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Seismic Design

Chapter 11.5 | June 2020

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ACKNOWLEDGEMENTS

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Funding for the NZ Wood Design Guides is provided by our partners:

wpma Wood Processors & Manufacturers Association of New Zealand
supported by forestgrowers commodity levy
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NZ TDS New Zealand Timber Design Society
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1. INTRODUCTION

All buildings must be designed to resist lateral loads from wind and earthquakes.

Wind loads, which occur frequently, result from positive or negative pressure on all exposed surfaces of buildings during windy conditions. Earthquake loads, which occur infrequently, result from ground motions which induce vertical and horizontal forces in buildings. Seismic design normally only considers horizontal forces, because most structures have inherent resistance to vertical forces. Exceptions to this may be buildings with large cantilevered balconies.

Earthquake loads are dynamic inertial forces which increase as the weight of the building increases, and as the stiffness of the structure increases. The level of earthquake load also depends on the level of structural damping which is related to the level of ductility a structure is able to possess.

This design guide gives the background to specific engineering design of timber buildings for earthquake loads, and describes how seismic calculations for bracing can be made. This guide covers seismic design for light timber framed buildings and mass timber buildings made from laminated veneer lumber (LVL), cross laminated timber (CLT) and glue laminated timber (glulam).

Earthquake loads are obtained from NZS 1170.5 which specifies loads and load combinations for both ultimate strength and serviceability limit states design.

The final section of this design guide has been written as a commentary on the seismic design section (Section 9) of the new standard for timber design, NZS AS 1720.1.

It is intended as a companion guide to NZS AS 1720.1 as this standard represents a change from previous approaches used in timber design in New Zealand.

1.1 RECENT EARTHQUAKES

Recent New Zealand earthquakes have shown the excellent resilience of timber buildings under seismic loading. Timber houses and other timber buildings generally showed excellent structural behaviour in the 2010 and 2011 Canterbury earthquakes and in the 2016 Kaikoura earthquake.

That excellent structural behaviour was observed in light timber frames houses, timber log-style houses, and timber commercial, industrial and educational buildings.

The few timber buildings which suffered significant damage had design flaws or were located on unsuitable sites with inadequate foundations.

The excellent behaviour of timber buildings in earthquakes results from the low weight of the buildings and the flexibility of timber structures.

One equation you may remember from your high school science class tells the story:

F = M x A (Force = Mass x Acceleration)

The lower the mass, the lower the seismic forces when buildings are subjected to ground acceleration in an earthquake. Timber buildings are generally very light weight (low mass), so the seismic forces are very low.

The seismic forces are further reduced in most timber buildings because they have reduced accelerations due to their flexibility and ductility, resulting in structural damping.

In addition, the flexibility of most timber buildings allows local movements to take place in nailed or screwed connections with no danger of catastrophic collapse, unlike buildings with more brittle materials which can fail suddenly.

Figure 1: The multi-storey mass timber Kaikoura District Council building uses Pres-Lam technology as its lateral load resisting system. It was used as the base of operations following the magnitude 7.8 2016 Kaikoura earthquake.

1.2 TIMBER HOUSES

Most houses in New Zealand are built with light timber framing consisting of timber studs and joists, nailed to each other and covered with sheathing such as plywood or Gib® board. Resistance to lateral loads from wind or earthquake is provided by structural walls, which work best when they are uniformly distributed throughout the house.

In order to provide sufficient bracing, timber stud walls are covered with nailed-on sheets of semi-rigid material such as plywood or Gib® board. The advantage of such bracing walls is that they are strong and stiff, providing excellent load resistance and some ductility to reduce the seismic forces.

A major earthquake will always cause some damage. An advantage of light timber framed housing is that most damage is repairable, provided that the lateral movements are not excessive.

The largest cause of damage to Christchurch houses in the recent earthquakes was soil liquefaction and foundation failure which affected houses of all materials. Many light timber houses had inadequate concrete foundations which suffered severe damage, leading to replacement or expensive repairs, regardless of the benefits of timber construction. It is worth noting, however, that despite these failures, all timber housing achieved the objective of life safety associated with strong levels of shaking.

These observations apply not only to single storey houses, but also to similar multi-storey timber buildings as used for apartments, or hotel and motel accommodation, up to three or four storeys in height.

1.3 TIMBER INDUSTRIAL AND EDUCATIONAL BUILDINGS

Timber structures supporting industrial and educational buildings suffered no significant damage In the Christchurch and Kaikoura earthquakes. Many school assembly halls, for example, with large span glulam portal frames, had sufficient flexibility to resist the shaking with no structural damage.

Damage following the 2011 Christchurch earthquake showing the extensive liquefaction that impacted residential housing.

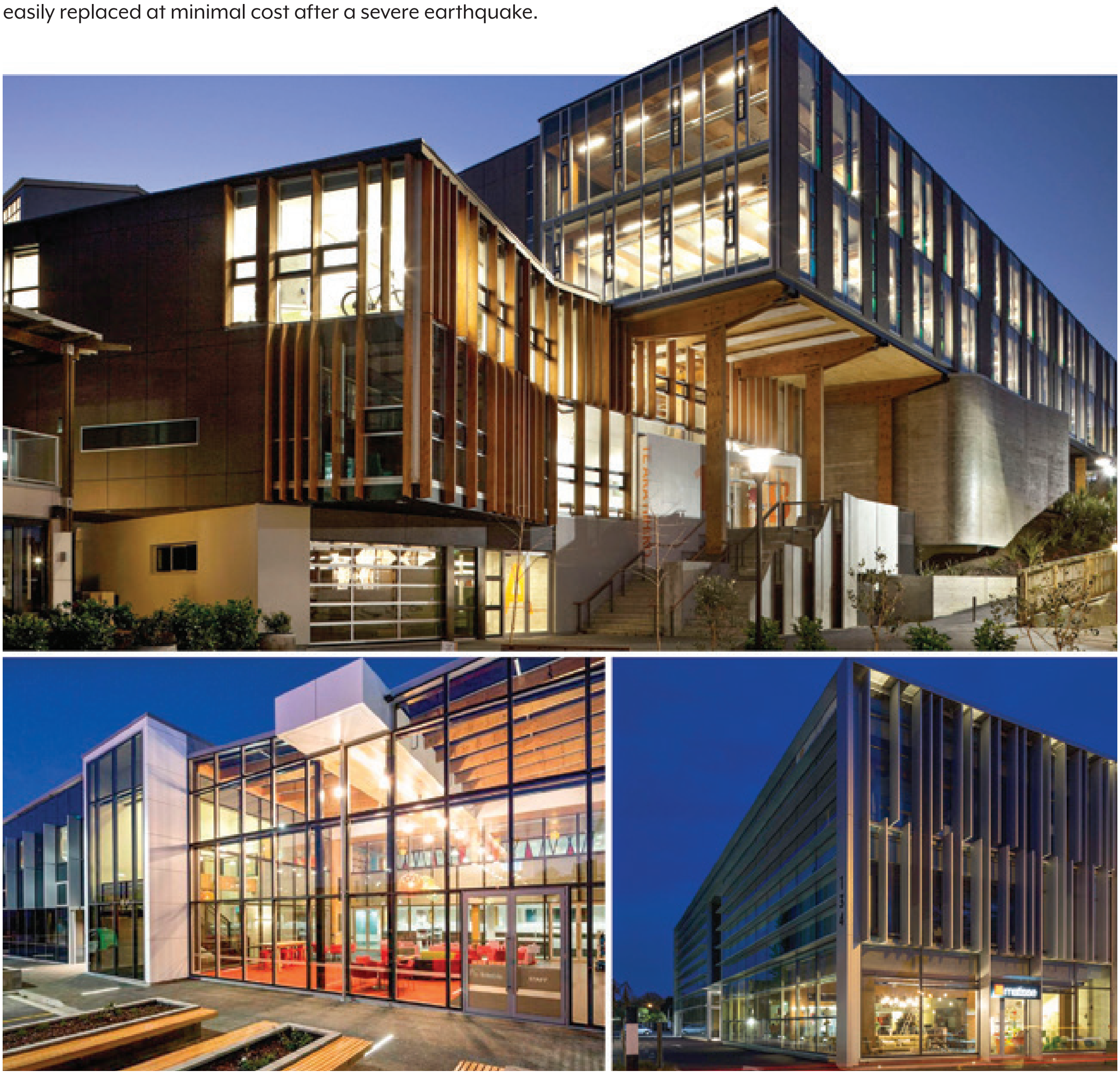
1.4 MASSIVE MULTI-STOREY TIMBER BUILDINGS

New types of engineered timber materials have been developed in recent years, leading to new forms of timber construction, often called “massive timber construction”. These new materials include Laminated Veneer Lumber (LVL) and Cross Laminated Timber (CLT), both manufactured in New Zealand and Australia.

Massive timber construction is ideal for multi-storey buildings. Innovative design methods allow massive timber to be used for tall buildings which will suffer very little damage under extreme loading conditions. Large residential buildings, often made from CLT, tend to have a large number of walls, providing excessive bracing and excellent performance.

Large commercial and educational buildings tend to have fewer bracing walls, so these have to work much harder. However there are still numerous methods to achieve the lateral load performance. One of these is a post-tensioned timber building technology, known as “Pres-Lam” which has been used in the design of recent multi-storey timber buildings including the three storey NMIT building in Nelson, the CoCA building at Massey University in Wellington, Young Hunter House and the Trimble building in Christchurch.

All of these buildings have first class earthquake resistance. The design and construction incorporates ductile “fuses” which can reduce acceleration by absorbing energy during a major earthquake. These fuses can be designed to be easily replaced at minimal cost after a severe earthquake.



(Clockwise from top) CoCA building at Massey University, Young Hunter House and Trimble Building.

2. LATERAL LOAD RESISTING SYSTEMS IN TIMBER

Lateral load resisting systems for buildings consist of vertical and horizontal elements which must be designed to transfer lateral loads to the foundations as shown in Figures 1 and 3. The most common lateral load-resisting elements, as shown in Figure 2 and summarised in Table 1, are:

- Moment resisting frames.
- Vertical cantilevers.
- Diagonal bracing.
- Shear panels.

There are other possible systems including solid timber walls acting as rocking panels. The lateral load resisting system will be selected on the basis of cost, convenience, efficiency and common sense, often after comparison of cost and feasibility.

In addition to the ductility considerations above, NZS1170.5 uses a Structural Performance Factor to reduce siesmic demand. For more information on the use of this factor in timber buildings refer to Section 8.6.1 8.

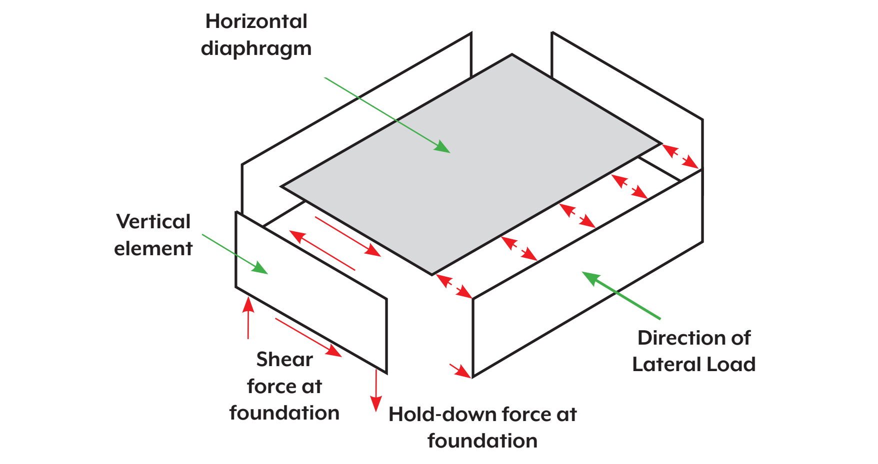


Figure 1. Elements of a lateral load-resisting system.

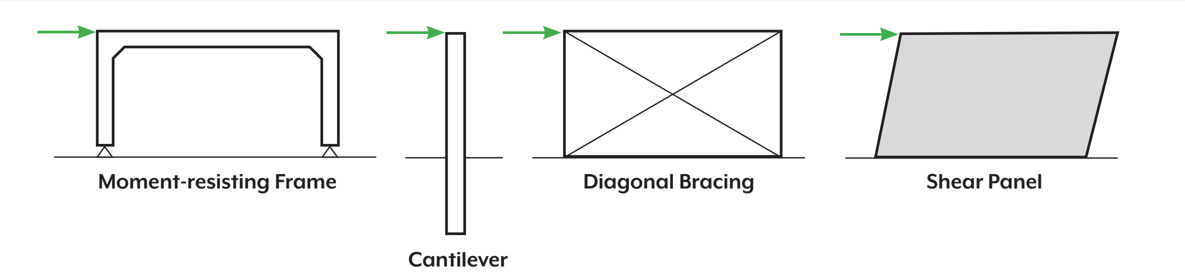


Figure 2. Types of a lateral load-resisting system.

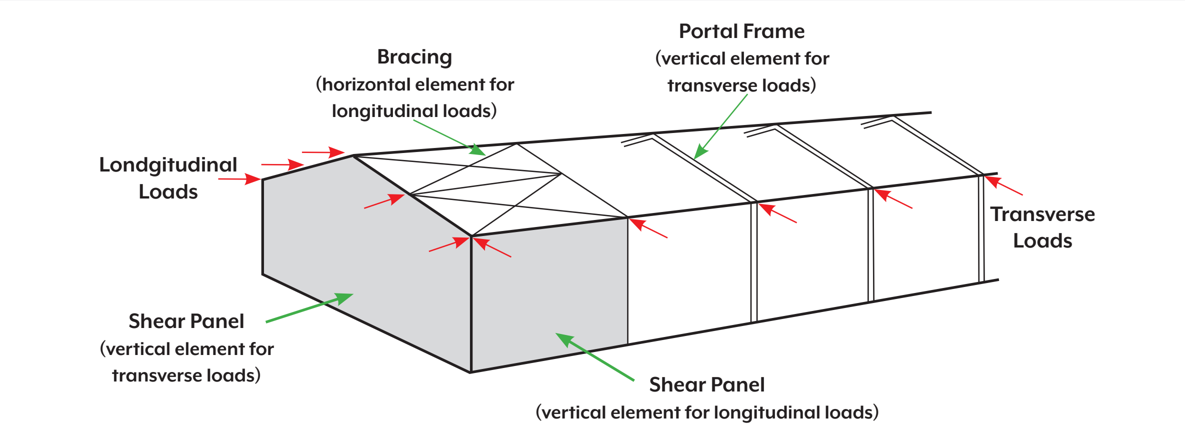


Figure 3. Typical industrial building using a combination of lateral load-resisting elements .

2.1 CANTILEVERS

A cantilever post fixed rigidly into the ground is the most simple system for resisting lateral loads. Timber poles are often used as cantilevers in single storey industrial or agricultural buildings, to support roof loads and to resist lateral loads.

Cantilevered poles are typically not capable of inelastic demand and are classed as brittle structures. More information on this type of structure can be found in Section 8.2.1.

2.2 DIAGONAL BRACING

Diagonal bracing is often the most economical vertical element for a lateral load resisting frame system. The bracing is located in the plane of a frame. The diagonal members may be designed to act in tension or compression or both, to form a vertical truss. Forces are hence transferred through a small number of members and their connections to the foundations. All parts of the vertical truss must be capable of carrying the imposed loads through the entire load path.

2.3 VERTICAL SHEAR PANELS

Vertical shear panels are walls which provide resistance to racking forces. If walls are required for other reasons they are often used for lateral load resistance. In light timber frame timber wall systems, shear panels consist of flat sheets of panel product fixed to timber framing. Shear forces are transferred through a large number of fasteners (nails, staples or screws) at all the panel edges. Overturning moments are resisted by the studs at the ends of the walls. The effects of window or door openings in the shear panels must always be considered.

Vertical shear panels can also be manufactured from solid timber such as LVL, CLT or glulam.

Vertical shear panels must be securely fixed to the foundations to prevent uplift and overturning.

Nominally ductile and limited ductile structural walls must be designed to form ductile failure mechanisms.

The design actions in walls must consider:

- All overstrength actions generated by Potential Ductile Elements (PDEs).
- Dynamic magnification factors for flexure and shear.

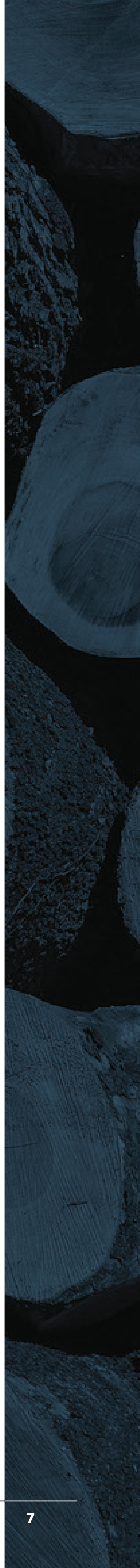
Distributed ductility in walls can be provided by metallic fasteners connecting the sheathing panels to the timber framing or concentrated in the connection at the wall-to-foundation interface, or in distributed ductile connections between panels. Yielding of hold down brackets or their bolted fasteners which resist both overturning and shear actions should be avoided.

2.4 MOMENT-RESISTING FRAMES

Moment-resisting frames are not often used in timber buildings because they often have inadequate stiffness to control deflections. They are most often used in single storey industrial buildings where they resist vertical loads from the roof as well as lateral loads. Moment-resisting frames can be expensive because of the high cost of moment-resisting connections.

In order to prevent a soft-storey collapse, nominally ductile and limited ductile moment-resisting frames must be designed to form a beam sidesway failure mechanism, except at the roof level. The design actions in columns must consider:

- All overstrength actions generated by PDEs.
- Dynamic magnification factors for flexure and shear (See Section 8.6.5).



Displacement compatibility is also important. Where elastic connections or elements are included in a moment-resisting frame system these connections or elements should also be checked to ensure that they can displace with the frame at the desired structural ductility level without significant loss of lateral strength or gravity load capacity.

2.5 USE OF PROPRIETARY INFORMATION AND PRODUCTS

Although not listed in the summary above, it is becoming increasingly common in design for an engineer to specify the use of a proprietary systems to resist seismic loading and/or provide inelastic response. Care should be taken by the designer when specifying these systems and although not exclusively limited to timber the following questions should be asked:

- Has the system been adequately tested within the context of this design or loading?
- If sourced from overseas, do design procedures or testing protocols fit within the framework of the New Zealand building code?
- What certification will be supplied to show compliance?
- What certification will be required for consenting?
- Who will be supplying any certification required?
- Are there any specific maintenance requirements that I need to make the building owner/occupier aware of?

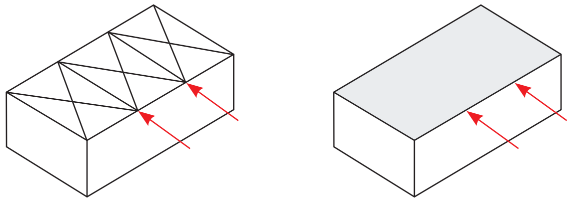
2.6 DIAPHRAGMS

All buildings require some form of horizontal diaphragm at each floor level and at roof level. The horizontal elements of a lateral load resisting system are typically either diaphragms or diagonal bracing, as shown in Figure 4. This also applies to sloping roofs which may not be exactly horizontal. Diaphragm design is not specifically covered in this guideline however they generally fall into two categories:

- A diaphragm fitted to a lateral load resisting system that is designed to respond elastically. In this case, some yielding may be appropriate for low rise buildings through careful design and special study.
- A diaphragm fitted to a lateral load resisting system that is designed to respond in a ductile manner. In this case, the diaphragm should be designed with overstrength/pESA forces from NZS 1170.5 and remain elastic. Limits on capacity design actions from Clause 9.3.5.7 can be applied to the pESA forces since, although designed elastically, a timber diaphragm can be designed with ductile fasteners capping design forces. To ensure ductile capacity however, it may be appropriate to provide an additional level of protection to potentially brittle members in the diaphragm.

There is the possibility for low rise construction that both the diaphragm and the lateral load resisting system are designed as nominally ductile however this should be done with care through special study.

Diaphragms are often one of the more challenging aspects of mass timber seismic design. Care should be taken with load transfer around openings and the avoidance of brittle failures along key load paths. Additional checks are also required to ensure that minimum spacing is adhered to in fasteners that are often being asked (i.e. in absence of dedicated chord beams) to act in shear at an angle to the grain.



Diagonal Bracing   Diaphram

Figure 4. Horizontal elements of a lateral load-resisting system

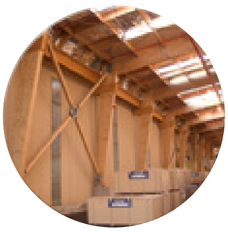
2.7 SUMMARY

The most common types of vertical elements in lateral load resisting systems for timber buildings, in rough order of increasing load capacity, are:

- Vertically cantilevered posts.
- Light metal diagonal bracing.
- Sawn timber diagonal bracing.
- Structural steel diagonal bracing.
- Moment resisting frames.
- Light timber frame walls with gypsum plasterboard sheathing as a shear panel.
- Light timber frame walls with additional plywood sheathing as a shear panel.
- Solid timber shear walls as a shear panel.

The relative costs depend greatly on specific building designs. For example, moment resisting frames may be the most cost effective lateral load resisting system if they are already installed to resist gravity loads. Diagonal tensile bracing is often an inexpensive option, but such systems generally have very little ductility because they become “sloppy” when yielding occurs in tension-only members. Shear panels may be the most cost effective if they use walls which are required for other reasons, either with light timber framing or solid timber panels. Table 1 gives a tabular comparison of the main alternative bracing elements.

Table 1 provides a wide range of solutions, some of which will only be efficient to a certain height and others that will not be efficient in some situations. It should also be noted that while not listed in the table below a timber gravity frame can be used in combination with a steel or concrete lateral load resisting system.



	STRENGTH	DUCTILITY	CONNECTIONS	ADVANTAGES	DISADVANTAGES
Vertically cantilevered posts.	Brittle fracture can occur in post.	No ductility.	Embedment in ground with or without concrete footing.	Inexpensive, simple.	High deflections, limited capacity, brittle failure.
Light steel strap bracing* (tension only)	Brittle fracture can occur at nail holes.	No ductility.	Nails at ends. End of strap may be folded over timber for more nails.	Available in long lengths. No cutting in. Can be tensioned.	Limited capacity. No compressive capacity.
Light steel angle bracing* (tension and compression)	Angle may have some compression strength.	Very limited ductility.	Nails at ends.	Remains straight. No tensioning required.	Limited lengths. Cut in to studs. Poor in compression.
Timber bracing (tension only)	Tensile strength depends on connections.	No ductility.	Nails, bolts, tooth plates or nail-on plates.	Remains straight. No tensioning required.	Bulky, difficult to position neatly May fail by buckling.
Timber bracing (tension and compression)	Compressive strength may depend on buckling or connections.	Ductile only if connections are ductile.	Nails, bolts, tooth plates or nail-on plates.	Remains straight. No tensioning required.	Connections may be difficult.
Structural steel bracing with rods, flats or angles (tension only)	High strength if connections are strong.	Very limited ductility.	Welded to nailplates or drilled through timber.	Long lengths available.	Tearing of holes. Can become heavy and difficult to fix.
Structural steel bracing with solid/hollow sections or buckling restrained braces (tension and compression)	High load capacity with strong connection Work in tension and compression.	Potenial for high levels of ductility.	Plates and fasteners typically designed as CPEs.	Can be placed on facade and still allow glazing. High stiffnees allows for smaller frame sections with not all bays braced.	Connections can be costly and a limiting design factor.
Thin steel cladding	Strength usually ignored.	No ductility.	Fixed to timber framing with nails, screws or clips.	Required as cladding material.	Low stiffness.
Glulam or LVL frames	High flexural strength.	Ductile with appropriate connections.	Steel or plywood gusset plates with nails Epoxied steel rods.	Little obstruction to open spaces.	Low stiffness.
Gypsum plasterboard panels*	Moderate load capacity.	Good ductility in nails or screws from plasterboard to framing.	Nails or screws at perimeter of all sheets.	Required as lining material. Can be fixed with screws.	Not as ductile as plywood.
Plywood panels	Large load capacity.	Excellent ductility in nails from plywood to framing.	Nails at perimeter of all sheets.	Other wood-based panels may also be used.	More expensive than gypsum plasterboard. Typically screws cannot be used.
Solid timber walls	Very large load capacity.	Specifically designed yielding elements.	Steel hold-downs or Pres-Lam.	Suitable for tall open-plan buildings.	More expensive than light timber framing.

Table 1. Comparison of main types of bracing elements.

* Proprietary products. Consult manufacturers' literature

3. WIND LOADS

Structural loading codes specify that the wind speed at each building site be determined on the basis of location, ground roughness and building height. A basic wind pressure is obtained from the wind speed, then the design pressure for each part of the building is determined using pressure coefficients for different building shapes. Wind loads are specified in AS/NZS 1170.2.

4. EARTHQUAKE LOADS

Modern structural loading codes specify several different methods of calculating seismic forces. The design guidance in this document is based on lateral forces obtained from the equivalent static method. More sophisticated design using modal analysis, time-history analysis, or displacement-based design will require specialist attention on a case by case basis. Seismic loads are specified in NZS 1170.5. Additional information on displacement based methods can be found in Section 4.5.1.

In the equivalent static lateral force method, the seismic forces are proportional to the weight of the building, dependent on the earthquake zone, the stiffness of the building, and the structural damping available.

4.1 LOAD COMBINATIONS

Structural load combinations are very important for seismic design, because the strength and behaviour of the structure can be greatly affected by the amount of permanent (gravity) and imposed (live) live load acting at the time of the design event. The "seismic live load" must be included in the mass at each floor level, for calculation of seismic mass. The overturning moment and the hold-down forces resulting from loading will depend greatly on the gravity loads acting on the structure. Gravity loads and load combinations are specified in AS/NZS 1170.1.

4.2 CAPACITY DESIGN

The capacity design process requires that structures be designed so that ductile yielding takes place in specially designated parts of the structure, with all other parts having sufficient capacity to prevent inelastic deformations or structural failure. In timber structures, ductility is usually obtained from non-linear behaviour of steel components in connections, with these connections being designated as the weakest part of the structure. It is unusual to design timber structures for high levels of structural ductility because elastic deformations normally reduce the amount of ductility which can be provided by the individual structural components.

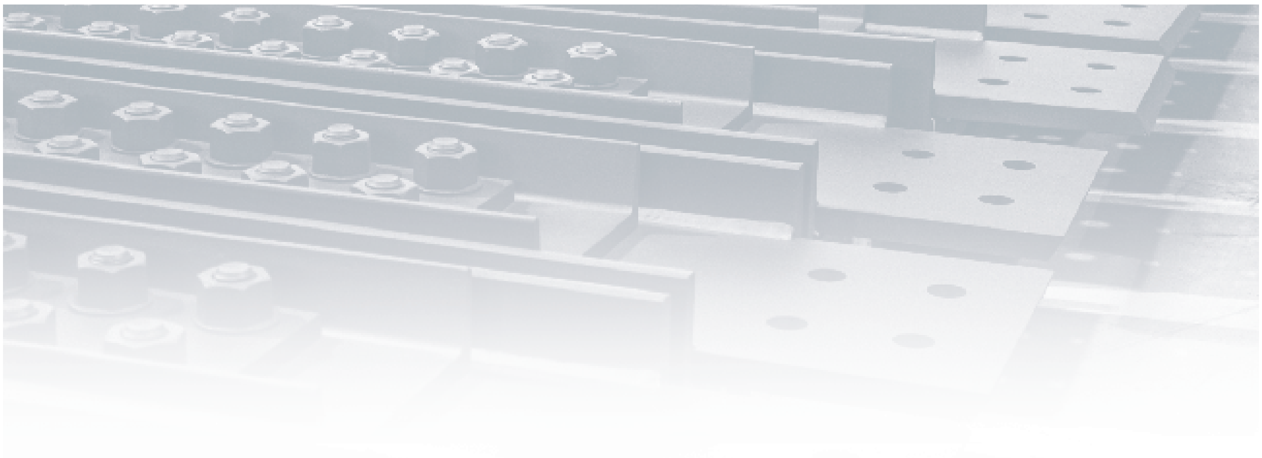
In NZS AS 1720.1, ductility is to be provided by using Potential Ductile Elements (PDEs) which are connections or components designed for inelastic or ductile response under ULS earthquake actions. Examples of PDEs include:

(a) Flat head nailed connections.

(b) Mild steel elements, sections, bolts, rods or bars.

(c) Connections shown by test or special study to achieve the required ductility under design displacements.

Capacity design is used to ensure ductile behavior of the structure. Capacity Protected Elements (CPEs) are designed to ensure the demands they experience are limited to the predicted overstrength of the associated PDEs, even at large lateral displacements.



4.3 DEFINITION OF DUCTILITY IN ELASTO-PLASTIC SYSTEMS

Figure 5 shows the idealised relationship between the total lateral force, P, applied at the top of a structure and the lateral displacement Δ for two different design strategies. The straight line shows elastic behaviour, and the bi-linear line shows inelastic behaviour. If the structure responds elastically to a given level of horizontal ground acceleration, it will be subjected to a maximum lateral displacement Δ_e which corresponds to a static lateral force of P_e on the structure. If the structure has been designed as an idealised elasto-plastic ductile structure which yields at a displacement Δ_y corresponding to a force P_y the same earthquake will subject it to a lateral displacement of Δ_d and the structural displacement ductility factor μ is defined as:

$$\mu = \Delta_d / \Delta_y$$

In most cases the two displacements Δ_e and Δ_d are similar. If they are equal, the lateral force P_y is related to the elastic response force P_e according to:

$$P_y = P_e / \mu$$

The area under the P vs Δ plot in Figure 5 is a representation of the amount of energy absorbed in the structure during the earthquake.

Considerations around the appropriate level of structural displacement ductility for timber structures is provided in Section 8.2.

It is important to understand that the structural ductility factor, μ, is the “global” ductility factor for the whole structure.

In order to achieve a certain value of global ductility, some individual components may have to achieve a much higher “local” ductility, depending on their geometry and location in the structure.

4.4 DAMPING AND DUCTILITY

The level of structural damping affects the seismic forces on a structure. Structural damping is most often provided by mild steel elements which are designed to yield, producing hysteretic damping. Ductile steel elements include multiple steel nails in plywood shear panels, hold-down devices designed for yielding, or ductile diagonal braces such as BRBs. Other possible forms of damping, not often used, include friction damping and viscous damping.

The benefit of increased damping is reduced seismic response of the dynamic system, hence reduced seismic actions in the structure. Damping is usually quantified as a percentage of critical viscous damping.

Note that because of the low mass of most timber buildings it is unusual to design for high levels of ductility. If ductility is needed, it is provided in connections in the vertical elements of the lateral load resisting system, not in the floor diaphragms or their connections.

The benefit of ductile design is that the building can be designed for lateral forces less than those required for elastic response. The disadvantage of ductile design is that unless steps are taken to avoid it, the ductile response may lead to damage that is uneconomical to repair.

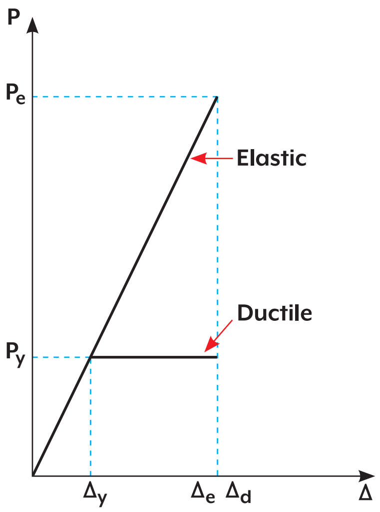


Figure 5. Lateral load vs displacement for elastic and ductile structures.

In addition to the ductility considerations above NZS1170.5 uses a Structural Performance Factor to reduce siesmic demand. For more information on the use of this factor in timber buildings refer to Section 8.6.1 of this guide.

4.5 THE CALCULATION OF BASE SHEAR FORCE IN TIMBER STRUCTURES

The calculation of the base shear force is a critical step in the seismic design of a timber structure. Several methods are available to designers, at differing levels of complexity. NZS 1170.5 refers to the following three methods for the calculation of base shear; the equivalent static method, the modal response spectrum method and numerical integration time history analysis. These are all force-based methods. The rules surrounding the application of these force-based methods are outlined in Section 6 of NZS 1170.5. Section 4.5.1 below provides a short overview of the force-based methods and Section 4.5.2 discusses displacement-based methods. Displacement-based methods can be offered as an Alternative Solution to show compliance with the New Zealand Building Code. These have some advantages over the force-based methods, especially in timber design.

4.5.1 Force based design methods

The most simple and common method for the calculation of the base shear force is the equivalent static method. This method uses two assumptions, the equal energy rule and the equal displacement rule to quantify the effect of period elongation, due to the sudden change in stiffness created by yielding, and energy dissipation from inelastic material behaviour. The applicability of these rules depends on the first period of vibration of the structure. For more information on the equal displacement and equal energy rules and their relationship to NZS 1170.5 refer to Dhakal (2011).

A feature of the equivalent static method is that a designer must first estimate the initial period of vibration of the building before calculating the base shear. This creates a design loop where the stiffness of a yet-to-be-designed structural system needs to be known in advance. Several 'rules of thumb' can be used for the calculation of initial period to ensure that the first estimate of base shear is roughly correct. One such rule of thumb can be found in the BRANZ guide, "Multi-story light timber-framed buildings in NZ: Engineering design for light timber frame buildings". The equivalent static method also requires the design engineer to select an appropriate amount of global ductility that the building will possess under severe cyclic loading. The selected design-level of ductility must be carefully checked to ensure it matches with the ductility capacities of the potential ductile elements (PDEs) in the final building design.

Clause 6.1.3.1 of NZS 1170.5 sets out the limitations of the equivalent static method, which is generally applicable for squat regular structures. Clause 6.1.3.2 establishes the modal response spectrum method as a possible design method in some cases where the equivalent static method is not appropriate. In the modal response spectrum method, the peak responses of enough modes to make up 90% of the modal mass of the building are combined to find member demand. In theory, this method has the advantage of providing a better representation of demand for irregular buildings. It should be noted however that the modal response spectrum method provides an envelope of design actions that may occur at different times during the earthquake shaking. The challenges created by this are described in Clause C6.3.1 of the commentary to NZS 1170.5. The modal response spectrum method also relies on a design engineer selecting the amount of global ductility for the building.

The only design method that does not have any limitations placed on it by NZS 1170.5 is numerical integration time history analysis. This method of analysis is complex and should only be used by design engineers who are confident in structural dynamics and have in-depth knowledge of the likely performance of the structural system which they are modelling. See Clause C6.3.1 of the commentary to NZS1170.5 for further considerations. The analysis typically requires a three-dimensional structural model, and needs consideration of hysteretic performance in the PDE. This can be particularly challenging with typical timber fastenings that display pinched hysteretic response. Testing of the PDE may be required to provide confidence in the model parameters used.

When used correctly, time history analysis is a powerful tool that will provide the engineer with excellent insight into the likely structural performance. For this method the design engineer will need to make an assumption about the amount of elastic damping available. Elastic damping is used to introduce damping which is not captured by the hysteretic model. This damping has several sources, the most important of which is the typical simplification that the hysteretic model has a perfectly linear response in the elastic range. Damping can also result from impact, deformation of foundations, and the interaction between structural and non-structural elements. A value of elastic damping of 5% is used in the definition of the elastic site spectrum in NZS 1170.5 and can be used for the time history analysis of timber structures.

For more information on response spectrum analysis and numerical integration time history analysis refer to Chopra (1995).

4.5.2 Displacement-based design methods

As discussed in Section 4.5.1, where the equivalent static method or modal response spectrum method is not appropriate, NZS 1170.5 leaves numerical integration time history analysis as the only method for the calculation of the base shear force. Time history analysis, although a powerful tool in design, requires a level of expertise in its application and can be time consuming and therefore costly to implement. Alternative displacement-based methods are Displacement Based Design (DBD) or Acceleration Displacement Response Spectrum (ADRS) analysis.

The fundamental different result of using a displacement-based method rather than a force-based method is a structure that would **achieve**, rather than be **bounded** by, a target performance limit state.

DBD is described in detail by Priestley et al. (2007). This book describes the application and foundation of the DBD method, but it is relatively light on its application to timber structures. More information on the displacement-based design of light timber framed buildings can be found in Filiatrault and Folz (2002). In general, the basic principles described in Filiatrault and Folz can be extended to other timber systems.

ADRS analysis is a non-linear static procedure which compares a pushover curve, converted from a multi-degree of freedom system into a single degree of freedom system, with the design spectrum. The performance point is then found at the intersection between the pushover curve and the design spectrum. The design spectrum is updated for the equivalent viscous damping at this performance point and iterated until convergence. For more information on the ADRS analysis method refer to Chopra, A. K. and Goel, R. K. (1999). The use of ADRS analysis often requires non-linear static modelling or the use of test results to establish the hysteretic damping capability of the structural system.

For guidance on the principles of using of displacement based design methods within the framework of the New Zealand Building Code, refer to the NZSEE “Guideline for the Design of Seismic Isolation Systems for Buildings” or “The Seismic Assessment of Existing Building: Technical Guidelines for Engineering Assessments.”

4.6 DUCTILITY OF CONNECTIONS

Because wood is inherently brittle, most ductility in timber structures is obtained by designing ductile steel connections. The amount of local ductility available in any connection depends on its type and geometry, and the level of displacement relative to the yield displacement. It is essential to have stiff and strong connections of the yielding devices to timber elements because flexible connections will not allow the required ductility to be activated before the structure reaches its ultimate displacement limit.

4.6.1 Effect of displacement

Consider the simplified load vs. displacement plot for a single fastener (a nail or a bolt) shown in Figure 6. This is the graph which would result from *monotonic* loading to failure, with no load reductions or load reversals. The initial linear elastic section of the graph is followed by a curve into an eventual yield plateau, and failure shown by the dotted line.

Depending on the definition of yielding, the yield load P_y is associated with the yield displacement Δ_y and the ultimate displacement is Δ_u and the available local ductility μ_c of the connection is

$$\mu_c = \Delta_u / \Delta_y$$

The available ductility will depend on many factors such as the end and edge distances of the fastener, and the withdrawal resistance.

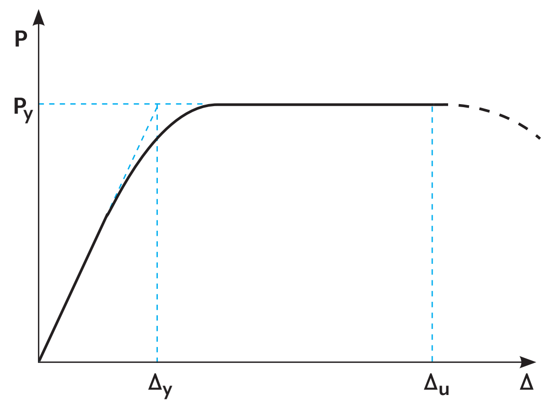


Figure 6. Idealised plot of load vs. displacement for a single fastener (a nail or a bolt).

Figure 7 shows how the local ductility will be utilised during reversed cyclic loading at various displacement levels. The three graphs show how the shape of the hysteresis loop changes as the displacement is increased. The area inside the loop is a representation of the amount of energy absorbed through hysteretic damping in the connection during the earthquake. Large displacements result in large energy absorption, but a problem with large loops of this shape is the large residual displacements when the load drops to zero.

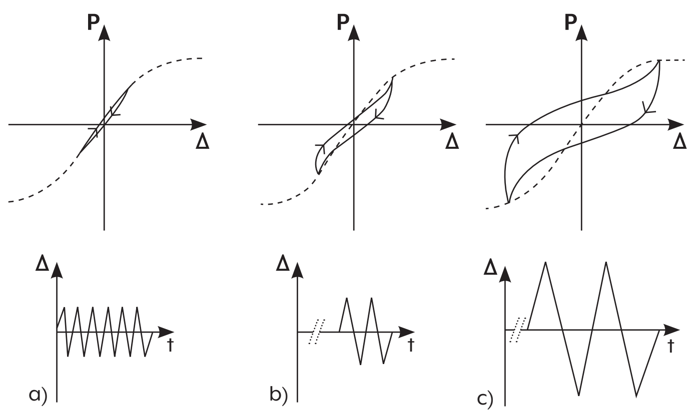


Figure 7. Hysteresis loops for cyclic loading of a fastener at various levels of displacement

4.6.2 Effect of connection geometry

Figure 8 shows the effect of connection geometry on the shape of the hysteresis loops. Figure 8(a) shows that small diameter steel fasteners develop ductility by flexural yielding of steel with only a small amount of wood crushing. Figure 8(b) shows the hysteresis loop for a system with crushing of wood fibres and no steel yielding. This results in a very "sloppy" connection (similar to a diagonally braced structure with tension-only members yielding). Figure 8(c) shows a connection relying for ductility on flexural yielding of steel, producing desirable fat loops, but again with large residual displacements when the load drops to zero.

New types of structural timber systems, such as Pres-Lam or the incorporation of timber elements and special dampers, are being developed to reduce residual displacements after an earthquake, based on prestressed steel tendons, which remain elastic for the duration of the earthquake and pull the building back to the original position, resulting in a flag-shaped type of hysteresis loop.

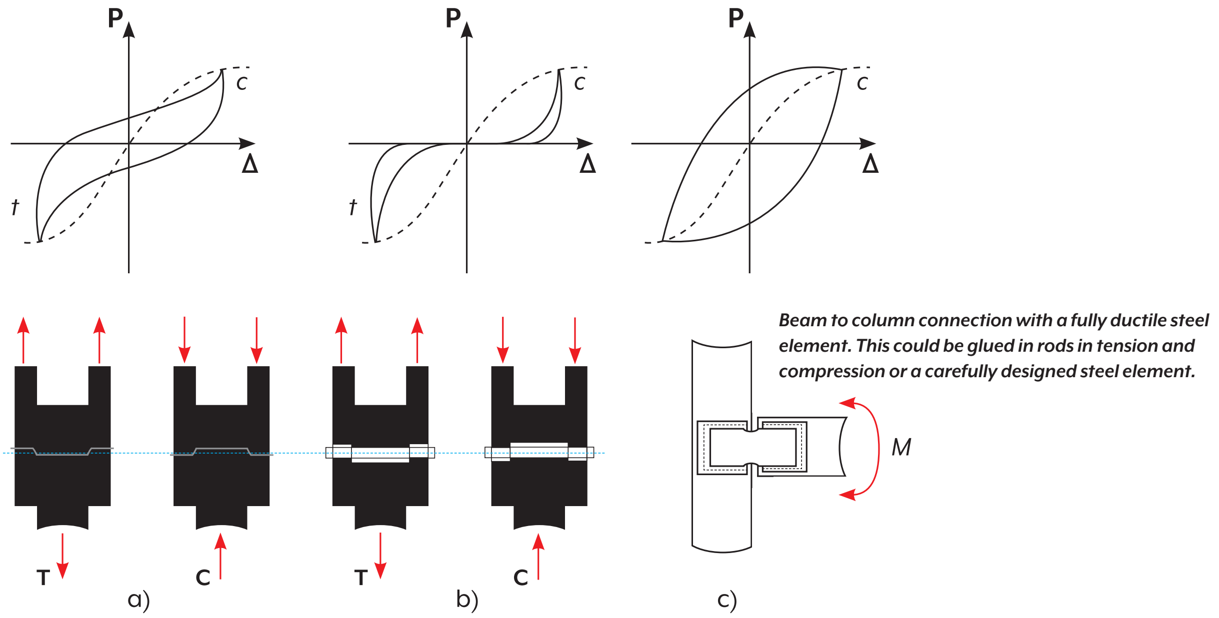


Figure 8. Hysteresis loops for cyclic loading of different types of fastener.

The capacity design procedure for any structure requires that the non-ductile parts of the structural system (CPEs) that could be subjected to actions from yielding elements be over-designed to avoid premature failure. This requires that CPEs be designed for the actions obtained from the initial analysis, multiplied by an *overstrength factor.* The value of the overstrength factor represents the amount by which the actual strength of the yielding components at large displacements may exceed the design strength. The overstrength factor depends on several factors:

- The strength reduction factor φ used in the initial design.
- Strain-hardening in the steel of the yielding components due to large displacements.
- The difference between the 5th percentile (characteristic) strength used for design of the yielding components and the 95th percentile strength value which could be present.
- The actual strength of the connection rather than its analytical strength (the strength provided by the formula in the design standard).
- Any other unintentional over-design of the weakest components.

For most timber structures the overstrength factor has a value of ϕ_{os} = 1.6 or more, which must be used for design of the non-yielding components of a ductile structure, according to the principles of capacity design.

5. STRUCTURAL ANALYSIS

For a building subjected to seismic forces, some form of structural analysis is required to assess the lateral forces at each level, and to allocate those forces to the lateral load resisting system.

5.1 HAND CALCULATIONS

For many simple buildings, it is possible to use hand calculations. Simple hand calculations can be used with the equivalent static method to assess the lateral forces at each level of the building. Those forces can be allocated to the elements of the lateral load resisting system, based on the stiffness of the floor diaphragms. For a rigid floor diaphragm the lateral forces should be allocated in proportion to the lateral stiffness of each element of the lateral load resisting system, but for a flexible floor diaphragm, the forces should be allocated in proportion to the tributary area associated with each component.

5.2 COMPUTER ANALYSIS

Most buildings will require the use of computer analysis to assess the lateral forces at each level of the building. A computer model can either be used with the equivalent static forces applied or with model analysis. The program will typically provide the results of this analysis as member demands.

The forces can be allocated to the elements of the lateral load resisting system based on the stiffness of the floor diaphragms, as described above, or using a three-dimensional structural modal of the building which takes into account the relative stiffness of all structural elements in the building.

When using computer modelling in the design of a timber structure (such as the example shown in Figure 9), care should be taken to obtain as accurate representation of member and connection stiffness as possible. How the programme is modelling shear stiffness of the timber members should also be carefully checked as some common software packages tie this to the Poisson's ratio that is not correct for wood based materials.

Some proprietary software programs, such as Gib Ezi-brace from Winstone Wallboards Ltd, provide a simple computer analysis for allocating lateral forces to elements of the lateral load resisting system.

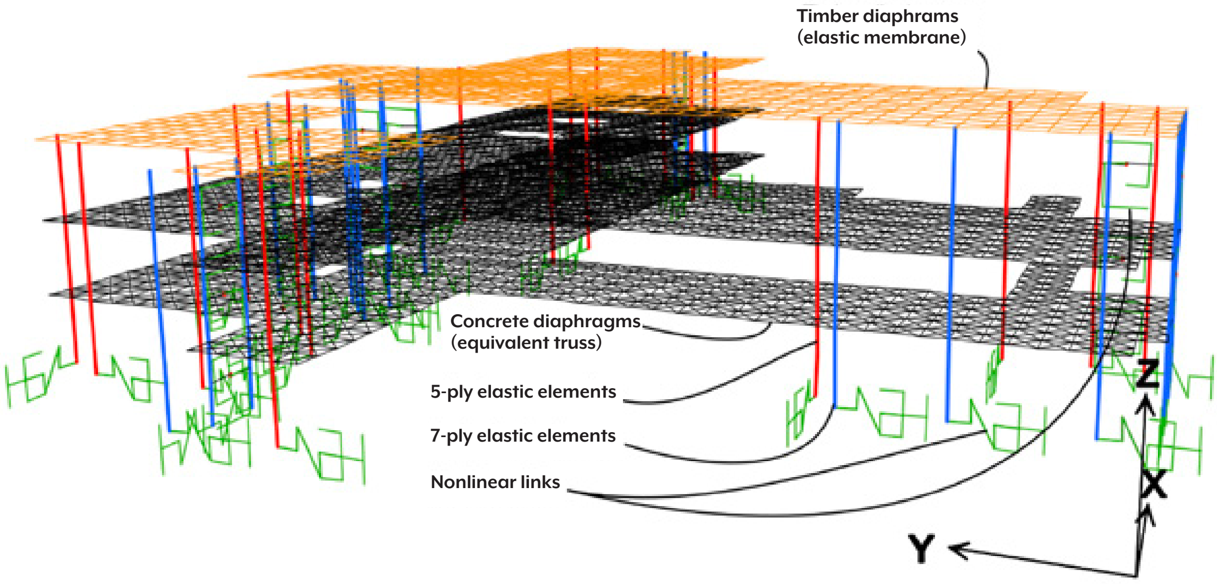


Figure 9: Schematic of computer model used to perform diaphragm analysis during the design of the 3 storey mass timber Peavy hall building in Corvallis, Oregon (courtesy of PTL | Structural consultants).

6. SPECIFIC DESIGN OF SHEAR WALLS

For buildings which require specific design, the structural engineering of shear panels on timber framing must be in accordance with the code requirements for wind and earthquake loadings, and must comply with appropriate material standards or proprietary manufacturers' information for the sheet material and fastener properties.

For more information on the design of light timber frame shear walls refer to the BRANZ guide, "Multi-story light timber-framed buildings in NZ: Engineering design."

6.1 SHEAR WALL GEOMETRY

6.1.1 Light timber framed walls

Typical geometry of a light timber framed shear wall is shown in Figure 10. The sheathing is usually arranged vertically. All vertical joints are on studs, and blocking or dwangs are provided at all horizontal joints. The sheathing is fixed to the studs and dwangs with nails or screws at close centers all round. The deformed shape of this wall assembly is shown in Figure 11 where it can be seen that the nails between the edges of the sheathing and the framing all resist the shear flow, and the end studs resist overturning moments. Shear deformation within the panel sheets is usually very small in comparison with the nail slip and the frame deformations.

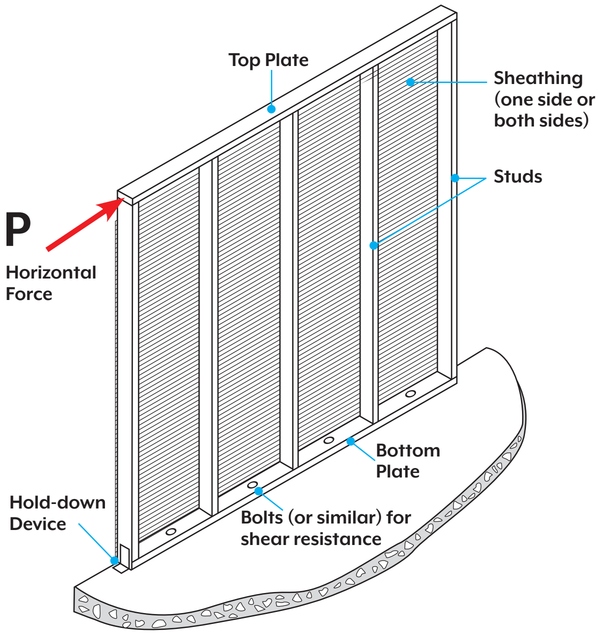


Figure 10. Typical light timber framed shear wall geometry.

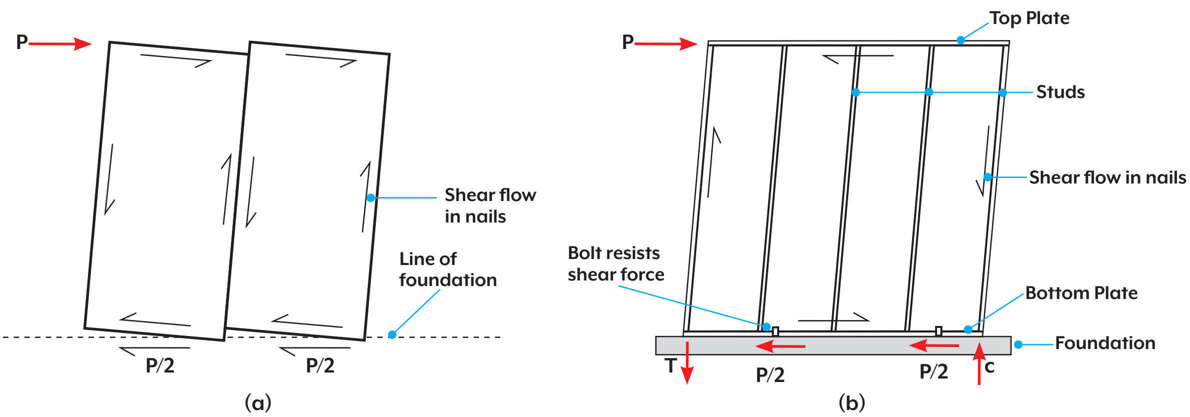


Figure 11. Deformation and forces in (a) the sheathing and (b) the framing under lateral loads.

6.1.2 Solid timber walls

Timber shear walls can also be manufactured from glued solid timber panels, including LVL, CLT and glulam. The ductility in solid timber wall systems must be designed to come from steel fastenings between wall panels, steel hold-downs, or specifically designed energy absorbing elements.

Holding down forces in one or two storey solid timber walls can be resisted by steel anchor plates nailed or screwed to the lower corners of the walls, or by internal prestressing cables in the Pres-Lam system. In multi-storey timber buildings, similar connections are required at horizontal joints between panels. Hold down forces could also be resisted using propietry dampers such as theTectonus® system.

6.2 PANEL SHEETING PRODUCTS

A wide range of panel sheeting products is available for use in bracing elements of light timber framed walls. These include:

- Plywood.
- Particle board.
- Medium density fibreboard (MDF).
- Gypsum plasterboard.

Plywood is the traditional material used in thicknesses from 7 mm to 20mm. Particle board, MDF, OSB and other wood-based panel products will behave similarly, but the sheets usually need to be thicker than plywood to resist local damage at the nail locations. A conservative approach is to use the 7 mm plywood load figures for other wood-based boards 12 mm or more in thickness. Characteristic shear stresses in plywood sheathing are given in Table 5.1 of NZS AS 1720.1, and summarised in Table 2. Consult manufacturers' literature for other materials.

Plywood thickness (mm)	Nail diameter (mm)	Characteristic strength (kN) per nail	Minimum nail length (mm)	Design strengths (JD5) of walls (kN/m) for nail spacing of:				Plywood design shear (F8) (kN/m)
				150mm	100mm	75mm	50mm	
7.0 12.5	2.8 3.15	545 680	35 50	4.7 5.9	7.1 8.9	9.5 11.8	14.2 17.7	23.3 39.9

NOTE: The following factors have been used in the calculation of the design nail strength: ϕ = 0.8, k_1 = 1.14, k_{13} = 1, k_{14} = 1, k_{16} = 1.1, k_{17} = 1.3. The following factors have been used in the calculation of the plywood design strength: ϕ = 1.0 (assumed overstrength load from nails), k_1 = 1.14, k_{12} = 1, k_{19} = 1, g_{19} = 1, f'_s = 4.2Pa

Table 2. Characteristic design strengths of nailed plywood walls.

Gypsum plasterboard can only be relied on if it has been tested specifically as a bracing material. Strength and stiffness are very different from those of wood-based panel products. Consult manufacturers' literature for design data. Winstone Wallboards and other manufacturers have a range of technical publications for both specific and non-specific design of shear walls and horizontal diaphragms with various gypsum plasterboard solutions.

6.3 PANEL CONNECTIONS

Panel products must be fixed to timber framing at all edges of every sheet. The usual fixing connection are flat head nails. Some gypsum plasterboard systems require special nails with large washers. Screws can be used for gypsum plasterboard systems because deformation occurs with local damage to the plasterboard at each screw location. Care should be taken in the use of screws for wood-based panel systems where ductility is required, because the screws will bend under reversed cyclic loading resulting in fracture of the screws and failure of the system.

This section is intended as a companion guide to the standard as it this presents a change from previous approaches used in timber design in New Zealand.

Nails or screws should be uniformly spaced around the perimeter of all sheets of panel sheathing. The studs in the centre of the panel sheets only require nominal nailing because these nails do not significantly resist the lateral loads.

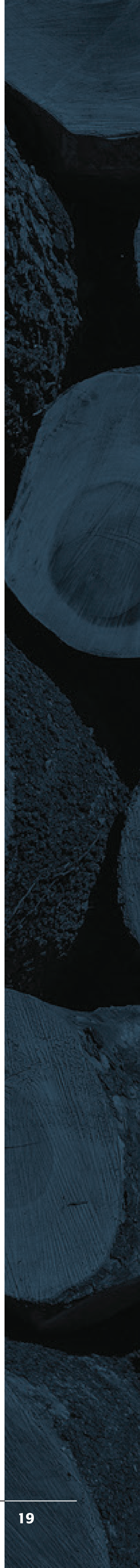
Minimum recommended nail length is 4 times the sheathing thickness, or 60 mm, whichever is less, to ensure that they do not pull out completely under cyclic loading. For ductile behaviour, the maximum recommended nail length is 6 times the sheathing thickness; longer nails will have less ductility because they are not able to withdraw slightly during reversed cyclic loading. The nail spacing is calculated from the shear flow along the edge of each panel and the capacity of the selected nails, using:

$$s = F / q$$

where:

s is the nail spacing (mm)
F is the capacity of each nail (kN)
q is the shear flow (kN/mm of panel length)

Care should be taken in the splicing of panels at internal studs to ensure two lines of fasteners will fit into the stud member.



6.4 LIGHT TIMBER FRAMING MEMBERS

The tension and compression chords of the shear wall should be designed so that design strengths, including overstrength where required, are not exceeded. These chords must be continuous for the height of the wall. Because of large shear deformation and nail slip, it can be shown that the internal framing members do not contribute to the in-plane flexural strength of the shear panel. For cantilever walls, the design strengths of the chords will be governed by the hold-down connections.

6.5 HOLD-DOWN DETAILS

The end chords of vertical shear panels must be securely fixed to the foundations with “hold-down” connections to resist vertical uplift forces, including overstrength where required. These connections must be designed to transfer chord forces into the foundations without inducing bending moments in the chord members. Some suitable details for light timber framed walls are illustrated in Figure 12. Many other proprietary systems are also available.

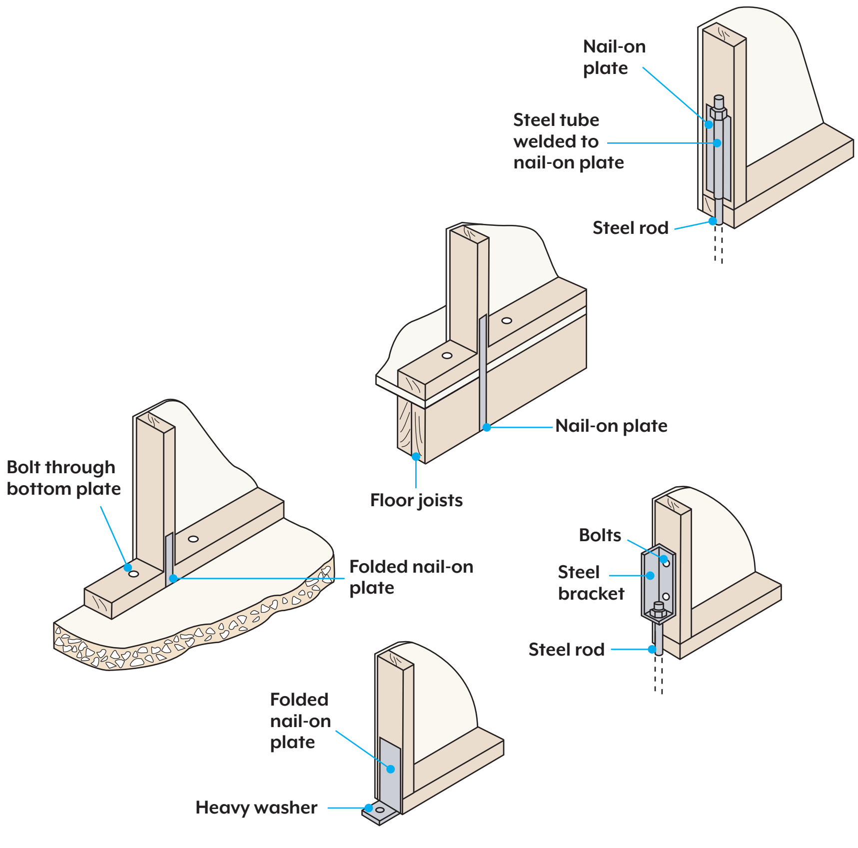


Figure 12. Some possible connection details for shear wall hold-downs.

6.6 OPENINGS

When a light timber frame shear wall or diaphragm has an opening, the shear flow interrupted by the opening must be transmitted into the sheathing by the adjacent framing members. This requires that the trimmer members on all sides of the opening be continuous, or incorporate connectors over the discontinuous sections.

For more information regarding the design of shearwalls with openings designers could refer to the APA technical note “Design for Force Transfer Around Opening (FTAO).” However care should be taken to adapt this method for New Zealand limit state design assumptions

6.7 SHEAR WALL DEFLECTIONS

The lateral deflection of a single-storey light timber frame wall with fixed sheathing can be determined by the following equations:

$$\Delta_h = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 \qquad \text{...9.2(1)}$$

where:

Δ_1 = inter-story flexural deflection as a cantilever (can be ignored for a single storey shear wall)

$$\Delta_1 = \frac{2VH^3}{3EAL_w^2} + H\theta \qquad \text{...9.2(2)}$$

Δ_2 = shear deformation of the wall

$$\Delta_2 = \frac{VH}{GL_w t}$$

Δ_3 = deflection of the wall due to fastener slip

$$\Delta_3 = 2\delta\,(1 + a)\,m \qquad \text{...9.2(4)}$$

Δ_4 = deflection of the wall due to connection deformation

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{L_w} \qquad \text{...9.2(5)}$$

where:

V = lateral load applied to the top of the wall

Lw = length of wall

E = elastic modulus of the chord members

A = cross-sectional area of a chord

H = height of wall

t = sheathing panel thickness

θ = flexural rotation at the base of the storey being considered in radians

G = shear modulus of the sheathing

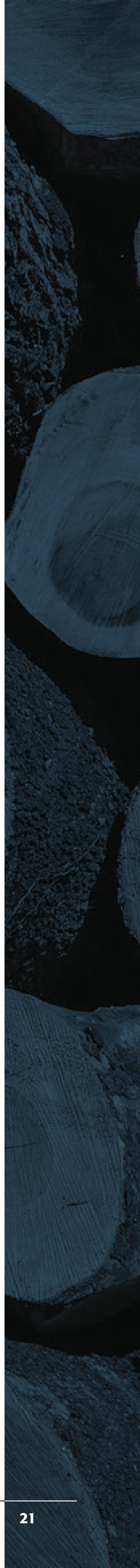
m = number of sheathing panels within the height of the wall

α = sheathing panel aspect ratio α = h/b, where h is the vertical dimension and b is the length in chord direction

δ = fastener slip of the panel-to-framing connection

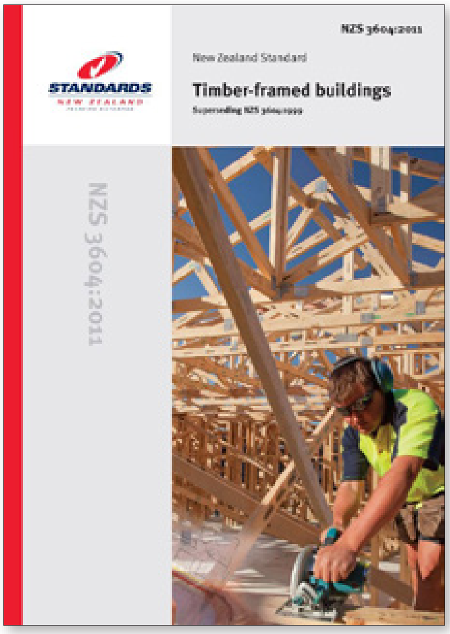
δc = connection slip compression chord connection

δt = connection slip tension chord connection



7. NON-SPECIFIC DESIGN USING NZS 3604

NZS 3604:2011 is the code of practice for light timber framed buildings up to two and a half storeys not requiring specific engineering design. NZS 3604 is called up with minor amendments in Acceptable Solution B1/AS1, to meet the requirements of Section B1 of the New Zealand Building Code.



For lateral loads, NZS 3604 specifies design of bracing systems using a tabular design method with loads expressed in “Bracing Units” (BU) depending on the location, weight and size of the building.

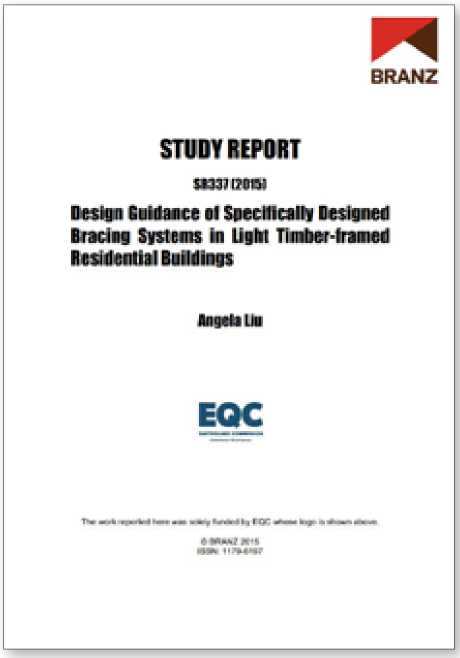
Proprietary tested bracing panels with a known BU rating (bracing units per metre of wall panel) must be located throughout the structure to resist the imposed loads. NZS 3604 requires that the bracing elements be distributed in a regular manner over the plan area of the building.

Bracing units are determined in a standard test known as the *P21 Test*, which can be carried out by a structural testing laboratory such as at BRANZ, Scion, or a university. The P21 test involves cyclic loading of a wall assembly and defines a point of performance of ultimate capacity.

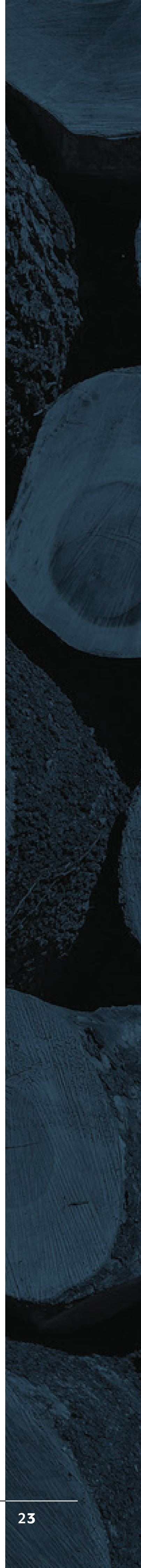
Most manufacturers of panel products can provide the number of Bracing Units per metre for various systems incorporating their products, as a result of P21 testing. Note that it is the assembled system and not just the panel material which has a BU rating. Care must be taken if using P21 results for a system that is otherwise specifically designed. Engineeres should seek the initial P21 results to check for suitability.

For comparison with specific design, 20 BUs equate to 1 kN of lateral load capacity.

Engineers are often required to combine the use of specifically designed steel frames within the framework of the bracing unit calculation. Guidance on the use of steel frames within NZS3604 can be found in the BRANZ paper “Design Guidance of Specifically Designed Bracing Systems in Light Timber-framed Residential Buildings”. This guide discusses the compatibility of the normally elastically responding frame and the non-linear response of the walls. The guide also discusses consideration of plan irregularity should a higher level of performance be desired.



8. STRUCTURAL DESIGN REQUIREMENTS OF NZS AS 1720.1



8.1 OVERVIEW

For many years, Verification Method B1/VM1 of the New Zealand Building Code has called up the Timber Structures Standard NZS 3603 for specific engineering design of timber buildings. Following recent changes to B1/VM1, NZS 3603 has been replaced by NZS AS 1720.1.

Section 9 of the New Zealand Appendix NZS AS 1720.1 sets out the minimum seismic design requirements for timber structures. This section of the Design Guide summarises the seismic design requirements of Section 9 of NZS AS 1720.1 and in general follows the layout of that standard NZS AS 1720.1.

NZS AS 1720.1 requires that the structure and its component parts be designed to have adequate strength, stiffness and ductility for the serviceability limit state (SLS) and ultimate limit state (ULS) earthquake load combinations as defined in NZS 1170.5.

8.1.1 Potential ductile elements (PDEs)

Timber as a material fails suddenly under tension stress (See NZ Wood Design Guide Chapter 1.2 Section 8.1). This sudden tension failure also impacts on the bending failure of timber which is also often sudden. This type of failure is considered brittle as it has no non-linear component. Brittle structures and factors governing their design are described further in Section 8.2.1, however, the brittle nature of timber means that in order to obtain ductility, elements capable of non-linear behaviour must be added.

NZS1170.5 describes potential inelastic zones and states that these zones shall be proportioned so that the design strength exceeds the design actions at these locations. This definition, however, is more suited to reinforced concrete or steel structures where the ductile failure mode is typically identified to be within the beam, wall or shear link.

In timber structures it is more appropriate to identify a Potential Ductile Element (PDE), often in the form of a steel connection, that provides inelastic capacity to the structure.

8.1.2 Capacity protected elements (CPEs)

NZS1170.5 requires that the areas outside of the inelastic zones have the capacity to sustain actions due to overstrength moments and shears from the PDE, in timber structures this typically means the timber beams, columns or walls.

These protected elements are called Capacity Protected Elements (CPEs).

When verifying a CPE against the overstrength demand from the PDE a strength reduction factor of $\phi = 1$ is used. For more information on overstrength refer to Section 8.6.3.

8.2 SEISMIC CLASSIFICATION

Structures subjected to earthquake forces are classified as:

- Elastic structures ($\mu = 1$).
- Nominally ductile structures ($\mu = 1.25$).
- Structures of limited ductility ($1.25 < \mu \leq 3$).
- Fully ductile structures ($3 < \mu$).

It is worth noting that although each of the seismic classifications below have a corresponding structural ductility factor (μ) stated, designers are free to use a lower value of μ in their design if they wish.

8.2.1 Elastic structures

Elastic structures contain primary seismic members or connections that are not capable of, or have not been designed for inelastic deformation and have not been designed using capacity design. Capacity design checks are not required for elastic structures and as such they have not been checked to assure that they possess ductile capacity. The impact of this is that elastic structures do not have the assured ability to redistribute seismic actions or limit seismic forces through period elongation or structural damping.

Some elastic structures may exhibit brittle failure under seismic loading which is greater than ULS loading unless additional design strength is provided, these are defined as brittle structures.

Some common examples of brittle structures include:

- Moment-resisting frames with glued knee joints.
- A cantilever post fixed rigidly into the ground to resist lateral loads. Timber poles are often used as cantilevers in single storey industrial or agricultural buildings, to support roof loads and to resist lateral loads. Tall cantilevered timber poles are flexible so other bracing may be required to limit lateral displacements, particularly where dynamic loads from cranes (in factories), or washing machines (in homes) are involved.
- Arch structures may be brittle where the point of maximum moment is often located away from connections and splices.
- Braced frames with cast steel elements that do not have the ability to assure ductile capacity. One way of introducing a limited amount of ductility to this type of system is to allow the bracing connection to provide yielding and design the brace itself for overstrength creating a tension only brace system.

Brittle structures are limited to one or two storeys in height. Primary seismic resisting members and connections must be designed to remain elastic when subjected to 1.5 times the ULS seismic actions, with material strains and deformations remaining within the elastic range, and no fracture occurring in timber members or the timber in connections.

A degree of engineering judgement should be used in the classification of ‘brittle’ and when the limitations of Section 9 apply. Brittle elements are classified as those that may exhibit sudden failure if loaded beyond their intended capacity. In general a structure should have a small margin of safety against collapse in the most likely severe earthquake shaking. Structures that have brittle failure mechanisms along critical load paths may not be able to provide this margin and hence the demand on the brittle mechanism should be increased. It is acceptable not to increase demand where alternative load paths exist or where the failure of a single member will not lead to full collapse (a closely spaced pole retaining wall for example).

8.2.2 Nominally ductile structures

Nominally ductile structures are those capable of sustaining small inelastic material deformations in PDEs or those that demonstrate a high level of hysteretic pinching during cyclic loading.

Some examples of nominally ductile structures include:

- Structures with joints having large diameter or squat steel dowel fasteners. This type of structure will have a tendency to have an embedment only failure in the PDE with the timber surrounding the dowel not having sufficient strength to force the dowel to yield. The consequence of this is that under cyclic loading, once the timber surrounding the dowel is crushed it will not provide resistance again until it is displaced beyond the displacement of the last cycle, creating a heavily pinched hysteretic response.
- Wall structures with screw-fixed sheathing. This type of structure is not always nominally ductile but should be classified in this way unless significant care is taken to ensure that the screws maintain their ability to provide ductile response under seismic loading. Screws are typically cold formed and in some cases have exhibited failure of their head followed by complete pull through under cyclic loading, limiting their ability to effectively act as a PDE.

- Structures with tension only braces. Similar to an embedment only failure mode, a tension only brace, once yielded, will not provide resistance until displaced beyond the displacement of the previous cycle, thus creating heavily pinched response.
- Structures with carpentry joints in compression. As with embedment only failure connections, carpentry joints will exhibit heavily pinched behaviour under cyclic loading. Extra care should be taken if this type of joint is to be nominated in a nominally ductile structure so that brittle failure of the timber in the connection is avoided.

Primary seismic resisting members or connections in nominally ductile structure that are not designated as PDEs must be designed to remain elastic when subjected to overstrength actions, with material strains and deformations remaining within the elastic range, so that no fracture occurs in timber members or in any timber in connections.

8.2.3 Limited ductile structures

Limited ductile structures are those capable of ductile behaviour under seismic loading beyond the design capacity of the PDE. Significant inelastic material deformations are required in PDEs to limit earthquake design forces.

Some examples of limited ductile structures include:

- Buildings with nailed plywood shear walls. This is perhaps one of the most common ways the achieve a reasonable level of ductility in a timber structure. Mild steel nails when displaced beyond their elastic limit yield in bending providing a stable source of hysteretic damping.
- Solid timber frames and walls with appropriate PDEs. The definition of appropriate PDE is varied and could be doweled brackets, either external or internal, nail plates, or steel hubs. Care should always be taken that brittle or sudden failure is avoided in establishing the local displacement of the PDE. The design engineer should also think about what the local displacement requirements will be in the PDE and if these are possible within the context of the surrounding structure.
- Structures with braced frames with braces that are designed to yield in tension and compression such as buckling restrained brace frames. Combined tension and compression braces within a timber moment frame represent a viable way of ensuring limited, or even fully, ductile global response. This type of structure will typically place a large amount of stress on the point of connection, particularly if connected at or through a beam-column joint and potential brittle failure should be assessed and avoided.

The strain or deformation demand on PDEs in a limited ductile structure must not exceed the limits specified in NZS AS 1720.1. Seismic resisting members or connections that are not PDEs are required to remain elastic when subjected to overstrength actions.

8.2.4 Fully ductile structures

Fully ductile structures with a structural ductility factor greater than 3.0 are not covered by Section 9 of NZS AS 1720.1 although many of the principles outlined in the section continue to apply.

Fully ductile structures can be difficult to achieve with timber due to the elastic deformation of the timber CPE. The elastic deformation of the CPE means that the local ductility requirement of the PDE cannot be achieved without the building moving beyond code specified maximum deformations.

The design of a fully ductile timber structure requires special study. This would typically consist of the modelling of all or part of the seismic resisting system to show that the global ductile capacity required can be accommodated by the local ductile response of the PDE. This analysis should include all significant sources of elastic deformation and focus on the assurance that potential brittle failure modes do not occur.

Due to the flexible nature of timber, it can be difficult to achieve a fully ductile structure while satifying Servicabilty Limit State deflection requirements.

8.3 LIMIT STATE DESIGN

8.3.1 Serviceability Limit State (SLS)

SLS exists to protect a building and its contents, enabling them to perform adequately for normal use under all expected actions. In the case of seismic design, a combination of importance levels and expected building life, as defined by NZS1170.0, is used to define the seismic hazard expected to occur during that building's specified life. As it is principally a limit state to avoid damage it is, in most cases, a deflection check. Recommendations for maximum deformations and inter-storey drifts due to application of SLS seismic actions can be found in NZS1170.0. This maximum drift will often depend on a building's linings and façade system. Due to the flexible nature of many timber systems the SLS deflection often governs the sizing of the timber members, however, this is more often the wind SLS requirements than the seismic SLS requirements.

Structures are required to satisfy the serviceability limit state requirements by remaining elastic when subjected to SLS seismic actions, so that materials remain within the elastic range, and timber members do not fracture.

8.3.2 Ultimate Limit State (ULS)

Structures are required to satisfy the ultimate limit state requirements such that the design strengths exceed the design actions. When ductility is used in design it is likely that, should this level of shaking occur, irreparable damage will occur to the elements providing this ductility.

There are also several limitations on structural deformations to meet the requirements of NZS 1170.5, with regard to inter-storey drift limits and P-delta stability coefficients, maximum permissible lateral displacement of the structure at site boundaries, and limits on maximum permissible strains in materials.

The maximum inter-storey deflection for ULS loading is 2.5% of the storey height. This level of displacement is expected to be accompanied by significant damage to non-structural elements and where possible, lower drift levels should be targeted.

In addition, any deformation incompatibilities between lateral displacements of structural elements must be checked to ensure that they do not compromise lateral or vertical load resisting mechanisms.

8.4 MATERIAL STRAIN CONSIDERATIONS

8.4.1 Redistribution of actions

The redistribution of seismic actions occurs because although the various lateral load resisting systems within a building are often analysed individually, they are tied together and will normally be subjected to similar displacement demands. This means that under seismic action an element that is locally ductile will reach a maximum capacity and seismic forces will be redistributed through to elements still able to resist load.

Redistribution requires ductile response and therefore is not permitted in elastic or nominally ductile structures. For limited ductile structures, the amount of redistribution is limited by the strain and deformation limits discussed in Section 8.4.2. Where redistribution is used the designer should check strain/deformation demands on both the element that the demand is being redistributed from and the element it is being redistributed to. The stiffness of the diaphragm must also be checked to ensure it is adequate to redistribute forces.

8.4.2 Strain and deformation limits

NZS AS 1720.1 requires that all timber elements, connections and other components designed to remain elastic, shall not be subjected to demands or deformations beyond the elastic range for either SLS or ULS loading.

For limited ductile structures, PDEs are required to sustain the cyclic strain or deformations required for ULS actions. Maximum material strains or deformations are summarised below:

- Mild steel nails, rods and bars are not to exceed limits given in Table 3.
- For PDEs consisting of other mechanical fasteners the cyclic ULS strain or deformation capability is to be assessed in accordance with a special study. One example of a special study would be testing in accordance with ISO 16670 that describes a testing method for joints made with mechanical fasteners.
- For PDEs constructed entirely of structural steel or reinforcing steel, material strain limits shall be in accordance with the appropriate New Zealand standards for steel design.

Potential ductile element	Displacement (mm)	Strain in steel (%)
Connections with mild steel nails	6	-
Mild steel rods/bars/bolts – tension only	-	8
Mild steel rods/bars/bolts – compression and bending[1]	-	5
[1] Mild steel rods/bars/bolts that are subjected to compression or bending shall be adequately restrained to prevent local or lateral torsional buckling.		

Table 3. Strain or deformation limits for specific potential ductile elements.

While the application of displacement and strain limits is an important verification in the design of a timber structure, judgement should be used in their application. It is often acceptable that a small number of individual fasteners are displaced slightly beyond the limits above provided that the average performance of the PDE is below these limits. For this reason, it is normally appropriate to perform the check on the average ULS displacement/strain of the fasteners and not the absolute maximum value.

An example of this is the verification a group of dowels working as a wall hold-down. While in reality the outer-most dowel will be subjected to the highest displacement demand under overturning, the deformation check can be done based on the center of the dowel group.

8.5 CALCULATION OF LATERAL DISPLACEMENTS

Lateral displacements of the primary seismic resisting structural system are to be determined for the SLS and ULS actions taking into account all significant components of deformation. For SLS, inter-storey drifts and local deformations are to be determined using elastic analysis, with no amplification required.

Clause C9.2.12 of NZS AS 1720.1 provides the formula for calculating the deformation of a single storey of a light timber frame wall. There are additional contributions that should be considered if for the upper levels of a light timber frame system that can be found in the BRANZ guideline "Multi-storey light timber-framed buildings in NZ: engineering design.

Clause 9.2.12.3 describes the requirements for the lateral displacement for nominal and limited ductile structures and includes the stipulation that displacement amplification factors should be allowed for where appropriate.

Typically the maixmum response of a ductile structure is calculated as its elastic response multiplied by its design ductility. Displacement amplification factors are used when this assumption does not adequately provide the maximum response. NZS AS 1720.1 refers to special study, experimental testing and/or time-history numerical modelling for the determination of displacement amplifications. Two possible methods are described on the next page:

8.5.1 The displacement amplification factor, k_{dt}

One potential reason for displacement amplification is that less hysteretic energy is dissipated during a cycle than in the elasto-plastic assumption that forms the basis of the common inelastic displacement calculation described above. Ductile timber structures using dowel type connections have less energy dissipating capacity and commonly exhibit a pinched force-displacement hysteretic response.

To account for increased seismic deformations that will occur due to the reduced energy dissipating capacity of timber structures, the lateral displacement for a given structural ductility factor can be amplified by a factor of $k_{dt} = \sqrt{\mu}$.

k_{dt} is derived by considering the spectral displacement resulting from energy-based equivalent viscous damping (ΔEVD) as defined by Priestley et al. 2007 and the spectral displacement resulting from application of the kμ factor from NZS 1170.5 ($\Delta_{k\mu}$) for a given fundamental period. The expression is derived as:

$$k_{dt} = \frac{\Delta_{EVD}}{\Delta_{k\mu}} \quad \frac{\sqrt{\dfrac{7}{2+\xi_{eq}}}}{\dfrac{1}{k_{\mu}}}$$

where:

$$\xi_{eq} = \xi_{el} + \xi_{hyst}$$

The elastic damping ξ_{el}, is taken as 5%. The hysteretic damping, ξ_{hyst}, is based on experimental test results from plywood sheathed shear walls with mild steel nail PDEs and is approximated as a linear relationship with ductility. The hysteretic damping is taken as 14% at a ductility of 3 as shown below. Therefore:

$$\xi_{hyst} = 7(\mu - 1)$$

The kμ factor from NZS 1170.5 is conservatively taken as μ. Therefore:

$$k_{dt} = \frac{\sqrt{\dfrac{7}{2+5+7(\mu-1)}}}{\dfrac{1}{\mu}} = \mu\sqrt{\frac{1}{\mu}} = \sqrt{\mu}$$

The use of the above equation is potentially conservative as it does not take into account the impact of period elongation in the reduction of load. It also does not consider that maximum displacement is a peak response and hence not as impacted by pinching.

8.5.2 The use of non-linear static methods

Non-linear static methods, such as the Acceleration-Displacement Response Spectrum (ADRS) method, are an appropriate special study to account for hysteretic properties and period elongation in displacement calculations. The use of non-linear static methods is described further in Section 4.5.1.

8.6 SEISMIC DESIGN ACTIONS

Clauses covering seismic design actions and the interaction of NZS AS 1720.1 with NZS 1170.5 are given in Section 9.3 of NZS AS 1720.1. Specifically, that section covers the calculation of seismic design actions for serviceability limit state (SLS) and ultimate limit state (ULS) in accordance with NZS 1170.5 by providing clauses around the use of:

- The structural performance factor, S_p.
- The structural ductility factor, μ.
- The dynamic characteristics of the structure.

While the following sections focus on the application of Equivalent Static Analysis to timber structures. Section 4.5.1 of this guide discusses other available verification methods and alternative solutions available to designers to calculate seismic design actions.

8.6.1 Structural performance factor

In seismic design, the structural performance factor is a multiplier which represents a number of other effects not explicitly considered. For more information on these effects refer to the Commentary of NZS 1170.5 (SNZ 2004).

In timber, for brittle structures, the structural performance factor is S_p = 1.5 for the reasons described in Section 8.3.3.

For SLS loading the structural performance factor is S_p = 0.7.

For all other structures, the structural performance factor Sp is defined in NZS 1170.5 using the structural ductility factor verified as being appropriate for the lateral load resisting system used.

8.6.2 Structural ductility factor

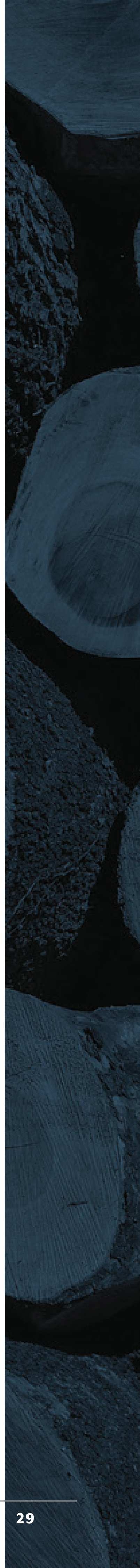
The structural ductility factor μ is the "global" ductility factor for the whole structure (where a small amount of non-linearity may already have occured). It is typically defined as the displacement at a point secant to yield divided by the maximum displacement of the structure during the earthquake. In order to achieve a certain value of global ductility, some individual components may have to achieve a much higher "local" ductility, depending on their geometry and location in the structure.

For the serviceability limit state the structural ductility factor should be taken as μ = 1.0.

For ultimate limit state design the maximum value of the structural ductility factor μ for the various types of structural classification are given in Table 4.

Type of structure	Structural ductility factor (μ)
Elastic structure	1.0
Nominally ductile structure	1.25
Limited ductile structure	3.0
Fully ductile structure	Specific design required

Table 4. Maximum structural ductility factor (μ) for ULS loading



Whenever a structural ductility factor μ greater than 1.25 is used, for example limited ductile design with μ = 3, it is necessary to check that all individual seismic load resisting elements, connections and joints within the complete structural system are able to deliver sufficient local ductility for the desired global ductility demand to be achieved. This assessment must take into account the hysteretic characteristics of the PDEs and all significant elastic deformation components of the primary seismic structural system.

For doweled type connections this means back calculating the non-linear dowel displacement demand during the ULS displacement check procedure to ensure that the maximum displacement or strain are within the limits described in Section 8.4.2. Typically the steps to do this are as follows:

Calculate the elastic displacement of the lateral load resisting system under ULS load:

- Find the inelastic displacement of the lateral load resisting system.
- Using an elasto-perfectly plastic approximation it can be assumed that all the inelastic displacement will come from the PDE. This will be conservative in situations where there is a post-yield stiffness and this is not the case.
- Back calculate to find what level of local displacement is required to achieve the required inelastic displacement of the lateral load resisting system.

For an example of this procedure for the design of light timber frame shear walls, refer to the BRANZ guideline "Multi-storey light timber-framed buildings in NZ: engineering design".

8.6.3 Overstrength actions

The overstrength actions for each PDE shall be determined on the basis of the detailing of the PDE and the critical load combinations which may occur.

The maximum likely strength of each PDE can be determined by multiplying its nominal strength by its overstrength factor, which is a function of several factors discussed in Section 4.6.3.

Checks on brittle failure modes within the PDE should be against overstrength actions.

For mild steel nails, screws and dowels, it is permissible to consider an overstrength factor of 1.6.

The maximum strengths of PDEs consisting entirely of structural steel and reinforcing steel shall be in accordance with the respective material standards.

8.6.4 Capacity design

Clause 9.3.5 of NZS AS 1720.1 provides specific rules for the application of capacity design principals described in Section 4.1. The clause requires the selection of the local ductile mechanism that is providing global ductility, ideally this would be stated clearly in a design features report.

Clause 9.3.5.6 outlines some of the demand considerations that should be considered when calculating the actions on CPE such as gravity loading if it is a wall or gravity beam as well as a laterally loaded member. In general terms it is desirable to separate the shear transfer and overturning moment connection demand at the base of a shear wall but a combination of these actions could be used if this separation is not possible.

Clause 9.3.5.6 also confirms that CPEs may use a strength reduction factor of 1.

Similar to reinforced masonry and steel design a limitation on capacity design actions is described in Clause 9.3.5.7. This states that the demand on a CPE need not exceed the design actions for an equivalent structure with a structural ductility of 1.

Worked Example: Use of the limit on capacity design actions

A doweled hold down connection is being used at the base of a shear wall to connect a wall to its foundation. The demand on the dowel group N* is calculated from the base shear of the wall with a ductility of $\mu = 3$. The capacity of the bolted joint is then calculated with a strength reduction factor of $\varphi y = 0.8$ and $\varphi s = 0.9$ for the timber embedment and the steel yielding, respectively. It is found that 3.2 dowels are required for the joint so the engineer specifies 4 dowels, already giving an overcapacity of the joint of 4/3.2 = 1.25. The engineer is then required to check the capacity of the CPEs along the load path of the PDE. To do this, they calculate the nominal capacity ($\varphi y = \varphi s = 1$) of the four dowel group and find that the difference between the design capacity and the nominal capacity is 1.45. The overstrength factor for mild steel dowels is 1.6 therefore the margin required between the demand on the connection and the demand on the CPE is 1.25 x 1.45 x 1.6 = 2.9 in accordance with NZS 1170.5. As this is a relatively squat building (say a period of 0.5s) the $k\mu$ factor for the structure is 2.43, as the structural performance factor remains as $S_p = 0.7$, the demand on the connection can be multiplied by 2.43 as the limit on CPE design actions.

While this limit is useful in design as it avoids high overstrength demands creating costly CPEs, the use of this limit requires engineering judgement to ensure that some margin is maintained between the PDE and the CPE. This is particularly the case for nominally ductile and limited ductility structures where it may not be appropriate, for example, to use the typical CPE strength reduction factor of 1.0.

8.6.5 DYNAMIC MAGNIFICATION FACTORS

The design actions in CPEs for nominally ductile or limited ductile structures must be amplified by a dynamic magnification factor where appropriate.

Dynamic magnification factors are used to account for "higher mode effects" that is the impact of high modes that are not allowed for by the first period assumption of the equivalent static method or a non-linear pushover analysis. They are also used to allow for the way in which a building changes its behaviour during an earthquake with the activation of a PDE, that is not accounted for with the use of linear numerical integration or response spectrum analysis.

Section C2 of NZS1170.5 discusses in more detail designing for higher mode effects, however, this tends to concentrate on tall concrete and steel structures being the more common tall building types in New Zealand. There is limited information available on appropriate dynamic amplification factors for timber structures however as the timber members themselves have typically no capability for inelastic deformation, the considerations listed in Section C2 of NZS1170.5 provide appropriate base level guidance.

The major impacting factor on the dynamic amplification is height and Clause 9.3.6 sets limits on the use of various design methods that can be used with a dynamic amplification factor of 1.0. For buildings beyond these limits it refers to special study.

As mentioned above there is limited information on dynamic amplification factors for timber buildings however the following information is available for Pres-Lam rocking systems and could be used for more general structural timber systems.

For moment resisting frame buildings dynamic amplification mainly impacts on the column moment and shear demand (Smith 2014). The dynamic amplification factor for higher mode effects for frames (ω_f) to apply to the column moments, in addition to the overstrength considerations above, may be calculated as:

$$\omega_f = 1.66 + 0.13\left(\frac{\mu}{\phi_o} - 1\right)$$

Where:

μ = Structural ductility factor
ϕ_o = Overstrength on associated PDE

As with the moment demand the shear demand on the column will include dynamic amplification for higher mode effects:

$$V^*_{col} = \phi_o V_E + 0.1 \mu V_E$$

Where:

V^*_{col} = The overstrength shear demand on the column
V_E = the shear demand on the column from elastic or non-linear static analysis

For cantilevered wall buildings dynamic amplifications mainly impact on shear demand and have only a minor impact on moment demand (Sarti 2015).

For cantilevered walls a design envelope should be used for both the moment and shear demand on the wall as shown in Figure 13:

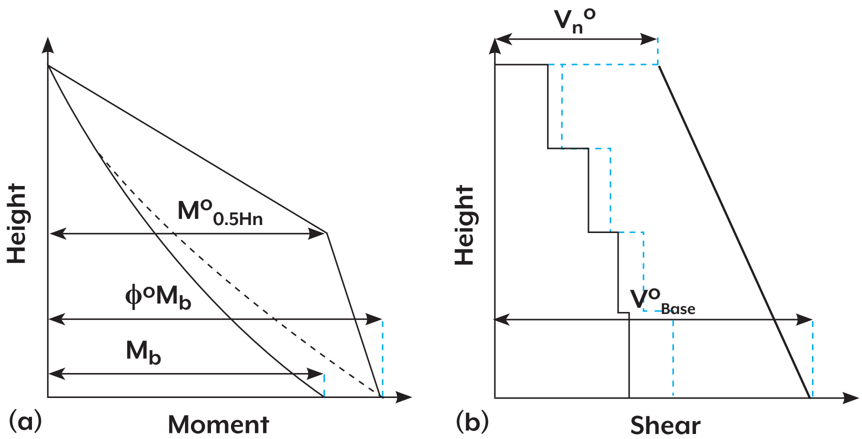


Figure 13: Simplified capacity design envelopes (Sarti 2015 modified after Priestley et al. (2007)

A bi-linear moment envelope (see Figure 13) was suggested by Priestley et al. (2007) and modified for Pres-Lam timber walls by Newcombe (2011) and Sarti (2015). While these envelopes were developed for Pres-Lam walls the dissipative rocking mechanism also dominates the performance of slender CLT walls with hold downs and as such can be extended to their design.

The bi-linear envelope is defined by the overstrength base moment capacity, $\phi_o M_b$, and the over-strength moment at mid-height, $M^o_{0.5Hn}$:

$$M^o_{0.5Hn} = C_{1,T} \phi_o M_b$$

Where:

$$C_{1,T} = 0.4 + 0.2\,(T_1 - 0.4) + 0.1\,(T_1 - 0.4)\left(\frac{\mu}{\phi_o} - 1\right)$$

The shear demand at the base of the wall can be calculated as:

$$V^o_{Base} = \phi_o \, \omega_V \, V_b$$

Where:

ϕ_o = the overstrength factor
V_b = the design base shear
ω_V = the dynamic amplification factor for walls:

$$\omega_V = \frac{\mu}{\phi_o} C_{2,T} + 1$$

Where:

$C_{2,T} = 0.062 + 0.4 (T_1 - 0.5) \leq 1.15$

Where:

T_1 = **elastic fundamental period of the structure**

μ = **structural ductility factor**

The shear demand at the top of the wall can be calculated as:

$$V_n^o = C_{3,T} V_{Base}^o$$

Where:

$C_{3,T}$ = $0.9 - 0.3T_1$

8.6.6 Concurrent actions for two-way lateral load resisting systems

For two-way primary seismic resisting systems, the effects of seismic actions occurring simultaneously along two orthogonal axes must be considered.

Members and connections for columns and walls which are part of a two-way lateral load resisting system must be designed to sustain the following concurrent actions:

- Overstrength actions in one direction amplified by the dynamic magnification factor, combined with the overstrength actions in the orthogonal direction with a dynamic magnification factor of unity and vice versa.
- Axial forces on columns or walls resulting from overstrength actions being generated simultaneously in two orthogonal lateral load resisting systems.

8.6.7 Displacement compatibility for systems acting in parallel

Where a combination of different lateral load resisting systems is used in a structure, rational analysis is needed to determine design actions on each CPE taking into account the relative stiffness and location of structural elements, overstrength actions from PDEs and dynamic amplification.

It is important to consider the displacement compatibility between different primary seismic resisting systems acting in parallel, especially for limited ductile deisgn.

For example, consider a shear wall and a moment-resisting frame (MRF) acting in parallel. The shear wall will generally be stiffer than the MRF, which will either increase the local ductility demand on the shear wall or reduce the ductility demand on the MRF. This incompatibility will also induce transfer forces in the floor diaphragm.

Displacement incompatibility between specifically designed elements and proprietary bracing elements is a common issue in the design of lightweight timber residential or commercial buildings.

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Further reading:

For information on the Pres-Lam system go to www.Pres-Lam.com
For information on Tectonus technology go to www.tectonus.com

Website trade literature is available from:

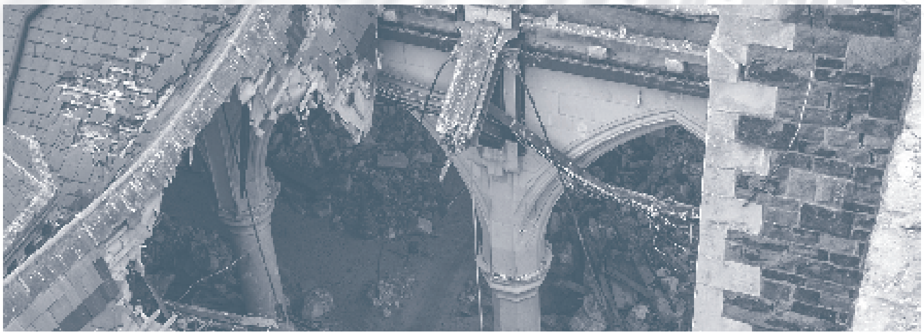
James Hardie Ltd. www.jameshardie.co.nz

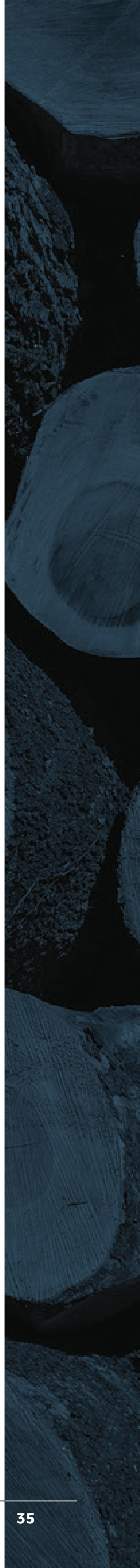
MiTek NZ Ltd. www.miteknz.co.nz

Pryda NZ Ltd. www.pryda.co.nz

Simpson Strong-Tie (USA) www.strongtie.com

Winstone Wallboards Ltd. www.gib.co.nz





Multi-unit residential housing that suffered a soft-storey collapse during an aftershock of the Christchurch earthquake.

Timber portal frames for resisting lateral loads. Light steel wall and roof bracing. No damage in the 2011 Christchurch earthquakes.

Light timber frame shear wall being tested.

Timber portal frame with timber cross bracing.

Light steel strap as bracing element in timber housing.

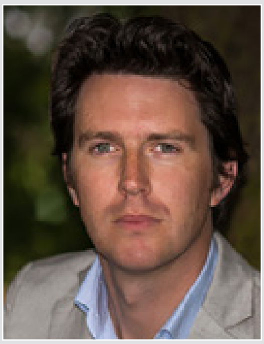
Damage to notched timber member following the Christchurch earthquakes.

Lateral load testing of a three-storey light timber frame shear wall.

A single storey light timber frame building with a plywood bracing panel.

Hold down and wall toe performance of a light timber frame shear wall.

ABOUT THE AUTHOR



Tobias Smith is a structural engineer and general manager of PTL | Structural Consultants. He attained a bachelor of civil Engineering from the University of Canterbury in 2007, and later, pursued a master's in civil engineering in 2008. Following a short period working as a structural engineer abroad, he commenced a doctorate in 2010 at the University of Canterbury in collaboration with the University of Basilicata in Potenza, Italy. During this time, he tested a three-story timber framed building on a shaking table. Since the completion of his doctoral studies in 2012 Tobias has worked for PTL in Christchurch.

Tobias has broad structural engineering consulting experience, having worked on a range of building projects such as residential, commercial, retail, and mixed-use within New Zealand and around the world. He has a special interest in timber buildings and their seismic design and is actively involved in the development of the New Zealand timber and seismic codes as well as the timber section of the European seismic design code, EC8. He has overseen various projects from the early concept design through to construction and the final sign-off.

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