The Seismic Assessment of Existing Buildings

Technical Guidelines for Engineering Assessments

Draft for Sector Briefings June 2016

Part C9: Timber Buildings

New Zealand Society for
Earthquake Engineering

Document Status and Amendments

Version	Date	Purpose/Amendment Description
Draft Version 2016_SB	30 June 2016	Draft for Sector Briefings

This document is likely to be incorporated by reference to the Earthquake Prone Buildings (Chief Executive's) Methodology to be developed under the provisions of the Building (Earthquake-prone Buildings) Amendment Act. It will also be endorsed by MBIE for use as guidance under section 175 of the Building Act to the extent that it assists practitioners and territorial authorities in complying with the Building Act.

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in the following file segments:

1. Contents
2. Part A – Assessment Objectives and Principles
3. Part B – Initial Seismic Assessment
4. Part C – Detailed Seismic Assessment

Updates will be notified on the above website.

The document will be formally released in early 2017, when the final form of the regulations and EPB Methodology associated with the Building (Earthquake-prone Buildings) Amendment Act 2016 is established.

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

Please contact the New Zealand Society for Earthquake Engineering via questions@EQ-Assess.org.nz if you require further information on these draft Guidelines, or if you wish to provide feedback.

Acknowledgements

These Guidelines were prepared during the period 2014 to 2016 with extensive technical input from the following members of the Project Technical Group:

Project Technical Group Chair	
Rob Jury	Beca
Task Group Leaders	
Jitendra Bothara	Miyamoto International
Adane Gebreyohaness	Beca
Nick Harwood	Eliot Sinclair
Weng Yuen Kam	Beca
Dave McGuigan	MBIE
Stuart Oliver	Holmes Consulting Group
Stefano Pampanin	University of Canterbury

Other Contributors	
Graeme Beattie	BRANZ
Alastair Cattanach	Dunning Thornton Consultants
Phil Clayton	Beca
Charles Clifton	University of Auckland
John Hare	Holmes Consulting Group
Jason Ingham	University of Auckland
Stuart Palmer	Tonkin & Taylor
Lou Robinson	Hadley & Robinson
Craig Stevenson	Aurecon

Project Management was provided by Deane McNulty, and editorial support provided by Ann Cunninghame and Sandy Cole.

Oversight to the development of these Guidelines was provided by a Project Steering Group comprising:

Dave Brunsdon (Chair)	Kestrel Group
Gavin Alexander	NZ Geotechnical Society
Stephen Cody	Wellington City Council
Jeff Farrell	Whakatane District Council
John Gardiner	MBIE

John Hare	SESOC
Quincy Ma	NZSEE
Richard Smith	EQC
Mike Stannard	MBIE
Frances Sullivan	Local Government NZ

Funding for the development of these Guidelines was provided by the Ministry of Business, Innovation and Employment and the Earthquake Commission.

Contents

C9. Timber Buildings

C9.1 General

C9.1.1 Scope and outline of this section

This section provides guidance for the detailed seismic assessment (DSA) of timber buildings to enable a consistent approach compared with the other materials addressed in these guidelines. In particular, it should assist by providing information on common forms of timber construction and estimation of the relevant member/element/system capacities. It builds on the section "Detailed Assessment of Timber Structures" in the previous version of these guidelines (NZSEE, 2006).

This section includes guidance for assessing:

- timber framed buildings where the timber framing in conjunction with lightweight materials providing bracing resistance to lateral loads, and
- engineered wooden buildings that incorporate elements such as timber portals.

When assessing buildings that are constructed primarily from other materials (such as unreinforced masonry or concrete) but include components such as timber diaphragms (refer Section C9.6.3) which may influence their seismic performance, this section should be read in conjunction with the relevant material sections (e.g. Section C8 Unreinforced Masonry Buildings and Section C5 Concrete Buildings).

Alert:

Timber framed buildings which are of lightweight construction are not expected to be targeted in the implementation of the earthquake-prone building legislation, and are therefore not expected to be subjected to DSA for that purpose.

A DSA of a timber building is typically performed after an Initial Seismic Assessment (ISA) has been undertaken in accordance with Part B of these guidelines. It should be noted that an ISA can identify high risk building components such as URM brick walls, heavy roofs, chimneys and poor foundation systems that can adversely affect the performance of a timber building. The mitigation or replacement of these undesirable features can increase the expected building performance, potentially making it unnecessary to undertake a detailed analysis/assessment.

C9.1.2 Definitions and abbreviations

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform inelastically before it fractures.
Cross laminated timber (CLT)	Engineered wood made from multiple layers of boards placed cross-wise to adjacent layers for increased rigidity and strength
Diaphragm	A horizontal structural element that distributes lateral forces to vertical elements, such as walls, of the lateral force resisting system

DSA	Detailed Seismic Assessment
Ductility	The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
European Yield Model (EYM)	Method for assessing connection design
Glulam	Glued laminated timber, a structural timber product made from layers of timber bonded with structural adhesives
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.
ISA	Initial Seismic Assessment (refer to Part B of these guidelines)
Laminated veneer lumber (LVL)	Engineered wood composite made from rotary peeled veneers, glued with a durable adhesive and laid up with parallel grain orientation to form long continuous sections
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil
Oriented strand board (OSB)	Engineered wood particle board formed by adding adhesives and then compressing layers of wood strands (flakes) in specific orientations
Plywood	Layered panel product comprising veneers of solid wood bonded to adjacent layers at right angles
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	Probable material strength (tension, compression, fracture, etc) and deformation (strain or ductility) capacities. Either as defined in these guidelines or relevant material standards, OR by testing. Probable material strength is defined in Section C1 as the expected or estimated mean material strength.
Sarking	Typically in New Zealand, timber construction board material fixed to timber framing to provide a diaphragm. Provides a surface to which other materials can be applied.
Shear wall	A wall which is subjected to lateral loads due to wind or earthquakes acting parallel to the direction of an earthquake load being considered (also known as an in-plane wall). Walls are stronger and stiffer in plane than out of plane.
Sheathing	The board, lining or panel material used in floor, wall and roof assemblies of both residential and commercial construction
Stressed skin panels	Structural flat plates which rely on composite action. Flexural strength is provided by the skins and shear resistance is provided by the filling of webs between the skins.

C9.1.3 Notation

Symbol	Meaning
%NBS	Percentage of new building standard (refer to Section C1).
A	Sectional area of one chord (mm^2)
a	Aspect ratio of each sheathing panel: • 0 when relative movement along sheet edges is prevented • 1 when square sheathing panels are used 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
B	Depth of diaphragm
B	Distance between diaphragm or shear wall chord members (mm)
b	Width of sheathing board
E	Elastic modulus of the chord members (MPa)
e_{n}	Nail slip resulting from the shear force V (mm)
F_{c}	Characteristic stress in the sheathing board in compression parallel to the grain
F_{n}	Nominal nail strength
G	Shear modulus of the sheathing (MPa)
H	Height of the storey under consideration (mm)
L	Span of a horizontal diaphragm (mm)
l	Spacing between joints or studs
m	Number of sheathing panels along the length of the edge chord
N	Total number of nails.
S_{p}	Structural performance factor in accordance with NZS 1170.5:2004
s	Nail spacing
t	Thickness of the sheathing board (mm)
V	Shear force in storey under consideration (N)
v'	• 74 N/m for 25 mm sawn boards • 148 N/m for 50 mm sawn boards, and • 222 N/m for tongue and groove boards
W	Lateral load applied to a horizontal diaphragm (N).
z	Section modulus of the sheathing board = $\frac{b^2t}{6}$, where t is the thickness of the board.
Δ_1	Diaphragm flexural deformation considering chords acting as a moment resisting couple (mm)
Δ_2	Diaphragm shear deformation resulting from beam action of the diaphragm (mm)
Δ_3	Deformation due to nail slip for horizontal diaphragm (mm).
Δ_4	Deformation due to support connection relaxation
Δ_5	Wall shear deformation

Δ_6	Deformation due to nail slip
Δ_7	Deformation due to flexure as a cantilever (may be ignored for single storey shear walls).
δ_c	Vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)
δ_t	Vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored).
θ	Flexural rotation at base of storey under consideration (radians)
ϕ	Strength reduction factor

C9.2 New Zealand Construction Practices

C9.2.1 Timber building types

Timber is a readily available building material in New Zealand. It has been used widely since the earliest European settlement for many different building types including residential, office, industrial and public buildings.

The two main categories of timber buildings are:

- timber framed structures such as those designed using non-specific design guides and standards, and
- engineered buildings such as halls, commercial and industrial buildings.

Some examples are shown in Figure C9.1.

Timber is also used in other building types (refer Section C9.2.4).

C9.2.2 Timber framed structures

C9.2.2.1 Frames and bracing

Timber framed structures employ what is commonly referred to as stick framing: small section timber such as 90 mm x 45 mm (historically 4” x 2”). These elements are combined to create wall frames with timber studs, and top and bottom plates. In older structures (prior to the introduction of NZS 3604:1978), bracing was commonly provided by the addition of let-in diagonal braces (typically 6” x 1” or 4” x 1”) or cut-in diagonal braces (typically 4” x 2” or 3” x 2”) between the studs. Occasionally, older buildings with timber framed walls rely on an internal lathe and plaster lining to provide the bracing rather than employing diagonal members. For a period between the use of lathe and plaster and sheet linings, around the early 1900s to 1920s, wide horizontal boards approximately 25 mm is often used as a backing for scrim and then wallpaper is applied over this.

Modern timber framed walls are likely to be reliant on their lining materials for providing bracing resistance. Linings include plasterboard, plywood, oriented strand board (OSB), particle board and sometimes fibre cement board.

C9.2.2.2 Floors

Floors of timber framed structures consist of multiple horizontal joist members between 400 mm and 600 mm apart. The span limits for sawn timber members tend to restrict the size of rooms in the building. More modern buildings may use engineered products for the floor system; such as engineered wood joists (I joists), laminated veneer lumber (LVL) joists, solid glulam panels or nail plated parallel flange truss joists. In these cases, the spans are likely to be greater than for sawn timber. The joists of ground floors are typically seated on timber bearers on piles. Upper floor joists are typically seated on the top plates of the walls or a ribbon plate side fixed to wall framing.

Older floors are generally constructed using tongue and groove strip timber members up to approximately 200 mm wide, fixed with two nails at each joist crossing. Some old commercial structures may have a “mill floor” which is a solid panel consisting of timber planks on edge and nail-fixed together. More modern floors are typically constructed with sheet materials such as particle board, plywood or fibre cement board products, fixed with nails or screws around sheet perimeters. Other fixings are generally used in the in-field area of the sheet at larger spacings to the intermediate joists.

C9.2.2.3 Roofs and ceilings

Older style roof framing includes rafters spanning between eaves and ridges (often supported at intermediate points by a propped under-purlin), overlaid with purlins to which the roofing products are attached. Sometimes, solid or hit-and-miss strip timber sarking is present as an alternative to purlins. Ceiling linings can be supported on ceiling joists (or the underside of floor joists in upper floors), which also span between walls. Ceiling joists are typically of a smaller depth than floor joists and intermediate support is often provided from above.

Modern roof construction typically consists of timber roof trusses with pressed metal tooth plate connectors spanning from outside wall to outside wall.

In both cases, bracing is provided in the roof space (either in the plane of the roof or in the roof space down to the top plates of internal walls), particularly if the roof has gable ends. Because of their shape, hip roofs generally support themselves against lateral loads and other bracing is not usually necessary.

C9.2.3 Engineered timber buildings

While engineered timber buildings can take many forms, their main characteristic is that they generally use larger member sizes, such as heavy posts and beams, to achieve greater spans. Systems include portal frames with moment resisting knee joints and bolted timber trusses. The portals may be constructed from glue laminated timber, round wood or LVL with glued, bolted or splice plated knee joints (e.g. steel and griplam nails or plywood with dowel type connectors, e.g. nails, screws, drift pins, bolts).

When heavy bolted timber trusses are used in conjunction with heavy sawn timber columns, a diagonal timber brace is often included at the connection between the truss and the column to provide a moment connection, thus creating portal-like action. In the orthogonal direction either steel angle or rod bracing is employed, or light timber framed walls with timber sheet material is used to resist the lateral loads.

Figure C9.1: Examples of timber buildings

C9.2.4 Timber in other building types

Timber has been used extensively in buildings, primarily constructed of other materials, for floor joists, roof framing, floors and sarking under roofs etc. If assessing such a building this section may need to be read in conjunction with the specific sections detailing the assessment of these other building types (Section C5 Concrete Buildings and Section C8 Unreinforced Masonry Buildings).

C9.3 Observed Behaviour of Timber Buildings in Earthquakes

C9.3.1 General performance

In general, low-rise timber framed buildings have performed extremely well with regard to life safety in large earthquakes.

Rainer and Karacabeyli (2000) carried out a survey of observed damage to timber framed buildings caused by eight significant earthquakes in the USA, Canada, New Zealand and Japan. Their study concluded that two-storey timber framed buildings largely met the life safety criterion required by design standards. The fatalities recorded in timber framed buildings were predominantly in larger (three-storey to four-storey) buildings or as a result of external hazards such as landslides. When subject to peak ground accelerations in excess of 0.6 g some Californian two-storey timber buildings exhibited soft-storey behaviour and suffered partial collapse; while in Kobe, Japan, minimal damage was observed.

In the 1987 earthquake centred at Edgecumbe in the Bay of Plenty, fewer than 50 of the nearly 7000 houses in the affected region suffered substantial damage and none collapsed (Pender and Robertson, 1987). The majority of buildings in this region were of 1ight timber frame construction and about two thirds were constructed during the period 1950 to 1979, prior to the introduction of NZS 3604:1978. Most of the significant damage occurred to building foundations. However, hundreds of houses suffered some lesser degree of damage including sliding off their foundations, damage to brick veneers, chimney collapse, and failure of foundation posts and roof struts.

The Christchurch earthquake of 22 February 2011 provided substantial evidence on building performance given that the majority of houses in the Canterbury region are light timber framed buildings. Buchanan et al. (2011a) summarised the observed damage to timber framed housing due to this earthquake, noting that single-storey and two-storey light timber framed buildings performed extremely well for life safety. The only recorded fatalities in timber framed residential buildings were attributed to rockfall.

The performance of engineered timber buildings was also reviewed by Buchanan et al. (2011b). The authors noted that these buildings generally performed well both for life safety and serviceability, with most buildings ready for occupation a short time after the event. Most of the damage that occurred resulted from lateral spreading, settlement resulting from liquefaction and unusually high levels of horizontal and vertical ground acceleration.

Other observations included that structural and non-structural damage was common but, in general, the structural integrity of these buildings was maintained. A small number of soft-storey failures were observed in older two-storey timber framed houses, but these typically did not result in collapse. These failures were often due to minimal bracing in the lower floors, potentially as a result of alterations.

Significant damage was observed to the internal wall linings of some timber buildings, particularly those with an asymmetric layout and large window openings. Damage to and collapse of brick veneers, unreinforced chimneys and heavy roof tiles was common in areas subject to high peak ground accelerations.

Concrete slab foundations generally performed well unless subject to liquefaction induced settlement or lateral spreading; although failure of the connection to a foundation wall or edge thickening was common where the slabs were unreinforced.

Foundations with short concrete piles and concrete perimeter walls generally performed very well, particularly when the perimeter walls were reinforced. Similar to slab foundations, damage was observed to pile foundations when subject to liquefaction induced settlement or lateral spreading.

Note:

The general good performance of timber buildings in earthquake is considered to be due, at least in part, to their relatively low supported mass and ability to deform considerably (via deformation in the connections) without loss of gravity load support. This means that while serviceability may not necessarily be achieved, it is unlikely that buildings of this type will create a significant life safety hazard even during severe earthquake shaking. Care should be taken, however, when there are elements within timber structures that either increase the mass (e.g. heavy wall partitions) or indicate a potentially vulnerable mechanism is present that would concentrate nonlinear behaviour (e.g. a cantilever or poorly cross braced sub-floor structure).

C9.3.2 Performance in the Canterbury earthquakes

A large number of school buildings in Christchurch are constructed from timber. These include both classroom-type buildings of one or two storeys and large span buildings such as hall and gymnasia. These school buildings were reviewed extensively following the Canterbury earthquake sequence of 2010/11 (Opus International Consultants, 2015), providing a platform for reviewing the performance of timber buildings generally.

Despite high levels of peak ground acceleration at a range of school sites across Canterbury during these earthquakes, no school structures collapsed and no serious injuries or fatalities were recorded. Therefore, timber school buildings performed well in the Canterbury earthquakes from a life safety perspective, confirming previous expectations. However, significant damage was caused by lateral spreading and liquefaction.

Further, the detailed engineering evaluations after the earthquakes of the Christchurch schools– many of which are expected to have been subjected to severe earthquake shaking – have shown that timber buildings performed better than conventional methods of theoretical structural analysis would suggest. This is because timber buildings (with the exception of portal framed structures typically used in warehouses, halls and gymnasia) generally have many additional load paths that are not easily quantifiable but that are able to carry and redistribute loads and deform significantly in a seismic event.

Alert:

The results of two separate full scale tests commissioned by the Ministry of Education and one by Housing New Zealand in 2013 (refer Appendix C9A) confirmed observations from the Canterbury earthquake sequence of the resilience of timber framed buildings. These tests confirmed the view held by many structural engineers that timber framed buildings constructed before the establishment of modern seismic codes have an inherent lateral resistance and deformation capacity beyond that which can be readily calculated. Timber framed buildings meeting modern seismic code requirements are expected to have earthquake resilience that meets or exceeds current minimum building code requirements for life safety.

Previous work by the Ministry of Education following the 1998 National Structural and Glazing Survey included the replacement of most heavy roofs on its school buildings. This action undoubtedly improved the performance of the lightweight building stock and has reduced the risk of serious damage during seismic events.

C9.4 Assessment Approach

C9.4.1 General assumptions and considerations

This section outlines the assessment approach for both timber framed structures and engineered timber buildings. The approach and extent of analysis will vary with the building's complexity and the degree of certainty regarding element capacities and should be in accordance with the objectives outlined in Section C1 and the procedures outlined in Section C2.

Also refer to Sections C1 and C2 for guidance on:

- documentation that should be sourced to undertake the assessment; inspection requirements to verify the design is in accordance with the design documentation;
- what, if any, intrusive testing should be considered; and
- the general assessment and analysis procedures that should be considered.

As noted in Section C9.1.1:

- Timber framed buildings and engineered timber buildings meeting modern standards are not expected to be earthquake prone unless a particularly vulnerable aspect is present and, even then, this would need to be one which would lead to a significant life safety hazard in the event of failure.
- An ISA typically performed before any DSA can identify high risk building elements such as heavy roofs, chimneys and poor foundation systems that can adversely affect the performance of a timber building. The mitigation or replacement of these can increase the expected building performance, potentially making it unnecessary to undertake a detailed analysis/assessment.

Alert:

Analysis of the results from full-scale testing of timber buildings (Brunsdon, et al (2014) Connor-Woodley (2015) and BRANZ (2015)) has indicated that the global seismic performance of these buildings is expected to be very good (when considered against life safety objectives) and far greater than the results of structural calculations may suggest.

As a result, a revised structural performance factor, $S_{p,}$ of 0.5 (a lower bound of 0.7 has typically been used) is recommended when completing a DSA for timber buildings as outlined later in this Section.

The following general assumptions and considerations should be used in the assessment of timber buildings:

- The assessing engineer should have access to relevant design standards including NZS 1170.5:2004, NZS 3603:1993, NZS 3604:2011 and AS 1720:1998.
- The assessor should identify the critical and controlling load paths, the strength hierarchy and likely mechanisms of the system to assist with determining the available ductility capacity using a rational analysis (where possible).
- An inelastic analysis is not considered necessary for the majority of timber buildings due to the flexibility of the diaphragms and ability to redistribute lateral load between timber elements of different stiffness on different bracing lines. However, an appreciation of the deformation capacity of timber elements is considered essential when these are being used in conjunction with elements of other material types and timber systems with significantly differing deformation capacities.
- The provision that 8% of the horizontal seismic base shear is applied at the eaves/roof level should only be considered if a heavy roof is present or a heavy wall is required to be propped at roof level.
- Assessment is based on probable capacities (i.e. the strength reduction factor (ϕ) is set to 1.0).
- The material properties and member/element capacities set out in Sections C9.5 and C9.6 respectively can be used as default values. The element capacities in Section C9.6

assume that the load path into and out of each member is complete and sufficient to transfer the required demands. This should be confirmed.

- Traditional sawn timber has a wide range of strength properties, with the lower fifth percentile strength used for characteristic properties. In structures with many contributing timber members the collective performance may approach a mean population strength, giving significant reserve strength capacity.
- Subject to sensitivity and complexity of analysis a greater degree of certainty may need to be obtained for some material properties. For example, variations in timber density can significantly influence connection stiffness and capacity.
- Where they are not visible and there is no drawing record, walls should be assumed to have no diagonal braces unless otherwise confirmed by site investigation.
- The specified lateral seismic deflection limits specified in NZS 1170.5:2004 are not expected to be relevant for typical timber buildings when the focus is on life safety. This is because the mass supported is typically low and considerable deflections can generally be sustained. However, this should not be considered a blanket relaxation as in some cases (e.g. when there is a large supported mass (roof or wall) careful appraisal will be required before the deflection limits are ignored.
- Portal framed structures are typically governed by deflection limits and non-seismic actions, so stiffness rather than strength will have governed section size. Joint strength and deformation capacity is then critical. Joints designed using dowel type fasteners typically have reserve strength capacity since fastener design capacities are usually limited by timber bearing/crushing, which is not a brittle mechanism.

For buildings constructed primarily of other materials (e.g. concrete or unreinforced masonry) but with timber elements that could affect their seismic performance, it is important to determine the state of the connection between the floors and the supporting walls and/or the sarking to the roof. This will have a direct bearing on whether or not the floors and/or roof can act as a diaphragm in distributing the seismic floor loads to the walls and whether the walls are tied together. Therefore, the state of the wall/diaphragm connection may determine the possible load paths for transferring seismic actions down to the foundations.

C9.4.2 Force based approach

A force based assessment approach is generally considered sufficient for most simple low-rise timber framed buildings.

Note:

A displacement based assessment approach is considered essential when timber elements of varying deformation capacity are being used in combination or when timber elements are being used in conjunction with elements of other materials.

For a force based assessment of a timber building it is generally acceptable to use a structural ductility factor where a ductile mechanism can be identified and the factor can be justified. If the mechanism(s) cannot be identified with certainty, the mechanism should be assumed to be brittle and the structural ductility factor limited to 1.25.

Alert:

In older timber buildings a capacity design is unlikely to have been undertaken, so brittle failure mechanisms may be present.

For timber framed buildings no more than two storeys high and with regular layouts, the bracing design provisions of NZS 3604:2011 can be adopted. This option should only be adopted if the distribution and spacing of bracing walls is generally in accordance with NZS 3604:2011. As bracing demands given in NZS 3604:2011 are derived from $\mu = 3.5$ and $S_{\mathrm{p}} = 0.70$, these demands should be scaled accordingly for other values of μ and S_{p}.

For engineered buildings, multi-unit buildings and complex layouts, earthquake demands should be calculated in accordance with Section C3 with the amended provision that the 8% allowance applied at eaves/roof level should only be considered if a heavy roof is present or upper support of a heavy wall is required.

A structural performance factor of S_{p} =0.5 is recommended in most cases for the assessment of timber buildings with a structural ductility factor $\mu \geq 2.0$.

The exception is buildings where the overall failure mechanism is assumed to be brittle (e.g. for buildings with weak or brittle sheeting or non-ductile configurations) and the structural ductility factor is limited to 1.25. For such buildings a higher structural performance factor of 0.9 should be used.

Note that a structural performance factor of less than 0.5 may be considered for particular buildings if justified by testing. Also refer to Section C9.4.3 below for additional considerations that may alter these values.

Alert:

The structural performance factor takes account of a number of effects including structural redundancy, additional energy dissipation, the likely short duration of peak load, and higher material strengths and connection capacities.

The value $S_{\mathrm{p}} = 0.5$ is considered reasonable based on observed behaviour in earthquakes and in the destructive testing of timber framed buildings.

A force based assessment will require determining the probable flexural, shear, axial and bracing capacities of the members, elements and connections using the information in Sections C9.5 and C9.6 and other references as necessary. In doing so, the potential failure mechanisms should be identified and, where relevant, plastic hinge rotation capacities and nail slip capacities determined by rational analysis.

It is emphasised that failures in timber connections can be brittle. Reference can be made to the European Yield Model (EYM) and Brittle Failure methods (EN 1995 and Quenneville, 2009) or other similar methods to determine the failure mode for connections.

The global seismic rating should be determined in accordance with Section C1 using the probable strength capacity of the global structure and the global base shear demand determined from Section C3.

C9.4.3 Displacement based approach

As noted earlier, a displacement based assessment approach taking account of nonlinear behaviour is recommended as limiting the capacity to first yield is likely to underestimate the capacity in many structures. It is also essential when attempting to combine together the contribution of systems of various nonlinear deformation capability and/or different materials. All assessment procedures outlined in Section C1 have as the first step completion of a SLaMA. This requires the assessor to have a good understanding of the deformation capacity of the various systems to ensure displacement compatibility issues, particularly when the deformations are in the nonlinear range, are addressed.

Appendix C9B provides further resources for this approach:

Alert:

To assist with using a displacement based approach, BRANZ (BRANZ, 2013; BRANZ, 2015) has tested a variety of wall systems commonly used in New Zealand timber construction to better understand the lateral load resisting behaviour of the various systems. These tests have provided probable strength and deformation capacities and stiffnesses for a range of bracing systems.

The global seismic score should be determined in accordance with the procedures outlined in Section C2

C9.4.4 Other issues

C9.4.4.1 General

Assessors should consider any particular vulnerabilities or weaknesses within the structure and use their engineering judgement to consider the effects of these.

Some likely issues include: horizontal irregularity, vertical irregularity, heavy roofs and masonry veneer claddings, building condition, foundations and slope considerations, geotechnical hazards, and stairs. These are discussed below, together with suggestions about how to alter the recommended ductility and structural performance factors accordingly.

C9.4.4.2 Horizontal irregularity

Where horizontal irregularities exist, the assessing engineer should consider the torsional behaviour of the building; in particular, the diaphragm performance with reference to Section C9.6.3

C9.4.4.3 Vertical irregularity

Vertical strength/stiffness irregularities, such as soft storeys, can occur in two and three storey, multi-unit residential buildings and also in buildings with garages on the ground floor. If a soft-storey mechanism is likely, the engineer should pay particular attention to the connections between bracing elements and the magnitude and consequences of P-delta actions. As a result, a reduced structural ductility factor may need to be assumed (for a force-based procedure) to take account of the likelihood of a shake-down scenario.

C9.4.4.4 Heavy roofs and masonry veneer claddings

The presence of a heavy roof and/or masonry veneer cladding may increase the dynamic response of a building due to the additional high mass. The engineer should ensure that the mass is appropriately accounted for in the analysis, and may consider increasing the minimum structural performance factor of 0.5.

C9.4.4.5 Building condition

Alterations, post-construction, are common in timber buildings but are not always visible. The engineer should undertake an appropriate level of inspection to provide confidence that any alterations, such as the removal of walls, have been identified. This may involve intrusive works in roof spaces, wall cavities and sub-floors.

Building damage, deterioration, corrosion of structural elements and the effects of biological decay (such as borer infestation and wood rot) should be considered and the capacities downgraded accordingly.

C9.4.4.6 Foundations and slope considerations

If a building is constructed on concrete perimeter walls and the sub-floor height is 0.8 m or less, it is considered reasonable to assess the building considering the ground floor as the base of the building (i.e. as if it were constructed on a slab-on-grade). If the sub-floor height is greater than 0.8 m, the ground floor mass should be included, but the mass of foundation perimeter walls should not be included to calculate the equivalent static forces to be applied at the upper levels of the building.

Inadequate or poor connection of the superstructure to the piles is common. Buildings which have a sub-floor height of 600 mm or less are unlikely to present a life safety hazard if they come off their foundations (although significant damage may result). Therefore, the capacity of the sub-floor in these buildings should not govern the *%NBS* seismic rating for the building.

The capacity of bolted connections in foundations should be calculated using the provisions of NZS 3603:1993, NZS 3604:2011, or EN 1995 and Quenneville (2009).

For timber framed buildings supported solely on pile foundations, it is recommended that a structural performance factor of 0.5 is applied when it can be determined that the piles are operating effectively as cantilevers from the soil and are well connected to the sub-floor framing. If this is not the case, a more conservative structural performance factor may need to be applied.

If a building is constructed on a slope greater than 1 in 8, as shown in Figure C9.2, this may require a review of the sub-floor bracing design and construction. If there are substantial foundation cross-bracing elements present or if the building is supported by a reinforced concrete or reinforced concrete block retaining wall that is not showing any signs of movement, then typically no further sub-floor assessment is considered necessary. If these elements are not present then further assessment should be undertaken. It is also recommended that the centre of rigidity for the subfloor system is checked in relation to the location of the centre of mass to check for potential torsional effects.

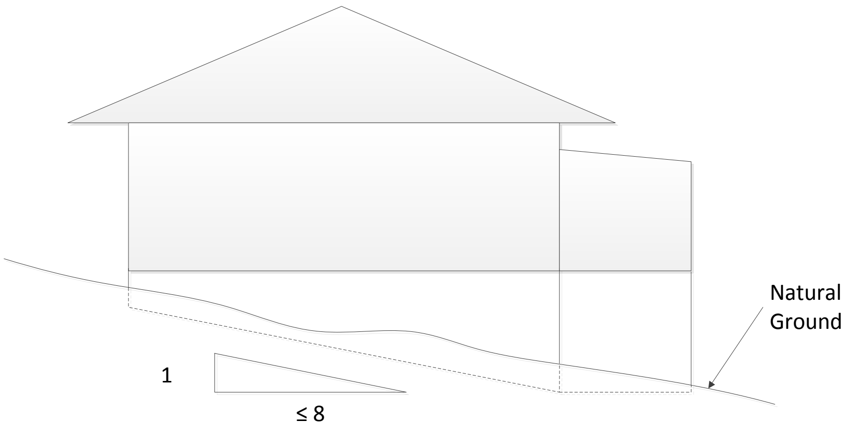

Figure C9.2: Definition of sloping ground

C9.4.4.7 Geotechnical hazards

As settlement of timber framed buildings caused by liquefaction is unlikely to lead to a significant life safety hazard, the effects of liquefaction should not govern the *%NBS* seismic rating. However, other geotechnical hazards such as slope failure that could lead to a significant loss of foundation support may be critical

Refer to Section C4 to assess potential geotechnical hazards that may be relevant to a particular site.

C9.4.4.8 Stairs

Internal stairs constructed of timber are unlikely to lead to a significant life safety hazard due to loss of egress but may contribute to an irregularity in structure stiffness.

External stairways, depending on their construction type, may be more vulnerable than internal stairs and should be checked.

C9.5 Material Properties

C9.5.1 General

Probable or expected values for the material properties, should be used when assessing an existing timber building to obtain the best estimate of the strengths and displacements of members, joints and connections.

It is intended that a ϕ factor of 1.0 should be used for timber components including poles and glulam members.

The effect of variations in material strength on the hierarchy of failure should be considered.

C9.5.2 Material strengths

Strength assessments of existing materials may be made from the results of tests. If no test results are available, the assessor should either conduct suitable tests or assess conservative values of strength by comparison with the properties of similar timbers as those given in NZS 3603:1993, the Timber Design Guide (NZTIF, 2007) or other recognised sources such as technical literature from manufacturers for products such as glulam and LVL.

For timber structures built before 2000, (reproduced from NZS 3603:1993) probable material strength values may be taken from Table C9.1. (Note that the values in this table vary from the values given in Amendment 4 to this standard.)

For timber structures built from 2000 onwards the probable material strengths provided in Amendment 4 to NZS 3603:1993 should be used. (Note that the timber in almost all buildings constructed during this period is either Radiata pine or Douglas fir.)

Table C9.1: Probable material strengths for visually graded timber (MPa) (from NZS 3603:1993)

Species	Grade	Bending	Compression parallel	Tension parallel	Shear in beams	Compression perpendicular	Modulus of elasticity (GPa)
1. Moisture condition – Dry (m/c = 16% or less)							
Radiata pine	No. 1 Framing	17.7	20.9	10.6+	3.8	8.9	8.0
Radiata pine	Engineering	24.5	24.2	12.2	3.8	8.9	10.0
Douglas fir	No. 1 Framing	17.7	22.1	10.6+	3.0	8.9	8.0
Douglas fir	Engineering	22.4	25.4	11.2	3.0	8.9	9.9
Larch	No. 1 Framing	22.7	27.1	13.6	3.5	8.9	9.6
Rimu	Building	19.8	20.1	11.8	3.8	10.9	9.5
Kahikatea	Building	14.5	19.5	8.6	3.0	5.9	6.8
Silver beech	Building	23.6	24.8	14.2	3.5	7.1	9.3
Red beech	Building	28.0	30.4	16.8	5.3	12.4	13.4
Hard beech	Building	29.5	26.6	17.7	5.0	14.2	13.6
2. Moisture condition – Wet (m/c = 25% or greater)							
Radiata pine	No. 1 Framing	14.8	12.7	8.9++	2.4	5.3	6.5
Radiata pine	Engineering	20.1	15.0	10.0	2.4	5.3	8.1
Douglas fir	No. 1 Framing	14.8	14.5	8.9++	2.4	4.7	6.5
Douglas fir	Engineering	20.1	17.1	10.0	2.4	4.7	8.0
Larch	No. 1 Framing	15.0	17.4	8.9	2.7	5.6	7.7
Rimu	Building	15.0	14.5	8.9	2.7	6.8	8.3
Kahikatea	Building	13.9	14.2	8.3	2.4	4.4	6.0
Silver beech	Building	20.7	19.2	12.4	2.7	3.8	7.5
Red beech	Building	25.1	18.3	15.0	3.8	7.7	11.3
Hard beech	Building	28.3	24.2	17.1	4.4	10.6	12.1

Notes:

+ Reduced to 8.8 MPa in 1996

++ Reduced to 7.4 MPa in 1996

Note:

The characteristic tension stress parallel to the grain was reduced for a number of the species in 1996, after new testing. However, the reduction in stresses for Radiata pine and Douglas fir are the only ones included here because very few, if any, of the other species would have been used in building construction after that date.

C9.5.3 Modification factors

The modification (k) factors given in NZS 3603:1993 should be used where appropriate.

C9.6 Element Capacities

C9.6.1 General

The structural systems of timber structures are typically made up of multiple members/elements which collectively define the strength and deformation capacity of the system as a whole. Behaviour of the elements (including shear walls, diaphragms, beams, columns, and braces) is dictated by physical properties such as: area; material grade; thickness, depth and slenderness ratios; lateral torsional buckling resistance; and connection details. Connected members include sheet products, planks, linear bracing, stiffeners, chords, sills, struts, and hold-down posts.

The actual physical dimensions of individual timber members/elements that are being relied for load transfer should be measured rather than relying on nominal sizing, e.g. nominal 100 mm x 50 mm stud dimensions are generally less due to choice of cutting dimensions and later machining and/or seasoning shrinkage. Modifications to member capacities can be caused by notching, holes, and in some situations splits and cracks. The presence of decay or deformation should be noted and allowed for.

The connections are an important aspect of timber systems and often determine the deformation capacity as a whole. The type, size, spacing and condition of fixings such as nails will often be critical when determining the capacity and although it will be difficult and impractical to confirm every fixing, checks should be made to confirm the general arrangements and condition.

The physical properties of the various components are needed in order to characterise building performance properly for a detailed seismic assessment. The starting point for establishing the properties should be the available construction documents. Accordingly, carry out a preliminary review of these documents to identify primary vertical (gravity) and lateral load-carrying elements and systems, and their critical members and connections.

Next, conduct site inspections to verify conditions and make sure that building alterations have not changed the original design. In the absence of a complete set of building drawings, inspect the building thoroughly to identify these members, elements, and systems, as described in Section C1. If reliable record drawings do not exist, an as-built set of building plans may need to be created. This may necessitate removal of linings to observe critical structural connections. Establish the extent, sizing and connection of connections.

C9.6.2 Timber shear walls

C9.6.2.1 General

The important failure modes for wood and light frame shear walls, are sheathing failure, connection failure, tie-down failure, and excessive deflection. The expected strength capacity of wood and light timber frame shear walls should be taken as the yield capacity of the shear wall assembly. For assemblies for which specific bracing test information is available (typically following test procedures such as the P21 test used in New Zealand to determine wind and earthquake ratings of bracing elements in timber framed structures built since 1980) the derived bracing ratings from those tests may be used. The probable capacity may be taken as the maximum capacity of the assembly, regardless of the deflection.

Note:

The behaviour of wood and light frame shear walls is complex and influenced by many factors; the most significant of which is the wall sheathing. Wall linings can be divided into many categories (e.g. brittle, elastic, strong, weak, good at dissipating energy, and those poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g. a wall sheathed with wood lath and plaster). Other factors that can influence the behaviour of shear walls include the fixing pattern and the hold-down connections.

Some older shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies – framing, covering, anchorage – been tested using cyclic loading procedures.

If different walls are lined with dissimilar materials along the same line of lateral-force resistance, the analysis should be based on the resistance of the individual elements maintaining displacement compatibility.

For overturning calculations on shear wall elements, stability should be evaluated in accordance with AS/NZS 1170.0:2002. Net tension due to overturning should be resisted by uplift connections at the ends of the element unless a rocking system can be justified.

It is important to consider the effects of openings in shear walls. This is because the presence of anything other than a small opening in a shear wall will cause a reduction in the stiffness and yield capacity due to a reduced length of wall available to resist lateral forces. Special analysis techniques and detailing are required assess the effects of openings. The presence of chord members around the openings will reduce the loss in overall stiffness and limit damage in the area of openings. Equally the effect on behaviour when these members are not present should be carefully considered.

C9.6.2.2 Types of timber shear walls

Transverse board lining

Transverse board lining consists of boards up to 25 mm thick and usually 100-200 mm wide, nailed in a single layer at right angles to the studs.

These walls tend to be overlaid with scrim material and wallpaper in residential construction. The sheathing resists the shear force caused by lateral loading. The perimeter members carry axial loading from the gravity loads and the lateral loading, whereas the intermediate studs are not loaded axially by the lateral loading but nevertheless provide support to the sheathing and the enable interconnection of sheathing elements.

The moment resistance provided by the nail couples at each stud crossing is the lateral load resisting mechanism. The resisting mechanism of the couplet is less effective with narrower boards but there are more couplets for the same wall height, meaning the wall result is similar for all board widths. Nail slip is the dominant cause of lateral deflection in shear walls of common dimensions. Flexural strains in the chord members and shear distortion in the sheathing itself may also contribute to the total deflection.

Single diagonal sheathing

The shear force applied to the shear wall is carried by tension or compression in the 45º diagonal sheathing and is transferred to the perimeter members by the nails.

This form of shear wall is likely to be found on external walls of warehouses, large school buildings and hall type structures between the column supports of portal frames or braced trusses.

Double diagonal sheathing

Two layers of sheathing on the same side of the framing significantly improves the shear characteristic of a shear wall. When double diagonal sheathing is used with one layer diagonally opposed to the other, one layer acts in tension and the other in compression and the shear is assumed to be shared; thus, the two layers act as a shear membrane.

Panel sheathing

This consists of wood structural panels (such as plywood or oriented strand board), gypsum plasterboard, or fibre cement board that is placed on framing members and nailed in place. Different grades and thicknesses of panels may have been used on one or both sides of the wall depending on requirements for gravity load support, shear capacity, and fire protection. Edges at the ends of the structural panels are usually supported by the framing members. Edges at the sides of the panels could have been blocked or unblocked.

Fixing patterns and fixing size can vary greatly. Spacing is commonly in the range of 75-150 mm on centre at the supported and blocked edges of the panels, and 250-300 mm on centre at the panel interior. In older construction, the fixings were usually nails. In more modern construction using gypsum plasterboard and some fibre cement board products, the fixings may be screws.

C9.6.2.3 Strength and stiffness of timber shear walls

An assessment of the strength of timber shear walls should be based on an assessment of the materials making up the particular shear wall and their individual strengths. Depending on the wall type, the formulae given in Appendix C9C can be used to determine the shear wall strength. In the absence of test results, the maximum values contained in Table C9.2 may be used in lieu of more detailed calculations.

Stiffness can be calculated using the formulae in Appendix C9D. For many shear walls the major component affecting the stiffness is the nail slip. It is acceptable to base the stiffness initially on the nail slip component of deformation unless the nail spacing is sufficient to induce large forces in the cladding.

Table C9.2: Probable strength values for existing timber framed wall bracing systems (based on 2.4 m wall height)

Bracing type	Strength values (ϕ = 1.0)
150 x 25 mm let-in brace at 45º	2.0 kN
150 x 25 mm let-in brace at 45° and sheet material* one face	2.5 kN
150 x 25 mm let-in brace at 45° and sheet material* both faces	3.7 kN
90 x 45 mm fitted brace both ways at 45º	2.0 kN
90 x 45 mm fitted brace both ways at 45º and sheet material* one face	2.5 kN
90 x 45 mm fitted brace both ways at 45º and sheet material* both faces	3.7 kN
90 x 45 mm dog leg brace (600 mm wall length)	0.75 kN
Timber framed stud walls with wood or metal lath and plaster.	1.5 kN/m each side
Timber framed stud walls with diagonal braces and wood or metal lath and plaster	2.8 kN/m
Gypsum plasterboard one side, and fixed at 300 mm centres (no diagonal timber braces included)	1.0 kN/m
Gypsum plasterboard one side, and fixed at 150 mm centres (no diagonal timber braces included)	2.5 kN/m
Gypsum plasterboard two sides, and fixed at 300 mm centres (no diagonal timber braces included)	2.0 kN/m
Gypsum plasterboard two sides, and fixed at 150 mm centres (no diagonal timber braces included)	3.0 kN/m
Match lining on one or both faces (no diagonal timber braces included)	1.25 kN/m
3.2 mm tempered hardboard fixed with clouts at 200 mm centres	3.0 kN/m
Horizontal board sheathing	1.0 kN/m
Horizontally oriented corrugated steel sheets	2.0 kN/m
Vertically oriented corrugated steel sheets	1.50 kN/m
140 x 20 mm bevel back weatherboard	0.30 kN/m

Note:

*Sheet material is defined as having a density of not less than 450 kg/m^3. It may be a wood based material not less than 4.5 mm thick or a gypsum-based material not less than 8 mm thick, both fixed to framing members not closer than 10 mm from sheet edges.

When assessing wall bracing probable capacity using the values in Table C9.2 the capacity of each bracing element should be calculated by multiplying by the length of the bracing element and adjusting for height in accordance with the following equation:

$$\frac{2.4}{\text{element height in metres}}$$

This equation is applicable for framing with sheet bracing products attached (and therefore it is not applicable for bracing systems such as horizontal sarking). Elements less than 2.4 m in height should be rated as if they are 2.4 m high. Walls of varying height should have their bracing capacity adjusted using the average height.

Where bracing units are used in place of force units (e.g. kNs), a conversion of 1 kN = 20 bracing units should be used.

Consideration should also be given to the aspect ratio of the wall element, i.e. its overall height to length ratio. If published indicative bracing ratings are being relied on, it should be ensured that the length of the element is applicable for the published value as failure mechanisms can change with aspect ratio, resulting in altered ratings per unit length. For narrow elements (height: length ratio > 2) consideration should be given to reducing the published capacity. It is suggested that a linear reduction of strength is applied from 1 times the published data for ratios of 2:1 to zero for ratios equal and greater than 3.5:1.

The total bracing capacity of a timber building is the sum of the element capacities. This assumes that the diaphragm has the capacity to distribute seismic loads. If this is not the case, assigning seismic loads on a tributary area basis may be required.

Where weaknesses in the load path have been identified, the capacity of each bracing element should be reduced accordingly.

C9.6.3 Roof and floor diaphragms

C9.6.3.1 General

Conventional structural analyses are based on the assumption that the roof and floor diaphragms are relatively rigid and that the weight of tributary areas on each level, including the diaphragm, can be lumped to act at points on relatively flexible shear walls. That is, the diaphragms are assumed to distribute the loads to walls parallel to the direction of lateral loading without significant out-of-plane loading of the walls perpendicular to the direction of loading. Refer Figure C9.3.

The expected probable strength of timber diaphragms should be taken as the yield capacity of the diaphragm assembly. The effects of openings in timber diaphragms also need to be considered. The presence, or lack, of chords and collectors will affect the load carrying capacity of the diaphragm. Connections between diaphragms and other components, including shear walls, drag struts, collectors, cross ties, and out-of-plane anchors, also need to be considered.

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size and spacing of fasteners, existence of perimeter chord or flange members, and the ratio of span length to width of the diaphragm. The presence of anything other than small openings in diaphragms will cause a reduction in the stiffness and yield capacity of the diaphragm due to a reduced length of diaphragm available to resist lateral forces. Special analysis techniques and detailing are required at the openings.

The presence or addition of trimming members around the openings will reduce the loss in stiffness of the diaphragm and limit damage in the area of the openings. The presence of chords at the perimeter of a diaphragm will significantly reduce diaphragm deflections due to bending, and will increase the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness due to chords in a single straight sheathed diaphragm is minimal due to the flexible nature of these diaphragms.

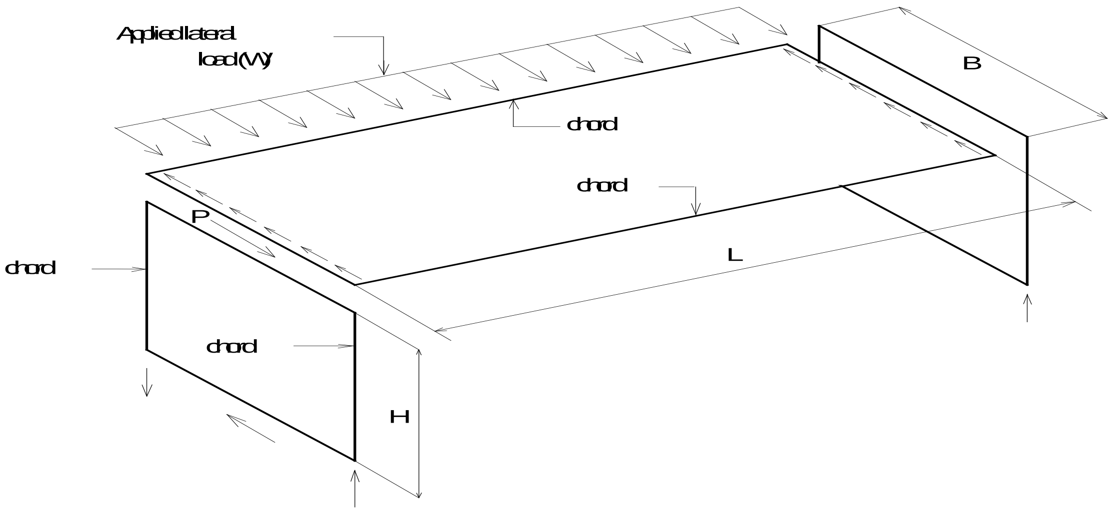

Figure C9.3: Distribution of loading for a horizontal diaphragm

C9.6.3.2 Types of timber diaphragms

Square sheathing

This type of diaphragm consists of 25 mm thick boards, usually 100-200 mm wide, nailed in a single layer at right angles to the cross members such as joists in a floor or rafters in a roof. In a floor, the boards are usually tongue and groove in order to improve the interconnection between the boards and thus improve the vertical load sharing ability of the system. In a roof, the boards are often square edged with no interaction between boards.

Note that sometimes the boards may be spaced with gaps between the boards as wide as the width of the boards. In such cases the diaphragm action will be less because of the smaller number of nail couplets per unit area.

The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, the sheathing was nailed with 60 mm or 75 mm long, 3.15 mm diameter jolt head nails, with two or more nails per sheathing board at each support. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple and some major axis bending of the sheathing boards. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints, which then work in weak axis bending.

Single diagonal sheathing

This consists of sheathing boards of 25 mm thickness and 100-200 mm wide, nailed in a single layer at a 45º angle to the cross members. This type of sheathing was generally only used in roof planes. It was common for the diagonal boards in some areas of the roof to be running at right angles to other areas in order to provide compression struts for loading in opposing directions.

This sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, the sheathing was nailed with 60 mm or 75 mm long, 3.15 mm diameter jolt head nails, with two or more nails per board at each support. The shear capacity of the diaphragm is dependent on the size, number and spacing of the nails at each sheathing board. This type of diaphragm has greater strength and stiffness than square sheathing.

Panel sheathing

This type of sheathing consists of wood, gypsum plasterboard or fibre cement structural panels (such as plywood or particle board) placed on framing members and nailed in place. Different grades and thicknesses of structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the structural panels are usually supported by the framing members. Edges at the sides of the panels may be blocked or unblocked.

Fixing patterns and fixing size can vary greatly. Spacing of fixings is commonly in the range of 75 mm to 150 mm at the supported and blocked edges of the panels, and 250 mm to 300 mm at the panel interior. In older construction, the fixings are generally nails. In more modern construction using gypsum plasterboard and some fibre cement board products, the fixings may be screws.

C9.6.3.3 Strength and stiffness of timber diaphragms

The assessment of probable strength for timber diaphragms should be based on an assessment of the materials making up the particular diaphragm and their individual probable strengths. Depending on the type of timber diaphragm, the formulae given in Appendix C9E can be used to determine its probable strength. In the absence of test results, the maximum values contained in Table C9.3 may be used in lieu of more detailed calculations.

Indicative values for diaphragm shear stiffness are also provided in Table C9.3. Diaphragm shear stiffness can also be calculated using the formulae in Appendix C9F. For many diaphragms, the major aspect affecting the stiffness is the nail slip. It is acceptable to start by basing the stiffness on the nail slip component of deformation.

Alert:

Recent investigations of square sheathed timber diaphragms by Giongo et al. (2014) have indicated that an accurate estimate of the diaphragm shear stiffness compatible with the seismic load is required, rather than relying on a bilinear schematisation characterised by an initial stiffness, yield strength and a ductility factor.

Softboard linings are considered to provide insufficient diaphragm action and should be ignored.

Table C9.3: Probable stiffness and strength values for existing horizontal diaphragms

Diaphragm type		Probable shear stiffness	Probable strength values
A1	Roofs with straight sheathing (sarking) and roofing applied directly to the sheathing – loading parallel to rafters	250 kN/m	4.0 kN/m
A2	Roofs with straight sheathing (sarking) and roofing applied directly to the sheathing – loading perpendicular to rafters	180 kN/m	3.0 kN/m
B	Roofs with diagonal sheathing and roofing applied directly to the sheathing	700 kN/m	10.5 kN/m
C1	Floors with straight tongue and groove sheathing – loading parallel to joists	285 kN/m*	4 kN/m
C2	Floors with straight tongue and groove sheathing – loading perpendicular to joists	215 kN/m*	3 kN/m
D	Floors and roofs with sheathing and existing gypsum plasterboard or fibre cement sheets re-nailed to the joists or rafters	4000 kN/m	Add 1.5 kN/m to the values for Items A1, A2, C1 and C2
E	Gypsum plasterboard ceilings fixed at 150 mm centres to the underside of roof framing (edges blocked) – loading parallel to rafters	7000 kN/m	6 kN/m

Note:
* Fair condition assumed

C9.6.4 Timber portal frames

Because there is a wide range of materials, connections and spans used for timber portal frames, it is not practical to provide a comprehensive table of probable capacities for these. Instead, establish the probable strength of a timber portal frame by using either generic material properties from Section C9.5 for solid timber sections or proprietary information from manufacturers, if known. Generic glue laminated timber properties may be taken from AS/NZS 1328.2:1998. The probable strength of spliced joints may be estimated using NZS 3603:1993, having regard to the connectors used and the splicing products.

C9.6.5 Timber trusses

Trusses in older buildings may use nailed plywood gussets at joints (smaller spans) or multiple member chord and web members with bolted connections (larger spans). The bolted connections may also be strengthened by the addition of split ring or shear plate connectors, but this will be difficult to establish. If the joints include connectors, proprietary strength information may be used if this is available.

C9.6.6 Connections

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections should be determined by a review of the plans and a field verification of the conditions. The connection between a timber diaphragm and the supporting structure is of prime importance in determining whether or not the two parts of the structure can act together. Except for light timber framed buildings, the form of connections is such that the flexural strength at first yield and their post-elastic stiffness can be determined by rational assessment.

In general, the determination of the capacity of connections should be undertaken in accordance with the provisions of NZS 3603:1993. If relevant, more detailed analysis of connection failure mechanisms can be determined using EYM methods. Refer to EN 1995 (2004) and Quenneville (2009).

In unreinforced masonry buildings the connections of timber elements to the masonry are often nominal and generally should not be relied upon for engineering purposes. However, the performance of such connections is influenced by the level of deterioration that may have taken place in both the masonry and the timber members, and by any corrosion of the bolts themselves. When assessing such connections, also refer to Section C8 Unreinforced masonry buildings.

Note:

Section C8 and Beattie (1999) contain further information about the likely performance of timber diaphragm to masonry wall connections.

C9.7 Improving the Seismic Performance of Timber Buildings

The process of conducting a DSA may identify structural weaknesses in the building that could be mitigated to improve its seismic performance.

For timber framed buildings, typical methods for improving seismic performance include:

- removing heavy elements such as concrete tile roofs, masonry veneer or chimneys
- replacing lining materials for existing wall bracing and diaphragm elements
- re-nailing or re-screwing existing structural wall linings
- adding supplementary bracing in the form of structural frames
- improving hold-down connections, and
- improving foundations, e.g. by adding additional cross bracing to existing foundation piles or anchor piles, and by improving the connections between the foundations and the superstructure.

For engineered timber framed buildings, methods of improving seismic performance include:

- enhancing connections at the joints in portal frame systems, e.g. by adding additional plates to the knee and apex joints
- fixing additional material to timber members to increase capacity, and
- enhancing foundation connections.

References

BRANZ (2013). *Study Report SR305 Bracing Ratings for Non-proprietary bracing walls*, BRANZ, Wellington, New Zealand.

BRANZ (2015). *Test Report ST1089 Gymnasium Wall Testing for MBIE and MOE*, BRANZ, Wellington, New Zealand.

Beattie, G. (1999). *Earthquake Load Sharing Between Timber Framed and Masonry Walls*, Proceedings of the Pacific Timber Engineering Conference, Rotorua, New Zealand, 1999.

Brunsdon, D., Finnegan, J., Evans, N., Beattie, G., Carradine, D., Sheppard, J. and Lee, B. (2014). *Establishing the resilience of timber framed school buildings in New Zealand,* Proceedings of the 2014 New Zealand Society for Earthquake Engineering Conference, Auckland, New Zealand, 2014.

Buchanan, A., Carradine, D., Beattie, G. and Morris, H. (2011a). *Performance of houses during the Christchurch Earthquake of 22 February 2011*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 44, No. 4, December 2011.

Buchanan, A., Carradine, D. and Jordan, J. (2011b). *Performance of engineered timber structures in the Canterbury earthquakes*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 44, No. 4, December 2011.

Connor-Woodley, P. (2015). *Destructive testing of a timber framed "multi" unit to determine realistic seismic assessment parameters*, Proceedings of the 2014 New Zealand Society for Earthquake Engineering Conference, Rotorua, New Zealand, 2015.

DBH (2011). *Practice Advisory 13: Egress Stairs – Earthquake checks needed for some*, Department of Building and Housing Wellington, New Zealand, September 2011.

EN 1995 (2004). *Eurocode 5: Design of Timber Structures*, European Committee for Standardisation, Brussels, Belgium.

FEMA P-807 (2012). *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings with Weak First Stories*, Applied Technology Council, Redwood City, California, USA.

Giongo, I., Wilson, A., Dizhur, D.Y., Derakhshan, H., Tomasi, R., Griffith, M.C., Quenneville, P. and Ingham, J.M. (2014). *Detailed seismic assessment and improvement procedure for vintage flexible timber diaphragms*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 47, No. 2, June 2014.

Ministry of Education (2016). *Structural and Geotechnical Guidelines for School Design*, Ministry of Education Engineering Strategy Group.

NZSEE (1996), *Assessment And Improvement Of The Structural Performance Of Buildings In Earthquakes*, Incl. Corrigenda 1 & 2, New Zealand Society for Earthquake Engineering, (NZSEE), Wellington, New Zealand.

New Zealand Timber Industry Federation Inc, 2007, *Timber Design Guide - Third Edition,* 2007.

Opus International Consultants (2015), *Canterbury Earthquakes Impact on the Ministry of Education's School Buildings*, Opus, Christchurch, New Zealand.

Pender, M. and Robertson, T., (1987). *Edgecumbe Earthquake: Reconnaissance Report*, Bulletin of the New Zealand Society for Earthquake Engineering, Wellington Vol.20:3, p. 201-249, September 1987.

Quenneville, P. (2009). *Design of Bolted Connections: A Comparison of a Proposal and Various Existing Standards*, New Zealand Timber Design Journal, Vol. 17, Issue 2, 2009.

Rainer, J.H. and Karacabeyli, E. (2000). *Wood - Frame Construction in Past Earthquakes*, Proceedings of the 6th World Conference on Timber Engineering, Whistler, BC, Canada, 2000.

SNZ (1993) *NZS 3603:1993 Timber Structures Standard*, Standards New Zealand, Wellington, New Zealand.

SNZ (1998) AS/NZS 1328.2 *Glue laminated structural timber – Part 2: Guidelines for AS/NZS 1328 Part 1 for the selection, production and installation of glue laminated structural timber*, Standards New Zealand, Wellington, New Zealand.

Appendix C9A: Full Scale Tests on Timber Framed Buildings

C9A.1 Tests on Classroom Blocks

In 2013, the Ministry of Education commissioned testing and invasive investigations of standard classroom blocks of timber framed construction to gather further evidence of the performance of these buildings. This included full scale destructive tests of two types of classroom block, an “Avalon” and a “Dominion” block. The findings from these investigations are summarised below and described in more detail by Brunsdon et al. (2014).

The first test involved two classrooms that formed part of a four-classroom Avalon block at South End School, Carterton, Wairarapa. Avalon timber framed blocks were commonly constructed in the late 1950s and early 1960s. They feature a front wall that is essentially fully glazed, with no recognisable structural bracing panels. The classroom ceiling features a high-level vertical glazed (or ‘clerestory’) section; again with no identifiable form of bracing.

The destructive test confirmed the general engineering expectation that timber framed buildings with older glazed facades have a strength and resilience significantly in excess of their calculated capacity. Test results indicated that failure of the glazing in the longitudinal direction occurred at more than five times the nominal calculated probable capacity of the building. A margin of three to four times was achieved in the associated test of a transverse wall.

The second test was undertaken on a Dominion block at Hammersley Park School, Christchurch. Dominion blocks were built in the 1950s and are timber framed buildings with brick veneer cladding to the walls with weather boards at gable ends and light weight corrugated steel cladding to the roof. The block selected for testing was constructed as a multi-classroom block.

Two adjacent classrooms at the western end were tested in the longitudinal direction, and a single classroom at the eastern end was tested in the transverse direction. This destructive test also confirmed the general engineering expectation that timber framed buildings with older glazed facades have a strength and deformation capacity significantly in excess of their calculated capacity. Test results indicate that failure in the longitudinal direction occurred at more than eight times the nominal calculated probable strength capacity of the building. A margin of two and a half to three times was achieved in the associated test of a transverse wall. Refer Figure C9A.1.

Figure C9A.1: Transverse test of the Dominion block showing high levels of drift

C9A.2 Tests on Housing Unit

In 2013, Housing New Zealand commissioned BRANZ Ltd to undertake a full scale test of a two-storey timber framed housing unit in Upper Hutt. This housing unit was constructed in the 1950s and consisted of four units separated by reinforced blockwork party walls. It had a significant number of wall openings at the ground floor which meant that a very short length of plasterboard lined walls was available to resist lateral load. The findings from these investigations are noted below and described in more detail by Connor-Woodley (2015).

The housing unit was tested in both the longitudinal and transverse directions. Similarly to the Ministry of Education tests, the results indicated significant capacity: in this case, a strength of over five times the calculated strength and a significant deformation capacity without creating a significant life safety hazard. Refer Figures C9A.2 and C9A.3.

Figure C9A.2: Longitudinal test of Housing New Zealand unit showing significant racking of ground floor walls

Figure C9A.3: Typical internal damage to plasterboard lined walls during test of Housing New Zealand unit

Appendix C9B: Strength vs Deformation Capacity Relationships for Generic Bracing Elements

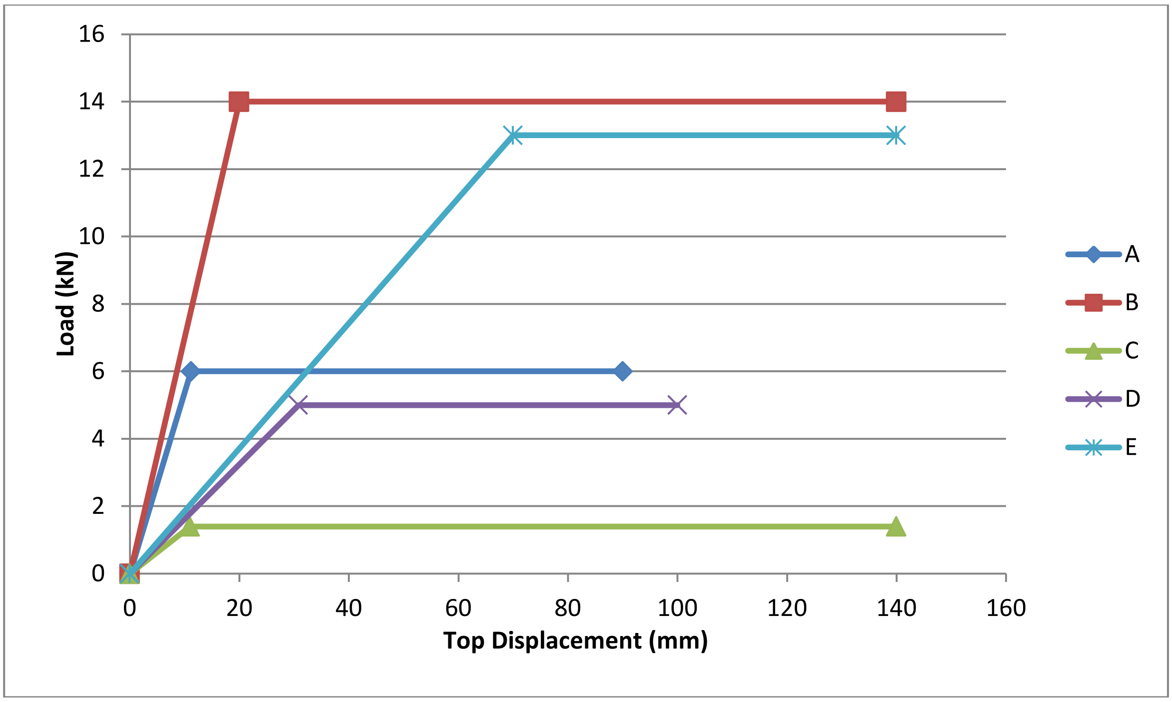

Figure C9D.1: Capacity relationships for timber framed walls with typical sheathing materials, for heights as noted below

A: Panels up to a height of 3.7 m for 12 mm particleboard and for heights between 3.7 to 5.5 m for 4.5 mm hardboard.

12 mm particleboard fixed with 40 mm x 1.5 mm jolt head nails at 300 mm maximum centres.

Hardboard fixed with 25 mm x 1.6 mm jolt head nails at 300 mm maximum centres.

B: Panels up to a height of 3.7 m for 12 mm particleboard to and for heights between 3.7 to 5.5 m for 4.5 mm hardboard.

12 mm particleboard re-nailed with 50 mm x 2.5 mm flat head galvanised nails at 300 mm maximum centres.

Hardboard fixed with 30 mm x 2.5 mm flat head nails at 300 mm maximum centres.

C: Panels up to a height of 5.5 m for 200 x 25 rusticated weatherboards (nett coverage per board 155 mm).

Weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

D: Panels up to a height of 3.6 m for full height interior 12 mm particleboard and exterior rusticated weatherboards.

12 mm particleboard nailed with 40 mm x 1.6 mm jolt head nails at 300 mm maximum centres.

Weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

E: Panels up to a height of 3.6 m for full height interior linings of 12 mm particleboard and exterior rusticated weatherboards.

12 mm particleboard re-nailed with 50 mm x 2.5 mm flat head nails at 300 mm maximum centres.

Weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

Note:

These relationships have been derived from BRANZ tests (Study Report SR305 (2013)) for 5.5 m high panels comprising 140 x 45 mm studs at 600 mm centres and nogs at 1200 mm centres between steel portal legs which were 4.4 m apart. Any resistance provided by the steel portals bending about their weak axis is not included and there was no contribution from any steel link beams between the portals.

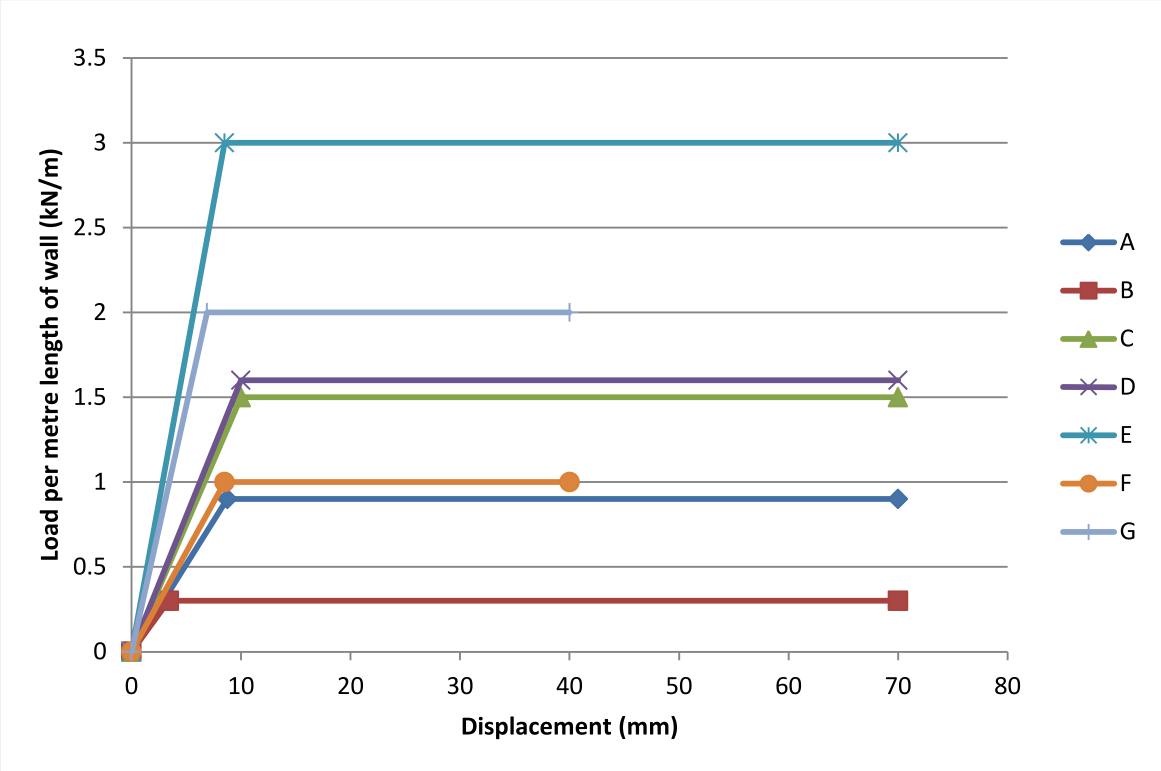

Figure C9D.2: Capacity relationships for 2.4 m high timber framed walls with sheathing materials as noted below

A: Horizontal 200 mm board sheathing.

1.8 m wide panel.

Three Studs at 600 mm centres.

200 mm wide horizontal boards on one face of frame (minimal gap between).

B: Bevel back weatherboard sheathing.

3 m wide panel.

Five Studs at 600 mm centres.

Bevel back weatherboards on one side of frame, fixed with one nail at 600 m centres.

C: Vertically oriented corrugated iron sheathing.

2.4 m wide panel.

Studs at 600 mm centres.

Nogs at 800 mm centres.

Vertical corrugated iron fixed through every third peak generally to plates and nogs.

D: Horizontally oriented corrugated iron sheathing.

3.0 m wide panel.

Studs at 600 mm centres.

Horizontal corrugated iron fixed through every third peak generally to studs.

E: 3.2 mm hardboard sheet with clouts.

1.2 m long panel.
Studs at 600 mm centres.
Hardboard on one side of frame, fixed with clouts at 200 mm centres.

F: Single side 10 mm plasterboard.
1.2 m wide panel.
Studs at 600 mm centres.
Plasterboard on one side of frame, fixed with 30 mm long FH galvanized nails at 300 mm centres.

G: Double side 10 mm plasterboard.
1.2 m wide panel.
Studs at 600 mm centres.
Plasterboard on both sides of frame, fixed with 30 mm long FH galvanised nails at 300 mm centres.

Note:

Results derived from BRANZ tests (Study Report SR305 (2013)).

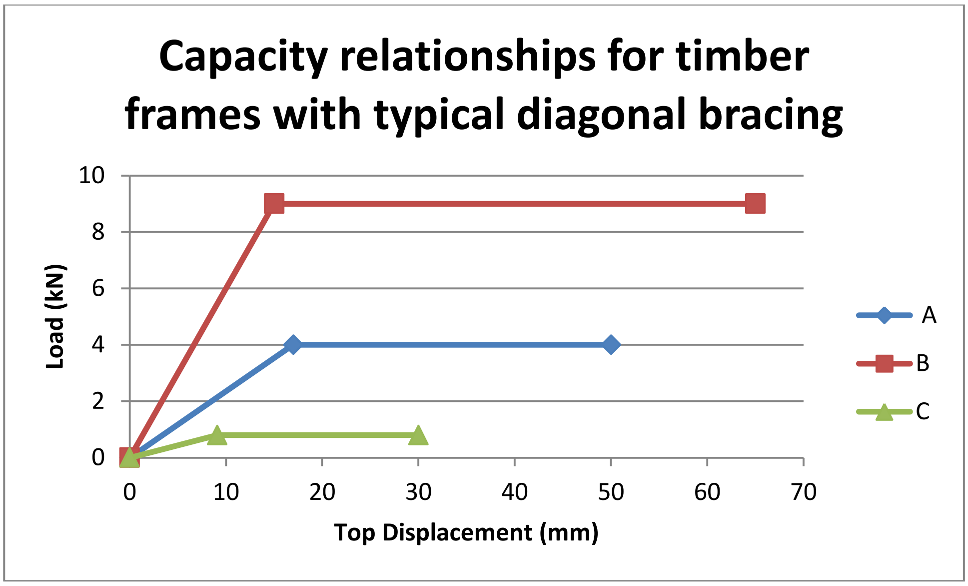

Figure C9D.3: Capacity relationships for 2.4 m high timber frames with diagonal bracing as noted below

A: Opposing 90 mm x 45 mm diagonal braces at 45^0 cut between studs.
Studs at 600 mm centres.

B: Two consecutive opposing 150 mm x 25 mm let-in diagonal braces at 45^0.
Studs at 600 mm centres.

C: 0.6 mm wide wall with 90 mm x 45 mm dogleg braces.
Nogs at 600 mm vertical centres.
Diagonal timber brace hand-nailed between adjacent nogs with two 75 x 3.15 mm jolt-head nails at each end.
Studs nailed to nogs with two 75 x 3.15 mm jolt-head nails at each end.

Note:

Relationships derived from BRANZ tests (Study Report SR 305 (2013)).

Appendix C9C: Timber Shear Wall Strength

C9C.1 Transverse Sheathing

The strength of transversely sheathed shear walls depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple $M = F_{\mathrm{n}}.s$, then the probable shear force in Newtons per metre length of wall, v_{prob}, that can be resisted is:

$$v_{\mathrm{prob}} = \frac{F_{\mathrm{n}}}{l}.\frac{s}{b} \qquad \text{…C9C.1}$$

and the probable shear strength, V_{prob}, in Newtons is:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}} sB}{bl} \qquad \text{…C9C.2}$$

where:

F_{n}	=	probable nail strength (N)
s	=	nail spacing (mm)
l	=	spacing between studs (m)
b	=	width of sheathing board (mm)
B	=	length of the wall (m)

If the boards have not shrunk apart, then friction between the board edges can be assumed to increase the probable strength by the addition of a term Bv', where

v' = 74 N/m for 25 mm sawn boards
= 148 N/m for 50 mm sawn boards, and
= 222 N/m for tongue and groove boards.

The probable in-plane strength of the sheathing in Newtons is given by the expression:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{b}} zB}{bl} \qquad \text{…C9C.3}$$

where:

F_{b}	=	the characteristic bending stress of the board (N/mm^2)
z	=	section modulus of the sheathing board = $\frac{b^2t}{6}$, where t is the thickness of the sheathing board (mm).

C9C.2 Single Diagonal Sheathing

The probable horizontal shear in Newtons, $(V_{\mathrm{prob}})_{\mathrm{i}}$, carried by each board is:

$$(V_{\mathrm{prob}})_{\mathrm{i}} = \frac{1}{\sqrt{2}} NF_{\mathrm{n}} \qquad \text{…C9C.4}$$

giving a total probable strength in kilonewtons of:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}} NB}{2b} \qquad \text{…C9C.5}$$

Since the axial force in the sheathing is the same on both sides of any intermediate stiffener, no load is transferred into the stiffeners from the sheathing. However, the perimeter members are subjected to both axial loads and bending and must be assessed for the combined stresses (see NZS 3603:1993). The bending in the plate members is caused by a universally distributed load, w in N/mm, of:

$$w = \frac{NF_{\mathrm{n}}}{b} \qquad \text{…C9C.6}$$

The probable in-plane strength of the sheathing boards, in Newtons, is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{c}}bt}{2} \qquad \text{…C9C.7}$$

where:

N	=	the number of nails fixing the board to the plate
t	=	thickness of the sheathing board (mm)
F_{c}	=	characteristic stress in the sheathing board in compression parallel to the grain (N/mm^2)

Other symbols are as defined in Appendix C9C.1.

C9C.3 Double Diagonal Sheathing

Based on the strengths of the nail pairs, the probable strength of the shear wall is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}}NB}{2b} \qquad \text{…C9C.8}$$

The probable in-plane strength in kilonewtons of the sheathing boards over the wall length is given by the expression:

$$V_{\mathrm{prob}} = F_{\mathrm{c}}Bt \qquad \text{…C9C.9}$$

The probable capacity of the chords in Newtons is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{c}}BA}{H} \qquad \text{…C9C.10}$$

while the probable capacity of the plates in Newtons is given by:

$$V_{\mathrm{prob}} = F_{\mathrm{c}}A_{\mathrm{p}} \qquad \text{…C9C.11}$$

where:

A	=	cross sectional area of the chord (mm^2)
A_{p}	=	cross sectional area of the plate (mm^2)
H	=	the height of the wall (m).

Other symbols are as defined inC9C.1 and C9C.2.

C9C.4 Panel Sheathing

The probable strength values in Table C9.1 should be used in assessing the strength of these elements; unless specific tests are carried out.

Appendix C9D: Timber Shear Wall Stiffness

The horizontal inter storey deflection in one storey of a shear wall Δ_w can be calculated from:

$$\Delta_w = \Delta_4 + \Delta_5 + \Delta_6 + \Delta_7 \qquad \text{…C9D.1}$$

where:

Δ_4 = deformation due to support connection relaxation (mm)
Δ_5 = wall shear deformation (mm)
Δ_6 = deformation due to nail slip (mm)
Δ_7 = deformation due to flexure as a cantilever (mm) (may be ignored for single storey shear walls).

For transverse sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…C9D.2}$$

$$\Delta_5 = 0$$

$$\Delta_6 = 2\frac{H}{s}e_n \qquad \text{…C9D.3}$$

$$\Delta_7 = H\theta$$

For single diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…C9D.4}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{…C9D.5}$$

$$\Delta_6 = 2\sqrt{2}e_n \quad \text{for the case where H} \leq \text{B, OR} \qquad \text{…C9D.6}$$

$$= 2\sqrt{2}\frac{H}{B}e_n \quad \text{for the case where H} \geq \text{B}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{…C9D.7}$$

For double diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…C9D.8}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{…C9D.9}$$

$$\Delta_6 = \sqrt{2}e_n \quad \text{for the case where H} \leq \text{B, OR} \qquad \text{…C9D.10}$$

$$= \sqrt{2}\frac{H}{B}e_n \quad \text{for the case where H} \geq \text{B} \qquad \text{…C9D.11}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \text{....C9D.12}$$

For panel sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \quad \text{...C9D.13}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \text{...C9D.14}$$

$$\Delta_6 = 2(1 + a)me_n \quad \text{...C9D.15}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \text{...C9D.16}$$

where:

a = aspect ratio of each sheathing panel:
- = 0 when relative movement along sheet edges is prevented
- = 1 when square sheathing panels are used
- = 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (i.e. vertical) (= 0.5 alternative orientation)

A = sectional area of one chord (i.e. end stud) (mm^2)
B = distance between shear wall chord members (mm)
= length of the wall
e_n = nail slip resulting from the shear force V (mm)
E = elastic modulus of the chord members (MPa)
G = shear modulus of the sheathing (MPa)
H = height of the storey under consideration (mm)
m = number of sheathing panels along the length of the edge chord
s = spacing of the nail couples in a board (mm)
t = thickness of the sheathing (mm)
V = shear force in storey under consideration (N)
θ = flexural rotation at base of storey under consideration (radians)
δ_c = vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)
δ_t = vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored).

Appendix C9E: Timber Diaphragm Strength

C9E.1 Square Sheathing

The probable strength of transversely sheathed diaphragms (i.e. diaphragms where the sheathing runs perpendicular to the diaphragm span) depends on the resisting moment furnished by nail couples at each joist/rafter crossing. If the nail couple $M = F_{\mathrm{n}}.s$, then the shear force in Newtons per metre length at the support, v, that can be resisted is:

$$v_{\mathrm{prob}} = \frac{F_{\mathrm{n}}}{l}.\frac{s}{b} \qquad \text{…C9E.1}$$

and the total probable shear strength in Newtons is:

$$V_{\mathrm{prob}} = \frac{2F_{\mathrm{n}}sB}{bl} \qquad \text{…C9E.2}$$

If the boards have not shrunk apart, friction between the board edges could possibly increase the probable capacity by the addition of a term, $2Bv'$, where:

v' = 74 N/m for 25 mm sawn boards
= 148 N/m for 50 mm sawn boards
= 222 N/m for tongue and groove boards.

The probable in-plane strength in the sheathing is given by the expression:

$$V_{\mathrm{prob}} = \frac{2F_{\mathrm{b}}zB}{bl} \qquad \text{…C9E.3}$$

where:

F_{n} = nominal nail strength (N)
F_{b} = the characteristic bending stress of the sheathing board, $\mathrm{N/mm^2}$
s = nail spacing (mm)
l = spacing between joists (m)
b = width of sheathing board (mm)
B = depth of diaphragm (m)
z = section modulus of the sheathing board = $\frac{b^2t}{6}$, where t is the thickness of the board ($\mathrm{mm^3}$).

C9E.2 Single Diagonal Sheathing

As above, the probable strength of the diaphragm depends on the resisting moment produced by the nail couples at each joint crossing. The probable capacity in kilonewtons is:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}}NB}{b} \qquad \text{…C9E.4}$$

where:

N = total number of nails.

The probable in-plane strength of the sheathing in kilonewtons is given by the expression:

$$V_{\mathrm{prob}} = F_{\mathrm{c}}Bt \qquad \ldots\text{C9E.5}$$

where:

F_{c} = characteristic stress in the sheathing board in compression parallel to the grain (N/mm^2)

t = thickness of the sheathing board (mm).

Other symbols are as defined in Appendix C9E.1.

The probable strength of the chord members need to be assessed for combined bending and axial stresses (refer to NZS 3603:1993).

C9E.3 Double Diagonal Sheathing

The probable capacity of the nail couples at each joist crossing is the same as for the single diagonal sheathing and the resulting in-plane capacity in kilonewtons of the sheathing is:

$$V_{\mathrm{prob}} = 2F_{\mathrm{c}}Bt \qquad \ldots\text{C9E.6}$$

All symbols are as defined in Appendices C9E.1 and C9E.2.

C9E.4 Panel Sheathing

The probable strength values in Table C9.1 should be used in assessing the strength of these elements – unless specific tests are carried out.

Appendix C9F: Timber Diaphragm Stiffness

The mid span deflection of a horizontal diaphragm, Δ_{h}, can be calculated from:

$$\Delta_{\mathrm{h}} = \Delta_1 + \Delta_2 + \Delta_3 \quad \text{…C9F.1}$$

where:

Δ_1 = diaphragm flexural deformation considering chords acting as a moment resisting couple (mm)

Δ_2 = diaphragm shear deformation resulting from beam action of the diaphragm (mm)

Δ_3 = deformation due to nail slip for horizontal diaphragm (mm).

For transverse sheathing:

$$\Delta_1 = 0$$

$$\Delta_2 = 0$$

$$\Delta_3 = \frac{Le_{\mathrm{n}}}{2s} \quad \text{…C9F.2}$$

For single diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \text{…C9F.3}$$

$$\Delta_2 = \frac{WL}{4EBt} \quad \text{…C9F.4}$$

$$\Delta_3 = \frac{(1+a)me_{\mathrm{n}}}{2} \quad \text{…C9F.5}$$

For double diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \text{…C9F.6}$$

$$\Delta_2 = \frac{WL}{8EBt} \quad \text{…C9F.7}$$

$$\Delta_3 = \frac{(1+a)me_{\mathrm{n}}}{2} \quad \text{…C9F.8}$$

For panel sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \text{…C9F.9}$$

$$\Delta_2 = \frac{WL}{8GBt} \quad \text{…C9F.10}$$

$$\Delta_3 = \frac{(1+a)me_{\mathrm{n}}}{2} \quad \text{…C9F.11}$$

where:

a	=	aspect ratio of each sheathing panel: = 0 when relative movement along sheet edges is prevented = 1 when square sheathing panels are used = 2 when 2.4 m x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A	=	sectional area of one chord (mm^2)
B	=	distance between diaphragm chord members (mm)
e_n	=	nail slip resulting from the shear force V (mm)
E	=	elastic modulus of the chord members (MPa)
G	=	shear modulus of the sheathing (MPa)
L	=	span of a horizontal diaphragm (mm)
m	=	number of sheathing panels or boards along the length of the edge chord
s	=	nail couplet spacing (mm)
t	=	thickness of the sheathing (mm)
W	=	lateral load applied to a horizontal diaphragm (N).