculate the compression strain in the timber using the strain rate presented for a range of timber species in Table 7.3. If the strain is higher than the proportional limit, the bearing plate should be increased in size and the calculation repeated.
4. Compensate for bearing area by dividing the result from Step 3 by the factor in Table 7.4, or If the bearing is within 75 mm of an end of a timber element, divide the result from Step 3 by 0.8.

Table 7.3: Strain rates of various common seasoned timber species[1].

Timber Species	Timber density at 12 % MC For tested samples	Strain Rate (mm/MPa)	Proportional Limit (mm)
Spruce	400	0.083	0.4
Radiata Pine	490	0.066	0.5
Douglas Fir	470	0.066	0.6
Cypress	690	0.026	0.6
Victorian Ash/ Tasmanians Oak	600	0.063	0.7
Flooded Gum	840	0.031	0.5
Silvertop Ash	890	0.031	0.5
Karri	880	0.019	0.5
Ironbark	1078	0.019	0.7
Blackbutt	1000	0.019	0.4
Spotted Gum	1169	0.015	0.7

1. Perpendicular to grain bearing test on common timber species, engineered timber and reinforcement methods, TDA 2018.

Factors in Table 7.4 are based on k7 from AS 1720.1.

Table 7.4: Multiplication factor of different bearing lengths.

Length of bearing (mm)	12	25	50	75	125	150
Strain factor	1.55	1.2	1.0	0.95	0.9	0.8

The extension of the rod can be controlled by choosing a rod with a large enough cross-sectional area, and the crushing under the bearing plate by selecting a large enough bearing plate with sufficient thickness.

7.7.4 Take-up devices

Shrinkage, crushing or creep can cause building shortening that accumulates over the height of the building and result in the gap shown in Figure 7.31. Take-up devices (also known as shrinkage compensators) can be used to close these gaps.

There are two types of take-up devices:

- expanding (Figure 7.32)
- ratcheting (Figure 7.33).

Expanding take-up devices

These devices use a spring that applies constant load onto the bearing plate and washer to ensure continuous connection at all times. The likely travel of the device needs to be estimated. It is a combination of:

- extension of the tie-down rod
- shortening of the building (Chapter 6)
- crushing of the timber under the bearing plate.

Take-up devices are generally located within the wall structure and so cannot be adjusted in service. Consult with the device supplier to ensure that it can accommodate the expected movement during the life of the building.

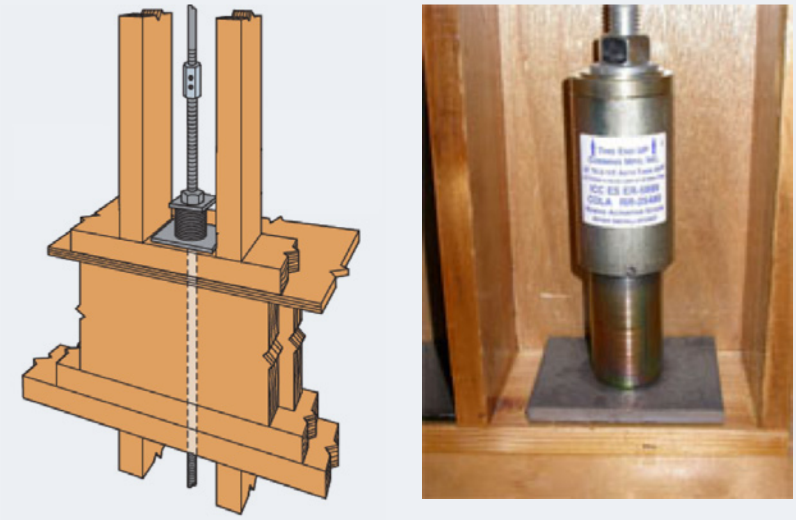


Figure 7.32: Expanding take-up device. (Image: Simpson Strong Tie, Photo: Jane Arnolda, Structerre)

Ratcheting take-up devices

These devices rely on the teeth within the device locking onto the thread on the rod; as the building shortens, the teeth ratchet to the next thread below it. The movement of the device is one way only. The advantage of this device is that there is no limit on movement nor is there a need to activate the device after it has been installed. The bearing plate size is determined by the same process as discussed for expanding device above. The shortening of the building does not have to be estimated to specify the correct ratchet take-up device.

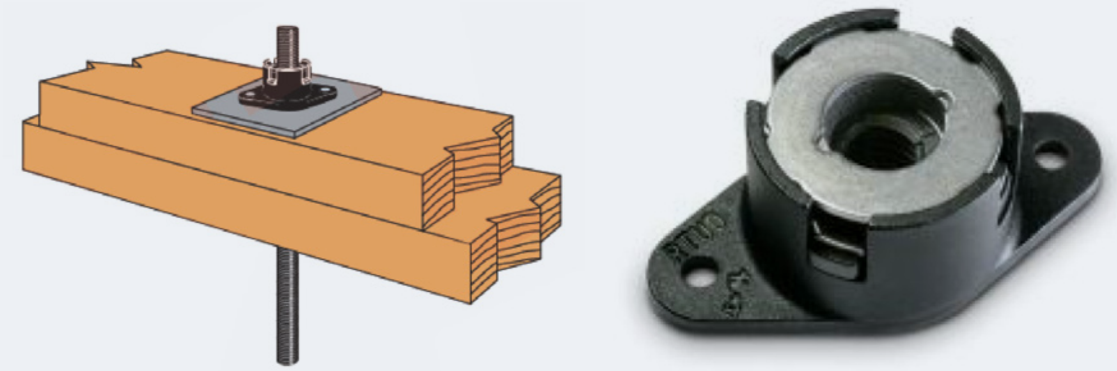


Figure 7.33: Ratchet take-up device. (Images: Simpson Strong Tie)

Location of take-up devices

Take-up devices should be positioned under the nuts that hold down the floor at each storey as shown in Figure 7.34. It is recommended that the rod joiner is positioned above the take-up device to avoid any interference as the building shortens. There are also take-up devices that function as both take-up devices and joiners.

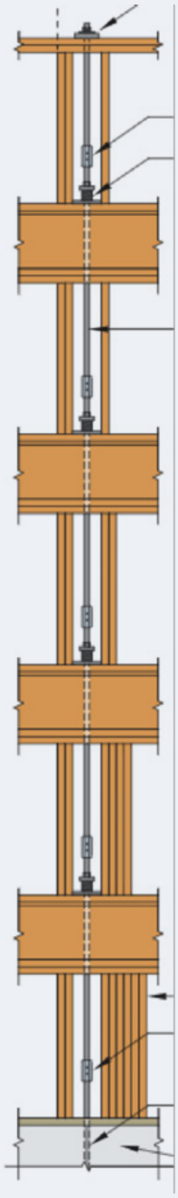


Figure 7.34: Positioning of take-up devices above each floor. (Images: Simpson Strong Tie)

8 Robustness Design

The principles of structural robustness are presented in the NCC and AS/NZS 1170.0. Both require that after some unspecified accidental damage the building is only affected locally "with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage". This is a requirement for all buildings, regardless of method of construction or material used.

The implications of this structural robustness requirement are that:

- accidental damage to individual structural components will only cause localised damage
- the damage should not spread too far within the whole building. This limits the extent of cascade failures following an initial loss of elements that lead to widespread collapse.

WoodSolutions Technical Design Guide #39 Robustness in Structures details design strategies to achieve the requirements for robustness. The main principles are:

Arrest collapse – direct loads through alternative load paths.

- Bracing walls act as deep beams to bridge over missing elements.
- Provide effective horizontal and vertical ties to mobilise catenary action through appropriately designed connections.
- Rim beams around top of wall panels support floors, span over gaps and redistribute load to support floors if a load bearing wall is damaged.
- Reinforce one-way spanning floors, e.g. strong backs in cassette floors to enable some two-way action in an emergency.
- Install drag straps at re-entrant corners of floor panels.

Conservative design – increase factor of safety.

- Where alternative load paths are not available, design elements with capacity to carry extra loads from increased tributary areas following failures of other structural elements.
- Duplicate some key elements if possible to introduce redundancy.
- Consider using a safety factor >2 on unprotected key elements.

Protect key elements – protect cores and load transfer structures such as transfer beams or trusses from accidental damage, e.g. vehicle impact.

Timber structural elements designed for normal loads typically have significant reserve capacity to achieve robustness requirements. Duration of load in emergency events is relatively short which gives an increase in capacity through the use of a high k_1 value.

Disproportionate damage

While much of structural design focuses on loads and the effect of those loads on the building, the concept of robustness ignores loading and focuses entirely on the consequences for the building.

The concept of disproportionate damage is:

- a significant amount of damage following an accidental load on a relatively small part of a building; or
- widespread damage to a number of floors or to a significant floor area after an accidental load has removed a supporting element.

The concept leads to a robustness objective: "An accidental load on a relatively small part of the building should lead to controlled damage."

A very significant accidental loading, for example aircraft impact, could be expected to produce very significant damage without compromising the robustness concept.

Progressive collapse is a parallel concept to disproportionate damage.

- If an accidental loading causes collapse of a small part of a building, the damage should not cause the failure to spread so that other parts of the building also collapse.
- Loss of some structural elements should not cause a 'domino effect' in the remaining elements, where the adjacent elements are overloaded, fail and distribute their loads to the neighbouring elements that are also overloaded.

The opposite of progressive collapse is part of the objective: "In the event of loss of a structural element, the building must absorb the load transfer without significant spread of collapse."

8.1. Accidental Loads and Events

Causes of accidental damage to buildings can include the following.

- **Unintended vehicle impact** – out-of-control or over-size vehicles making direct contact with the structure. In many cases, vehicle impact damages, breaks or removes structural elements such as beams, columns or walls. Figure 8.1 shows an unexpected vehicle impact with a timber-framed building. While the wall that was hit by the vehicle sustained significant damage, widespread collapse was avoided.
- **Explosion** – gas leaks or criminal acts causing blast loadings on floors, walls and columns. It is generally not possible to design all structural elements to resist unspecified blast loadings, which means that a blast may result in loss of the floor, wall or column.
- **Unexpected building movement** – movement of the structure that is greater than the construction tolerances and bearing lengths. Loss of bearing under some floor or roof panels can lead to collapse of the supported element. This local collapse has been historically experienced in buildings that have used prefabricated elements, so is also relevant for mid-rise timber buildings.
- **Design and construction** – tolerances and uncertainties introduced relative the assumptions made in structural design lead to load paths which may differ from those used in design.

Figure 8.1: Accidental structural damage due to unexpected vehicle impact.

Rather than modelling the load that caused the damage, in most cases the design for robustness anticipates the loss of some elements and designs the structure to continue to transmit normal gravity loads using alternative load paths. (See Section 8.3 for some methods of checking the capacity of the alternative load paths in mid-rise timber buildings.)

8.2 Achieving Robustness in Mid-rise Timber Buildings

8.2.1 Preventing accidental damage from causing a collapse

Protected Elements

Some elements in a structure are vital in carrying loads to the ground and have no redundancy. As accidental damage to these elements is likely to cause a significant collapse in the building, they can be designated as 'Protected elements'. The design strategy is to prevent them from sustaining any accidental damage.

Some examples of protected elements include: the following.

- **Cores** – may be protected by configuring ground storeys and basements so that it is not possible for vehicles to get close to them and design them to so that they can resist accidental damage from other impacts. This strategy is applicable to all stair and lift cores.
- **Load -transfer structures** – large load- transfer beams or trusses can be protected by strategies including: ensuring that there are lower elements in the basement to prevent over-height vehicles from contacting them; or designing them with redundancy so that if one element is removed other elements can still carry the load (refer to Section 8.3.4). In mid-rise timber buildings, load- transfer beams or trusses are likely to be in the concrete structure at the first storey or ground level.

Resilient elements

In some cases, it is possible to design important structural elements to carry loads from increased tributary areas. Some eExamples include: the following.

- designing ***one-way floor panels*** to span an increased span if one line of support fails (larger deflections are acceptable);
- designing ***load -bearing walls*** to withstand additional loads due to an increased tributary width;
- providing ***additional redundancy*** in design (e.g. ties in load- bearing walls that allow an upper floor to support a lower floor).

The design loads for estimating the performance of these resilient structural elements should be the same as those the fire limit state as shown in Section 8.3.1: $G+\psi_l Q$. The reduced load makes it possible to carry load from a larger tributary area.

8.2.2. Arresting a collapse

If a structural element is missing because of a local collapse, a more widespread collapse can be prevented by directing loads through an alternative load path.

FPInnovations Technical Guide for the Design and Construction of Tall Wood Buildings in Canada has examples of design concepts to help designers mitigate the possibility of progressive collapse including the following.

- **Arranging interior walls to provide additional support in wall- bearing structures**. In this case, walls that may have been designed as bracing walls can act as deep beams to bridge over missing elements below them. (The walls normal to these walls provide stability for them by acting as a deep beam.)
- **Providing a system of ties along the principal elements in the building**. Where there are local failures in any of the principal elements, there will be large deflections that will mobilise tensions in the connections. The strong ties between elements allow structural elements to also function as catenaries and carry some of the load in tension. There needs to be a load path elsewhere in the building to resist the forces generated by the alternative load path. Connection design in such a case is strength governed. Future developments to AS1720.1 include incorporation of the European Yield Model (based on Johannsen's theory) into connection capacity design which will allow for more refined interpretation of expected connection failure modes (ductility).
- **Reinforcing one-way-spanning floor elements to sustain loads in the perpendicular direction as an alternate load path.** Strong backs in floor cassettes fill this role and CLT floor panels have some capacity in bending perpendicular to the face grain because of the inner layers of timber in the panels.

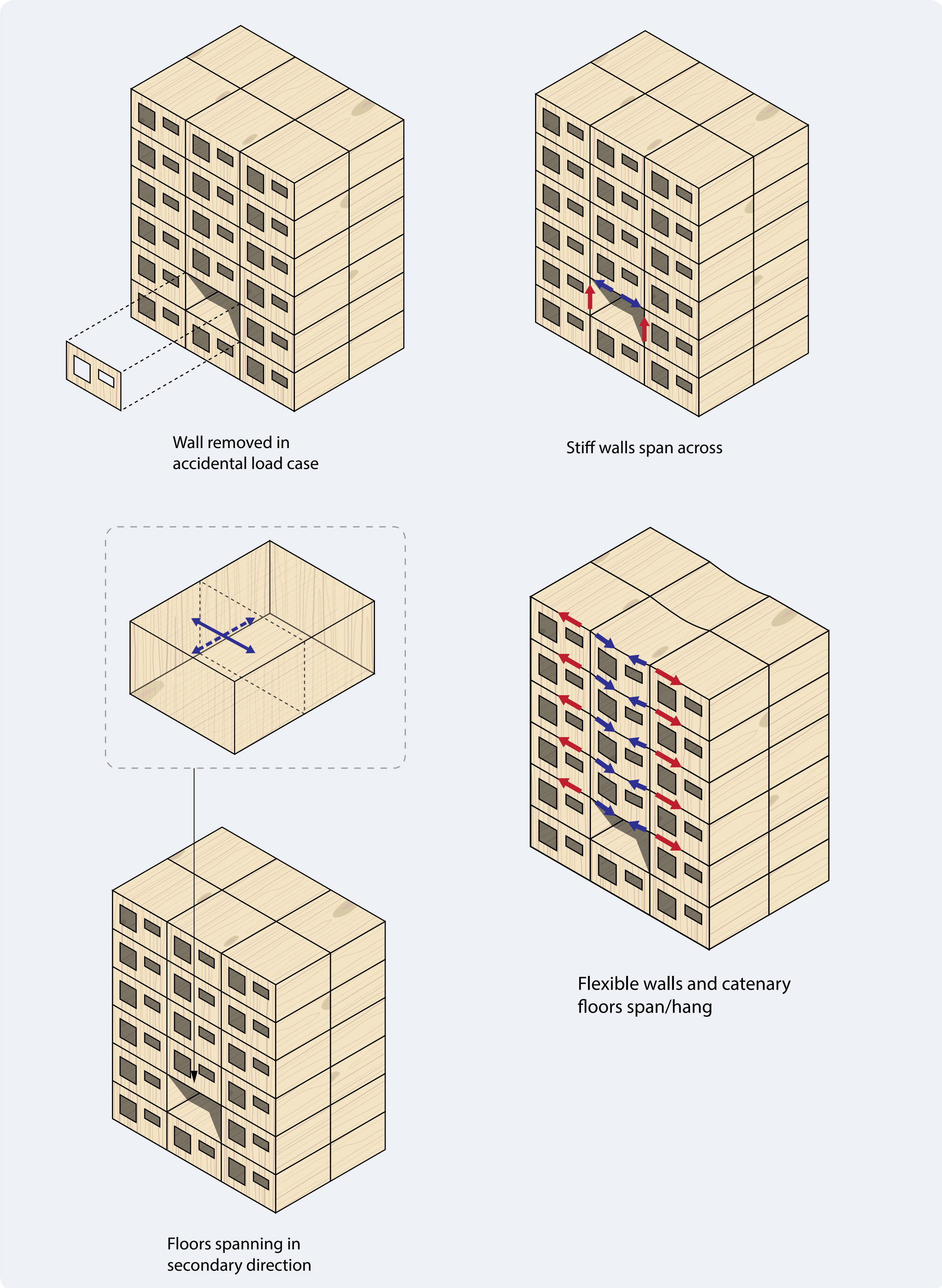


Figure 8.2: Alternative load paths.

For roof structures under gravity loads, the roof cladding itself can function as a folded plate structure and continue to resist load after the loss of significant structural elements such as trusses.

8.2.3 Timber framing – ring/rim beams

Some alternative load paths place tension in the floors, so there needs to be an element at the outside of the building that resolves the tension. Ring beams provide:

- a compression flange that enhances the in-plane bending capacity of the floor
- a stiffener at the edge of the floor plate that facilitates the floor to safely carry compression
- assistance to the deep beam bending of the wall above or the wall below if there is loss of support.

Ring beams must be continuous and well connected at corners as illustrated in Figure 8.3.

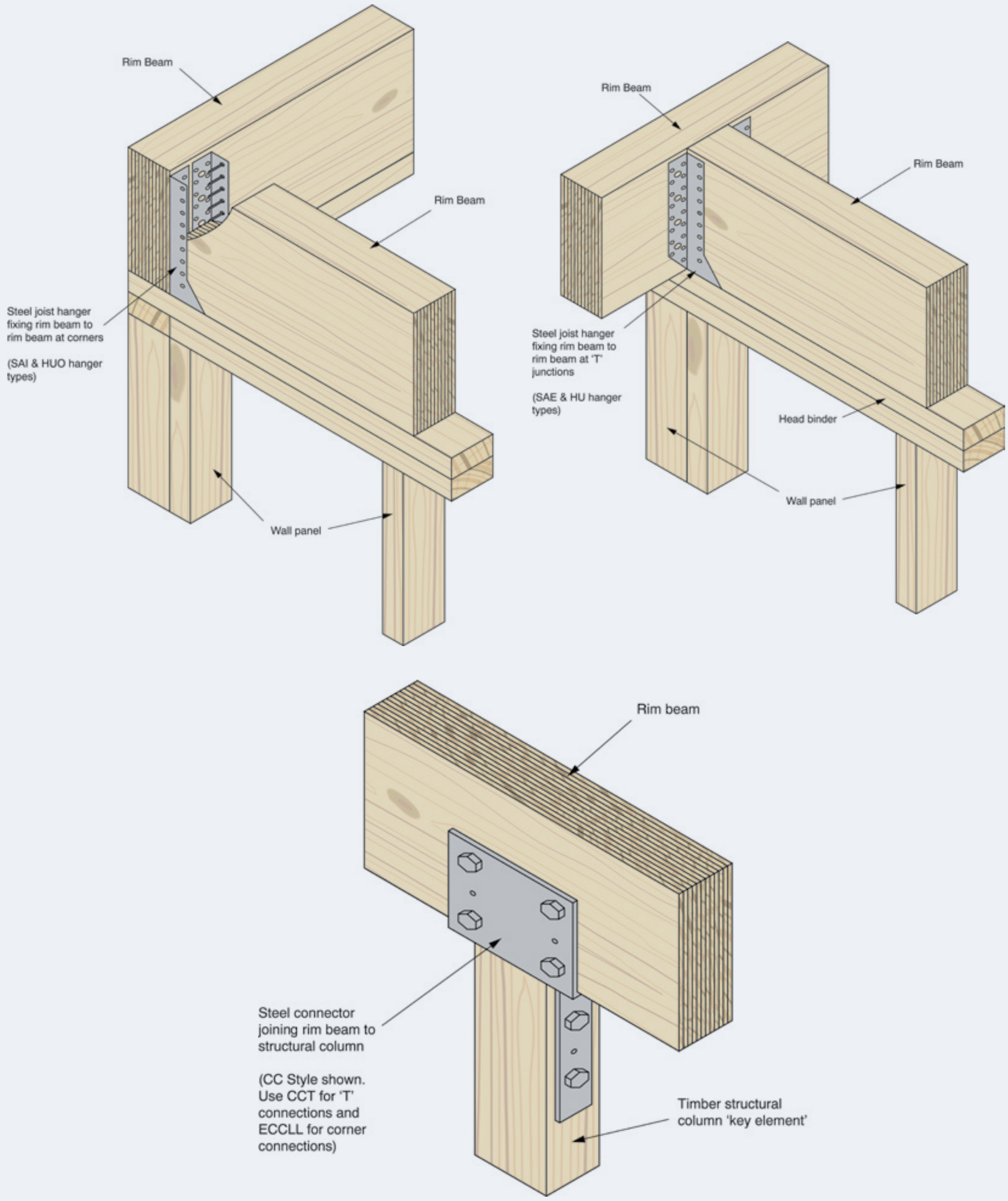


Figure 8.3: Ring beam connections (from WoodSolutions Technical Design Guide #39 Robustness in Structures).

8.2.4 Mass-timber panels

There are many challenges in designing a mass timber building to be compliant with the NCC clauses around robustness, largely stemming from two issues:

- the structure is made up of multiple discrete elements which all need to be connected together
- the connection capacity between mass timber elements is typically more limited in capacity and less ductile that other materials like steel and concrete.

The challenge to design a robust mass timber building needs to be considered from the start of a project as it can inform how buildings are set-out and what structural systems are chosen. A typical CLT honeycomb apartment has a lot of potential for alternative load paths but a post and beam mass timber building is a much trickier prospect that may rely on strapping or ties embedded into the structure to accommodate potential failure modes.

Simple guidance principles for designing more robust buildings include:

- Designing floors to allow them to 'double span' in the event of a support failure

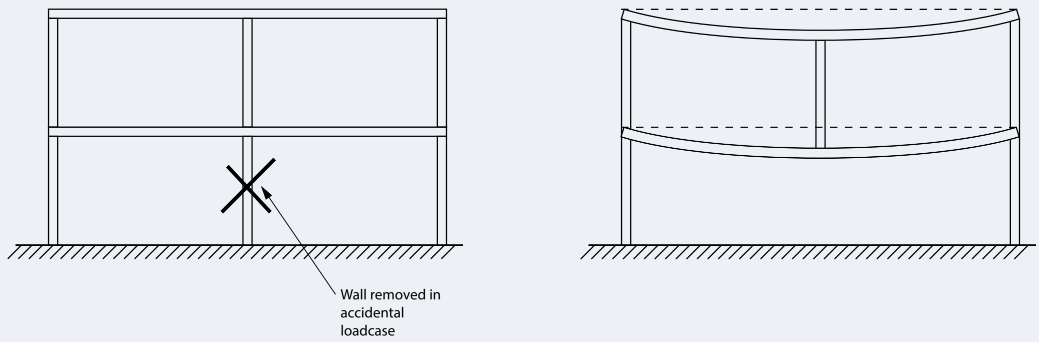


Figure 8.4: Removal of a load-bearing wall in a CLT building. (from WoodSolutions Technical Design Guide #39 Robustness in Structures)

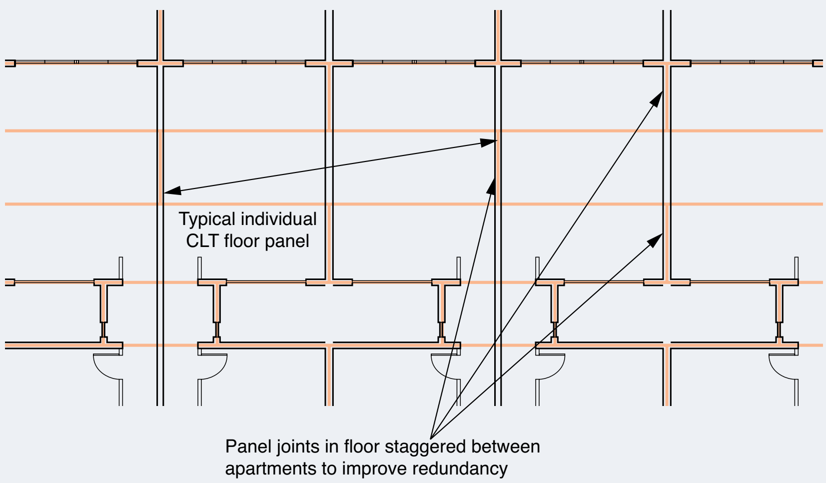


Figure 8.5: Staggering floor joists in CLT buildings is good practice for robustness. (from WoodSolutions Technical Design Guide #39 Robustness in Structures)

Ensure connections have capacity in secondary directions (i.e. shear connections for a beam to post should also have some tension capacity in the event that they need to work in catenary action to support the floor). In the example below the brackets providing the shear connection between the wall above and the floor are designed to support the floor in tension if the lower walls is removed. This ability to accommodate large rotations in timber joints can be limited as failures can often be more brittle. The use of metallic connectors will to some extent add ductility into the system but care should be paid to where the failure mechanisms will occur, with them ideally being designed to occur in the metal sections.

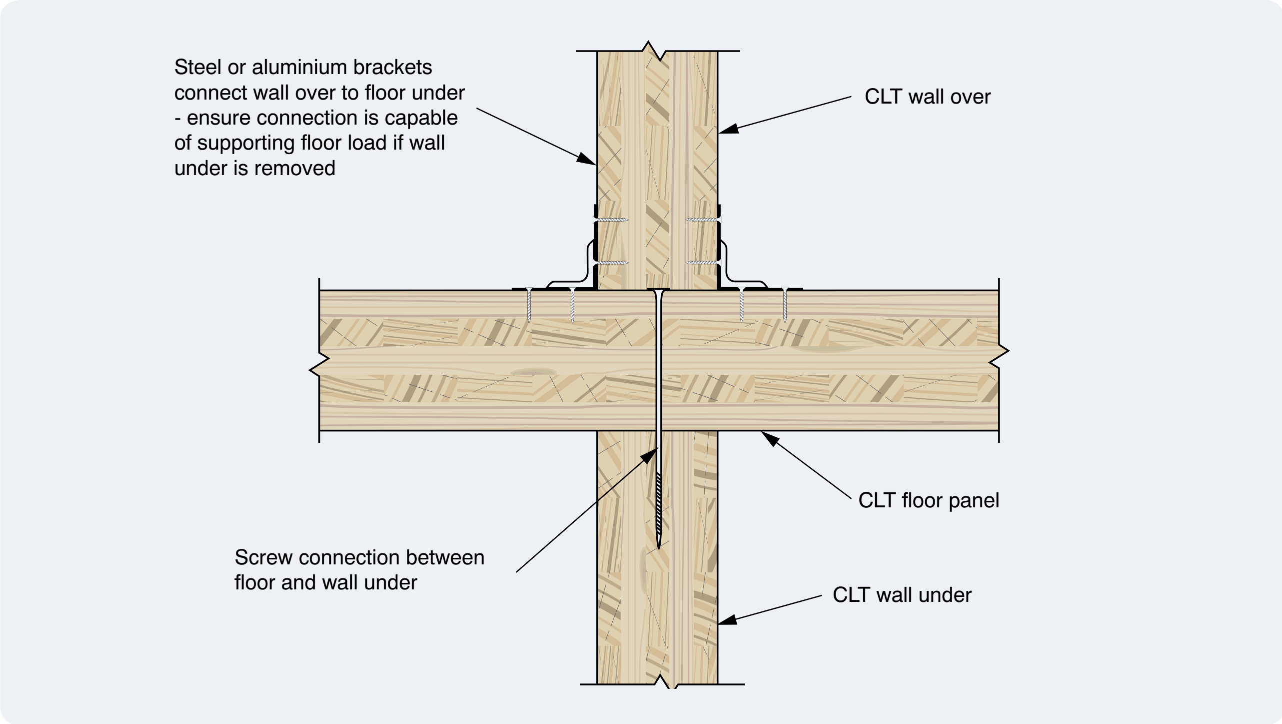


Figure 8.6: Typical wall-to-floor connection in a CLT building. (from WoodSolutions Technical Design Guide #39 Robustness in Structures)

- Designing wall panels as deep beams in temporary cases to allow them to span over accidentally damaged panels below.

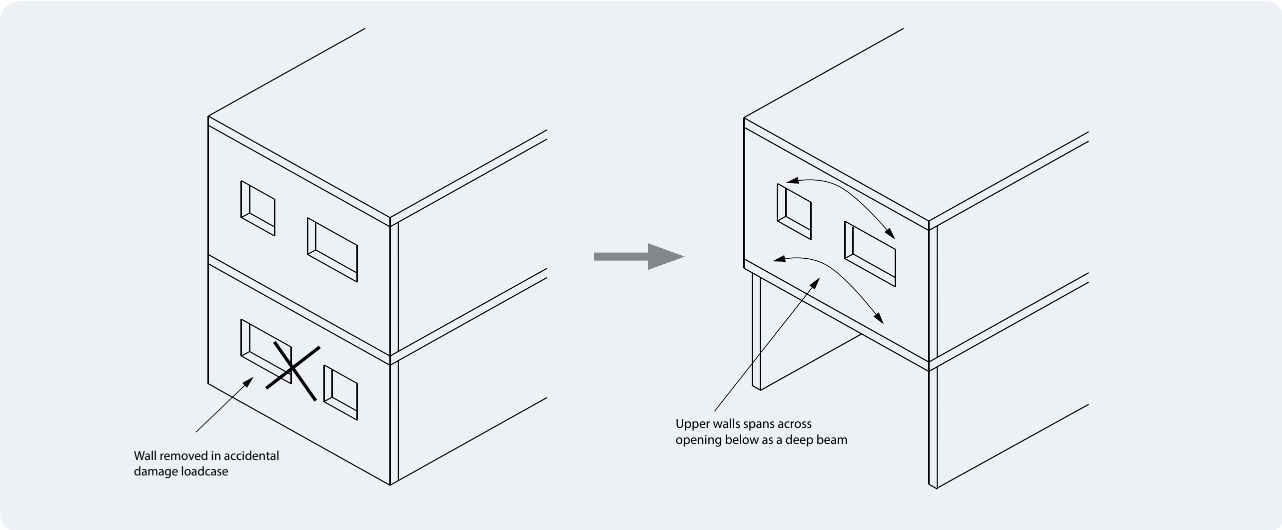


Figure 8.7: Spanning of walls in accidental damage load cases. (from WoodSolutions Technical Design Guide #39 Robustness in Structures)

For more information, read WoodSolutions Technical Design Guide #39 Robustness in Structures, which provides more information on how to design robust structures across all materials and looks in more detail at the NCC's requirements.

8.3 Demonstrating Robustness During Design

WoodSolutions Technical Design Guide #39 Robustness in Structures presents a flow chart that shows methods of demonstrating that the robustness requirements have been satisfied. For Class 2 and Class 3 buildings, the requirements can be satisfied by:

- applying gravity loads to a model of the damaged structure and demonstrating that they can be safely resisted by the following mechanisms
- providing appropriate ties between building elements (principally floors and walls)
- demonstrating that loads can be carried by alternative load paths using an analysis that models a missing structural element
- protecting key elements.

8.3.1 Loads for demonstration of robustness

The loads specified for robustness are not the unknown loads that cause the accidental damage to the building. They are the loads that have to be resisted by the partially damaged structural system.

AS/NZS 1170.0 defines the fire limit states load combination as $G+\psi_{\ell}Q$. These loads are compatible with the minimum lateral load resistance and minimum connection capacity for robustness in Section 6 of AS/NZS 1170.0 $G+\psi_c Q$. (European standards define loads for robustness calculations as $G+\psi_e Q$. The European robustness loads are slightly lower than the loads provided in AS/NZS 1170.0 as typically $\psi_{\ell} = \psi_c = 0.4$ and $\psi_e = 0.3$.)

For timber structures, the load combination must also be accompanied by an appropriate duration of load factor, k_1. While neither AS/NZS 1170.0 or AS 1720.1 give guidance on this, it is a short-term emergency load event, similar to the fire limit state, and for that load combination, $k_1 = 1.0$. This is also the value recommended in Technical Design Guide #39.

Check for robustness using a load combination of $G+\psi_c Q$ with $k_1 = 1.0$.

This example shows that there is a significant reserve of strength in timber structural elements to accommodate robustness in normally designed floors and supporting elements.

Typically floors will be designed for $1.2\,G + 1.5\,Q$ with $k_1 = 0.8$.

The same elements will be checked against robustness provisions using $G + 0.4\,Q$ with $k_1 = 1$.

Comparing these two load combinations gives the performance ratio (load combination/k_1) shown in Figure 8.8.

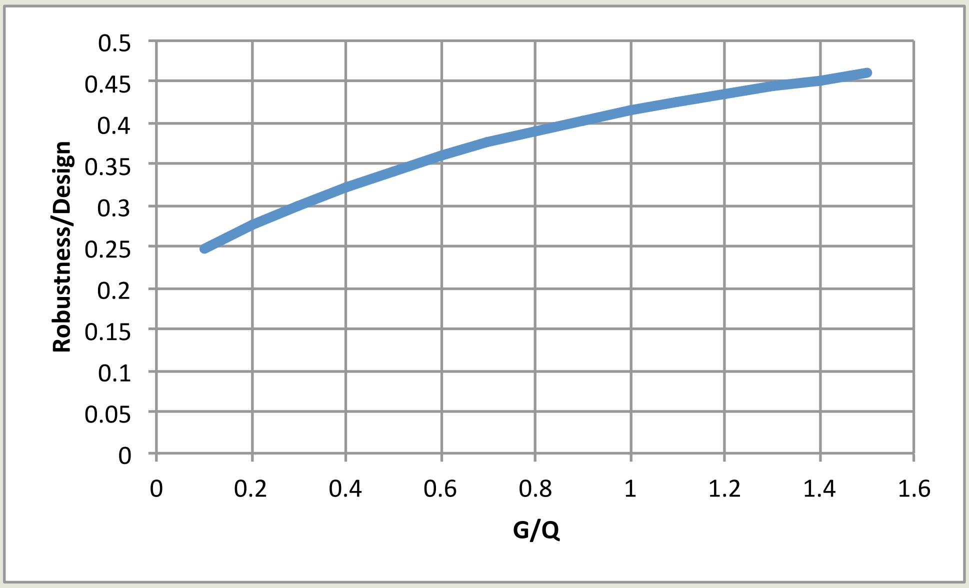


Figure 8.8: Performance ratio for Robustness compared with normal design.

Figure 8.8 shows that for normal G/Q ratios for timber, the robustness load case gives a significantly lower structural demand than the normal design conditions. This means that elements have the capacity to support longer spans under robustness loads compared with other design conditions. The combination of the capacity to carry longer spans and alternative load paths can be used to demonstrate structural integrity after removal of a support. Caution is needed on the provision of ties to ensure the accidental load path can be maintained.

8.3.2 Providing effective ties

AS/NZS 1170.0 indicates that all parts of the structure must be tied together and the connections designed to resist 5% of the vertical robustness load as a lateral tying force. The flow chart for robustness given in WoodSolutions Technical Design Guide WS TDG#39 Robustness in Structures indicates that satisfying this provision is all that is required for mid-rise timber Class 2 or Class 3 buildings above basement level. Basements must demonstrate alternative load paths or have key structural elements designed as outlined in Section 8.3.4.

The tie force is evaluated as 5% of a gravity force ($G+\psi_c Q$) applied horizontally for the connection of floors to walls or floors to other floor elements (or vertically between wall elements on adjacent storeys). Check the capacity of the connections for robustness against the AS 1720.1 connection capacities with $k_1 = 1$.

Connections between elements should be designed for the largest lateral load:

- assembly on site and maintenance of construction tolerances;
- requirements for lateral load transfer in the structure under wind or earthquake loads; or
- robustness considerations.

The same connection detail can be used to serve all these functions independently. It is not required to apply loads from all these scenarios simultaneously.

The European minimum tie force for robustness (40% of the robustness load with a minimum of 75 kN) is significantly greater than 5% of gravity forces required in AS/NZS 1170.0.

8.3.3 Demonstrating alternative load paths

The structure should perform adequately with the following components removed from the load path:

- an external wall with few windows
- an internal wall supporting a floor element with a long span
- a larger floor panel.

The loads used should be the combination outlined in Section 8.3.1, and compared with the capacity of members and connections evaluated with short-term loads (k_1 = 1 for members and connections).

In each case, modify the structural model of the building by removing an element and checking the load paths and the capacity of surrounding elements to resist the loads:

- remove structural elements as required
- apply the robustness loads to the entire structure
- evaluate deformations and stresses in surrounding elements
- check the proportion of load that can be carried in secondary load paths (e.g. catenary behaviour) in elements with large deflections
- check the capacity of surrounding members and connections with k_1 = 1.

Removal of a wall panel

Use the structural model of the building to model the loss of a supporting element such as a wall. The alternative load paths may require the use of different elements or modelling methods to correctly model the secondary effects such as:

- tension in deformed floor plates
- in-plane bending of walls acting as deep beams.

Removal of a floor panel

The loss of a floor may not affect the nearby walls, but it may contribute extra load to the floor below or increase the wall effective length. This effect is modelled as follows:

- Remove one floor section (generally a room-sized floor section that contains a number of panels).
- Apply an increased load to the floor below: $2G + \psi_c Q$. This combination accounts for the dead load from both floors but only one live load. (This combination is based on the loading used in the European robustness requirements for floor loss).
- Check the capacity of the floor system to support the applied robustness loads with k_1 = 1 for members and connections.

Removal of a roof structural element

The loss of a significant structural roof element (such as a truss) will require the roof cladding to be modelled as plate elements. In this case, the structure below the roof level is external to the analysis, and the robustness model can be based on the roof on rigid supports.

8.3.4 Designing key elements

If it is not possible to design an alternative load path for a structural element, then it can be treated as a key element. The element can demonstrate robustness using the following.

- **Protection** – positioning the key element behind sacrificial features so that it is protected from accidental damage.
- **Redundancy** – introducing adjacent elements that can carry the load if the key element is accidentally damaged.
- **Conservatism** – designing the key element with a larger factor of safety so that it is able to perform its intended function after sustaining some damage.

Protection

Protecting key elements from exposure to accidental loads is important to provide robustness to timber cores and transfer beams or trusses because timber elements have limited ductility.

Vehicle impact and criminal acts involving large amounts of explosive are the main threats to these elements. In both cases, separating the key elements from vehicular traffic will reduce the loads on those elements in an attack and therefore improve the robustness of the building. The separation may be achieved by enclosing these elements within spaces that are not accessed by vehicles, e.g. positioning transfer beams within rooms rather than in parking areas, and surrounding lift and stair wells with storerooms or lobbies.

Redundancy

Key elements in mid-rise timber buildings can be duplicated. Under normal circumstances, they each carry half of the applied load, so are under-utilised. However, in special circumstances, the load can be successfully carried even if one element is removed:

- an element can be replaced without compromising the building performance.
- there is an alternative load path if the structure is damaged.

This strategy would only be used for some key structural elements where it was not possible to provide protection. It is not a viable option for all structural elements.

Conservatism

Some key elements cannot be doubled up, but by increasing their size, the factor of safety on the member can be increased so that it is better able to carry the robustness loads in case of:

- damage to that element so that its capacity has been compromised or
- damage to other elements in the structure so that load paths elsewhere in the structure have been changed and the load on the key element has increased.

9 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.
Axial Shortening	Reduction in length of vertical elements caused by shrinkage, compression deformation, creep and settlement.
Box beam	A built-up beam with solid timber flanges and plywood or wood-base panel product webs.
Bracing	An assembly used to resist racking forces.
Buckling	Sideways deflection of a structural member under compression.
Building element	A principal part of a building, such as a roof, wall or floor.
Capacity factor	A factor used to multiply the nominal capacity to obtain the design capacity.
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.
Cladding	Coverings to external wall surfaces.
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.
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.
Continuous Span	A member supported at three points and separated so the span between the supports is not less than half of the other span.
Creep	Increase in deformation following prolonged loading.
Cross Laminated Timber (CLT)	Wood panel products manufactured from gluing layers of solid-sawn timber together, with each layer orientated perpendicular to the adjacent layers.
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.
Deemed-to-Satisfy Provisions	Provisions that are deemed to satisfy the Performance Requirements of the NCC.
Deemed-to-Satisfy Solution	A method of satisfying the Deemed-to-Satisfy Provisions in the NCC.

Term	Explanation
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.
DfMA	Design for manufacturing and assembly. In isolation, Design for Manufacturing refers to making the manufacturing and CNC machining of timber elements as simple as possible, without due regard to how it might be installed onsite. Whereas, Design for Assembly focuses on the erection and construction process without due regard for how long it might take to manufacture in the factory. Design for Manufacture and Assembly considers both manufacturing and onsite erection. When properly coordinated and executed, DfMA ensures that the overall process is as efficient as possible
Discontinuous construction	A wall having a minimum 20 mm cavity between two separate leaves. A staggered stud wall is not deemed to be discontinuous construction.
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.
Duration of loading	Period during which a member, a structural element or a complete structure is subject to a specified load level
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.
Effective height	The vertical distance between the floor of the lowest storey included in the calculation of rise in storeys and the floor of the topmost storey (excluding the topmost storey if it contains only heating, ventilating, lift or other equipment, water tanks or similar service units).
Effective Span	The distance between the centres of areas of bearing or connections.
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 (EWP)	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. CLT, Glulam, plywood, LSL, LVL, I-beams, and OSB are engineered wood products.
Equilibrium moisture content (EMC)	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.
F-grade	A stress grade of timber for which the specific suite of design properties given in AS1720.1 Table 2.4
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.

Term	Explanation
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-protected timber	Fire-resisting timber building elements that comply with NCC Volume One Specification C1.13a. Includes the use of fire-resistant lining e.g. fire-rated plasterboard.
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.
Floor cassette system	A floor system assembly of floor joists, flooring and associated members prefabricated off-site, then transported to site and installed by crane into the building. Very quick and easy to install and provides higher levels of workplace safety for on-site workers reducing fall from height risks. Panel sizes generally set by transport limits, typically up to 12 m long and 3 m wide.
Floor load width (FLW)	Floor load width is the width of floor contributing load to the supporting member, measured horizontally.
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 (Glulam or GLT)	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.
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.
Hygroscopic	A material that gives off and absorbs moisture to remain in equilibrium with the surrounding atmosphere.
Insulation	In relation to an FRL, the ability to maintain a temperature on the surface not exposed to the furnace below the limits specified in AS 1530.4
Integrity	in relation to an FRL, the ability to resist the passage of flames and hot gases specified in AS 1530.4.
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 (LSL)	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.

Term	Explanation
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	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.
Lining	Coverings on internal wall surfaces.
Loadbearing Walls	Walls supporting loads other than self-weight.
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.
Massive timber construction	In the NCC, "massive timber means an element not less than 75 mm thick as measured in each direction formed from solid and laminated timber". This can include cross laminated timber, laminated veneer lumber and glued laminated timber.
Member	A beam or column within a structure or assembly.
MGP	A grade of softwood structural timber whose properties are defined In Appendix H of AS1720.1. It should be noted that MGP 10 and MGP 12 grades are generally available, but MGP 15 has a restricted supply (check with suppliers before specifying MGP 15).
Mid-rise buildings	A building of four or more storeys but with an effective height of less than 25 metres.
Modified resistance to the incipient spread of fire (MRISF)	The ability of a covering (i.e. fire-rated plasterboard) to insulate mass timber elements (greater than 75 mm thick), so as to limit the temperature rise to a level that will not permit ignition of the timber. The MRISF is expressed in minutes and indicates the time the covering will maintain a temperature below specified limits. See Wood Solutions Technical Design Guide #37 for a more detailed explanation of MRISF requirements
Moisture content	The amount of moisture contained in wood, expressed as a percentage of the oven dry mass.
Nail laminated timber (NLT)	Sections of sawn timber nailed together to form larger, more structurally reliable timber elements. The sections may be 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 90-degree 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.
National Construction Code (NCC)	The NCC is Australia's primary set of technical design and construction provisions for buildings. As a performance-based code, it sets the minimum required level for the safety, health, amenity, accessibility and sustainability of certain buildings. The NCC has three parts. NCC Volume 1 Building Code of Australia covers the design and construction of mid-rise buildings.
Noggings	Horizontal members fitted between studs in a wall frame.
Non-combustible	Material determined by AS 1530.1 to be non-combustible.
Non-loadbearing Wall	A partition wall that does not support load other than self-weight.
Oriented Strand Board (OSB)	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.
Particleboard	An engineered wood product made from timber particles combined with adhesive bonded together under heat and pressure into sheets.

Term	Explanation
Performance Requirement	Under the NCC, a requirement that states the level of performance that a Performance Solution or Deemed-to-Satisfy Solution must meet.
Performance Solution	A method of complying with the Performance Requirements of the NCC other than by a Deemed-to-Satisfy Solution (previously known as an Alternative Solution)
Permanent action	Dead loads, such as the self-weight of the structure or fittings, ancillaries and fixed equipment.
Platform frame construction	A building method where the floor system is supported directly on top of the wall frames. The next level of wall frames is then built/installed off this platform.
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.
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.
Resistance to the incipient spread of fire (RISF)	The ability of a covering (ie fire-rated plasterboard) to insulate voids and timber framed elements, so as to limit the temperature rise to a level that will not permit ignition of the timber and the rapid and general spread of fire throughout any concealed spaces. The RISF is expressed in minutes and indicates the time the covering will maintain a temperature below specified limits. *The NCC gives a concession for mass-panel timber construction and therefore a Modified Resistance ton the Incipient Spread of Fire (MRISF) value applies.*
Rim-board	A solid wood member around the perimeter of a wood frame, that provides restraint to the ends of the floor joists are attached. Solid rim-boards also are used to assist in transferring upper level wall loadings through the floor system.
Rise in storeys	The greatest number of storeys calculated in accordance with C1.2 of the NCC.
Seasoned timber	Seasoned timber is timber supplied at a moisture content less than 15%.
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.
Single span	The span of a member supported at or near both ends with no intermediate support.
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.

Term	Explanation
Sole-occupancy unit (SOU)	A room or other part of a building for occupation by one or joint owner, lessee, tenant, or other occupier to the exclusion of any other owner, lessee, tenant, or other occupier and includes: (a) a dwelling (b) a room or suite of rooms in a Class 3 building that includes sleeping facilities.
Spacing	The centre-to-centre distance between structural members, unless otherwise indicated.
Span	The face-to-face distance between points capable of giving dull support to structural members or assemblies.
Staggered stud wall	An acoustically separating lightweight timber framed wall that utilises top and bottom plates wider than the studs and each alternate stud is staggered to either sides of these plates. Wall linings are fixed effectively every second stud in the wall, so noise vibrations are not transferred directly through the wall, the only path is through the top and bottom plates.
Standard Fire Test	The Fire-resistance Tests of Elements of Building Construction as described in AS 1530.4.
Stick-built	A method of on-site construction using individual pieces of timber to construct wall or roof frames.
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).
Stress grade	Sawn or engineered timber that have properties contained in AS1720.1 e.g. F17, MGP10, GL18.
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.
Structural adequacy	In relation to an FRL means the ability to maintain stability and adequate loadbearing capacity as determined by AS 1530.4.
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.
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.
Tie-down	A connection to resist uplift or overturning forces.
Timber	A general term for natural or sawn wood in a form suitable for building or structural purposes.
Timber-framed: construction	A construction method utilising lightweight timber elements including wall studs and plates, roof trusses, floor trusses/joists.
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.

Term	Explanation
Type of construction	NCC defines three ‘Types of construction’ for protecting buildings from fire: Type A (most fire resisting), Type B and Type C (least fire resisting). The building class in conjunction with the building height, expressed in terms of the ‘rise in storeys’, is used to determine the Type of construction. The NCC requires Type A construction for all buildings four or more storeys.
Ultimate limit state	Limit states associated with collapse or other forms of structural failure that may endanger the safety of people.
Unseasoned timber	Timber in which the average moisture content exceeds 25%. Unseasoned timber should not be used in mid-rise construction.
Volumetric construction	Three-dimensional usually fully finished building module units manufactured in under controlled factory conditions, then transported to site and installed.
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