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Robustness in Structures

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1 Introduction

As a response to structural collapses around the world, a number of international structural design codes or regulatory standards have advocated the specific consideration of robustness or avoidance of disproportionate collapse in structural design. In Australia, the National Construction Code (NCC) (2019) and previous revisions have always had a requirement that a structure should:

"... be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage."

Figure 1.1: NCC Clause BP1.1(a)(iii).

The application of this clause has often varied between designers and rarely given much attention. Guidance can be found in Section 6 in the AS/NZS 1170.0 (2002) (Structural design actions, Part 0: General principles), although it deals with robustness in a relatively generic way. The supplementary commentary to AS/NZS 1170.0 (2002) gives some more information but the subject of structural robustness has remained an ambiguous and largely forgotten requirement.

The NCC 2019 and accompanying Structural Robustness Handbook (National Construction Code 2020) describe a number of ways that compliance with Clause BP1.1(a)(iii) (Figure 1.1) can be demonstrated. This guide gives supporting information on the subject of structural robustness and provides practical advice to building designers on how robustness can be met in timber buildings.

Structural robustness is about considering a variety of extreme events that may occur during a building's lifetime and ensuring that a building's design will guard against damage to the structure disproportionate to its cause. These events may be natural disasters (earthquakes, landslides, floods, hurricanes, etc) or man-made (vehicle impacts, terrorist attacks, explosions). Designing a robust structure is a key factor in designing a fire-safe structure, as fire is one of the most likely causes of accidental damage.

Most timber design codes around the world have been based on a domestic scale of construction where robustness is much less of a concern. As modern engineered timber allows ever-larger structures in wood, it is important to consider robustness in their design.

Considering robustness early in the design stages can result in a building with enhanced resistance to disproportionate collapse without significant impact on the structure or the time required for the design. This guide is intended to be a best practice guide for Australian timber buildings. It offers technical advice, design methodologies and details for typical building types. Examples on how robustness was considered on actual buildings are also provided.

This guide is not a substitute for a detailed risk assessment to determine any likely accidental damage events that should be considered as part of the design process. It does not cover the specific requirements of large structures, such as sports stadiums, bridges and large towers, and it is not intended to address buildings where there are specific risks of certain events, e.g. terrorist attacks at government buildings, explosions at chemical plants or fire hazards in hospitals. For these more specialist applications, the appropriate expert advice should be sought.

1.1 Why Design for Robustness?

The buildings under construction today may experience changes in usage, changes in environmental conditions, unforeseen events or material degradation over their lifetime, and there is a reasonable expectation that the structure should not be unduly damaged as a result. High-profile events like the World Trade Centre collapse have increased worldwide interest in, and attention to, the robustness of buildings and designers have a social responsibility to explicitly consider it in their work.

As buildings become taller and more pressure on designers leads to structures being designed ever more keenly, there is an increasing requirement to consider robustness more explicitly. In addition, the trend towards subcontractor design for different elements of the building can mean that overall robustness of the building is lost in a fragmented design philosophy.

Figure 1.2: Ronan Point, UK – progressive collapse of one side of building due to gas explosion.

Arguably the most famous example of a progressive collapse occurred at Ronan Point in the UK in 1968 (Figure 1.2).

An explosion due to a faulty gas stove on the 18th floor of this loadbearing precast apartment building caused the flank walls in one corner to be blown out. This caused the collapse of the floors above, which overloaded the floors below and caused the progressive collapse of an entire corner of the building. Four people died and 17 people were injured. The primary fault was that the joints between the precast walls and floors had inadequate strength for any blast load but, more importantly, they were also not sufficiently well tied together. The entire building behaved as a stack of cards when one loadbearing element was removed.

Following this event, there were a number of changes to the structural legislation in the UK. The main outcome was that buildings had to be "constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause". In addition, guidelines on the minimum requirements were introduced for tying building elements to one another and for various forces and pressures to be considered in a damage event.

Timber construction is innovating rapidly. The brittle nature of the timber material, criticality of connections and the one-way spanning form of typical timber construction present specific challenges to robustness. Safe, robust design is crucial to the development of mass timber construction using engineered wood products.

1.2 Code Requirements and where AS/NZS 1170.0, AS 1720.1 and NCC/BCA Fall Short

When structural engineers think of robustness, they typically consider Section 6 of AS/NZS 1170.0 (2002), which deals with robustness largely by dictating a minimum lateral load to be applied to the structure (1% for buildings over 15 m high or 1.5% for buildings less than this). In the vast majority of buildings, this lateral force will be significantly less than either the design wind or earthquake loads and is often neglected by the designer. The application of a minimum lateral load has as much to do with ensuring the building is able to cope with construction and out-of-plumb tolerances.

The Australian Building Codes Board (ABCB) has recognised the importance of robustness in buildings and the 2019 edition of the NCC has the following clause:

BV2 Structural robustness

Compliance with BP1.1(a)(iii) is verified for structural robustness by—

(a) assessment of the structure such that upon the notional removal in isolation of—

(i) any supporting column; or

(ii) any beam supporting one or more columns; or

(iii) any segment of a load bearing wall of length equal to the height of the wall,

the building remains stable and the resulting collapse does not extend further than the immediately adjacent *storeys*; and

(b) demonstrating that if a supporting structural component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

Figure 1.3: NCC 2019, Volume 1 – Clause BV2.

The effect of this clause is that designers need to be able to demonstrate that the structure is not overly reliant on any one particular element. However, the accompanying 2020 ABCB handbook on structural robustness (National Construction Code 2020) precises that “while the Verification Method is generally applicable to all buildings, it is expected that it will be used only for buildings of high importance, when the risk of disproportionate collapse is high or when a DTS [Deemed-to-Satisfy] Solution is not applicable (e.g. for new materials)”.

New forms of timber construction, such as mass timber buildings, are rarely covered by the DTS. Additionally, Clause 6.1 of the AS/NZS 1170.0 (2002) states that “structures shall be detailed such that all parts of the structure shall be tied together both in the horizontal and the vertical planes so that the structure can withstand an event without being damaged to an extent disproportionate to that event”. Clause 6.2.3 quantifies the tie forces by stipulating that the structure shall be interconnected and the connections shall be capable of transmitting 5% of the gravity and reduced live loads. With the elements tied together, it is commonly believed that buildings are inherently robust. Additionally, the timber design standard AS 1720.1 (2010) also falls short compared to other design codes, such AS 4100 (2020), which provides for instance minimum design actions for connections, or AS 3600 (2018), which stipulates minimum eccentricities for columns. Specific requirements for robustness in steel and concrete international standards result in inherently more robust structures (Gilbert et al. 2022).

In practice, therefore, progressive collapse would only be looked for in “high importance” buildings, as mentioned in the ABCB handbook (2020) on structural robustness, and ties may be detailed without further consideration. However, as will be developed later in this document, ensuring horizontal and vertical ties does not guarantee robustness for mass timber structures to the same extent it does for reinforced concrete and steel buildings. Mass timber buildings need to be specifically designed against progressive collapse, regardless of their importance level.

Furthermore, progressive collapse is essentially a dynamic event. Therefore, when using static analyses (as commonly practised) to assess the structural robustness by notionally removing load-bearing elements, the inertial loads may be modelled. This is commonly achieved through a Dynamic Increase Factor (DIF), which is, for all types of materials, neither explicitly considered in the accidental load combination in the AS/NZS 1170.0 (2002) nor in the NCC (2019, 2020) recommendations. Appendix A of the AS/NZS 1170.0 (2002) recommends to use special studies to consider the dynamic effects. The DIF for timber structures is further discussed later in this document.

The understanding of how to design against disproportionate collapse is well advanced in the US and Europe, and in the UK in particular, and this guide looks to their examples. It takes a number of the principles of the US Department of Defence (DoD) (UFC 4-023-03 2016), Eurocode 1 (EN 1991-1-7 2006) and the IStructE (2010)’s Practical Guide to structural robustness and disproportionate collapse in buildings, discusses them in an Australian context, gives further practical guidance on how to comply with the Section 6 of AS/NZS 1170.0 (2002) and provides simple ways to improve the robustness of buildings.

The ideas of notional removal of elements and systematic risk assessments, the specificities of timber buildings in resisting progressive collapse, as well as methods for demonstrating robustness, are discussed in more detail in the rest of this document. Clause BV2 of the NCC (2019) (Figure 1.3) shares a number of similarities with the European approach and a more detailed comparison between the two is explored later in this document. Alternative methods for demonstrating robustness will also be recommended.

1.3 Recent research

To assess timber structures for notional element removal, it is necessary to understand the post-failure behaviour of timber structures. This guide incorporates the findings of recent research carried out through the ARC-funded Future Timber Hub that examined the behaviour of post-and-beam construction with mass timber floors following various column removal scenarios (Cheng et al. 2021, Lyu et al. 2022, Lyu et al. 2021a, b, Lyu et al. 2020).

This research has highlighted that there are features of engineered timber buildings that can present specific challenges to achieving continuity, redundancy and energy dissipating capacity (ductility), and that there is a lack of relevant testing for propriety connectors. However, this research demonstrated that design methods can be applied safely to verify alternative load paths.

It is recommended that all mass timber buildings be explicitly assessed for robustness by adopting the approach described in the ABCB handbook on structural robustness (National Construction Code 2020) and drawing on the information in this guide.

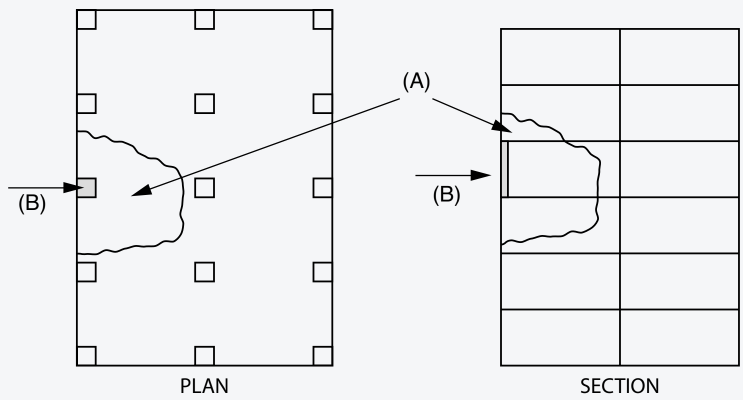
2

Robustness Concepts

2.1 Disproportionate Collapse

Following the loss of a load-bearing element, the damage must not be excessive, considering the cause. For instance, the loss of a single wall should not cause the collapse of an entire section of the building, as in the Ronan Point example. The NCC's (2019) requirement is that the building remains stable and the resulting collapse does not extend further than the immediately adjacent storeys. It does not provide an upper limit on the area of floor allowed to collapse, although BV2(b) places a limit of a single member carrying more than 25% of a total structure before a systematic risk assessment is required. Therefore, what constitutes 'disproportionate' is a matter for consideration for the individual engineer and a judgement should take into account the probability of a collapse and the consequences of any failure. For example, a lightly trafficked industrial building should be considered quite differently to an auditorium.

By comparison, the EN 1991-1-7 (2006) approach (Figure 2.1) is more prescriptive and provides a limit of 15% of the floor area or 100 m2 (whichever is smaller) and the local collapse should be limited to no more than two adjacent storeys. The US Department of Defense (DoD) (UFC 4-023-03 2016) refers to the ASCE 7 (2016) description of what constitutes 'disproportionate': "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".



Key

(A) Local damage not exceeding 15% of floor area in each of two adjacent storeys
(B) Notional column to be removed

Figure 2.1: Eurocode definition of recommended limits for admissible damage.

Typically, it is desirable to design and detail the building in such a way that the loss of any one supporting element does not lead to the likely collapse of any remaining part of the structure. Where this is impossible to achieve, the designer should assess the individual building and determine the most appropriate method to follow. While the NCC (2019) guidance could theoretically be interpreted to allow the collapse of up to 25% of the total structure, this could amount to a significant loss of structure in a large building and it may be prudent to reference the European approach and limit areas of collapse to 100 m^2. There are cases where the designer may allow the entire collapse of a non-critical building (e.g. a remote, rarely used barn) or no collapse for some structures (e.g. post-disaster structures). In all cases, a conscious decision should be made and communicated to the client and certifying authorities.

Both the NCC (2019) recommendations and the EN 1991-1-7 (2006) consider the collapse of two adjacent floors to be acceptable. Although not stated in the code or associated guidance, there is the implication that the first intact floor below this level should be capable of supporting debris from the collapsed floor(s) above.

This can be done either by designing each floor to support the combined dead load of two floors and a reduced live load based on accidental load and material factors or designing a 'strong floor' that is capable of supporting the weight of multiple collapsed floors. It will often prove simpler and more efficient to design and detail the structure so that the collapse of two adjacent floors is prevented.

2.2 Redundancy

One of the simplest ways to improve the robustness of a structure is to design it to be statically indeterminate. Beams running continuously over/past supports is a simple way to ensure that the loss of one particular load-bearing element may not cause the collapse of the entire floor structure. Due to the nature of mass timber buildings (i.e. brittle material in bending and tension, lack of continuity of the structural elements associated with the prefabrication process and the lack of ductility in some of the connections), upon the loss of a load-bearing element, alternative load paths should be clearly identified in the design phase to ensure that the accidental loads are solely resisted by one or more of these load paths.

2.3 Insensitivity to Construction Tolerances

A large part of designing a robust structure is ensuring that it is not overly sensitive to construction tolerances, thermal movements, support settlements, etc. The minimum notional horizontal loads defined in AS/NZS 1170.0 (2002) will cover this in some ways but it is wise to consider the effects of tolerance in the detailing. Consider the steel connection in Figure 2.2.

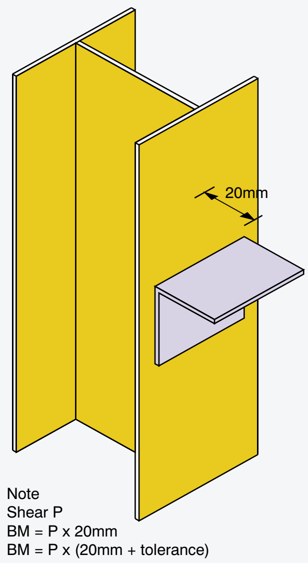


Figure 2.2: Bracket with high sensitivity to construction accuracy (adapted from IStructE (2010)).

A small cantilever support bracket is designed for a column. A credible construction tolerance of 20 mm will double the moment induced on the cantilever and this might be considered in the design. Clearly, if the cantilever was 1 m long then this tolerance is insignificant. Consider the precast beam detail in Figure 2.3.

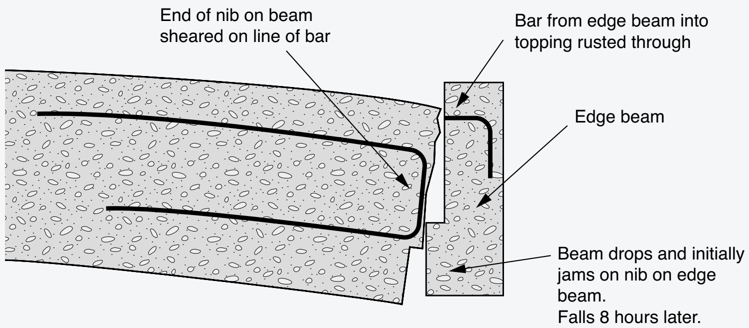


Figure 2.3: Precast connection with high sensitivity to building movement (adapted from IStructE (2010)).

In this case, the precast beam is seated on a narrow corbel on the edge beam and connected with a bar at the top through an in situ joint. This detail is from a real building in the UK – the top bar became corroded through a weakness in the in situ joint and the beam became reliant on the bearing detail. The beam underwent some torsional and lateral movement and the precast slab collapsed.

Buildings should be able to accommodate these sorts of movements and credible material degradation without suffering disproportionate collapse.

2.4 Accidental Damage Load Case

Designing a structure for robustness involves calculating loading and member capacity for certain situations. In these accidental damage cases, it is considered acceptable to use reduced partial load factors and reduced material strength reduction factors appropriate for each material and load as defined in the Australian Standards. In addition, the building's serviceability checks are no longer of concern and only the strength needs be checked to ensure the structure remains intact.

Given that most structures are designed based on serviceability requirements, significant reserves of strength available in the structure before collapse are likely. The robust design of a structure is not expected to add significant – if any – costs to the structure; it is an exercise in good detailing and the engineer's clear understanding of robustness concepts.

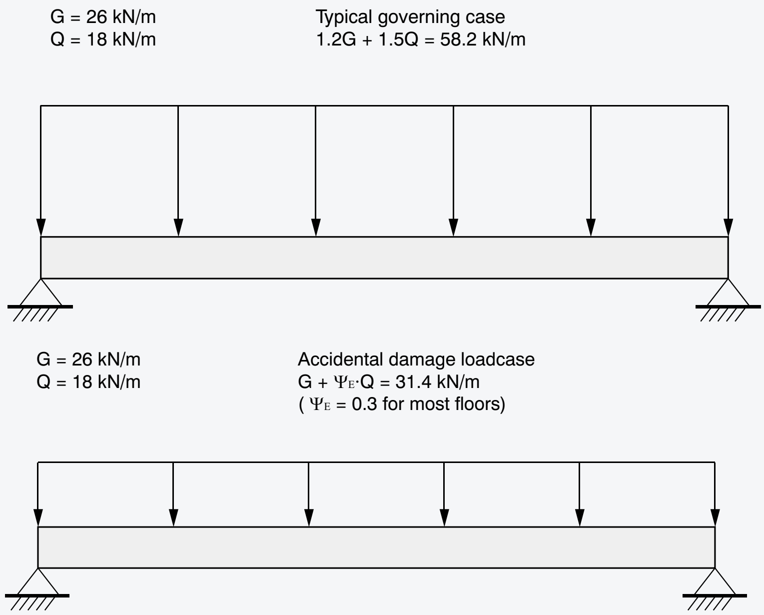


Figure 2.4: Comparison of ultimate vs accidental loads for a typical beam in the AS/NZS 1170.0 (2002).

Looking at timber design as an example, in addition to the reduced loads the AS 1720.1 (2010) and the Eurocode 5 (EN 1995-1-1 2004) allow one to assume either a higher design strength of material, a reduced load duration or both in the accidental damage situation. In AS 1720.1 (2010), the k1 factor goes from 0.8 for a medium-term loading to 1.0 for a short-term load, effectively gaining 20% capacity. The Eurocode 5 allows significantly more for the material strengths in an accidental design verification, with a typical nominal material safety factor increasing by up to 70%. When all these factors are considered, and with deflections no longer being a consideration, the reduction in design loads in these cases is very significant, which helps allow some very long span solutions to become viable.

This simplified approach does not fully consider dynamic effects that may erode some of these benefits. Increasing the accidental load combination on the bays adjacent to the removed element by a DIF of 2.0, as suggested by the DoD (UFC 4-023-03 2016) for timber structures, may offset the reduction in design loads. Such inertial loads need to be properly considered in design as later developed in this document.

3 How to Design for Robustness

3.1 NCC Compliance

As noted in the introduction, mass timber construction is unlikely to fall within DTS requirements, and compliance with BP1.1(a)(iii) must be demonstrated as a Performance Solution. The design may be verified by Verification Method BV2, specifically by considering notional removal, in isolation, of listed structural elements or by systematic risk assessment. The ABCB Handbook explains, "the Verification Method is one way, but not the only way, to demonstrate compliance with the NCC Performance Requirements". Part A2 of NCC Volume One and Two detail all possible Assessment Methods available to develop a compliance solution.

It is recommended that the certifying engineer adopts the following risk-based strategy to consider the appropriate methodology for compliance. This strategy draws on international best practice. The UK approach is specifically mentioned in the ABCB Handbook.

3.2 Risk-based Strategy

3.2.1 Building Classification

The first task is to consider the use of the building. Typically, buildings are categorised based on their size and use. Structural engineers in Australia will be most familiar with the importance levels set out in Section 3.3 of AS/NZS 1170.0 (2002). The Eurocode is based on the old UK Building Regulations designations of Classes 1, 2A, 2B and 3 in increasing consequence of failure. The DoD uses five risk categories of buildings and other structures. The three sets of classifications (Table 3.1) show a good degree of correlation so it is proposed to use the importance levels from AS/NZS 1170.0 (2002). More details on which building types fall into these categories can be found in AS/NZS 1170.0 (2002).

Consequences of failure	Description	Importance level	Comment	Eurocode Classification	DoD Classification
Low	**Low** consequence for loss of human life, or **small or moderate** economic, social or environmental consequences	1	Minor structures (failure not likely to endanger human life)	1	I
Ordinary	**Medium** consequence for loss of human life, or **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels	2A	II
High	**High** consequence for loss of human life, or **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)	2B	III
		4	Post-disaster structures (post disaster functions or dangerous activities)	3	IV
Exceptional	Circumstances where reliability must be set on a case by case basis	5	Exceptional structures	3	V

Table 3.1: Building classifications.

These classifications reflect the consequences of failure and it is fair to take a different view of the robustness requirements for an isolated industrial shed compared to that for an office building. In larger buildings, it may be sensible to consider different uses in different areas and approach robustness differently in each area (e.g. a large retail store with a restaurant may have different requirements to the attached warehouse).

Where a low classification building is immediately next to a high classification building, it may be necessary to design the low classification building to a higher standard of robustness if there is any risk of damage to the other building. This is recommended by the Eurocode where the adjacent building is within 1.5 times the height of the lower classification building.

3.2.2 Assess Extreme Event Risks

There are two basic strategies for designing a building against accidental events: to identify and design to resist specific accidental actions or to ensure the extent of localised failure will be limited. A simplified flowchart to assess the robustness of structures is given in Figure 3.1.

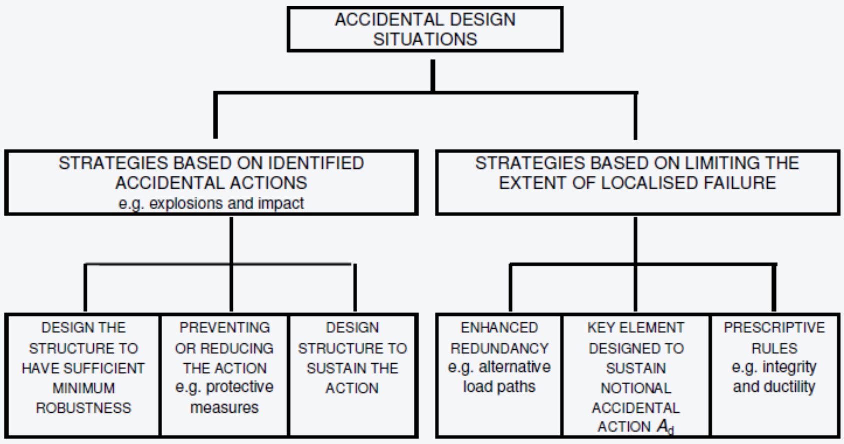


Figure 3.1: Decision flow chart for robustness design (Source: EN 1991-1-7 (2006) and IStructE (2010)).

Buildings where there are more significant risks of certain accidental actions should be subject to a systematic risk assessment and look to either prevent or reduce the action. An example would be an office building built on a deck over a road. The potential risk would be a vehicle damaging a column supporting the deck putting the entire structure at risk of collapse. An appropriate response could be to move the columns away from the road edge and to place a physical barrier between the road and column to reduce the possibility of an impact. If these specific events can be reduced in likelihood then the building could be designed to comply with the relevant requirements for its class.

Particular extreme events may include (but not be limited to):

- gas/chemical explosions
- terrorist attack
- vehicular impact – Figure 3.2
- natural disaster (flood, landslide, earthquake) – Figure 3.3
- deliberate or accidental removal of a load-bearing structural element
- fire.

Figure 3.2: Designing for disproportionate collapse is about anticipating the unexpected.

Figure 3.3: Disproportionate collapse of an apartment block in Venezuela following landslides.

The relationship between fire and disproportionate collapse is important; in a lot of cases it is the most likely cause of the damage of the loadbearing element. The design of a robust structure will typically also improve the performance of the building in fire.

3.2.3 Select an Appropriate Design Response

As the importance level of the building increases, the designer's response to the robustness of the structure should also increase. While the NCC (2019) does not explicitly treat different classes of buildings differently in its design approach, distinctions are made in the Eurocodes (EN 1991-1-7 2006), IStructE (2010) and DoD (UFC 4-023-03 2016). These references require typical details to achieve a basic level of robustness, and buildings of greater importance are subject to specific assessment. The resulting design methods can be classified as indirect and direct.

An indirect design method applies a set of requirements to tie elements together, providing "minimum levels of strength, continuity and ductility" as specified in the ASCE/SEI 7-16 (2016), rather than modelling the consequences of removing a load-bearing element. This is typically suitable where consequences of failure are low and accidental actions are not severe. For mass timber buildings, as detailed in Section 3.4.1, ties may not be sufficient to guarantee robustness and should not be the only measure to resist progressive collapse.

By contrast, a direct method considers the resistance to progressive collapse either by modelling the consequences of losing a load-bearing element or by modelling the building, or part of it, to resist the threat. The 'alternative load path' (ALP) method involves designing the building to span over the missing element and designing 'key elements' to ensure that load-bearing elements can resist the accidental loads. A detailed review of these approaches can be found in the review study by Arup (2011) or in Adam et al. (2018).

For the ALP, static and dynamic analyses can be run, with some guidance provided in the DoD (UFC 4-023-03 2016). Static analyses would be commonly adopted in practice and likely considered as one of the most practical and best-suited procedures to assess the structural robustness. Either linear static or non-linear material and geometric analyses can be run.

The flowchart in Figure 3.4 details the IStructE (2010) design response to each different Importance Level of buildings. Table 3.2 compares the design responses in both the IStructE (2010) and DoD (UFC 4-023-03 2016).

Classification			Design approach	
AS	**EU**	**DoD**	**IStructE (2010)**	**(UFC 4-023-03 2016)**
1	**1**	**I**	No special requirements	No special requirements
2	**2A**	**II**	Provide horizontal ties and anchorage to walls	Provide horizontal and vertical ties and design the corner and penultimate load-bearing elements as key elements OR Design for Alternate Load Path
3	**2B**	**III**	Provide horizontal and vertical ties OR Design for Alternate Load Path OR Design for key elements	Design for Alternate Load Path and design all perimeter first storey load-bearing elements as key elements
4	**3**	**IV**	Undertake systematic risk assessment	Provide horizontal and vertical ties and design for Alternate Load Path and design all perimeter first storey load-bearing elements as key elements
5	**3**	**V**		

Table 3.2: Design approaches proposed in IStructE (2010) and DoD (UFC 4-023-03 2016).

This recommended approach may be summarised as follows:

Importance Level 1

For an Importance Level 1 building, it may not be necessary to specifically consider robustness, provided there is no abnormal risk of an extreme event.

Importance Level 2

For Importance Level 2, the building can be designed:

- with a minimum set of horizontal ties as described in Section 3.4
- for notional element removal as described in Section 3.5
- for key or protected elements as described in Section 3.6.

Importance Level 3

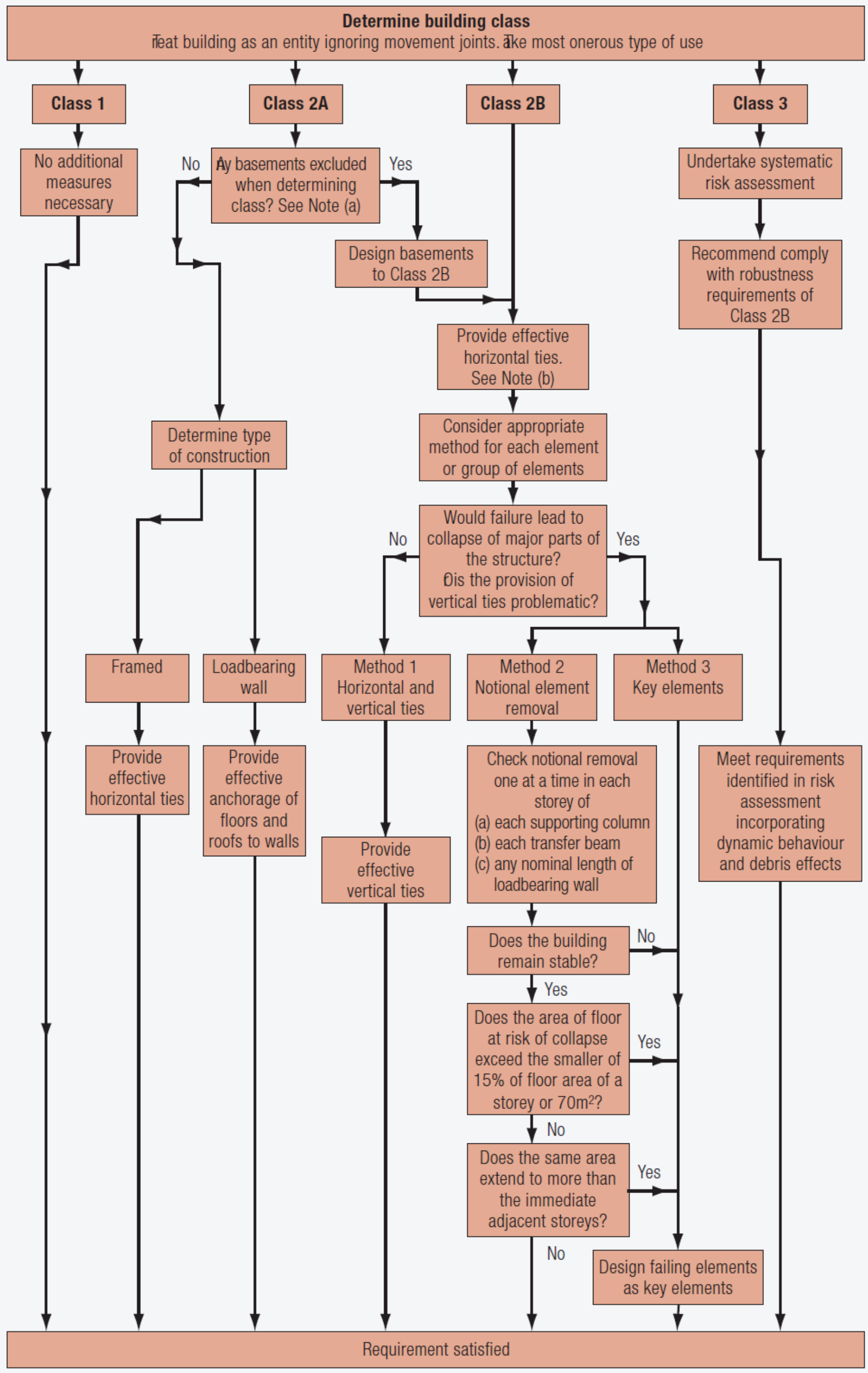
For Importance Level 3, the building can be designed:

- with a minimum set of horizontal ties as described in Section 3.4.3 and vertical ties as described in Section 3.4.4
- for notional element removal as described in Section 3.5
- for key or protected elements as described in Section 3.6.

Importance Levels 4 and 5

Buildings in Importance Levels 4 and 5 are outside the scope of this document and a detailed risk assessment and consideration of all potential collapse events is recommended. As a minimum, they should be treated as an Importance Level 3 building and consider other specific risks.

Figure 3.4 details the design response to each different Importance Level in the IStructE (2010). Note that Importance Levels are referred to as Classes in the IStructE.



Notes

a Rules on the exclusion of basements vary in the building regulations of the various regions. When designing to EN1991-1-7[5.2] basements are included in the storey count. However in England and Wales and Scotland and Northern Ireland, if they satisfy the Class 2B requirement, they may be excluded from the count.

b In Class 2B, for horizontal ties, rather than using physical ties in some cases alternative methods may be demonstrated by test.

Figure 3.4: Robustness design flowchart (Source: IStructE (2010)).

3.3 Choice of Structural Form

The initial design decisions made for the structural form can have a significant effect on the robustness of a building. Designers of multi-storey timber buildings adopt a range of approaches to achieve robust structures. These have typically focused on designing structures that provide alternative load paths –through either primary frames or secondary elements such as floors. Approaches include:

- Double-span or multi-span floors so that panels, joists or cassettes are supported in at least three locations, such that loss of one support does not cause collapse.
- Multi-span beams in post-and-beam structures, for the same reason, usually configured as two parallel beams passing either side of columns. This can also be configured as a continuous beam with splices designed for shear and tension only that also offer continuity in event that a support is lost.
- Use of perimeter 'rim' beams, intersecting load-bearing walls and posts in multi-storey stud-framed structures, drawing on practice developed by the UK timber frame industry (UKTFA Special Project 2008).
- Introduction of discrete steel ties or drag bars fastened to timber frame, which may also be part of the design of floor diaphragms.
- Adoption of post-tensioning in beam design, as developed by University of Canterbury, NZ, which provides a structure capable of post-failure catenary action (Buchanan et al. 2011).
- Adoption of rocking walls, also developed by University of Canterbury, NZ, that allows connections to dissipate energy without ultimate failure.
- Use of continuity in sheeting or tertiary elements with appropriate nailing.

The following section describes structural principles and analytical approaches that may be applied. Section 4 provides specific guidance for timber structures.

3.3.1 Redundancy

Reliance on a single column to hold up a significant proportion of a building creates a potentially unnecessary dependence on a single element. Any accidental damage involving the column puts a disproportionate area of the building at risk of collapse.

Consider the example in Figure 3.5. The central column potentially holds up the entire roof structure and heavy vehicles are constantly moving around it. Potential options to improve the robustness could be:

- place barriers around the column to reduce the chance of vehicle impact
- change the roof structure to be able to span across the building if the column is damaged
- change the roof structure from a radial structure to one spanning across the building
- design the column as a protected element to resist vehicle impact

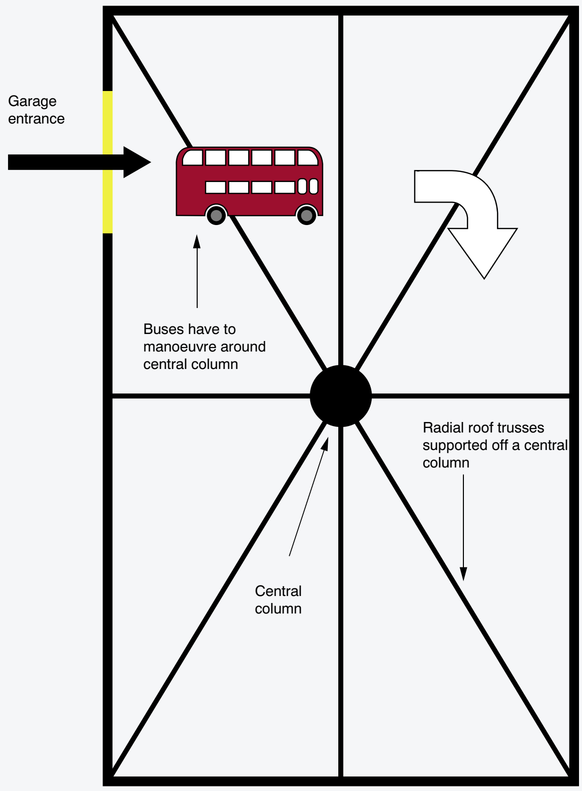


Figure 3.5: Example of a structure with a lack of redundancy (Source: IStructE (2010)).

3.3.2 Transfer Beams

Transfer beams can create a heavy reliance on a particular structural element. The Murrah Federal Building in Oklahoma was the subject of a terrorist bombing in 1995 (Figure 3.6). At the third storey there was a large transfer beam supporting the floors above where the column spacing halved. The bomb destroyed one of the lower columns and caused the transfer beam span to suddenly increase. The lack of sufficient ties caused the failure of the transfer beams and along with it all adjacent bays on the floors above.

Figure 3.6: Oklahoma City's Murrah Building after the bomb attack.

Clause 6.2.2 of the AS/NZS 1170.0 (2002) requires that all structures are designed for minimum lateral loads equivalent to:

- 1% ($G+\psi_c Q$) for structures over 15m tall
- 1.5% ($G+\psi_c Q$) for all other structures

The intent of this load is to ensure that buildings that may otherwise not attract significant wind load would have some measure of resistance against unforeseen horizontal loads. In the vast majority of cases, this load is likely to be significantly less than the applied wind or seismic loads and is often forgotten by most designers.

The Eurocode maintains the application of notional horizontal loads as a requirement but its rationale is based on it accounting for lack of tolerance in the building construction, particularly the out-of-plumbness of columns. Figure 3.7 shows that the lateral load generated by a column with an out-of-plumb tolerance of 1/200 is equivalent to a lateral load of 0.5% of the vertical load in the column.

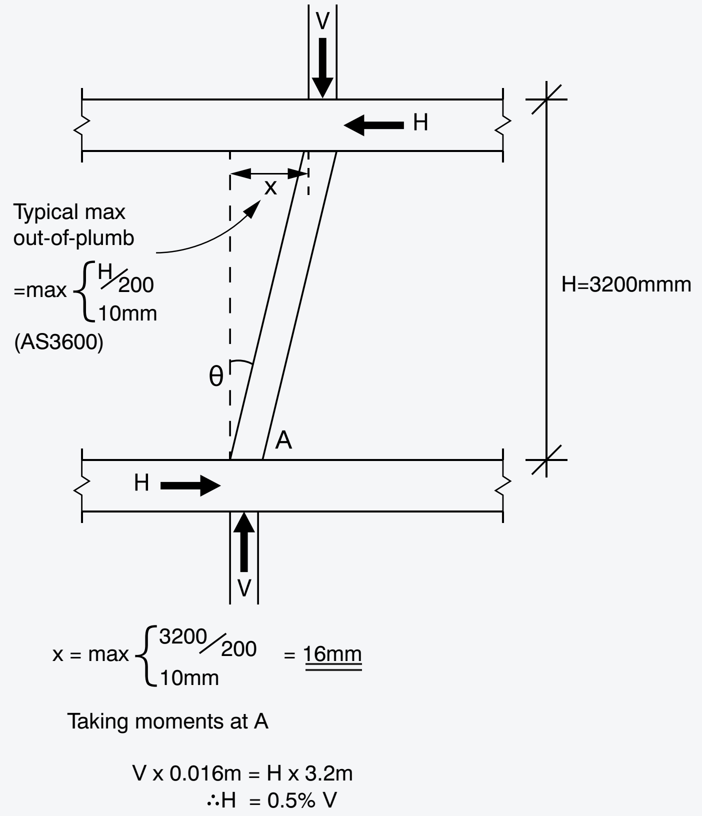


Figure 3.7: Out of plumb column leading to net lateral load.

For this reason, the application of notional horizontal loads is recommended for all buildings. Complying with this alone does not guarantee a robust structure, but does mean the structure is capable of withstanding some construction tolerance.

3.3.3 Compartmentation

Compartmentation is an approach that can increase robustness by combining different robustness design methods at different scale levels. The principle is to isolate progressive collapse to compartments, therefore avoiding the spread of the damage throughout the entire building.
For instance, in the building shown in Figure 3.8 and discussed in Voulpiotis et al. (2021), five compartments are created with the main elements of these compartments designed as key elements. The structure inside each compartment is isolated from the other compartments and designed for robustness under a load-bearing element removal scenario. The whole building is equivalent to a system of robust compartments. This sort of approach may be useful when designing for fire-safety, as for Atlassian's Headquarters in Sydney, Australia.

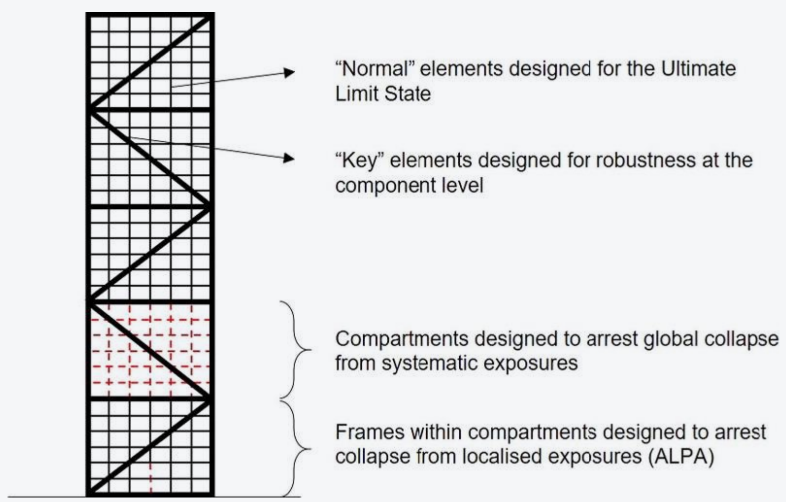


Figure 3.8: Example of a building with robust compartments (Source: Voulpiotis et al. (2021)).

For long-span timber elements, this philosophy can also be used when designing the secondary structure that links them – for example, purlins on long-span roof structures – to ensure they and/or their connections are sufficiently weak and prevent the transfer of accidental loads from one element to another. This prevents progressive collapse in case of systematic design or construction errors. The collapse of the Ballerup Siemens-arena in 2003, due to incorrect connection design, was limited to two glulam trusses out of 12 as the purlins did not transfer the load from one truss to the next (Munch-Andersen and Dietsch 2011). In such cases, the members would have to be designed as ‘key elements’ (see Section 3.6).

3.4 Detailing and Tying

The concept of providing ties within a structure is not specifically mandated within the NCC but mentioned in Clause 6.2.3 of the AS/NZS 1170.0 (2002). Ties are used within the Eurocodes as a practical and simple way to improve building robustness. Ties can be presented as a potential alternative way to demonstrate compliance with the NCC, and are particularly useful in both steel and concrete buildings.

3.4.1 Why We Include Ties

The purpose of tying elements of a structure together is twofold: to constrain the elements during an event and to create a statically indeterminate structure that is capable of resisting the loss of a load-bearing element through catenary action.

Ties can be classified as horizontal (peripherical and internal) and vertical. The Eurocode considers these independently, each serving a different purpose in the event of accidental damage. The DoD requires that both horizontal and vertical ties be incorporated into the design. Vertical ties are intended to share load over a number of levels if a load-bearing element is removed at a lower level.

An implicit requirement for horizontal ties is that after the loss of a load-bearing element, large deformation will take place, enabling the horizontal structural elements to be in tension and resist the vertical load through catenary action. In reference to Figure 3.9, structural integrity is provided if:

$$\text{Gravity Load} \leq 2 \cdot \times \text{ Tie strength } \times \cdot \sin\theta$$

Enough rotation must take place at the connections for the above equation to be satisfied with the tie strength requirements provided in standards. A minimum typical rotation of 0.2 rad is required, as stipulated in the DoD (UFC 4-023-03 2016). Therefore, ties require connections to be ductile enough to allow rotation and for catenary action to fully develop.

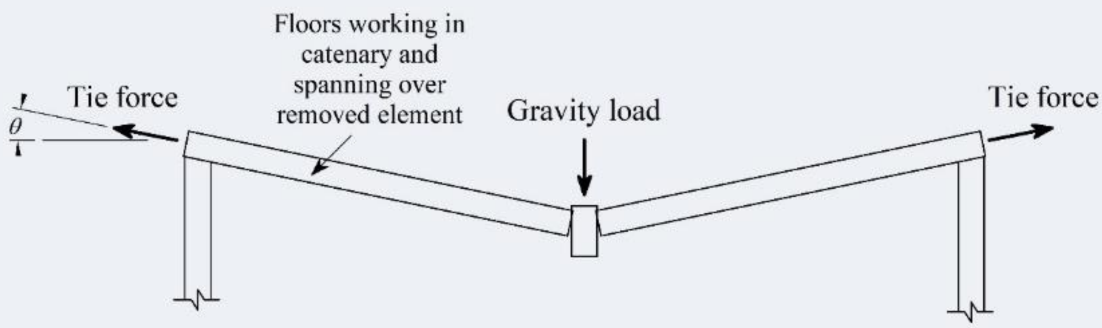


Figure 3.9: Resisting mechanisms of horizontal ties (catenary action).

Figure 3.10 shows that in a building with ties, alternate load paths can be found after removal of a column. Ties distribute loads elsewhere in the structure and avoids disproportionate collapse.

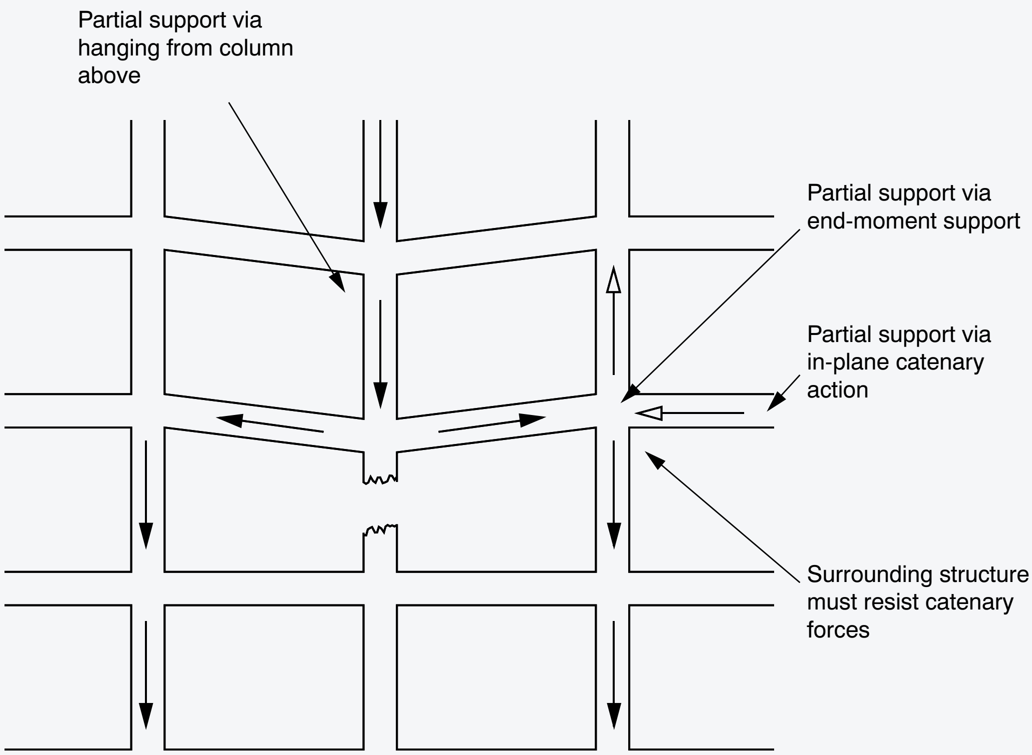


Figure 3.10: Alternate load paths available in a building designed with vertical and horizontal ties (Source: IStructE (2010)).

3.4.2 Limitations of Ties in Mass Timber Buildings

While rotation of connections and supports can be achieved in the more ductile reinforced concrete and steel buildings, this is more problematic for more brittle mass timber connections. Indeed, commercially available connectors for timber structures are rarely designed with robustness in mind and the rotational ductility is generally not provided or verified by the manufacturers.

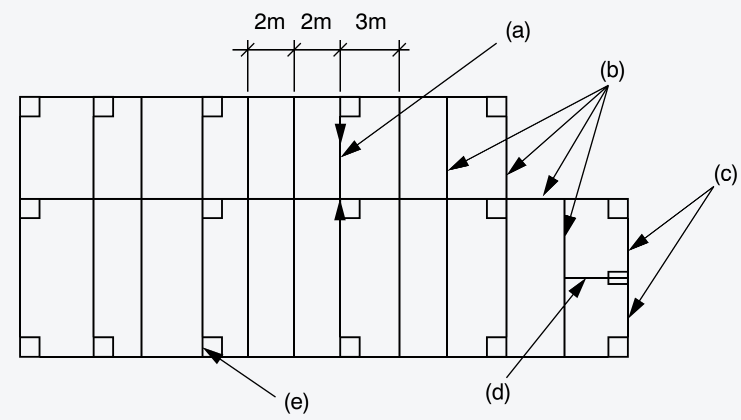
Research has shown that failure may occur before sufficient rotation takes place to allow catenary action to fully develop (Lyu et al. 2020, Masaeli et al. 2020). If connections are not especially designed to be ductile under a column removal scenario, extreme caution is needed for mass timber buildings when relying solely on beam connections to provide horizontal and vertical ties.

Post-tensioned frames developed in New Zealand would likely achieve catenary action and the PT tendons would also provide excellent tying action. The robustness of post-tensioned timber frames is outside the scope of this guide but further information on this type of construction is available through the Expan website.

Additionally, the inclusion of ties within a structure is good practice and will help safeguard the structure against unknown hazards. However, designers of mass timber buildings may take a more thorough approach and look at the removal of notional elements, as discussed in Section 3.5. This is appropriate for post-and-beam buildings.

3.4.3 Horizontal Ties

Peripherical and internal horizontal ties should be continuous across the building and around the perimeter, and should be in a straight line where possible. Where ties are required to be cranked, their tendency to straighten should be considered and appropriate restraints provided. In a typical regular-framed structure, these ties will be largely catered for by the main floor beams and the only remaining consideration is to ensure that the connections have sufficient capacity to transfer the tie loads. Figure 3.11 shows an example of typical tie layout.



Key
(a) 6m span beam as internal tie
(b) All beams designed to act as ties
(c) Perimeter ties
(d) Tie anchored to a column
(e) Edge column

Figure 3.11: Typical tie layout for a framed building (Source: Way (2011)).

3.4.4 Vertical Ties

Vertical ties should be continuous from the foundations to the roof level. This is usually easily achieved in framed structures, provided the tie forces are transferred through any column splices.

The additional load from any collapsed floors can be spread through other floors and reduce the likelihood of collapse of the local area. The other floors deflect significantly but the structure itself may remain intact.

3.4.5 Tie Design Forces

As mentioned earlier, tie design forces in Australia are given in Clause 6.2.3 of the AS/NZS 1170.0 (2002), which stipulates that the connections shall be capable of transmitting 5% of the gravity and reduced live loads. No distinction is made between vertical and horizontal ties.

In the DoD (UFC 4-023-03 2016), tie design forces vary with the structural form, such as framed or flat plate buildings, one- or two-way slabs.

The tie design forces prescribed by the Eurocode (EN 1991-1-7 2006) are more prescriptive than the AS/NZS 1170.0 (2002). While they are based on the theories outlined above, the actual forces generated in an event are unknown. The Eurocode splits them into the following:

Horizontal ties

Tie forces recommended by the Eurocode are:

Internal Ties- $T_i = 0.8(G + \psi_E Q)sL$ *or 75kN (whichever is greater)*

Perimeter Ties- $T_p = 0.4(G + \psi_E Q)sL$ *or 75kN (whichever is greater)*

Where:

G = Characteristic dead load

Q = Characteristic live load

ψ_E = Combination factor for accidental actions

s = Tie spacing

L = Tie length

These equations relate the tie force to the area of floor supported and effectively require the tie force transferred by the connection to be a minimum of around 80% of the factored reaction at the end of the beam.

Vertical ties

Vertical ties for a framed building should be sized so that the columns and their splices are capable of resisting an accidental design tensile load equal to the largest design vertical permanent and variable load reaction applied to that column from one storey. This load need not be applied at the same time as any other permanent and variable actions.

For load-bearing wall construction, the Eurocode recommended design tie force T to be:

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \text{ N or 100 kN/m (whichever is greater)}$$

where:

A is the cross-sectional area in mm² of the wall measured on plan, excluding any non-load-bearing leaf

H is the wall height

T is the wall thickness

Ties should be at maximum 5 m centres along the wall and occur at no more than 2.5 m from an unrestrained end of the wall. This is fairly onerous for a lot of construction types, including load-bearing precast buildings.

An alternative would be to treat vertical ties in a load-bearing wall construction in the same way as in a framed building (Figure 3.12) and to design the walls and connections to resist a minimum of the design vertical permanent and variable action applied to that wall at any one level. This would allow the walls below to hang the floors in an accidental damage situation and increase the number of alternative load paths.

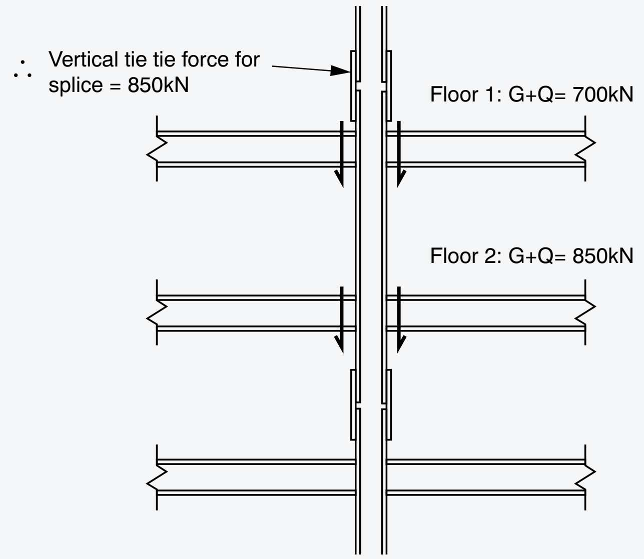


Figure 3.12: Proposed vertical tie force for a framed building.

While all these forces should be relatively achievable in the case of steel or concrete buildings, they are more difficult to achieve in typical masonry or timber buildings due to the connection capacity limitations inherent in these materials, and details should be considered carefully.

3.4.6 Tie Design Details

The tie details will differ greatly between materials. By their very nature, reinforced concrete structures are relatively robust and well tied through normal detailing practices and robustness will not be a key design consideration. By contrast, a precast concrete building connected together with weld plates or dowels is naturally significantly less robust than its in situ equivalent and the localised connections make it more difficult to demonstrate robustness. A notional element removal method may be more appropriate.

Deemed-to-satisfy tie details for mass timber buildings have not yet been developed. The designer should not rely solely on the axial capacity of beam connections without detailed consideration, as these are not always able to withstand large deformations and test data may not be available. Therefore, in mass timber buildings, alternative design methods may be required. Examples of connections designed with robustness in mind are shown in more detail in the following sections of this guide.

3.5 Notional Element Removal

3.5.1 Procedure

As an alternative to providing ties a designer can look at the notional removal of each load-bearing element, one at a time. The word notional emphasizes the fact that it is an imaginary scenario.

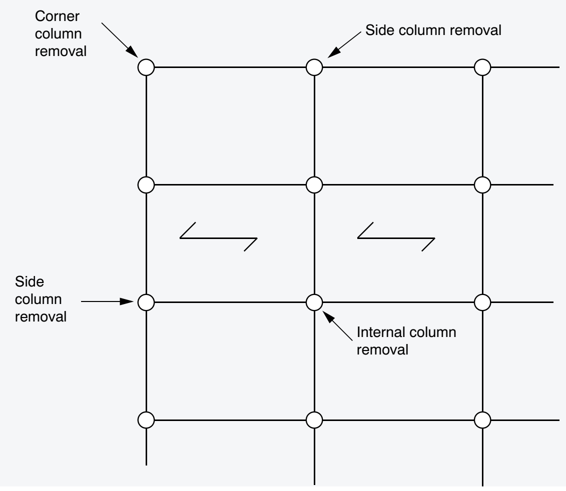


Figure 3.13: Notional load-bearing element removal – framed structures (Source: IStructE (2010)).

In the case in Figures 3.13 and 3.14, each individual column, length of load-bearing wall or transfer beam is imagined to be removed and the design methodology focuses on examining the consequence of losing the load-bearing element rather than its cause. The building should remain stable and the connected floors should remain intact, with the exception of any localised damage being permitted as discussed in Section 2.1. The behaviour of mass timber buildings under a load-bearing element removal scenario has been studied by researchers and is described further in Section 4.

The length of the load-bearing wall to be notionally removed varies between guidelines and is equal to the storey height H in the NCC (2019), 2H in the DoD (UFC 4-023-03 2016) and 2.25H in the IStructE (2010), as defined for the latter in Figure 3.14.

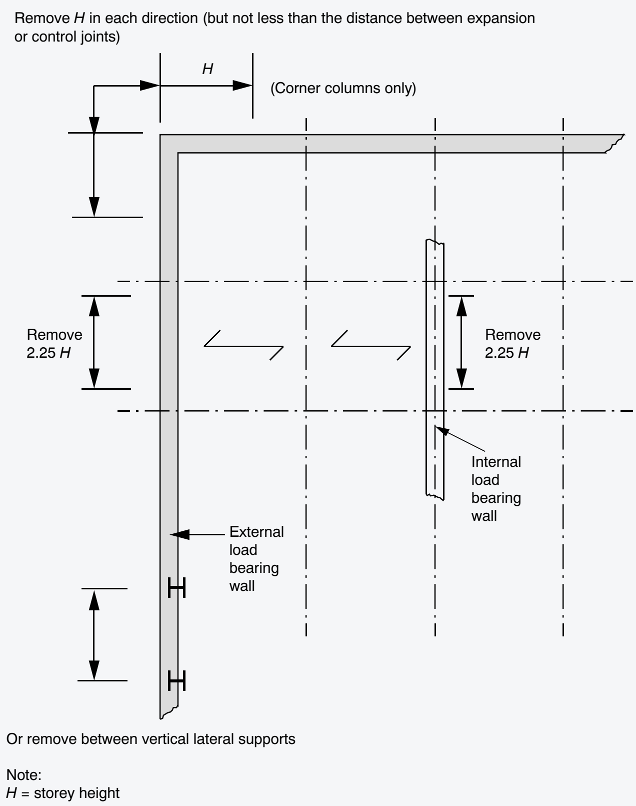


Figure 3.14: Notional element removal – load-bearing wall structures (Source: IStructE (2010)).

When a load-bearing element is removed, several resisting mechanisms could develop, providing alternative load paths. Firstly, it is conceivable that if the beams/floor over the removed element are sufficiently well tied together, the floor may hang in catenary from the adjacent structure and remain intact, albeit with significant deformations in the floor structure (see Figure 3.15). This is challenging to achieve for mass timber buildings and would depend on a number of design considerations:

- The beam-beam connection through the column remaining intact and having sufficient tie strength to support the catenary loads.
- The surrounding structure being able to support the horizontal tie forces generated.
- The connections have sufficient ductility to allow rotation at the supports to allow the beams to work in catenary action.

While the accurate calculation of the catenary action is complex in reality and dependent on a number of variables, the provision of the ties is intended to provide this alternative load path.

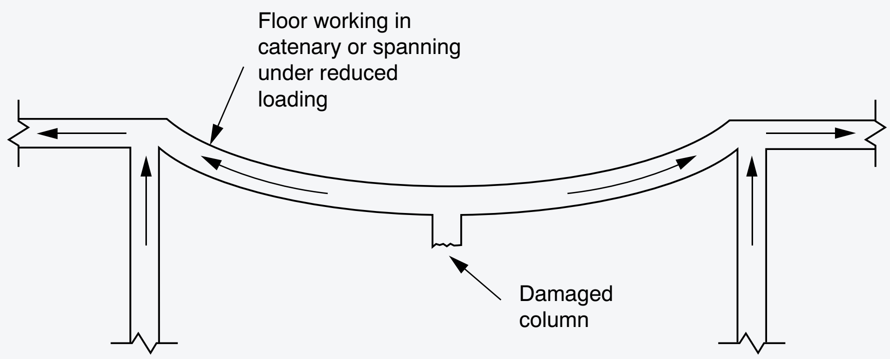


Figure 3.15: Floor working in catenary over removed column.

Alternatively, depending on the stiffness and loading of the floors relative to each other, the load can be shared between floors, as shown in Figure 3.16. Frames may act as Vierendeels (Figure 3.17); or walls may act as deep beams (Figure 3.18).

These alternative load paths may not be readily mobilised in mass timber buildings, and the designer must pay special attention. Removal of certain elements may also be problematic for certain situations where there is irregular geometry, re-entrant corners or large transfer structures. In these cases, the layout of the building may need to be changed or the element designed as a protected element (see Section 3.6).

Instead, the design of horizontal elements to span more than one bay is a simple and efficient solution to provide continuity through a mass timber building and offer an alternate load path in the event of localised failure. Figure 3.19 shows CLT floors spanning two bays and under a removed load-bearing element scenario, either spanning over the missing element or acting as a simply supported floors with overhangs.

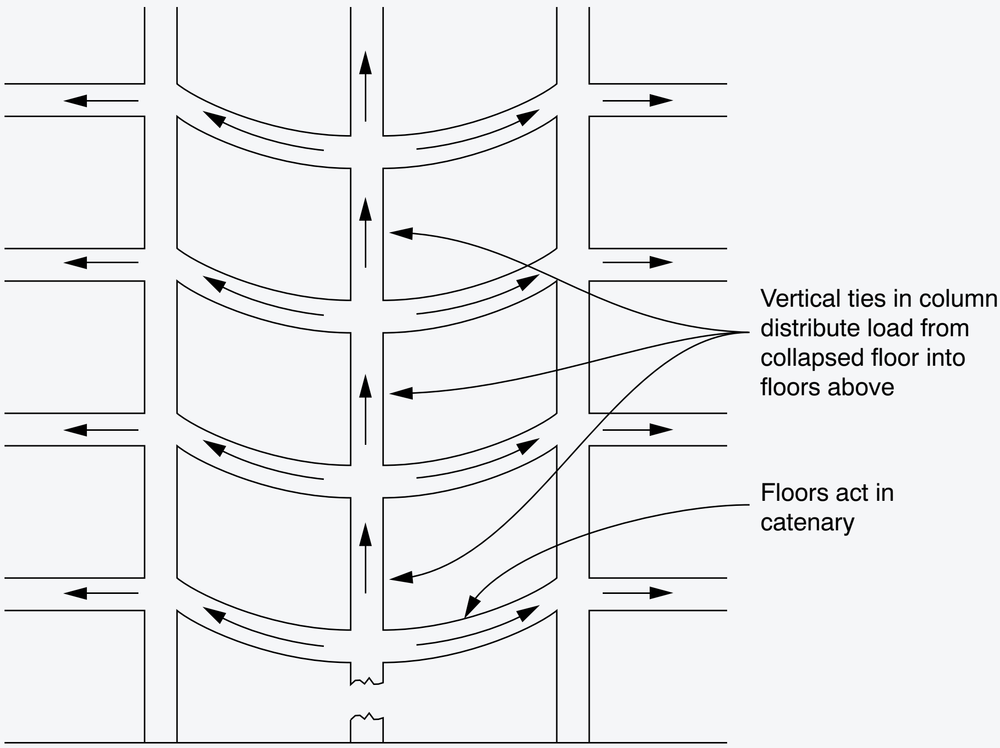


Figure 3.16: Sharing the load between floors over removed columns.

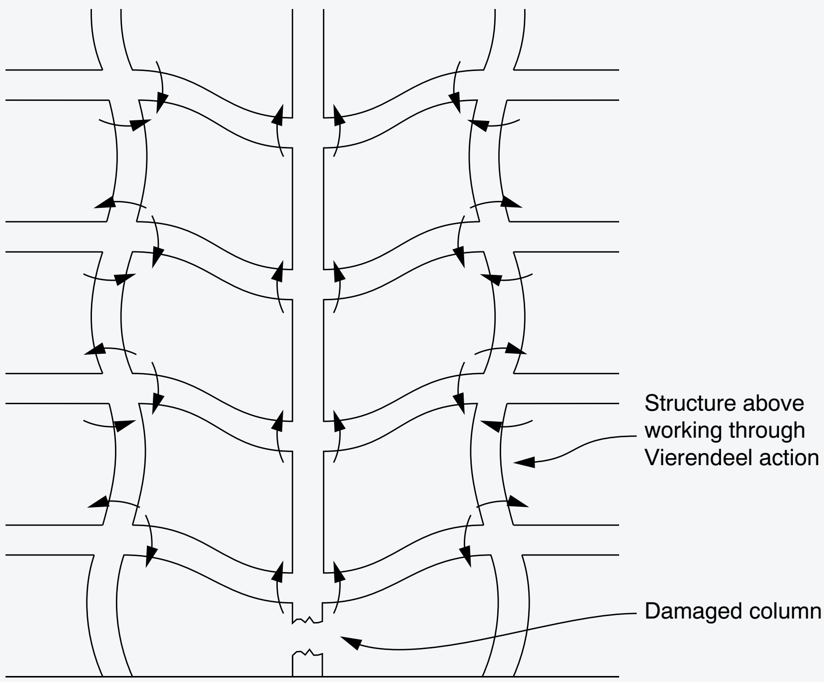


Figure 3.17: Frames acting as Vierendeels over removed elements.

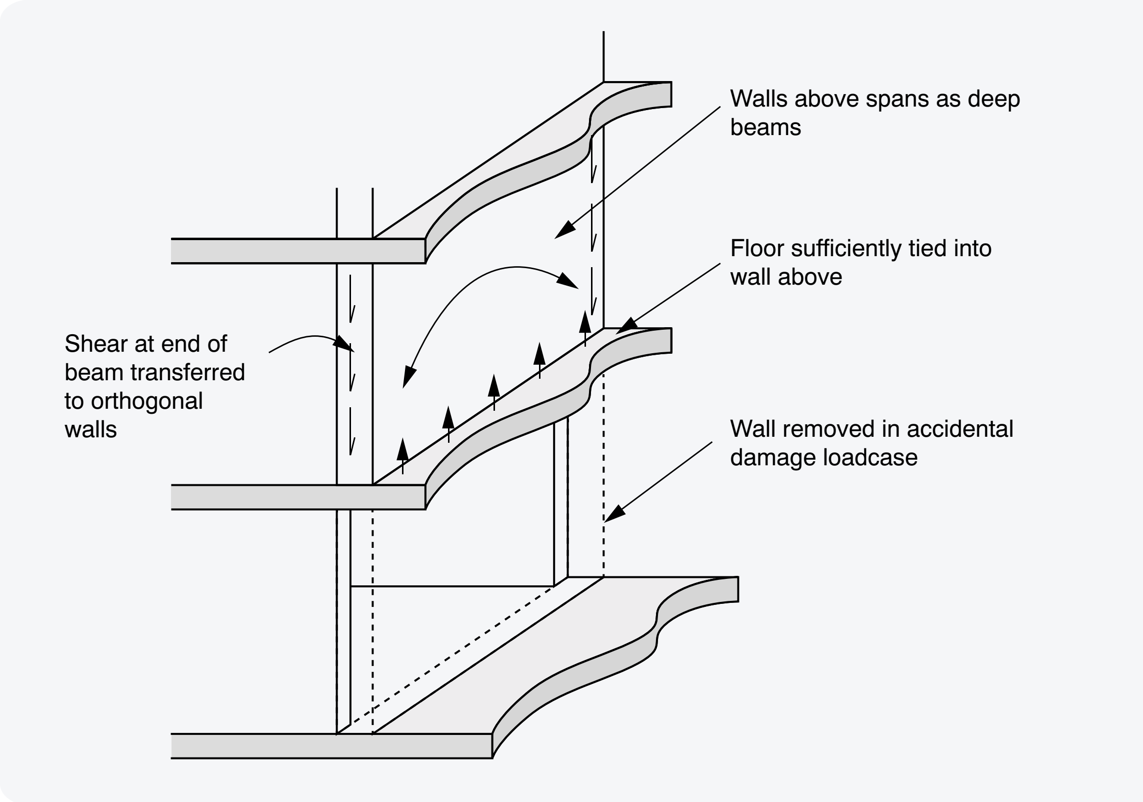


Figure 3.18: Walls acting as deep beams to span over removed elements.

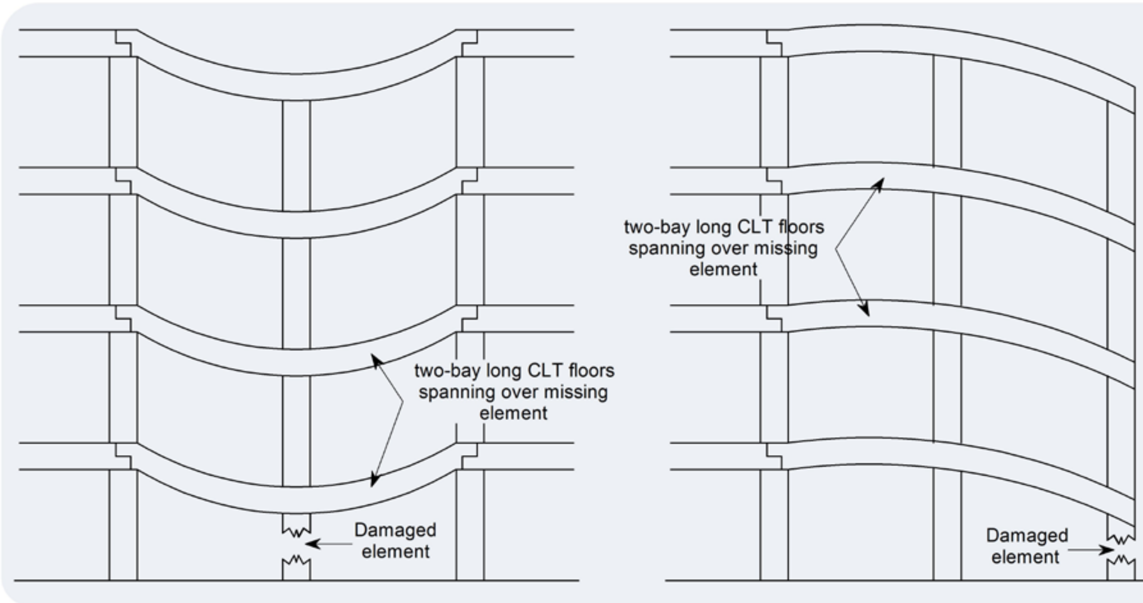


Figure 3.19: Two-bay long CLT floors creating an alternative load path for mass timber buildings.

3.5.2 Calculations

For most reinforced concrete and steel buildings in design classes 1-3, detailed calculations may not be needed for every load-bearing element being removed, provided that sufficient simple checks are made to ensure that the structure has some feasible alternative load paths available. Both steel and concrete structures will tend to behave plastically in a failure event and redistribute load to other elements.

Timber behaves in a brittle and elastic manner, and therefore building structures are less ready to redistribute forces. They are more reliant on discrete connections and a more detailed consideration of robustness is required. A methodical design approach involves the identification of at least one alternate load path for the notional removal of each load-bearing element, such as the one illustrated in Figure 3.19, and to assess the structural elements in that load path for the accidental load case.

In such an approach, the building must be modelled without the load-bearing element under consideration, and all members and connections of the alternative load path are verified to have sufficient strength to resist the accidental loads. This is repeated for each notional element subject to removal.

Simple models based on structural mechanics principles can be used for this purpose (see Section 4). If this approach proves unsuccessful, either a more detailed model that considers all alternative load paths would be needed or the building would need to be designed for protected/key elements. A more detailed model would have to consider non-linear material and geometric analyses to correctly redistribute the load through the various alternative load paths. Various types of suitable analyses with various degrees of complexity (linear static, non-linear static and non-linear dynamic) and accuracy are discussed and detailed in Arup (2011) and the DoD (UFC 4-023-03 2016).

Considering all alternative load paths in the analysis will not be necessary in the majority of buildings, although the sorts of structures outside the scope of this design guide may find it necessary. Some modelling guidance for these more complex buildings can be found in the guidelines by FPInnovations (Gilbert et al. 2022).

3.5.3 Dynamic Increase Factor

Progressive collapse is a dynamic event, with the building structure subjected to inertial loads after the sudden removal of a load-bearing element. When performing static analyses, these inertial loads may be considered by increasing the vertical loads by a Dynamic Increase Factor (DIF), so that the resulting static deformation matches the actual maximum dynamic deformation. The static results would reproduce the maximum level of stress encountered by the structures during the dynamic event.

Only the DoD (UFC 4-023-03 2016) explicitly considers the inertial loads in its design methodology by increasing the accidental loads applied to the bays adjacent to the removed load-bearing element by a DIF of 2.0 for timber structures. Appendix A of the AS/NZS 1170.0 (2002) mentions that dynamic effects should be considered through special studies, without further explanations. In Eurocode 1, by comparison, dynamic effects are mentioned for impact loads in Part 1-7: General actions – Accidental actions (EN 1991-1-7 2006) and a DIF factor of 2.0 is recommended in Part 1-6: General actions – Actions during execution (EN 1991-1-6 2005) under "local failure of final or temporary supports". Dynamic effects are not considered in the IStructE (2010).

While the inertial loads are always present in progressive collapse, this is unfortunately not always assessed in practice, likely due to the lack of guidance. The designer must make informed choices on how the inertial loads are considered in the design and, if using static analyses, on the appropriate DIF value to be used. The DIF of 2.0 proposed in the DoD and Eurocode 1 Part 1-6 corresponds to a theoretical upper value. The actual DIF would be lower for most systems. A DIF of 1.5 has been proposed for timber structures by Palma et al. (2019), a value also found experimentally in Cheng et al. (2021). However, larger DIF values have been encountered in specific cases (Arup 2011) and when brittle failure modes developed at the timber connections (Cheng et al. 2021). Brittle failure modes under accidental loads must be avoided. As developed in the DoD, the DIF are only applied to the floors that deform dynamically, i.e., corresponding to the bays adjacent to the removed load-bearing element.

3.6 Protected (or Key) Elements and Systematic Risk Assessment

In cases where certain elements are unable to be notionally removed without the risk of collapse of a disproportionate area of the building, it is necessary to design these elements as protected elements. The element should be designed to resist both their normal applied loads and the anticipated load from the accidental damage event. It is a method that a designer should use as a last resort and apply with a great deal of caution because it relies on the consideration of a lot of variables in terms of force, point of application and structural response.

The NCC requirement is for a systematic risk assessment to be undertaken and critical high-risk components identified. They then need to be designed to cope with the identified hazard or protective measures chosen to minimise risk. Accidental damage events are, by their nature, difficult to define and the forces applied in these events are hard to assess. Specialist advice may be needed. It is anticipated that designing a member as a protected element in an accidental damage load case will be the critical load case for this member, sometimes by an order of magnitude larger than the normal in-service design condition.

The Eurocode (EN 1991-1-7 2006) gives some guidance on the forces to apply, although the actual application of these forces is poorly defined. The UK national annex calls for an accidental damage uniformly distributed notional load of 34 kPa "applicable in any direction to the key element and any attached components (e.g. claddings, etc)". Defined following the Ronan Point disaster, 34 kPa is a static force representing the force generated by a gas explosion. There is no real guidance on how best to apply this load and it is clearly a large force that will be onerous in a number of cases and un-conservative in others.

Take the example of an isolated internal column. If a 34 kPa force is applied horizontally to the face of the column, it clearly has a very small net load due to the small surface area. However, in the case of columns supporting cladding, the horizontal force applied to the column may be large as the accidental pressure will be also applied to the cladding until it fails. For instance, if the cladding and its connections can resist 10 kPa, then this pressure applied to the column and cladding will have to be used to design the column. Vertically, if we apply the same 34 kPa force to the slab that transfers load to the column, this load is very large indeed and most likely would cause the slab and connections to fail long before the load reaches the column. Note that the vertical load to be applied to the column is the maximum that the slab can transfer before failure.

The IStructE's (2010) guide notes that this is a limitation and recommends taking the upper boundary of the connection capacity for the floor to the column as the largest vertical load transferred to the column. The lateral load on the column is also recommended to be a point load applied at the worst-case location, 250 kN at ground floor and 150 kN at all other floors.

Designing protected elements requires a lot of risk assessment and engineering judgement. Consultation with specialists in the field of blast engineering and transport impacts may be prudent in a number of cases. Cost-effective design strategies may often include a combination of notional element removal scenarios and protected elements.

4

Timber

4.1 Mass Timber or Stick Frame Construction

Modern timber construction can mean anything from domestic residential to large-scale commercial buildings. Each material type and building use can require a different approach to robust design. In general, this guide groups them into two categories:

4.1.1 Mass Timber Construction

Mass timber refers to a more recent trend in larger-scale engineered-timber buildings. Engineered-timber products like glulam, laminated veneer lumber (LVL) and cross-laminated timber (CLT) are making it possible to build much larger buildings – and ones that are much more likely to need consideration of robustness in their design. Mass timber construction can be used for a wide array of building types and sizes: medium-rise, multi-residential (like Forte, Melbourne – Figure 4.1), public buildings (like Library at the Dock, Melbourne), offices, schools, etc.

Figure 4.1: Forte, Melbourne, nine storeys of CLT on concrete podium.

Mass timber buildings can take various construction forms, such as:

- Platform-type: buildings having the floors of each storey directly resting on the walls below, creating a platform for the storey above (Huber et al. 2020, Mohammad et al. 2019). The walls are not continuous and the floors are loaded perpendicular to the grain.
- Balloon-type: buildings having continuous walls extending multiple storeys with non-continuous floors attached to them (Chen et al. 2020).
- Post-and-beam: open-space buildings with continuous columns linked by beams. The floors sit on the beams and typically span one-way.
- Flat plates: buildings having the floors directly resting on the columns with special connectors, with no beams and walls, such as the UBC Brook Commons in Canada.

The inherent robustness and recommended design procedures are different for each construction form. Mass timber construction performs well, if designed correctly, for a few reasons:

- While panels are typically designed for one-way spanning, they have the ability to work in two directions.
- There are typically a large number of load-bearing elements, offering a high level of redundancy.
- Wall panels have the ability to span as beams.
- High connection capacities can be developed along panel joints.

These behaviours are discussed further in the following section.

4.1.2 Stick Frame Construction

This covers traditional timber frames, joisted floors, stud walls, prefabricated roof trusses. This construction is most often used in domestic residential (Figure 4.2) and low-rise multi-residential buildings.

Figure 4.2: Typical timber-framed domestic construction.

This form of construction exhibits good robustness due to a very high level of redundancy in the large number of individual studs, joists and rafters. Loss of individual components can typically be accommodated readily, however the loss of wall frames in tall multi-storey framed construction can cause disproportionate collapse without specific measures to provide alternate load paths, such as the introduction of rim beams and posts within walls.

4.2 How is Timber Different?

The main difference between timber structures and reinforced concrete or steel structures is that timber does not behave in a plastic manner and is therefore much less able to redistribute load or utilise catenary action. It is also made up of a number of individual components that are connected at discrete locations. These connections define how the structure is able to transfer load in an accidental damage situation.

Specifically, it is noted that:

- Timber is a brittle material.
- Timber is typically greater in strength than the characteristic design value by a larger margin than for steel, which reflects the variability due to natural properties of timber. Timber also exhibits greater strength when loaded for very short durations such as those found in accidental load cases.
- Structural elements are typically one-way spanning and can often have limited continuity.
- Continuity of structure can be limited by manufacturing and transportation constraints.
- Post-and-beam structures use deep members, causing large deformations at connections when subject to the accidental loss of support and related joint rotations, requiring high levels of joint ductility to maintain continuity between beam and column.
- Timber structures are typically lightweight, meaning imposed and wind loads are significant compared to gravity loads, and extreme loads are more likely to cause failure.

For these reasons, the design of robust structures in timber often requires more attention and some more explicit design checks to be made.

In timber structures' favour are that they are typically much lighter weight than the equivalent building in other materials and the forces generated during accidental damage events are much less severe and easier to design against.

4.3 Robustness in Mass Timber Construction

Mass timber construction is gaining momentum in Australia. Buildings like Forte and the Docklands Library in Melbourne led the way and more timber buildings have been built since or are in construction. Timber construction has been relatively commonplace in Europe for a few decades, although robustness concepts have only really been taken on board with the adoption of the Eurocodes.

While different materials are often used in combination, this section splits its consideration of mass timber construction into the main two types of construction forms, i.e. multi-residential platform-type construction with CLT and post-and-beam construction with glulam/LVL.

The behaviour of mass timber buildings under a load-bearing element removal scenario has been currently studied numerically for platform-framed (Huber et al. 2020, Mpidi Bita et al. 2018) and flat-plate buildings (Mpidi Bita and Tannert 2019), and experimentally for post-and-beam buildings (Cheng et al. 2021, Lyu et al. 2021a, Lyu et al. 2020). The studies showed that despite the brittle response of the timber material, alternative load paths exist for these buildings, consisting of:

- For all structural systems: structural elements (such CLT floors and beams) spanning more than one bay are efficient in resisting progressive collapse by providing continuity throughout the building.
- For platform-framed buildings: (1) localised support can be provided by the CLT floors, at the floor-to-wall interface, to the CLT walls above the missing element and (2) providing that there is enough shear capacity at the wall-to-wall connections, CLT walls can be used at deep beams. See Section 4.3.1.
- For post-and-beam buildings: (1) beam-to-column connections do not typically have enough capacity/ductility to resist progressive collapse by themselves but can provide localised supports in shear to the CLT panels under large deformations, and (2) the layout of the CLT panels plays an important role in making the building robust. See Section 4.3.2.
- For flat-plate buildings: (1) the axial tension capacity of the column-to-column connections, (2) the rotation ductility and the axial tension capacity of the floor-to-column connections, and (3) the axial and shear capacities of the floor-to-floor connections are all important factors to consider in designing a robust building.

4.3.1 Platform-type construction

Platform-type construction is perhaps the most inherently robust of all the massive timber construction forms for a few reasons:

- the high number of walls provides the opportunity to create alternative load paths
- wall panels have the ability to span as beams
- high connection capacities can be developed along panel joints.

Careful design and detailing is needed to ensure that it will perform sufficiently robustly in an accidental case and the locations of panel joints will be crucial to establishing this. Due to their potential high level of structural redundancy, platform-type structures are best designed with the notional element removal method, which will often lead to the most economic answer in terms of connection design and speed of construction.

Figure 4.3 presents a typical residential platform-type structure. All walls and floors are constructed from CLT and are connected with screws and brackets.

Figure 4.3: Typical CLT apartment layout

Two strategies prevail to design robust platform-type construction through two alternative load paths:

- Having CLT floor panels spanning two-bays and bridging over the missing wall as simply-supported or cantilevered floors, as described below and presented in Figure 3.19.
- Having the walls acting as deep beams and bridging over the missing wall.

For the first alternative load path, consider the removal of a typical wall panel either at the middle of the two-bay long CLT floor or at its end, as shown in Figure 4.4. As noted in Section 3.4.2: (1) in the IStructE (2010), the minimum length of the wall to be considered as removed should be the larger of 2.25H (about 6.75 m in this case) or the distance between vertical support for external walls, and (2) in the NCC (2019), the storey height H. For the following examples, we assume that the wall below the entire width of one CLT floor has been removed and so no partial support is provided to the floor.

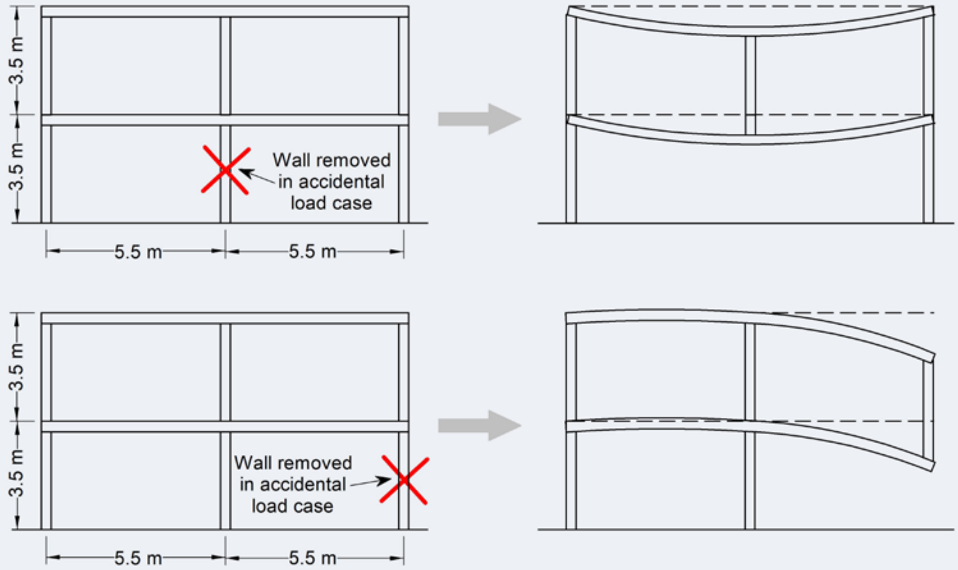


Figure 4.4: Removal of load-bearing wall in a platform-type building.

The floor is now spanning across double the distance or cantilevering over one bay (Figure 4.4). If all floors are loaded the same and have the same bending stiffness, each of the floors above the removed wall want to deform the same and no axial load will be present in the walls above the removed one. However, as often is the case, the roof will be thinner and have a different live load value than the floors. This difference will generate an axial load in the walls above the removed wall. The value of this axial load, assuming stiff floor-to-wall connections, is discussed in the FPInnovations modelling guidelines (Gilbert et al. 2022). Depending on the relative stiffness difference between the roof and the floors, the walls above the removed one will either be in tension or compression. A separate design check would therefore be needed for the roof and floors.

Assuming in this example that the roof and floors have the same bending stiffness and accidental load combination (i.e. no axial load in wall above the removed one), the free body diagrams of the CLT floors in Figure 4.4 are plotted in Figure 4.5. The free body diagrams for different roof and floors are provided in the FPInnovations modelling guidelines (Gilbert et al. 2022).

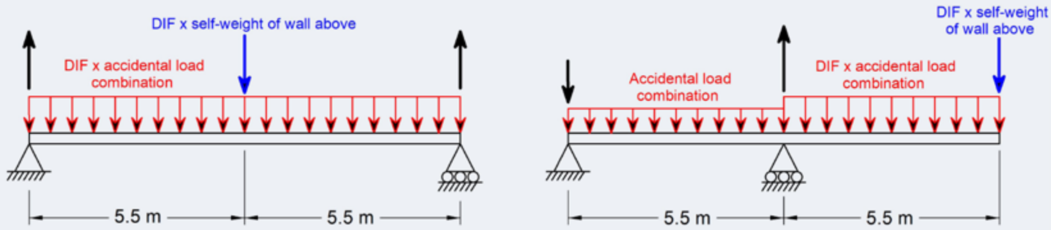


Figure 4.5: Free body diagram of CLT floor after removal of load-bearing wall in a platform-type building (wall removed in middle (left) and at end (right) of two-bay long CLT floor).

For the permanent design case, the floor was designed as 5-layer 205 mm thick, based on deflection and dynamic requirements. In the accidental case, the floor can be designed based on reduced factors and loads. The deflection of the floor is now unimportant and the governing design check will be the bending moment. The values of the floor dead load (self-weight and superimposed dead load) G_k, live load Q_k and self-weight of wall above G_w are given as:

$G_k = 2.29$ kPa, $Q_k = 1.5$ kPa and $G_k = 2.5$kN/m

Therefore, the accidental load ω_{acc} from the accidental load combination in the AS/NZS 1170.0 (2002) is given as:

$\omega_{acc} = G^k + 0.4Q_k = 2.80$ kPa

So, for a 1 m wide CLT panel and using a DIF of 1.5 as suggested by Palma et al. (2019) (see Section 3.4.4), the free body diagrams for the two removal scenarios in Figure 4.5 give a maximum bending moment M_{acc} of:

$M_{acc,d} = 73.8$ kNm when a wall is removed at the middle of the floor

$M_{acc,d} = 84.1$ kNm when a wall is removed at the end of the floor

According to the CLT handbook (Popovski et al. 2019), based on the short-term loading factor k_1 of 1.0, material partial factors Φ of 0.9, an effective section modulus $S_{eff,y}$ of 5.87×10^6 mm^3, a characteristic bending strength f'_b of 24 MPa and a strength reduction factor $K_{rb,y}$ of 0.85 for bending about major strength axis, this gives an allowable bending moment resistance $M_{r,y,acc}$ of:

$M_{r,y,acc} = \Phi \times k_1 \times S^{eff,y} \times f'_b \times K_{rd,y} = 107.7$ kNm

The bending capacity of the material is not exceeded in the accidental case and the floor remains standing, albeit with large deflections. It is good practice to stagger the joints in floor panels where possible as shown in Figure 4.6 (this has the added bonus of making it easier to demonstrate diaphragm action in the floors).

Other design checks would include where applicable: (1) the compressive capacity of the CLT walls which support the two-bay long CLT floors and to which the additional loads are transferred; (2) the shear capacity of the CLT floors; (3) the compression perpendicular to the grain of the CLT floors; and (4) the wall-to-floor connections if tension forces develop at these connections. It also needs to be ensured that under large deformation, the horizontal movement at the supports is small enough for the two-bay long CLT floors to rest on the walls below.

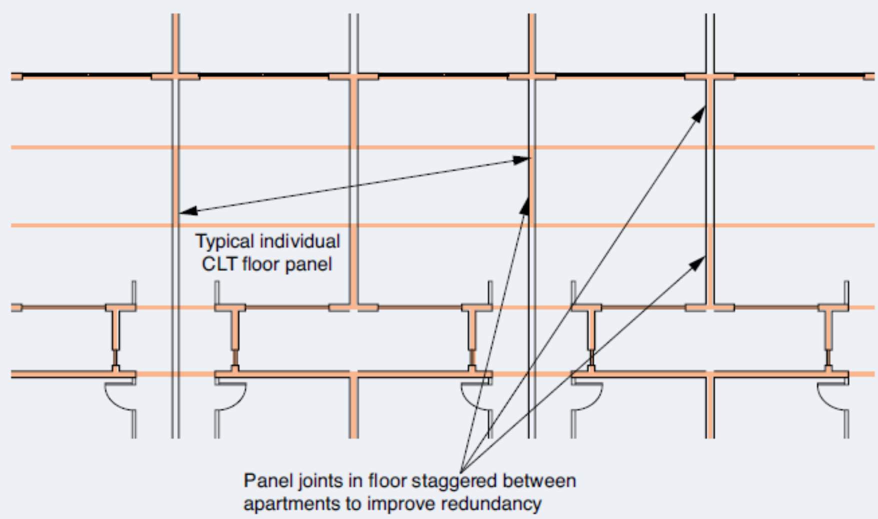


Figure 4.6: Staggering floor joints in CLT buildings is good practice for robustness.

Another useful benefit of CLT construction is being able to use walls as deep beams to span over walls removed below. The depth of the walls means they will work well as spanning elements if called for and it remains to ensure the connections are capable of resisting the loads and the walls are sufficiently well restrained.

Consider the example in Figure 4.7.

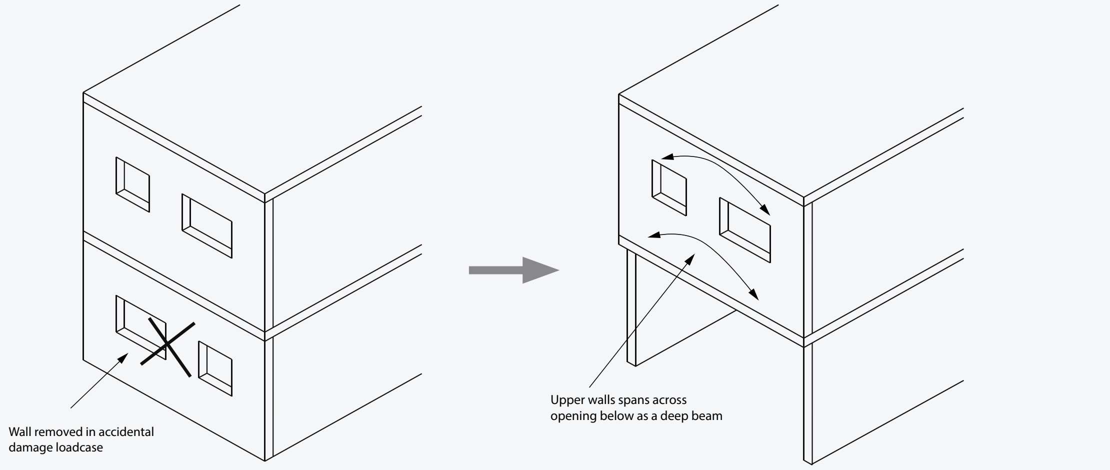


Figure 4.7: Spanning of walls in accidental damage load cases.

The load-bearing wall at the lower level has been removed, and the wall spans 7 m. The floor is also connected into the wall above, assuming a 6 m long tributary area for the floor. A very basic design check on the wall shows the stresses are very low and the wall buckling is unlikely to be at risk as the wall is restrained top and bottom by the floors.

$G = 6\text{ m}\times G_k + G_w = 6\text{ m}\times 2.20\text{ kPa} + 2.5\text{ kN/m},\ Q = 6\text{ m}\times Q_k = 6\text{ m}\times 1.5\text{ kPa}$

This leads to an accidental load ω_{acc} applied to the wall from the accidental load combination in the AS/NZS 1170.0 (2002) of:

$\omega_{acc} = 19.3\text{kN/m}$

and a bending moment $M_{wall,acc}$ and bending stress $\sigma_{wall,acc}$ applied to the wall of (using an effective section modulus of the wall of 1.34×108 mm3):

$M_{wall,acc} = (19.3\times 72)/8 = 118.2\text{ kNm}$

$\sigma_{wall,acc} = 0.88\text{ MPa}$

Next, it is important to check if the floor-wall connection is capable of hanging the lower floor. In the accidental load case, the weight F of the floor hanging from the wall would be:

$F = 16.8\text{ kN/m}$

This load is within the right magnitude for the capacity of a typical floor-wall connection detail and is also notable for being significantly lower than the vertical tie force, had that method been chosen and verified to apply. If proprietary brackets are used, it is important to ensure that the chosen bracket has sufficient tension capacity. Some are only rated for tension loads when installed on both sides of a wall, as shown in Figure 4.8.

Additionally, it is wiser to install screws rather than nails in these brackets as nails have nominally little tension capacity.

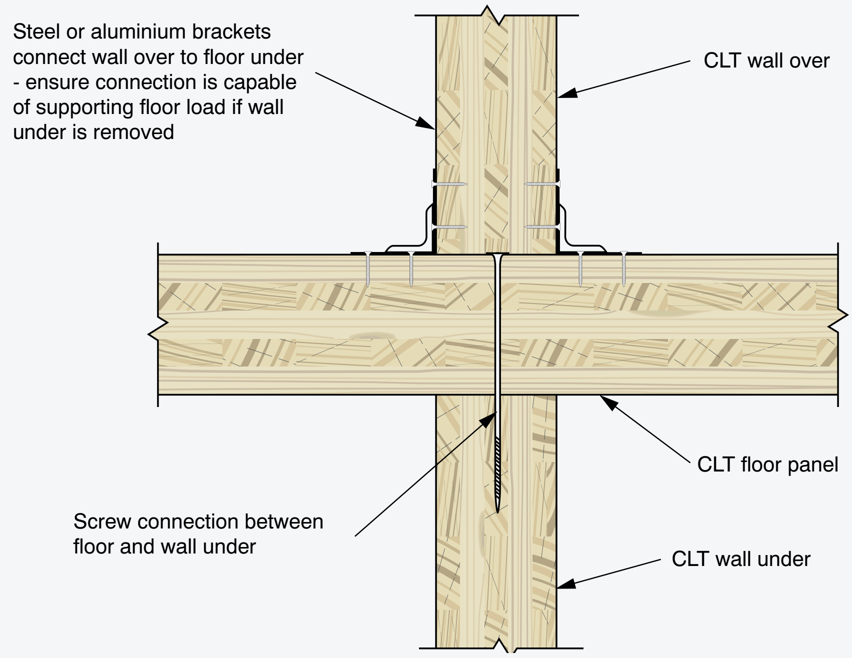


Figure 4.8: Typical wall-floor connection in a CLT building.

Finally, for a wall to act as a deep beam, the wall-to-wall connections must be able to withstand the shear forces applied to them in the accidental load case. In the example, for the 3.5 m-high wall, the accidental shear force V applied to the wall-to-wall connections would be:

$V = 19.4$ kN/m

4.3.2 Post-and-Beam Construction

At a basic level, the construction of a timber post-and-beam building is similar to a steel-framed one. The building is built in a similar way – with columns, primary beams and a strongly single-direction spanning floor system. Figure 4.9 shows an example of post-and-beam buildings.

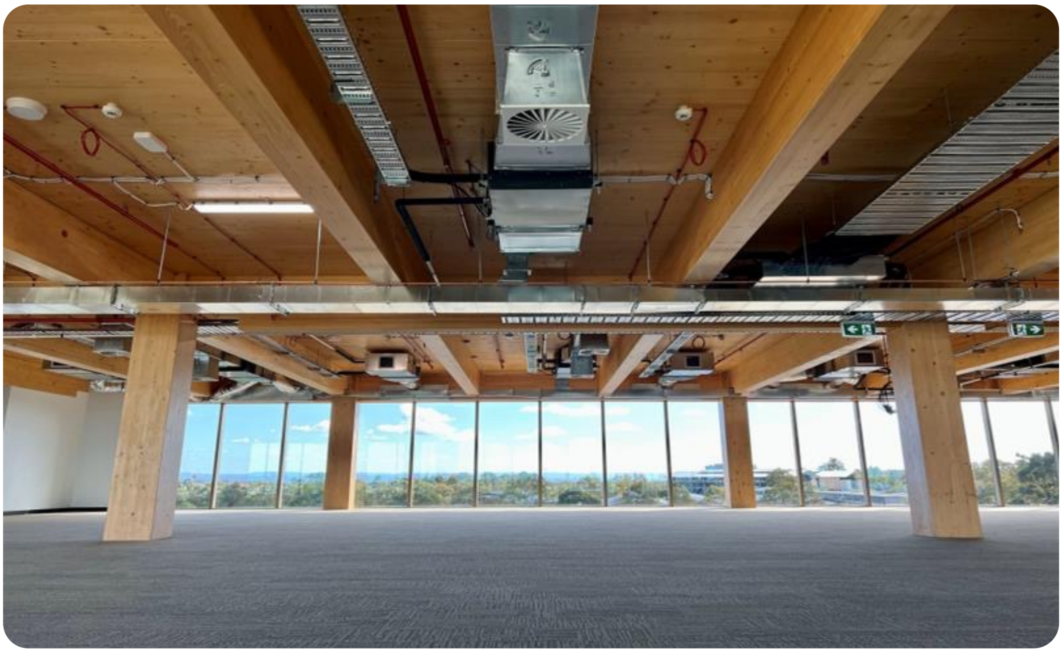


Figure 4.9: Timber post-and-beam construction – The Bond, Norwest NSW.

The main challenge in attempting to design a robust timber post-and-beam building is that it will perform fundamentally differently to a steel building. It will be more elastic and there is much less capacity for rotation at joints, and little catenary action, etc.

The type of construction also offers less redundancy than platform-type construction. When designing for alternative load paths, the designer must conceive the building with clear alternative load paths and design for them. This means that some more explicit design checks need to be made.

Moreover, beam-to-column connections are typically designed as shear connections, not moment connections, and do not contribute to resist the horizontal loads. The ability of three types of commercially available beam-to-column connections to provide enough rotation and enable catenary action to develop under a column removal scenario was experimentally investigated by Lyu et al. (2020). The tests showed that the examined connection types did not allow catenary action to fully develop and resist the accidental load by themselves. They therefore do not constitute an alternative load path and designing post-and-beam buildings solely under the tie force requirements (Section 3.3) must be avoided at all costs unless beam-to-column connections are proven to be robust. Figure 4.10 shows the concept of a beam-to-column connection designed to allow sufficient rotation to occur through the slotted holes and catenary action to mobilise. The connector alone was shown to resist up to 1.4 times the accidental load combination in the AS/NZS 1170.0 (2002), without considering the DIF, under a column removal scenario (Lyu et al. 2020). The connection used sets of aluminium plates connected offsite, with overdesigned dowels, to the beam and column. On site, the set of plates connected to the beams is bolted to the one connected to the columns, allowing ductile failure to occur at the bolted aluminium-to-aluminium connection.

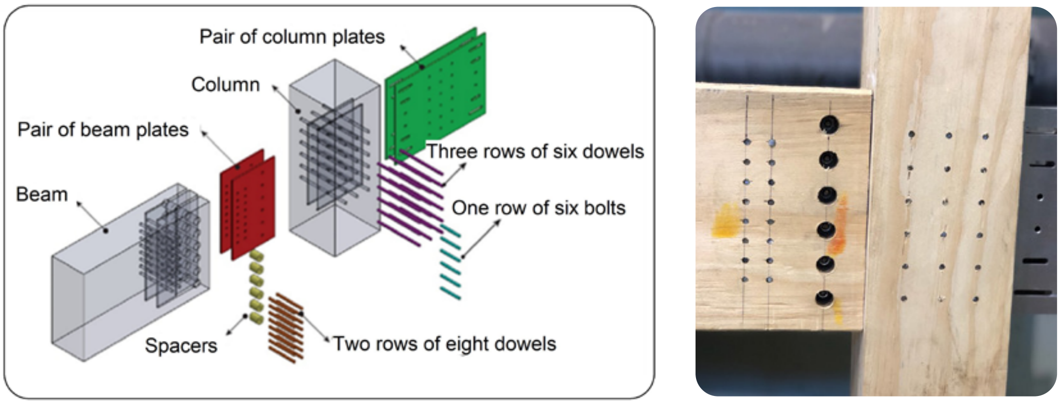


Figure 4.10: Concept of post-and-beam beam-to-column connection designed for robustness (Lyu et al. 2020).

Two alternative load path strategies prevail to design robust post-and-beam construction:

- Similar to platform-type construction, having CLT floor panels spanning two-bay and bridging over the missing element as simply-supported or cantilevered floors.
- Having the beams spanning more than one bay and bridging over the missing column. The beams can be either 1.5-bay long and splice-connected or 2-bay long, as later illustrated.

3D experimental tests performed on corner and edge column removal scenarios have shown that post-and-beam buildings offered additional alternative load paths to the two listed above (Lyu et al. 2021a, b), with the tested structures resisting up to 10 times the value of the accidental load combination. Nevertheless, the two-bay long CLT floors and their layout were found to be critical in designing robust post-and-beam buildings. If one-bay long CLT floors cannot be avoided, such as for long-span floors, it is essential to either provide continuous beams, design columns as key elements or prove through 3D analysis that the building provides additional alternative load paths. 3D Finite Element models for the latter purpose are described in Gilbert et al. (2022).

Key element design may represent the most appropriate design method, even if alternate load paths are present in the building when (Gilbert et al. 2022):

- column and beam dimensions are relatively large due to architectural requirements
- column and beam dimensions are relatively large due to design requirements for a gravitation load case or
- columns can directly transfer loads via bearing into the floor diaphragm.

For some post-and-beam structures in one of the above scenarios, if the primary elements and their connection details can resist the key element design loads without further or significant modifications or enhancement, this approach may be more economical than an alternative load path approach. ALP methods can often result in complex post-and-beam connection details, and dense fixing requirements at floor-to-beam and floor-to-floor connections.

An ALP, different to the ones mentioned previously in this section, can also develop in post-and-beam buildings. This load path consists of a localised support provided to the CLT floors at the connections linking the beams to the columns adjacent to the removed column, this occurring despite these connections failing in bending and undergoing large displacement. This alternative load path were found by Lyu et al. (2021a,b) to transfer about 25% to 30% of the load applied to the bays adjacent to the removed column to the foundations. If the shear capacity of these connections undergoing large rotation can be quantified, a designer could take advantage of this load path through 3D analysis.

Figure 4.11 and Figure 4.12 show the deformation of post-and-beam substructures experimentally investigated under corner and edge column removal scenarios, respectively. In the figures, the alternative load path of the beam-to-column connections provides localised support to the CLT panels.

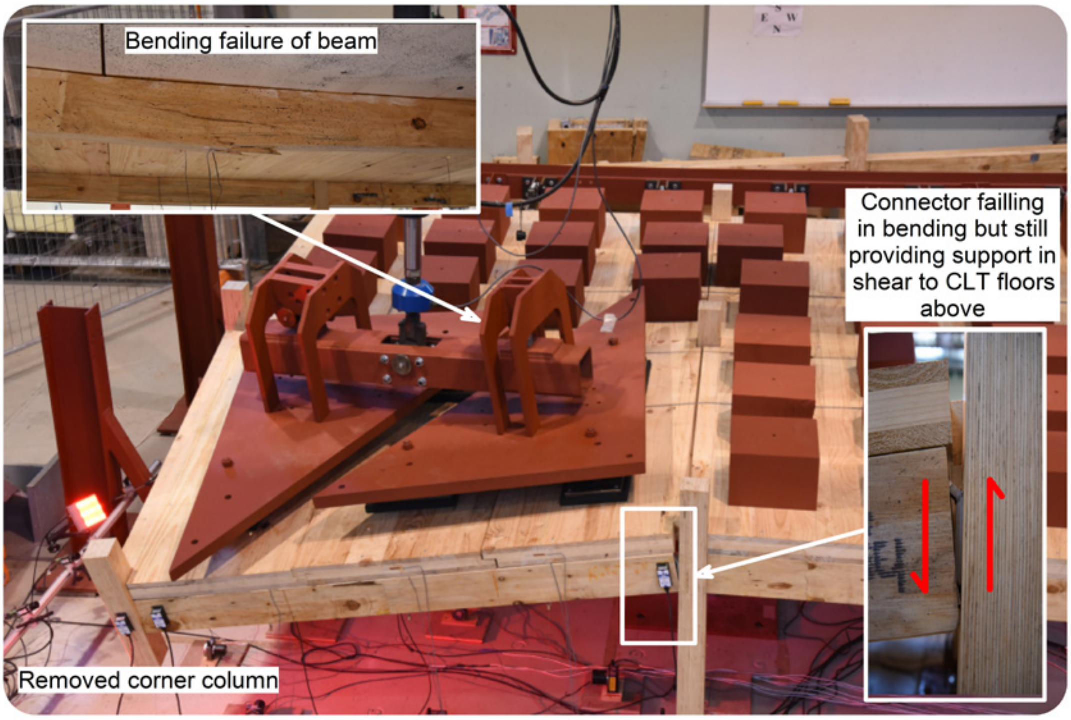


Figure 4.11: Timber post-and-beam substructure tested under a corner column removal scenario.

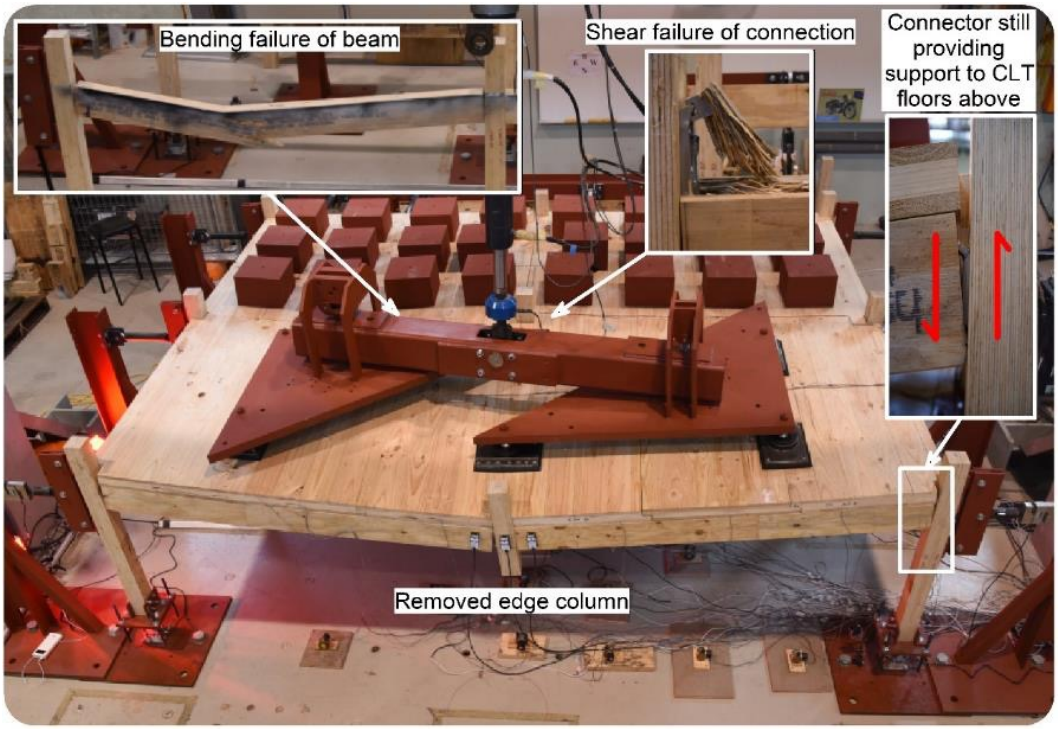


Figure 4.12: Timber post-and-beam substructure tested under an edge column removal scenario.

The ultimate load of the post-and-beam substructures shown in Figures 4.11 and 4.12 was reached when either failure of the beams supporting the middle of the two-bay long cantilevered CLT floors occurred in bending or failure of the beam-to-column connection in shear. When using two-bay long CLT floors as an alternative load path, both the strength of the CLT floors and their supporting elements must be verified.

The following examples are based on a structure similar to the Library at the Dock in Melbourne, although some changes have been made to demonstrate different robustness concepts. The beams are detailed as double sections running either side of the column, which is notched to partially seat the beams on. The examples are based on the alternative load paths.

Using a double beam arrangement either side of the column is an efficient way to provide continuity and the associated alternative load path. The continuity also helps reduce the beam deflections. However, the double sections provide more timber surfaces exposed to potential fire.

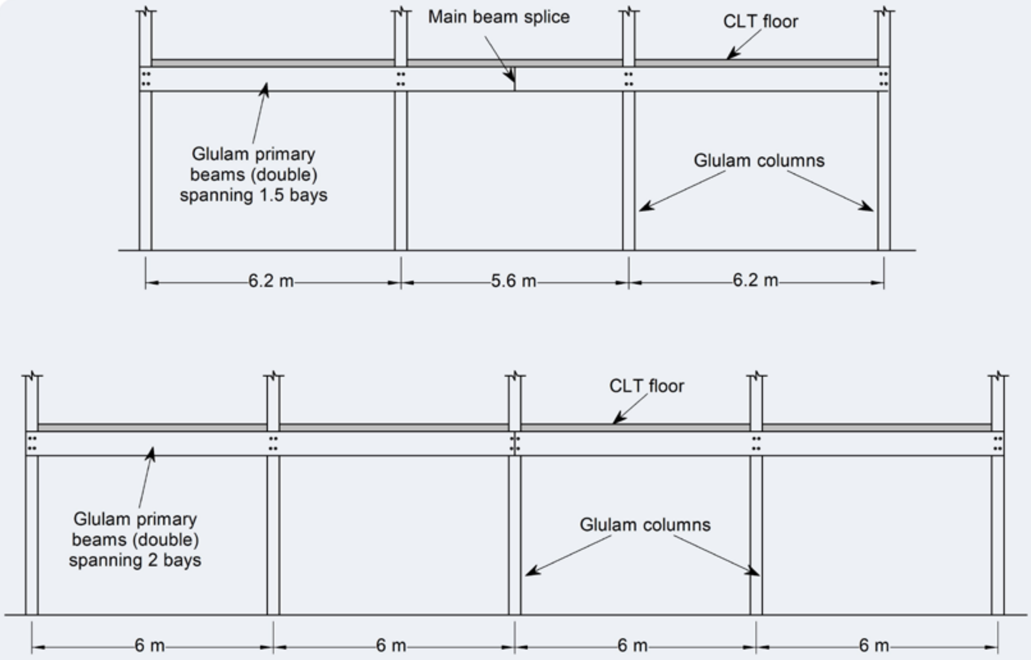


Figure 4.13: Post-and-beam construction with double sections continuous either over 1.5 or 2 bays.

In designing for the continuous beams as the alternate load path, let us consider the removal of either an exterior or interior column in Figure 4.13. In a structural system with a beam spanning only one bay and single-span CLT floors, the loss of a column would potentially lead to the loss of both connecting primary beams and likely two entire floor bays. This would likely extend through the building and would clearly constitute a disproportionate collapse.

The free body diagrams of both the 1.5-bay and 2-bay long beams in Figure 4.13 are plotted in Figure 4.14 under the two column removal scenarios.

In the case of losing a central column, for the 1.5-bay long beams, the beams now span to the central splice location while the other beam supports a significant additional load on the end of the cantilever. In this case, the right-hand beam would usually experience a large increase in moment over the internal support. Typically, the floor design will be governed by serviceability requirements for normal conditions and therefore beams may have reserves of moment capacity that will aid design. However, if member capacity is insufficient, options to reduce this bending moment would need to be considered. Options include detailing floors to maintain some continuity to reduce the loads or to detail some composite capacity in the floors to increase the effective section depth over the column. In these cases, the main beam splice will need to be designed to carry the shear forces associated with the accidental load scenario.

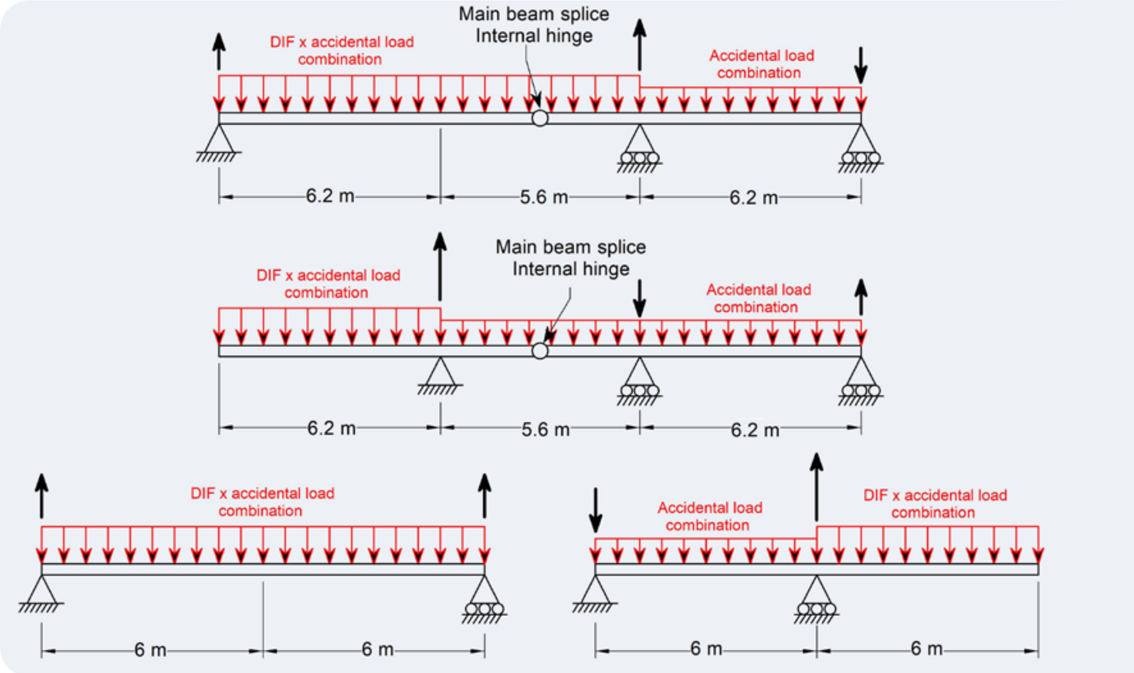


Figure 4.14: Free body diagram of double section beams after removal of a column in a post-an-beam building (central column and 1.5-bay long beams (top), end column and 1.5-bay long beams (middle), central column and 2-bay long beams (bottom left) and end column and 2-bay long beams (bottom right)).

The other option, as mentioned earlier, is to have two-bay long CLT floors to resist the accidental load combination. In such cases, it assumed that beams connected to the removed column cannot resist the load and that the CLT floors are solely carrying the accidental load combination. To design the floors, both the free body diagrams shown in Figure 4.5 (but without the self-weight of the wall above) and the design checks performed in Section 4.3.1 apply. As failure was experimentally shown to occur at the beams supporting the two-bay long floors, these critical beams and their connections would also need to be checked. The reactions obtained by the free body diagram in Figure 4.5 are therefore reported as UDL to these beams. Note that when the column is removed at one end of the CLT floors, the reaction needs to be calculated by considered the next CLT floors which are also supported by the same critical beams. The design principles and free body diagrams of the beams are illustrated in Figure 4.15 and Figure 4.16. For the tested substructures presented in Figure 4.11 and Figure 4.12, this design approach was found to be conservative and underpredicted the failure load by more than a factor of 2.0. Indeed, it assumes that the load is only resisted by one alternative load path and it ignores other load paths, such as the ones described earlier in this section, also contributing to resist the load.

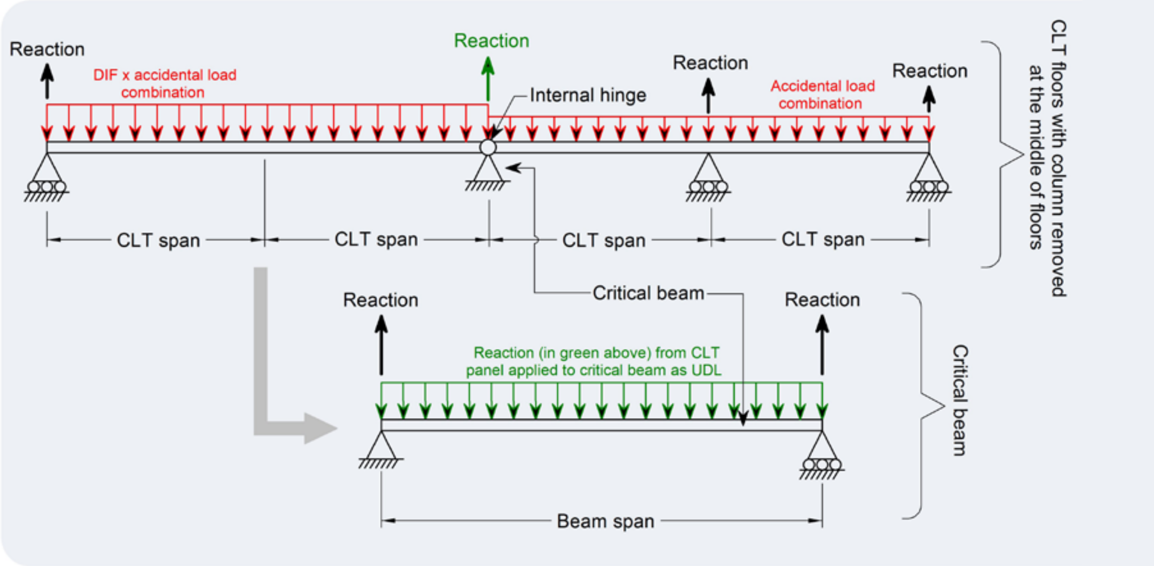


Figure 4.15: Principle and free body diagram to check beams supporting two-bay long CLT floors used as the alternative load path, column removed at the middle of CLT.

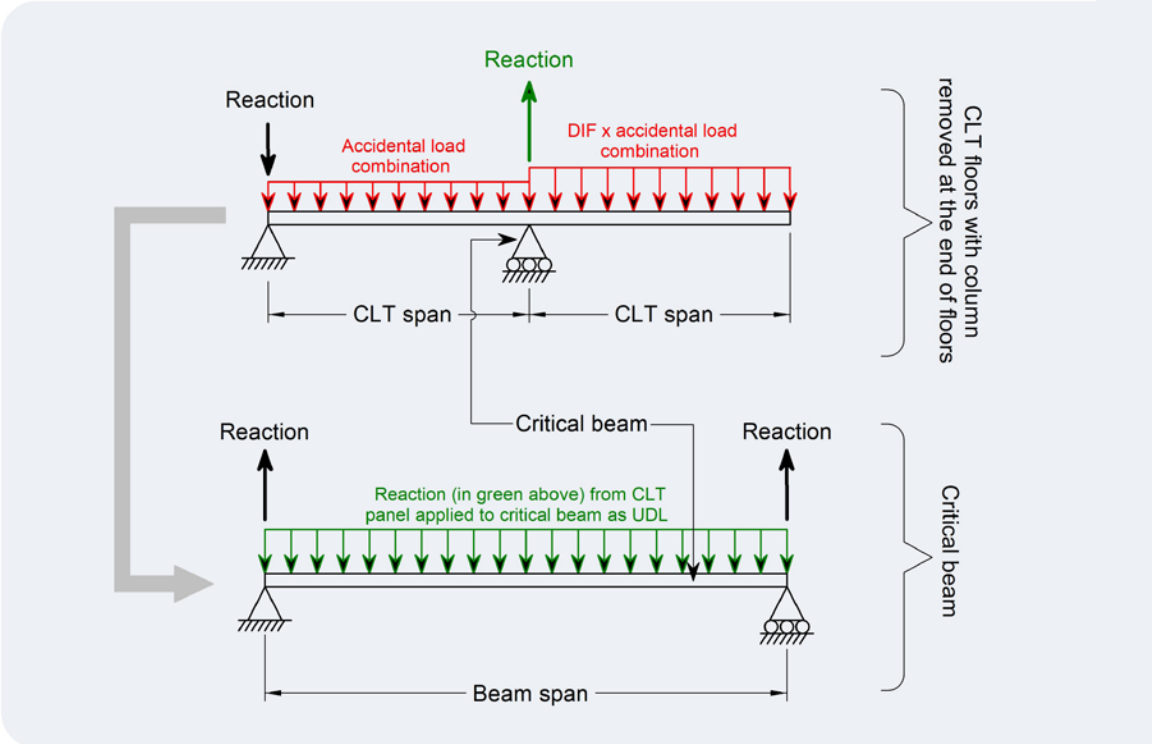


Figure 4.16: Principle and free body diagram to check beams supporting two-bay long CLT floors used as the alternative load path, column removed at the end of CLT.

In all alternative load paths, the axial capacity of the columns supporting the additional loads must be verified.

4.4 Robustness in Stick Frame Construction

Stick frame refers to the frame construction familiar to most engineers and builders. Built from stud walls, joisted floors and roof trusses, it is most often seen on domestic and multi-residential buildings, sometimes making up the top few storeys on a concrete-framed apartment building. Currently, the tallest multi-residential project in Australia constructed in this manner is Fraser Property's 'The Green' in Parkville, Melbourne. Its five storeys were built from largely prefabricated components and shows this form of construction is viable for larger buildings.

Robustness design principles established in this document will apply to this type of construction, but it is anticipated that the most effective method would be to provide horizontal and vertical ties in accordance with Section 3.4 and to utilise a system of rim beams around the top of wall panels to support the floors in the event of the loss of the wall below. The tie forces established are lower than the corresponding values for steel and concrete and this is a reflection of both the lightweight nature of these buildings and the fact that they are unlikely to be economic above 5-6 storeys and therefore the consequences of failure are less. The provision of ties method and the tie forces would need to be revisited if the floor becomes more heavyweight or the consequences of failure increase (either the building use becomes more sensitive of the height increases).

In the UK in 1999, the BRE (Building Research Establishment) carried out a series of tests on a full-scale, 6-storey, timber-framed, brick-clad building. This largely focused on fire resistance testing and disproportionate collapse testing. A number of fire scenarios with the complete removal of certain walls were tested to see how the structure performed. The results led to a number of typical details being developed to constitute best practice for multi-storey timber frame. These are included in Section 4.4.2.

4.4.1 Stick Frame Robustness Concepts

All the typical robustness concepts detailed in Section 3 are relevant for stick-framed buildings although some can be difficult to apply.

Tie forces

Tie forces considered in Section 3.3 are unlikely to be viable in timber-frame buildings due to the magnitude of forces and the difficulty in achieving high capacity connections with small timber sections. The tie forces have been generated through consideration of heavyweight concrete and steel construction and the forces likely to be experienced by a stick frame structure are significantly less.

The UK Annex to the Eurocode (EN 1991-1-7 2006) recognises this and reduces the maximum tie forces to be considered in timber framed buildings as below:

Class 2 Buildings

$$Internal\ Ties - T_i = 0.8(G + \psi_E Q)sL\ or\ 7.5kN\ (whichever\ is\ greater)$$

$$Perimeter\ Ties - T_p = 0.4(G + \psi_E Q)sL\ or\ 7.5kN\ (whichever\ is\ greater)$$

Class 3 Buildings

$$Internal\ Ties - T_i = 0.8(G + \psi_E Q)sL\ or\ 15kN\ (whichever\ is\ greater)$$

$$Perimeter\ Ties - T_p = 0.4(G + \psi_E Q)sL\ or\ 15kN\ (whichever\ is\ greater)$$

where:

G = Characteristic dead load
Q = Characteristic live load
ψ_E = Combination factor for accidental actions
s = Tie spacing
L = Tie length

Ties in timber buildings

The UK Timber Frame Association (UKTFA Special Project 2008) has developed a set of typical details designed to ensure that the buildings maintain a minimum degree of robustness. These include minimum requirements for fixing density, member sizes, etc, and were detailed as a Deemed-to-Satisfy response to robustness. Two of the primary details included in Figure 4.17 indicate these fixings but Australian designers should seek to satisfy themselves that these details are appropriate for their own situation.

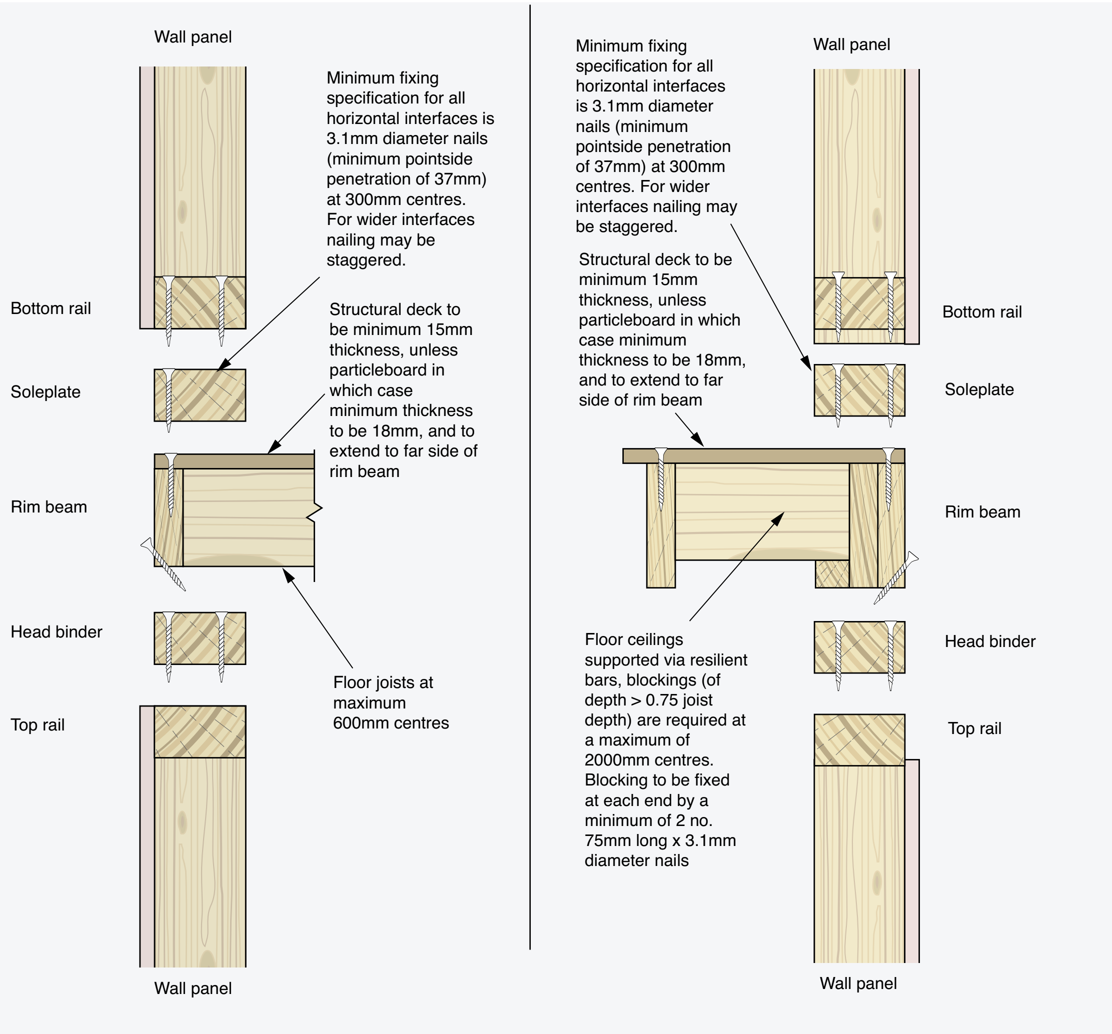


Figure 4.17: Extract from British Standards showing minimum sizes for wall-floor connections.

Rim beams

The most commonly used robustness measure in timber stick-framed buildings is to detail continuous rim beams around the top of the walls. These allow for the support of a floor if the load-bearing wall is damaged by spanning over any gaps and redistributing loads into the adjacent walls.

Figure 4.18 shows typical requirements for rim beams in a timber-framed building.

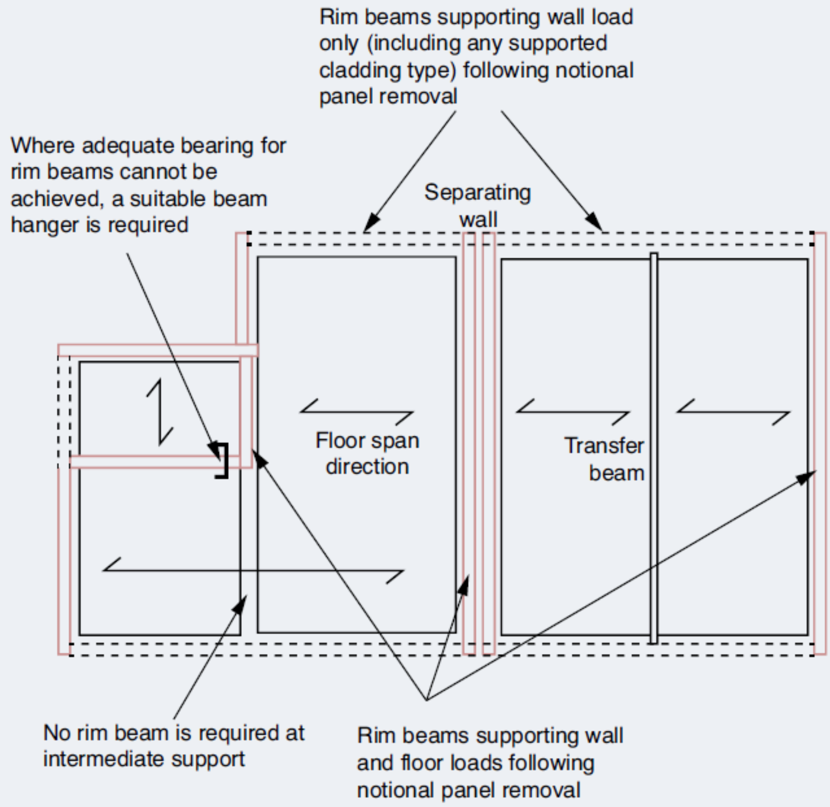


Figure 4.18: Plan of a typical timber-framed apartment building.

Section 3.4 defines the removal of a notional length of load-bearing wall as 2.25H so for a typical apartment building with a floor-floor height of 3.2 m this translates to a maximum length of 7.2 m.

G_k = 1.5kPa, Q_k = 1.5kPa, Load width = 4.5m (external wall), rim beam 90x300 LVL

$$w_{acc} = 9.45\text{N/m}$$

$$M_{acc} = \frac{9.45 \times 7.2^2}{8} = 61.2\text{kNm}$$

$$\sigma_{acc} = \frac{61.2 \times 10^6}{1.35 \times 10^6} = 45.3\text{MPa}$$

This applied stress is high but achievable in a high-grade engineered timber. This calculation is for the maximum length of wall removed and in most wall designs the length of wall between returns is likely to be less than 7.2 m. There is also the potential to mobilise the walls above as a deep box beam with the rim beam acting as the bottom flange. Although this is difficult to demonstrate, it was an observed phenomenon in testing of the TF2000 framed building.

When detailing the rim beams, ensure that the beams would be supported if the loadbearing panel was removed (Figure 4.19). If this is difficult to achieve with a corner stud group detail, it may be necessary to include a joist hanger or another mechanical fixing to ensure the support to the rim beam is maintained.

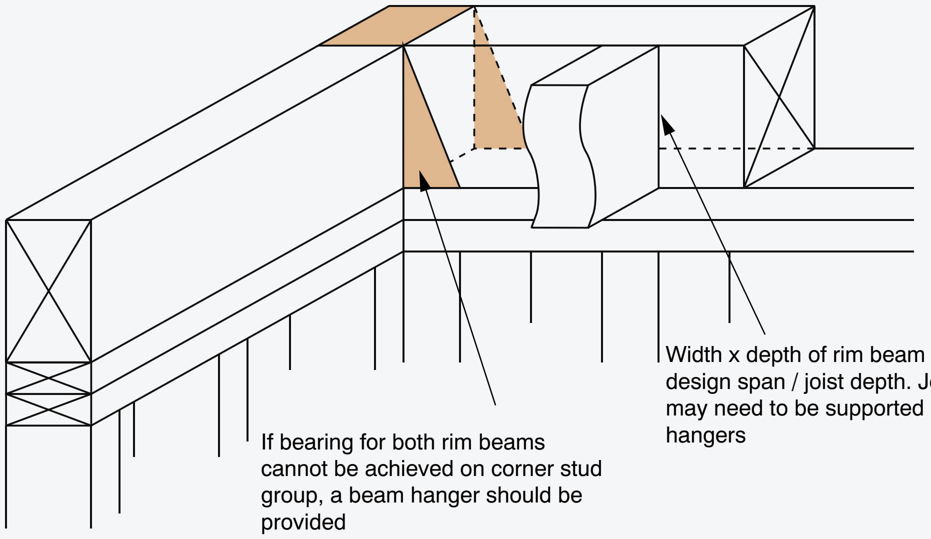


Figure 4.19: Rim beam connection with joist hanger at corner.

Notional removal of load-bearing walls

One fire test on the TF2000 project in the UK looked at the fire resistance of a wall. Following the test, the fire brigade put the fire out. Several hours later, the fire re-appeared on the floor above that has been smouldering within the wall cavity. It is thought that this was due to poor installation of the cavity fire barriers.

In multi-storey timber frame construction, there is a possibility that the loss of the load-bearing stud wall could extend over more than one floor if a fire does penetrate into the cavity. It is recommended that Class 3 and above buildings should consider the removal of load-bearing walls and rim beams over two floors. The normal concepts of rim beams and alternate load paths/ties still apply for the structure but it is likely that the loss of two floors will need to be accounted for within the design. The building should still comply with the limits set out in Section 2.1.

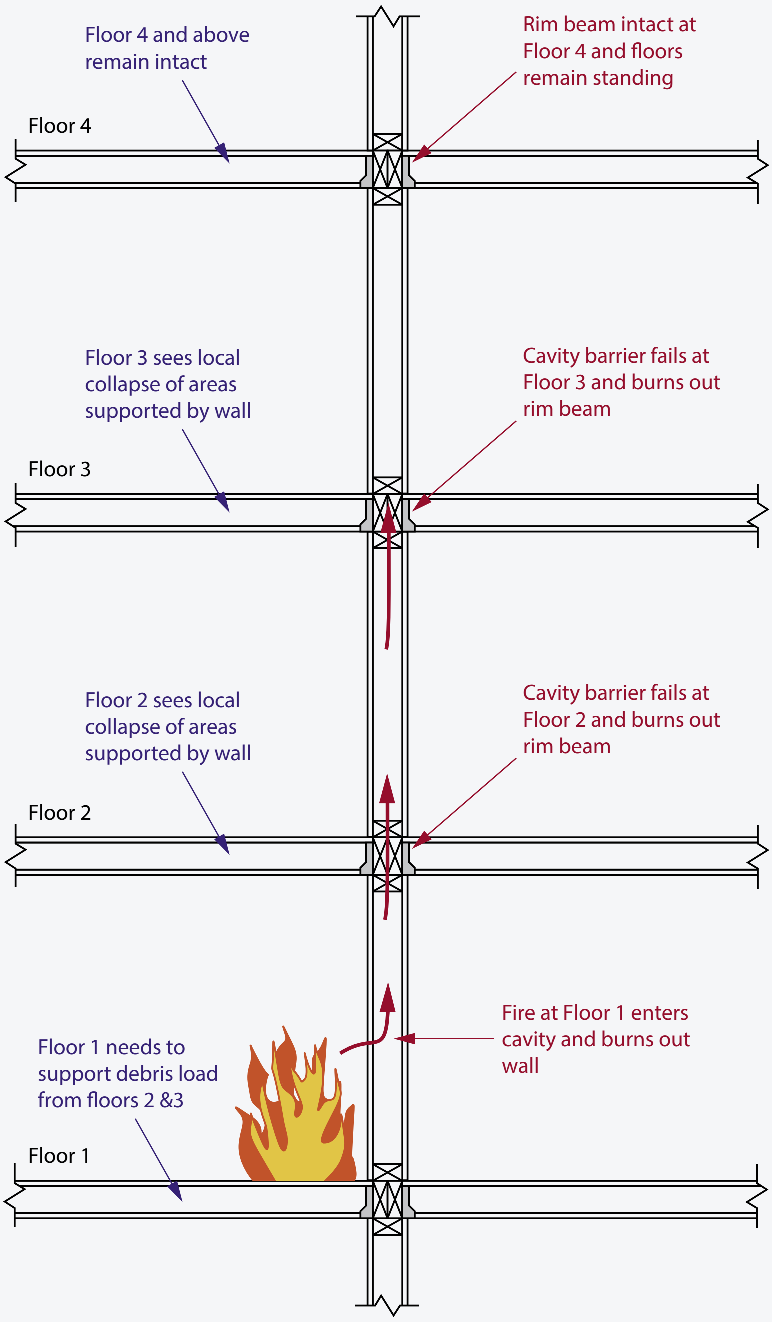


Figure 4.20: Loss of two storeys of load-bearing walls in a load-bearing timber stick-framed building.

The concept of strong floors requires consideration in this particular accidental damage load case. Allowing the collapse of two floors implies that the lower floor should be capable of supporting the debris load from the floors above, which is likely to govern the floor design. Rather than increase the design load on every floor, it is possible to design a strong floor every other level to allow for the debris loading and reduce the impact of any potential increase in materials.

Diaphragm action

Suitable diaphragm action should be provided in all timber buildings but as timber stick-framed buildings start to increase in size, the diaphragm action should be checked more explicitly. It is important to ensure that all the loads can be transferred out of the floors and into the walls. The design of diaphragms in these buildings is well covered in the EWPAA's *Structural Plywood and LVL Design Manual*, which sets out the member and connection checks required.

Other methods of designing against disproportionate collapse

Other methods that can be potentially used to design buildings against disproportionate collapse include:

- wall panels designed to work as deep beams (difficult to do once openings are introduced but useful for partition walls
- floor cassettes designed to double span in accidental damage load cases.

4.4.2 Typical Stick Frame Details

The following details were developed from the TF2000 project in the UK and represent typical robust wall/floor connections for different classes of timber stick-framed building.

Figure 4.21 shows details for Class 2 buildings.

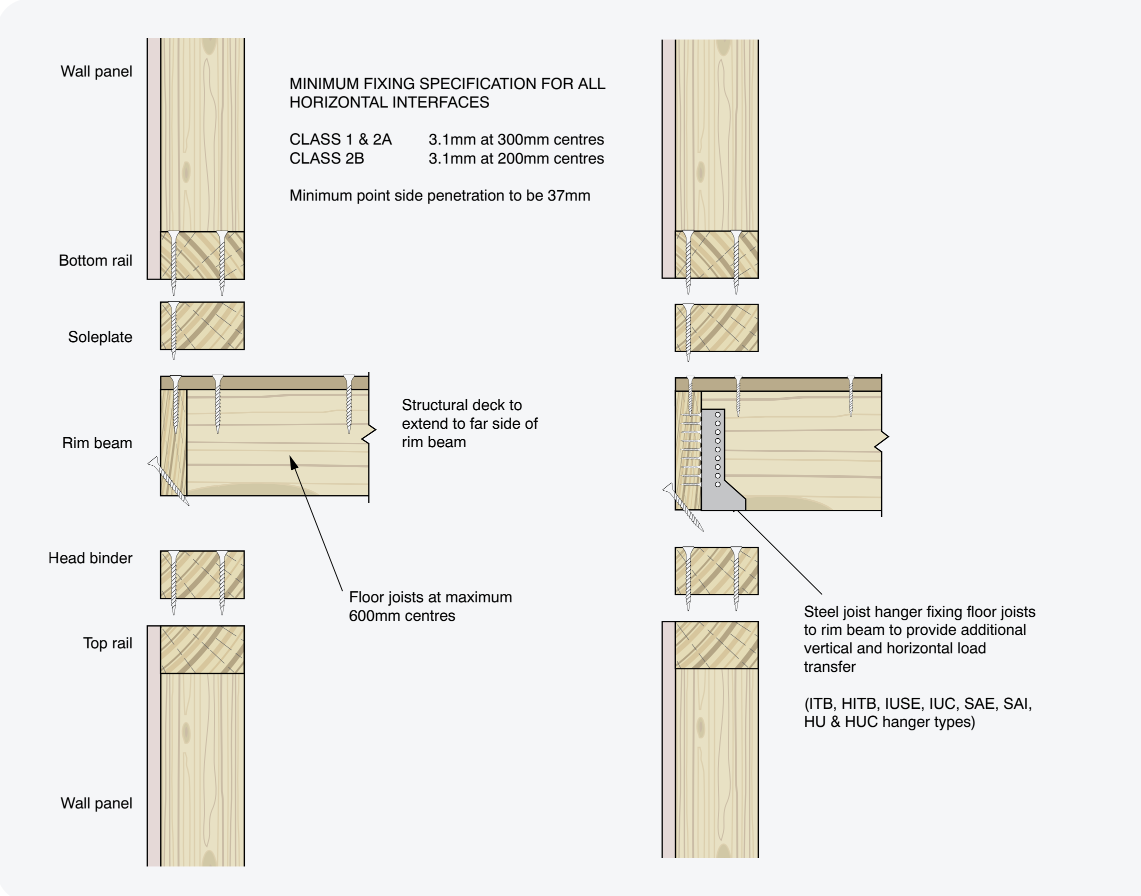


Figure 4.21: Class 2 building, typical floor-wall connection details.

Figure 4.22 shows the same detail for Class 3 buildings and introduces an additional rim beam member as part of the floor system. This provides some additional redundancy and strength to the system as well as making it well suited to prefabricated floor units as it provides a robust edge member to the unit.

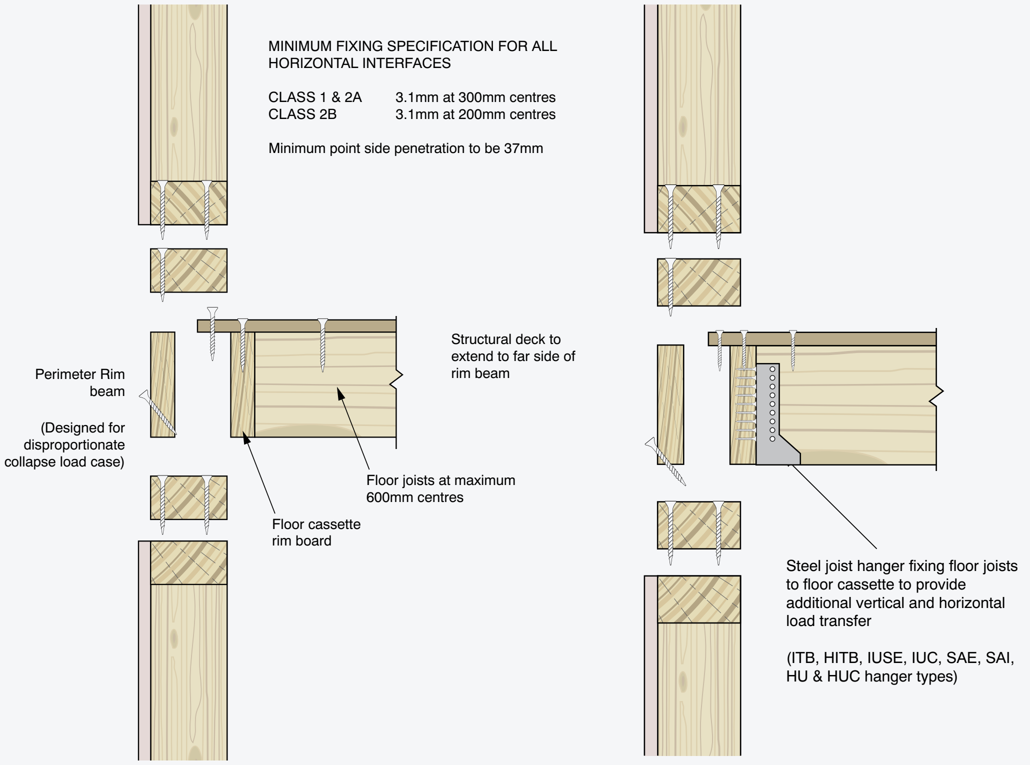


Figure 4.22: Class 3 building, typical floor-wall connection details.

Detailing of rim beams is critical to the robustness of stick-framed buildings in accidental damage load cases. Ensuring that they remain supported when the wall panels are notionally removed is vital. The details in Figure 4.23 show typical details of rim beam connections with joist hangers at corners to provide support in the event of removal of the supporting walls below.

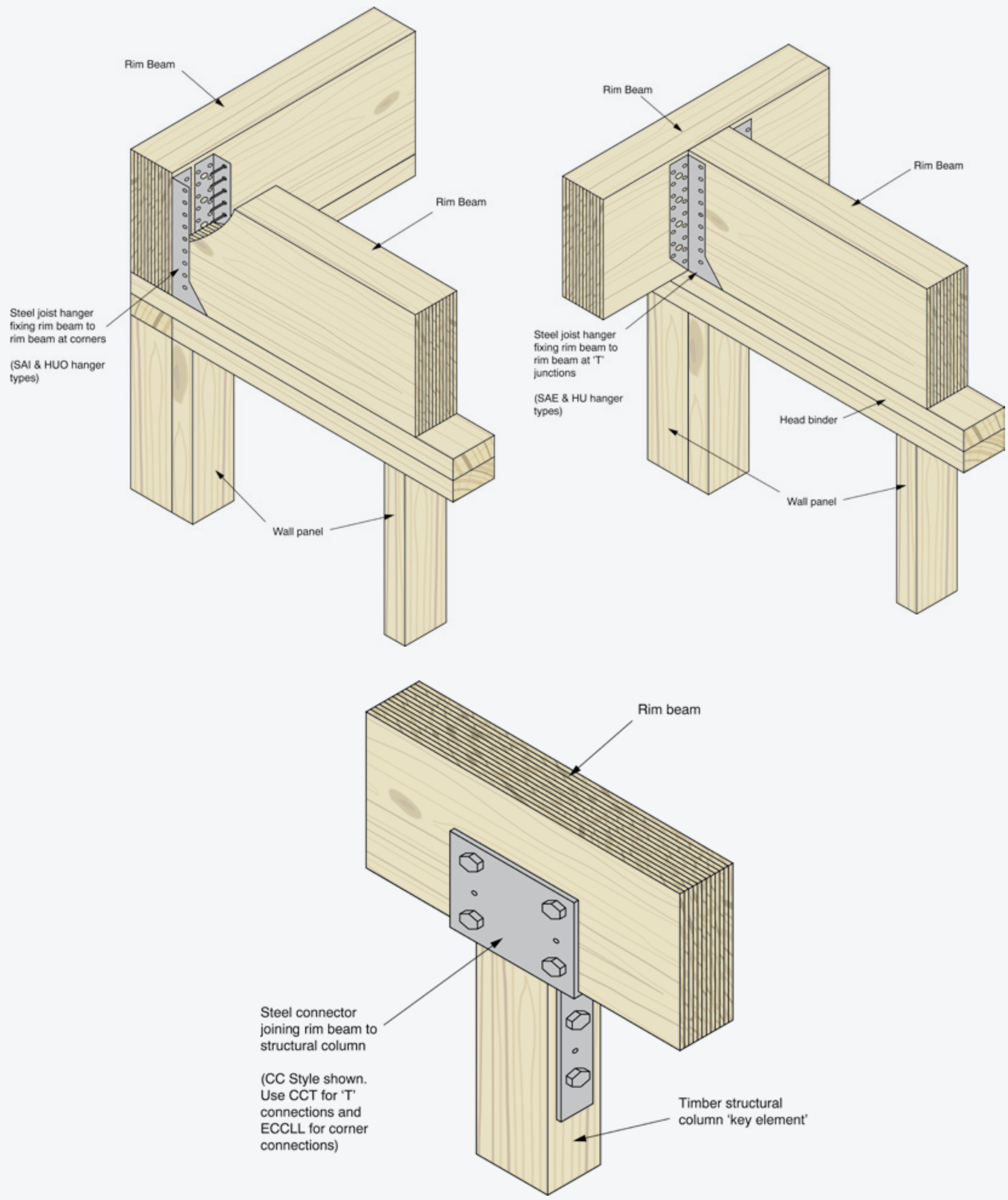


Figure 4.23: Rim beam connection details.

Transfer beams will require the same sorts of precautions as other structures and, where an accidental damage load case causes the potential loss of a transfer beam supporting more than two storeys, this may require designing as a key element. The details in Figure 4.24 show such a case.

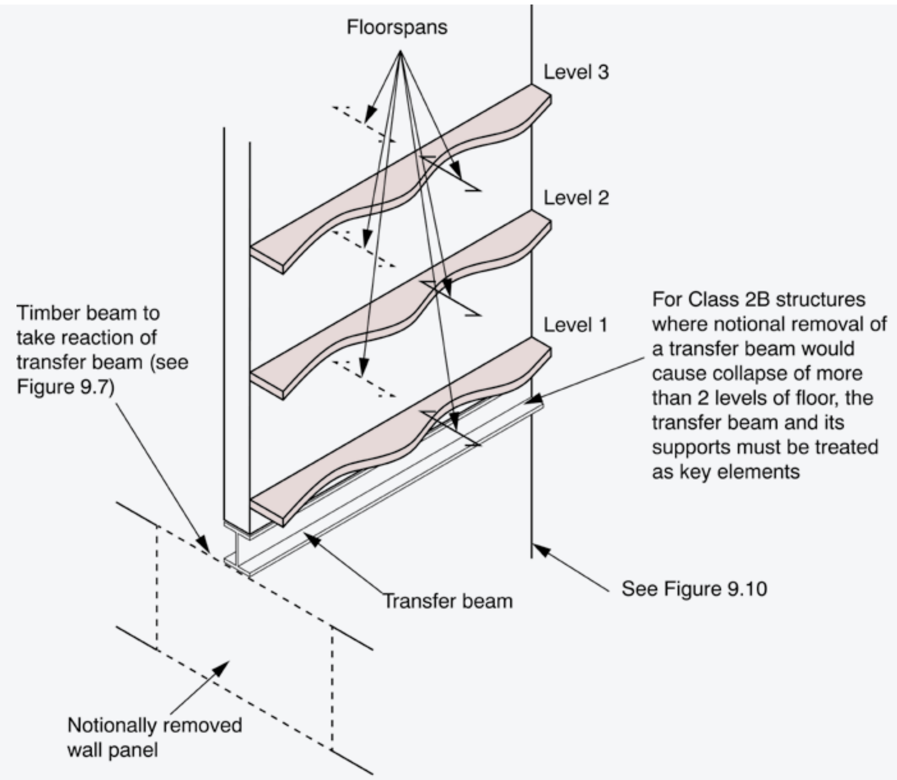


Figure 4.24: Transfer beam in timber frame multi-storey building.

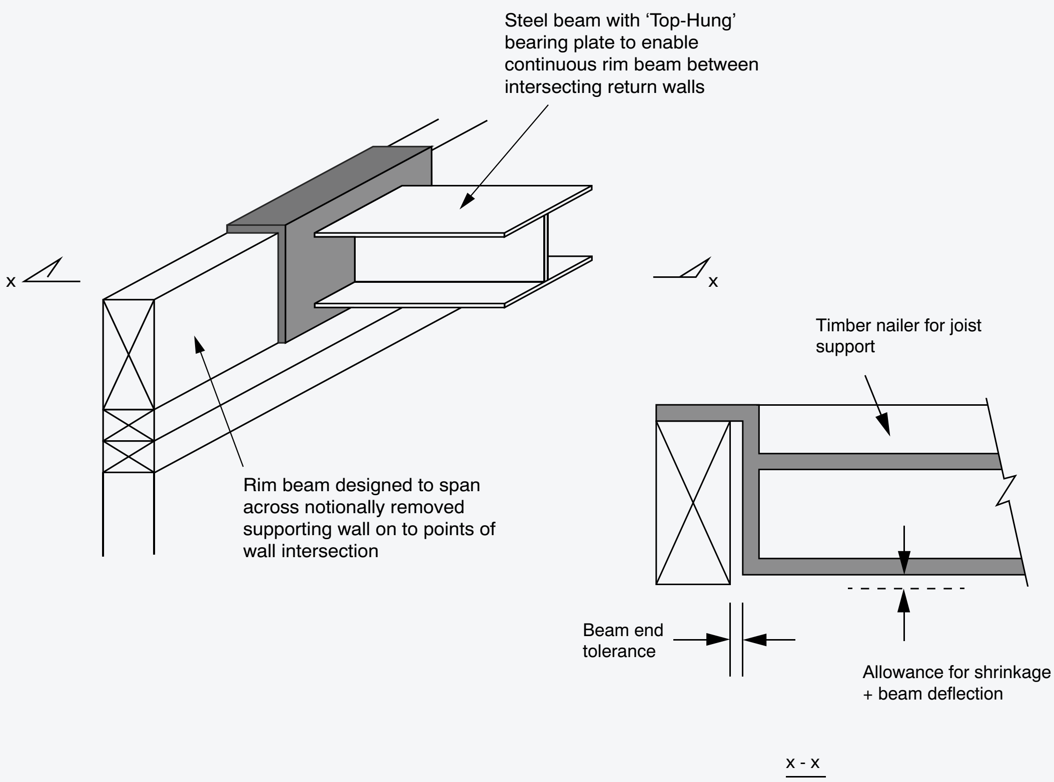


Figure 4.25: Transfer beam connection to allow rim beam to span continuously to supports.

5

Examples of Robustness in Recent Buildings

5.1 Overview

This section introduces how robustness was considered in actual built cases and reflect best practices in Australia.

5.2 Macquarie University Ainsworth Building

5.2.1 Overall view of the building

The Ainsworth Building is a teaching and learning space for the Macquarie University's Medicine and Health Sciences Faculty. It is a four-storey timber structure, comprising CLT floors and walls with glulam beams and columns on a typical 2.4 m x 15 m grid. Arup's team, made up of more than 15 disciplines, worked alongside architecture and design practice Architectus to ensure sustainability was at the heart of the development and to realise the ambitious rapid construction programme. Following the tender stage, Arup's civil, structural and acoustic teams were engaged by Buildcorp, the design and build contractor, with the firm also carrying out a peer review role on the building services design for Macquarie University.

The 3,325 m^2 facility provides students and staff with a dynamic and flexible place to work and study, and houses multiple state-of-the-art lecture theatres and team-based learning spaces. The adoption of an almost entirely mass-timber structure above ground level had multiple benefits. The structure has low embodied carbon; it was able to be fabricated off site to a high degree of accuracy; construction was quick and efficient; and the construction methods used were far quieter than those adopted for non-timber structures, meaning construction did not disturb the adjacent sensitive buildings, including Macquarie University Hospital.

The building integrates three types of engineered timber: glulam European spruce for the internal columns and beams; CLT European spruce for the internal floors, lift core and shear walls; and glulam Victorian ash hardwood for the external columns. The structural frame was formed from a 2.4 m × 15 m grid of glulam columns (800 mm × 350 mm) and beams (1,380 mm × 350 mm). The glazing mullions were aligned with the columns at 2.4 m centres, helping to minimise the depth of the edge beams and enabling the double-glazed glass façade to draw optimal daylight into the interior spaces.

Figure 5.1: Macquarie University Ainsworth Building (Credit: ARCHITECTUS (Architects), Image: Brett Boardman Photography)

5.2.2 Robustness concept and design

Robustness of the Ainsworth Building was considered by assessing element removal and substantiating alternate load paths.

In the typical floorplate, 15 m long glulam beams span onto perimeter columns spaced at 2.4 m centres. Staggered two-bay CLT floor panels were used to ensure that vertical load could be spread to adjacent glulam beams and supporting columns, in the event of column removal.

The design featured an inclined glulam W-column that supported the base of the building. Its removal was also considered. A steel beam joining the tops of the columns was designed to redistribute the vertical load in the event that a column was removed. The remaining glulam columns were assessed for this load case.

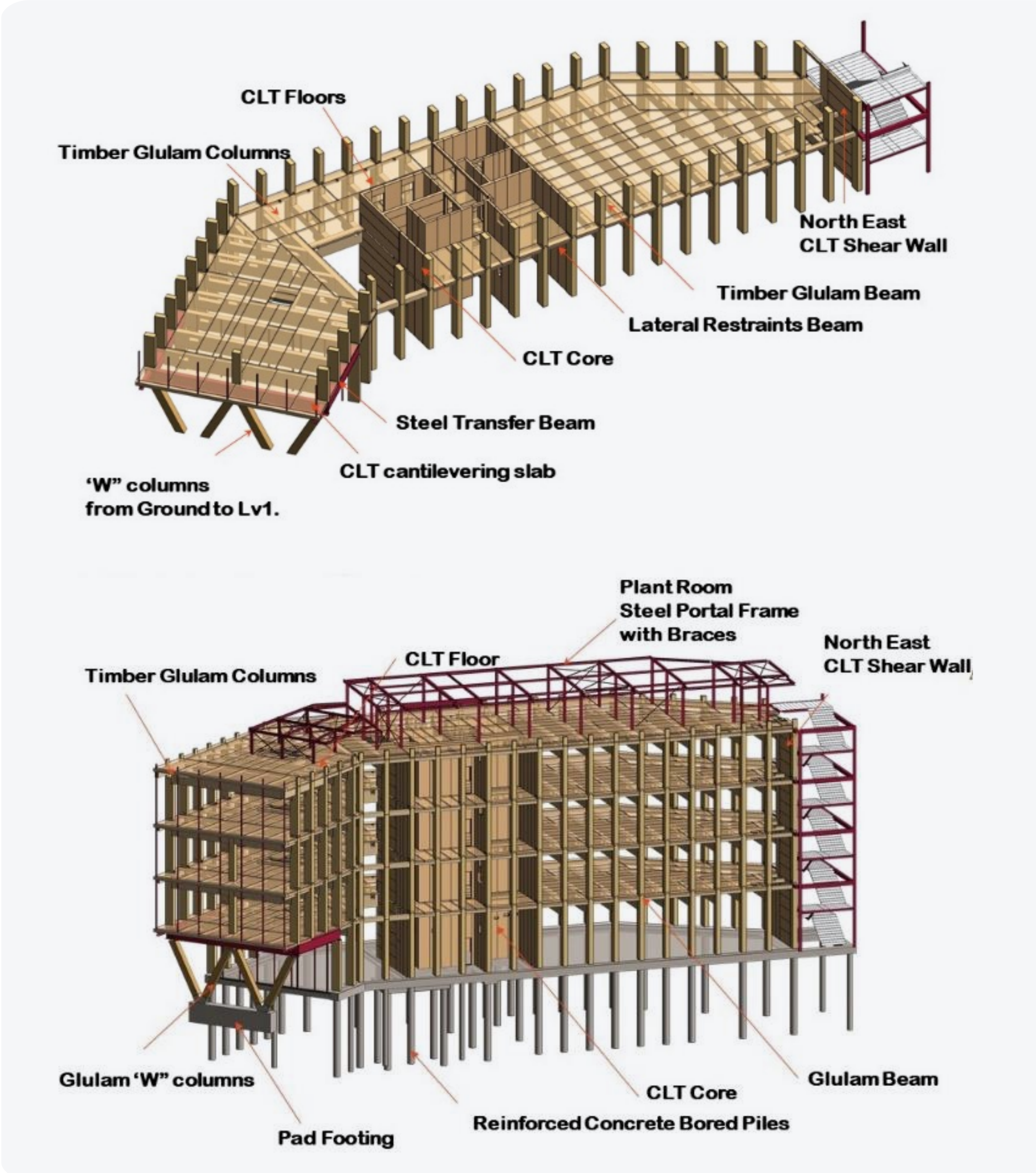


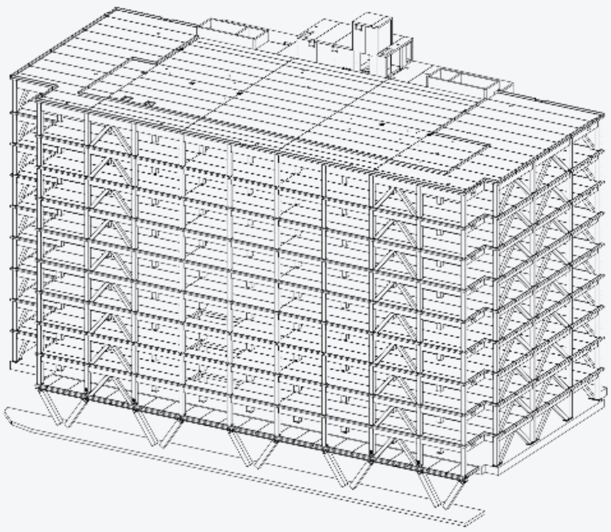
Figure 5.2: Macquarie University Ainsworth Building structural system (Image: Arup)

5.3 25 King Street

5.3.1 Overall view of the building

25 King is a commercial office building made from engineered timber, located in the heart of the Royal National Agricultural and Industrial Association of Queensland (RNA) Showgrounds, Brisbane, Australia. Designed by architect Bates Smart and with engineering services provided by Aurecon and Lendlease, the ground plus nine-storey tower includes three bespoke ground level retail tenancies. The building is an anchor to the growing King St precinct, where well-being and sustainability are a cornerstone of the planning intent for the area. The timber construction is completely visually exposed in the office space, and is expressed through the glazed curtain wall façade, complimenting the historic timber pavilions that are retained in the showground precinct. At the time of completion in 2018, 25 King St was Queensland's first 6-star green star design and as-built building, and the largest engineered timber office building in the world.

Although originally conceived in concrete, the site overlaps a key road tunnel in the Brisbane network, adding incentive to adopt a lighter weight building solution.



a)

b)

Figure 5.3: a) Timber manufacturing model, b) as-built project (Image: Lendlease)

The structure has eight full-CLT floors, with CLT walls and softwood glulam columns, beams and lateral bracing. The building uses two types of engineered timber, CLT and glulam, with both being manufactured using European spruce. The timber floors rise from a single concrete podium level, in a broadly 6 m x 8 m grid across the main floor. Along the longer edges of the building, shorter 4 m grids are used to reduce beam heights and provide simple service distribution across the floor plate. The floor plan is approximately 28 m x 60 m, and the building has an overall height of 46.8 m, providing a total Global Floor Area (GFA) of 16,446 m^2.

The structural frame was formed from a grid of glulam columns (600 mm^2 at the lower levels, 480 mm^2 at the upper levels) and beams (480 mm x 760 mm typical primary beam). The diagonal braces (320 mm x 400 mm typically, 2 bays on each elevation) provide stability for the structure. On the front elevation, a single level colonnade of raking glulam columns provide support to the floors above, as well as a striking entrance space for the building. The core is completely formed from CLT walls, as is the floor, which is double spanning across two bays of supporting glulam frame.

Site conditions permitted pre-assemblies of core wall elements to be completed safely and accurately at ground level before being lifted into final position as a single module, saving time from the main crane. This resulted in an average nine day floor cycle for a complete floor, and also enabled the façade to follow the main structure only one floor behind the leading deck.

5.3.2 Robustness concept and design

The staged approach to robustness involved verifying minimum tie forces were achieved, applying two-bay floor panels, and then further investigation into notional element removal that demonstrated that localised failure resulted in the collapse of an acceptably small proportion of the overall floor area.

The approach adopted European design codes and approaches, as no criteria is provided in the NCC. Specifically, the design process adopted minimum horizontal and vertical tie forces calculated according to EN 1991-1-7 (2006), Annex A, for Class 2b building type.

The design then investigated notional element removal. Four design cases of element removal were considered to understand the behaviour of the CLT panel and beams upon the removal of a column. The structure features glulam beams on a 6 m grid, and consequently design cases included CLT spanning 12 m or cantilevering 6 m. The CLT panels and remaining beams were shown to be satisfactory against ULS checks when subject to minimum load set out by AS/NZS 1170.0 (2002) clause 6.2.2.

The findings of these analyses were compared to European limit of 15% of the total floor area or 70 m^2, whichever is smaller.

The robustness design also considered the inclined colonnade columns that supported the front face of the building. A steel beam joining the tops of the columns was designed to redistribute the vertical load in the event that a column was removed. The remaining glulam columns were assessed for this load case. The columns are also set atop a concrete plinth, as an added robustness measure against a vehicle collision from the road adjacent.

Figure 5.4: Glulam colonnade with concrete plinth bases. (Image: Lendlease)

5.4 The Bond

5.4.1 Overall View of the Building

TTW was engaged to provide structural, timber, civil and facade engineering expertise for the construction of The Bond, located in Norwest, north-west of Sydney, NSW. This building is Mulpha's latest next-generation workplace where world-leading wellness initiatives are redefining what a workplace can be. The building provides commercial, retail and childcare spaces, and facilities for medical services with an oncology bunker within the basement and an IVF clinic on the level 2 timber floor plate. TTW was engaged throughout the duration of the project, working with architects, Fitzpatrick+Partners, and Buildcorp constructed the project. The building involved the construction of three levels of basement car parking, including the facilities for the medical bunker, then a semi-basement for the ground floor retail and parking, and a concrete podium at level 1. Above this, there are six levels of mass timber structure which then supports steel framing and concrete topping for a rooftop plant room area.

TTW was responsible for the overall structural design and documentation for the project, which included full design and detailing of the mass timber, steel, and concrete structures. The mass timber, which is supplied as European spruce, has glulam beams and columns supporting cross-laminated timber (CLT) floor panels. An 8.4 m × 8.4 m typical grid was adopted to give flexibility for leasing of the floor plate, but also minimise the transfer structures at the concrete level as the structure transitions into car parking layouts. Glulam columns support the 8.4 m span primary glulam beams, with secondary glulam beams at 2.8 m centres. On top of the beams, the CLT floor panels triple-span continuous over the top of the secondary beams.

Figure 5.5: Visualisation Render. (Image: Mulpha)

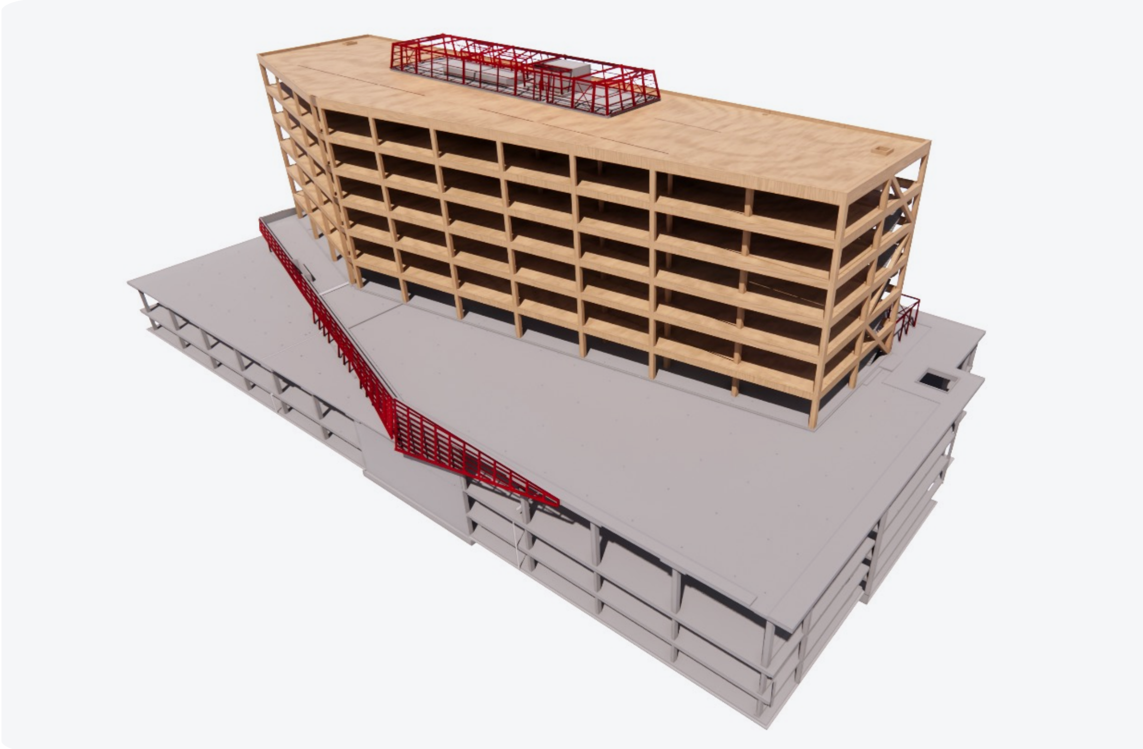


Figure 5.6: Structural Model. (Image: TTW)

5.4.2 Robustness Concept and Design

The robustness method that was initially adopted was the horizontal and vertical tie method. With continuous column lines and a regular series of beams internally and around the perimeter providing clear tie lines throughout. The initial connection details proposed allowed for sufficient rotation to occur at the joints while maintaining tie-load carrying capacity. When the timber supplier was appointed post-tender documentation, their preferred and proprietary connection type for beam-to-beam and beam-to-column interfaces would not have been sufficient to provide the horizontal tie force transmission under the displaced shape expected with catenary action. As such, the robustness strategy was changed to adopt the notional element removal method. With the longer spans, thin CLT floor panels and no floor panel continuity over primary beam lines, the removal of a beam or column would result in more than the allowable collapse prescriptive limit of 15% of the floor area or 100 m^2 in the EN 1991-1-7 (2006). As a result, the CLT floor panels, secondary beams, primary beams and their supporting columns were designed as critical high-risk components in the NCC (2019), in reference to the ABCB Hanbook (2020). The 34 kPa accidental action load under the key element design approach in the IStructE (2010) (as discussed in Section 3.6) was followed. TTW systematically applied the accidental action load onto each element and worked with the timber supplier to specify additional screw reinforcement or through-bolts at connections interfaces to resist the end reactions that resulted from the 34 kPa loads.

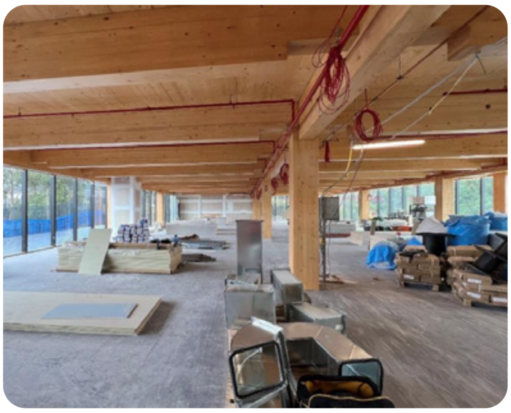


Figure 5.7: Internal Construction. (Image: TTW)

Figure 5.8: External Construction. (Image: TTW)

5.5 T3 Collingwood

5.5.1 Overall View of the Building

For your next-gen workplace, and Hines's first 'T3' project in Australia, you cannot look past the project at 36 Wellington St, T3 Collingwood.

T3 stands for timber, transit and technology – T3 Collingwood will epitomise this, bringing tenants the benefits of biophilia through its exposed timber structure, while they enjoy the dynamism of vibrant Collingwood, located less than 2 km from the Melbourne CBD.

As always for Hines, sustainability is at the core of this project – targeting 5.5-star NABERS Energy and 6-star Green Star ratings. The mass timber structure plays a big role in these sustainability credentials and is supplied locally using resources from sustainably managed forests and plantations by local manufactures Australian Sustainable Hardwoods (ASH) and XLAM Australia (XLAM).

T3 Collingwood project is set to become Australia's tallest mass timber hybrid commercial office tower standing approximately 63 m above ground level. The project, delivered by Icon as Head Contractor for Hines, is a sustainable commercial office building (21,900 m^2 GFA) designed by architects Jackson Clements Burrows and structural engineers AECOM. The building will present occupiers with the next generation of flexible and creative office space, featuring a wellness centre, seamless tech integration, and premium end-of-trip facilities that support cyclists, joggers and walkers using alternative ways to travel to work.

The new tower will consist of:

- two levels of basement parking
- ground floor retail – food & beverage and/or retail tenancies
- premium ground floor lobby and end-of-trip facilities
- PCA A grade commercial office space from L1 to L14 (L6-L14 are mass timber)
- Outdoor areas on the podium and L12 Terraces
- 5.5 NABERS
- 6 Star Greenstar.

The mass timber office spaces, with a typical 9.6 m × 5.5 m grid, are built with ASH's MASSLAM 45 glue-laminated timber (glulam) columns and beams and XLAM's cross-laminated timber (CLT) floor panels.

ASH – Australia's largest and most advanced hardwood manufacturer – has been involved in the design and supply of many of Australia's largest and most prominent mass timber projects. With the MASSLAM range, ASH offers highly attractive, high-performance mass timber solutions that are unrivalled in Australia. ASH's range of mass timber glulam is the largest, most experienced, and trusted in Australia.

XLAM is Australia's leading manufacturer and supplier of CLT. With state-of-the-art manufacturing facilities, XLAM has the capacity to supply projects of any scale, across government, commercial, and residential sectors.

Figure 5.5: Visualisation Render. (Image: Mulpha)

5.5.2 Robustness Concept and Design

With such scale, a robust structure is of paramount importance. Mass timber and pre-fabrication go hand in hand, and as such, discontinuity of discreet elements results in a heavy reliance on joint connectivity for load transfer and alternate load paths. This requires a well-considered robustness strategy. AECOM's structural team developed a pragmatic strategy for robustness/disproportionate collapse taking guidance from:

- Former edition of the present WoodSolutions *Technical Design Guide #39: Robustness in structures*
- The Institute of Structural Engineers: *Practical guide to structural robustness and disproportionate collages in buildings* (IStructE 2010)
- The Institute of Structural Engineers: *Manual for the design of timber building structures to Eurocode 5* (IStructE 2019)

The robustness strategy includes:

- Continuity in CLT floor panels, providing ALP.
- Timber-to-timber seated bearing connections that accommodate significant over-load (in ambient conditions) and rotation.
- Robustness ties in the glulam structure.
- Columns designed as key elements to resist high lateral point loads in combination with short-term gravity load combinations.

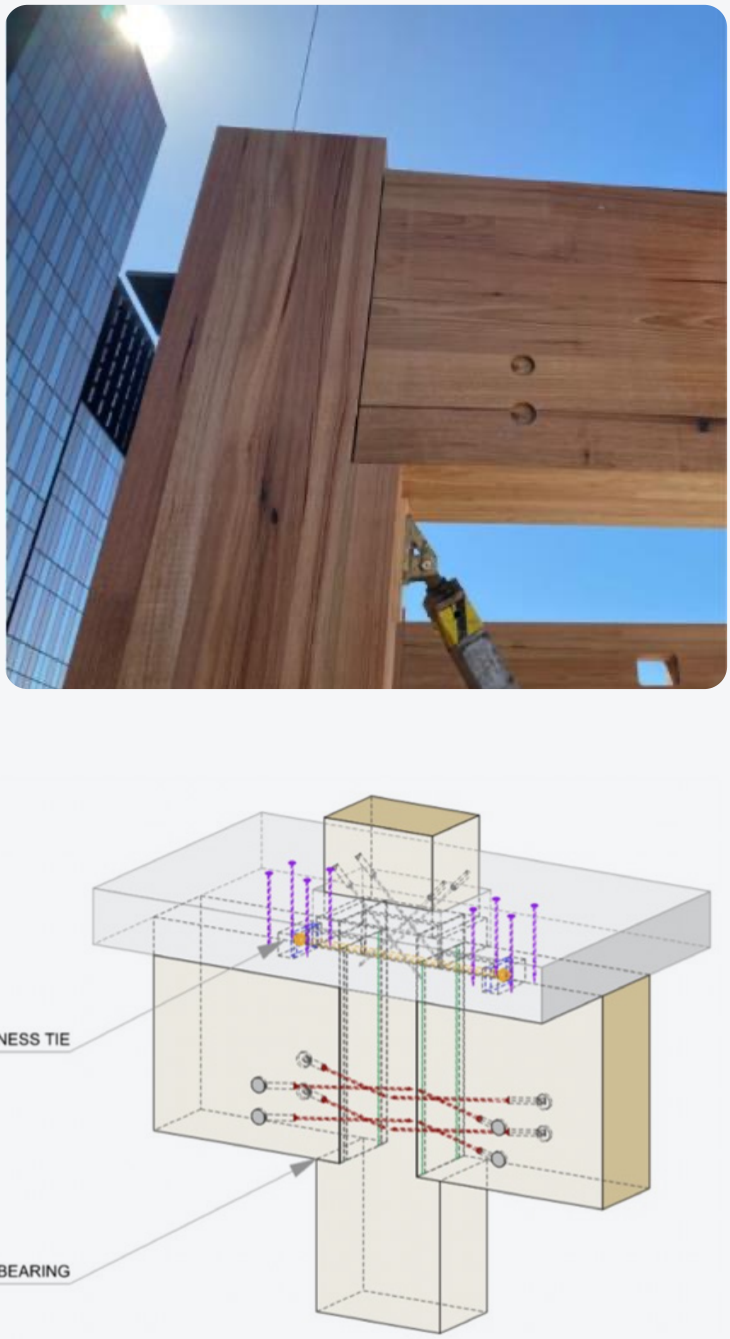


Figure 5.10: T3 Collingwood Beam-to-Column Seated Bearing Connection. (Photograph: Icon; Detail: ASH)

6

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