

PART C

Timber Buildings **C9**

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- 2 Part B – Initial Seismic Assessment
- 3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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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 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 provides bracing resistance to lateral loads, and
- engineered timber buildings that incorporate elements such as timber portals.

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

Note:

Timber framed buildings and engineered timber buildings meeting modern design 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.

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 acronyms

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform inelastically before it fractures.
Capacity design	A design process for new buildings that identifies zones where post elastic response is acceptable and details these accordingly. All other parts of the primary structure are then designed to ensure other undesirable inelastic response mechanisms are suppressed.
Cross laminated timber (CLT)	Engineered wood made from multiple layers of boards placed cross-wise to adjacent layers for increased rigidity and strength
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the walls around it and distributes earthquake lateral forces to vertical elements, such as walls, of the lateral force-resisting system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes 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.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
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.
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, with grain direction orientated 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 part of the primary gravity structure.

Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
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 resists lateral loads along its primary axis (also known as an in-plane wall)
Sheathing	The board, lining or panel material used in floor, wall and roof assemblies
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Stressed skin panels	Structural flat plates which rely on composite action for resistance to out-of-plane loads. Flexural strength is provided by the skins and shear resistance is provided by the filling of webs between the skins.

C9.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
A	Sectional area of one chord (mm^2)
a	Aspect ratio of shear walls: <ul style="list-style-type: none"> • 0 when relative movement along sheet edges is prevented • 1 when transverse 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)
A_p	Sectional area of the plate (mm^2)
B	Depth of diaphragm (mm)
B	Distance between diaphragm or shear wall chord members (mm)
B	Length of the wall
b	Width of sheathing board (mm)
E	Elastic modulus of the chord members (MPa)
e_n	Nail slip resulting from the shear force V (mm)
F_b	Probable bending strength of a board (N/mm^2)
F_c	Probable strength of a sheathing board in compression parallel to the grain (MPa)
F_n	Probable nail strength (N)
F_{prob}	Probable (strength) capacity of a diaphragm
G	Shear modulus of the sheathing (MPa)
H	Height of the wall or storey under consideration (mm)
k_i	Modification factor from NZS 3603:1993
L	Span of a diaphragm (mm)

l	Spacing between joints or studs (mm)
M	Moment couple formed by a pair of nails
m	Number of sheathing panels along the length of the edge chord
N	Total number of nails across the width of a diaphragm, or number of nails fixing a board to a plate as appropriate
$P - \text{delta}$	Structural actions induced as a consequence of the gravity loads being displaced horizontally due to horizontal effects
s	Nail spacing (mm)
S_p	Structural performance factor in accordance with NZS 1170.5:2004
t	Thickness of the sheathing board (mm)
V	Shear force in storey under consideration (N)
v'	Probable interboard friction capacity for timber floors
$(V_{\text{prob}})_i$	Probable horizontal shear in Newtons
V_{prob}	Probable shear strength
v_{prob}	Probable shear force in Newtons per metre length of wall
W	Lateral load applied to a horizontal diaphragm (N)
w	A universally distributed load causing bending in the plate members of a diaphragm
z	Section modulus of the sheathing board = $\frac{b^2 t}{6}$, where t is the thickness of the board (mm ³)
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 (mm)
Δ_5	Wall shear deformation (mm)
Δ_6	Deformation due to nail slip (mm)
Δ_7	Deformation due to flexure as a cantilever (may be ignored for single storey shear walls with H/B ratios less than 1.0)
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_h	Mid span deflection of a horizontal diaphragm
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)
Δ_w	Horizontal inter-storey deflection in one storey of a shear wall
θ	Flexural rotation at base of storey under consideration (radians)
μ	Structural ductility factor in accordance with NZS 1170.5:2004
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. Since the earliest European settlement it has been used widely 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 to Section C9.2.4).



Figure C9.1: Examples of timber buildings

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 of sheet linings, around the early 1900s to 1920s, wide horizontal boards approximately 25 mm were often used as a backing for scrim and then wallpaper was 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 typically 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, nail plated parallel flange truss joists, solid glulam panels, or cross laminated timber (CLT) panels. 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.

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, flooring 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 (e.g. 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 light 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 in 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 (pole frame structures) 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 earthquakes 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 of timber framed school buildings 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 halls and gymnasias. 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 during these earthquakes at a range of school sites across Canterbury, 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, detailed engineering evaluations of these schools after the earthquakes 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 gymnasias) generally have many additional load paths that are not easily quantifiable but are able to carry and redistribute loads and deform significantly in a seismic event.

Note:

The results of two separate full scale tests commissioned by the Ministry of Education and one by Housing New Zealand in 2013 (refer to 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 procedures in Section C2.

Also refer to Sections C1 and C2 for guidance on:

- documentation that should be sourced to undertake the assessment, and 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 design 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 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 can increase the expected building performance, potentially making it unnecessary to undertake a detailed analysis/assessment.

Note:

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 1720.1:2010.
- The engineer 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 which are taken as the nominal capacities defined in NZS 3603:1993; i.e. taking the strength reduction factor, ϕ , equal to 1, but using probable material strengths.
- The probable material strengths and member/element capacities set out in Sections C9.5 and C9.6 respectively can be used as default values in the absence of better available data that is specific to the building. 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 characteristic (lower fifth percentile) strength used in the design of new timber structures. In structures with many contributing timber members the assessment of the collective capacity should be assessed using the appropriate modification factors, k_1 , from NZS 3603:1993.
- 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 can be ignored.
- Portal framed structures are typically governed by deflection limits and non-seismic actions, so stiffness rather than strength will likely have governed section size. Joint strength and deformation capacity is then likely to be critical. The capacity of joints with dowel type fasteners is usually limited by timber bearing/crushing, which is a ductile mechanism.

For buildings constructed primarily of other materials (e.g. concrete or URM) but with timber elements, such as floors or roofs, 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 significantly 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.

Note:

In older timber buildings and some modern buildings a capacity design may not 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 $S_p = 0.70$, 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 for the assessment of timber buildings.

Note:

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 $S_p = 0.5$ is considered reasonable based on observed behaviour in earthquakes and on 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 considered when assigning the available ductility in the system.

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 1995-1-1:2004 and Quenneville, 2009) or other similar methods to determine the failure mode for connections.

The global earthquake 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

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 of different materials.

All assessment procedures outlined in Section C1 have completing a SLaMA as the first step. This requires the engineer 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.

Note:

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 their lateral load-resisting behaviour. These tests have provided probable strength and deformation capacities and stiffnesses for a range of bracing systems. Examples are included in Appendix C9B.

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

C9.4.4 Other issues

C9.4.4.1 General

Engineers 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 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 that valid load paths exist to transfer the inertial loads from source into the structure.

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 floor framing 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 earthquake 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 1995-1-1:2004 and Quenneville (2009).

When assessing the foundations for timber framed buildings, the value of S_p to be used for the calculation of the earthquake score for the foundation should be that appropriate for the foundation system.

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.

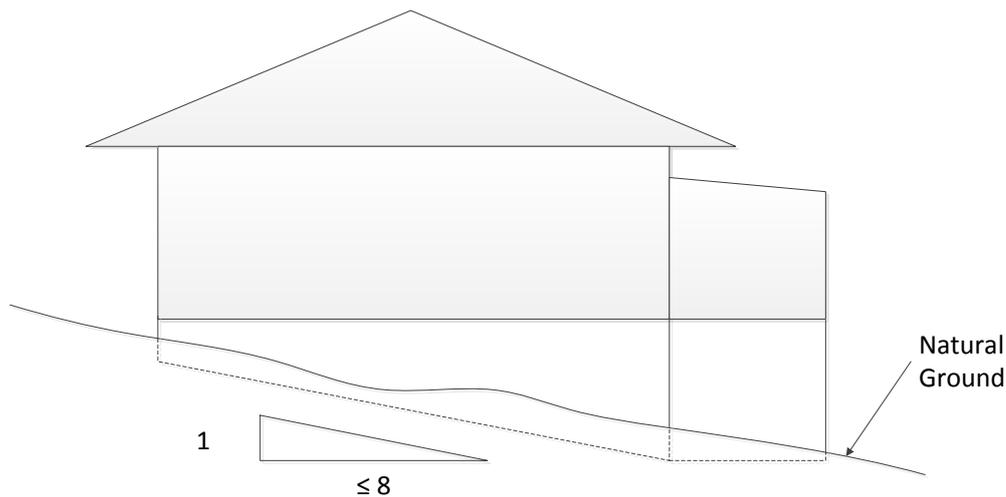


Figure C9.2: Definition of sloping ground

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 earthquake 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.

C9.5.2 Material strengths

An assessment of the probable strengths of existing materials may be made from the results of tests. If no test results are available, the engineer should either conduct suitable tests or assess conservative values of strength by comparison with the properties of similar timbers to 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 the engineer may take probable material strength values as the characteristic strengths given in NZS 3603:1993 and reproduced in 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 for Radiata pine and Douglas fir may be taken as the characteristic strengths given in Amendment 4 to NZS 3603:1993. (Note that the timber in almost all buildings constructed during this period is either Radiata pine or Douglas fir.)

Note:

The characteristic strength for tension 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 factors (k_i) given in NZS 3603:1993 should be used where appropriate.

Table C9.1: Probable material strengths for visually graded timber (MPa) (characteristic strengths 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

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, and struts.

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 drying 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 DSA. The starting point for establishing the properties should be the available construction documents. Accordingly, the engineer should 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 produced. This may necessitate removal of linings to observe critical structural connections.

The intention of these guidelines is that the earthquake scores/ratings are based on probable capacities of elements. The probable strength capacity is assessed using probable material strengths as outlined in Section C9.5, and taking a strength reduction factor, ϕ , of 1. The probable deformation capacity of timber elements is likely to exceed other practical constraints.

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 probable strength capacity of wood and light timber frame shear walls should be taken as the nominal capacity of the shear wall assembly from NZS 3603:1993, i.e. using a strength reduction factor, ϕ ,

equal to 1 but using probable material properties. 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.

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 timber framed buildings the wall linings were not expected to act as bracing (e.g. lath and plaster linings). Engineers should verify the presence of diagonal bracing behind such linings if possible. 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 are required to assess the effects of openings. The presence of chord members around the openings, with linings well fixed to them, will reduce the loss in overall stiffness and limit damage in the area of the 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 sheathing

Transverse sheathing or 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 enable the 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 capacity.

Single diagonal sheathing

The shear force applied to the shear wall is carried by tension or compression in the 45° 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 probable strength of timber shear walls should be based on an assessment of the probable strengths of the materials making up the particular shear wall. Depending on the wall type, the formulae given in Appendix C9C can be used to determine the shear wall probable strength. In the absence of test results, the probable strength values contained in Table C9.2 may be used in lieu of more detailed calculations.

The deflection at the notional yield 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 sufficiently close 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	Probable strength values
150 x 25 mm let-in brace at 45°	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 45°	2.0 kN
90 x 45 mm fitted brace both ways at 45° and sheet material* one face	2.5 kN
90 x 45 mm fitted brace both ways at 45° 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 determining the probable wall bracing 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. This is because 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.

Note:

The bracing units apply to the capacity of an individual wall panel. Any weak links or issues with the stiffness of the diaphragms which may limit or determine the extent to which individual panels are able to contribute to the overall building capacity should be identified.

C9.6.3 Roof and floor diaphragms

C9.6.3.1 General

The probable strength of timber diaphragms should be taken as the probable capacity of the diaphragm assembly determined from a rational assessment of the individual elements. 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 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.

Note:

The actions on the individual elements of a diaphragm will depend on the relative stiffness of the diaphragm compared with the lateral stiffness of the connected vertical elements. The relative stiffness will change if the vertical elements are loaded into the nonlinear range, at which point a timber diaphragm could be considered as rigid. The analysis of diaphragms is discussed further in Section C2 and for URM buildings in Section C8.

C9.6.3.2 Types of timber diaphragms

Transverse 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 will have been 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 45° 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 transverse sheathing.

Panel sheathing

Panel 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 deflections 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.

Softboard linings are considered to provide insufficient diaphragm action and any contribution to strength or stiffness 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 and assuming the probable strengths are the nominal strength ($\phi = 1$), 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 assuming that the probable strength is the nominal strength ($\phi = 1$). If relevant, more detailed analysis of connection failure mechanisms can be determined using European Yield Methods. Refer to EN 1995-1-1:2004 and Quenneville (2009).

In URM buildings the connections of timber elements to the masonry are often nominal and generally should not be relied upon for engineering purposes. Further, 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
- 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
- enhancing foundation connections.

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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, 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. 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 lateral 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 to 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 to 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

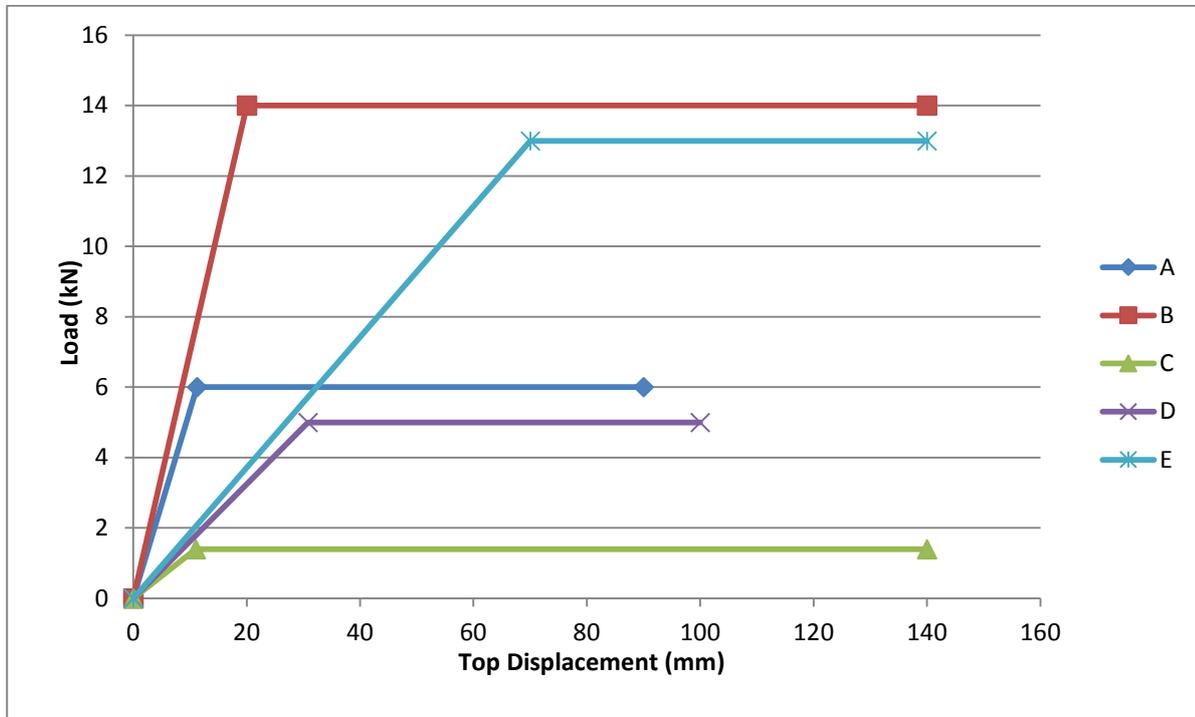


Figure C9B.1: Capacity relationships for timber framed walls with typical sheathing materials, for heights as noted below

- A: 5.5 m high panels with 12 mm particleboard up to a height of 3.7 m and 4.5 mm hardboard between 3.7 and 5.5 m
 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: 5.5 m high panels with 12 mm particleboard up to a height of 3.7 m and 4.5 mm hardboard between 3.7 and 5.5 m
 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 of 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 of full height 12 mm particleboard for the interior face and full height rusticated weatherboards for the exterior face
- 12 mm particleboard nailed with 40 mm x 1.6 mm jolt head nails at 300 mm maximum centres
 - Rusticated 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 of full height 12 mm particleboard for the interior face and full height rusticated weatherboards for the exterior face
- 12 mm particleboard re-nailed with 50 mm x 2.5 mm flat head nails at 300 mm maximum centres
 - Rusticated 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 SR 305 (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.

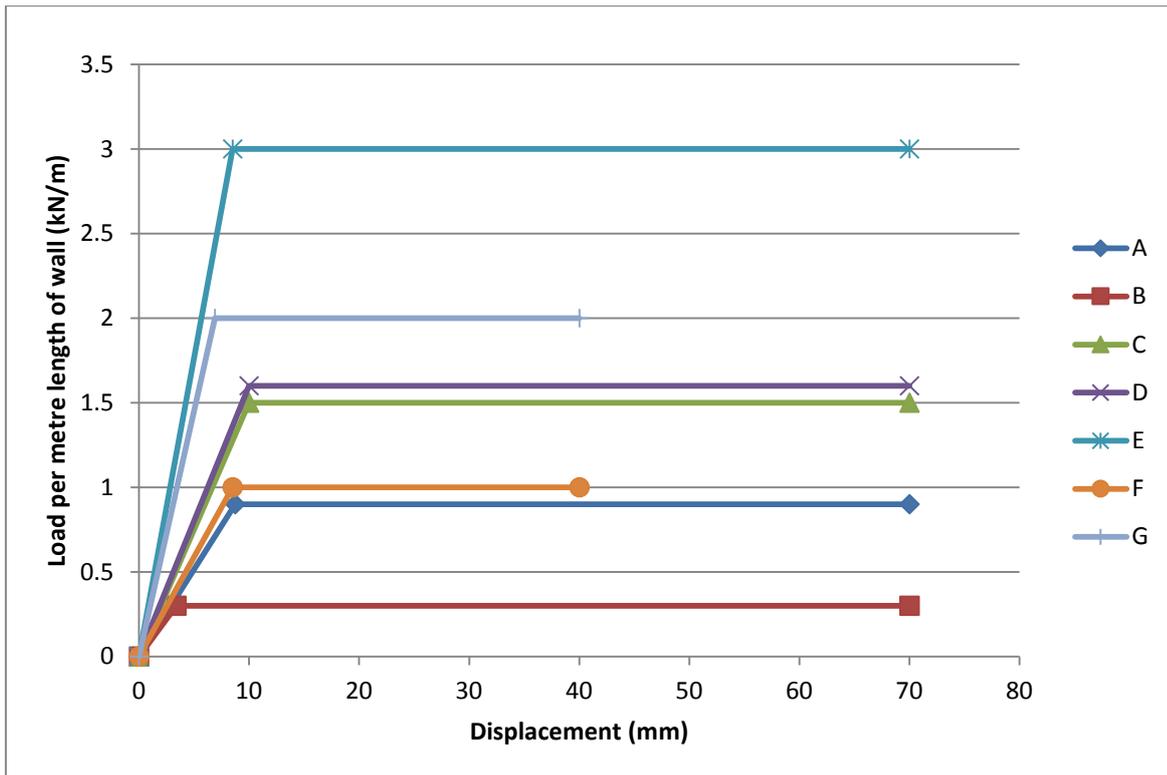


Figure C9B.2: Capacity relationships for 2.4 m high timber framed walls with sheathing materials as noted below

- 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 mm 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 SR 305 (2013)).

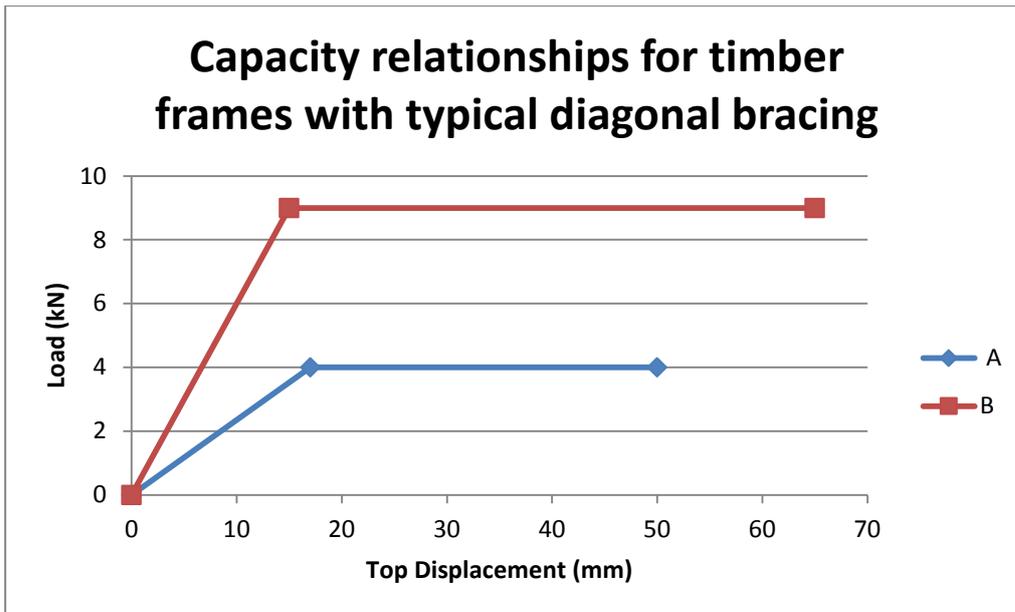


Figure C9B.3: Capacity relationships for 2.4 m high timber frames with diagonal bracing as noted below

- A: Opposing 90 mm x 45 mm diagonal braces at 45° cut between studs
Studs at 600 mm centres.
- B: Two consecutive opposing 150 mm x 25 mm let-in diagonal braces at 45°
Studs at 600 mm centres.

Note:

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

Appendix C9C: Timber Shear Wall Strength

C9C.1 Transverse Sheathing

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

$$v_{\text{prob}} = \frac{F_n}{l} \cdot \frac{s}{b} \quad \dots\text{C9C.1}$$

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

$$V_{\text{prob}} = \frac{F_n s B}{bl} \quad \dots\text{C9C.2}$$

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).

Friction between the board edges can be assumed to increase the probable strength of the wall 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
	=	222 N/m for tongue and groove boards.

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

$$V_{\text{prob}} = \frac{F_b z B}{bl} \quad \dots\text{C9C.3}$$

where:

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

C9C.2 Single Diagonal Sheathing

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

$$(V_{\text{prob}})_i = \frac{1}{\sqrt{2}} N F_n \quad \dots\text{C9C.4}$$

giving a total probable strength in kilonewtons of:

$$V_{\text{prob}} = \frac{F_n N B}{2b} \quad \dots\text{C9C.5}$$

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:

$$w = \frac{NF_n}{b} \quad \dots\text{C9C.6}$$

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

$$V_{\text{prob}} = \frac{F_c b t}{2} \quad \dots\text{C9C.7}$$

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 Section C9C.1.

C9C.3 Double Diagonal Sheathing

Based on the strengths of the nail pairs at the end of each sheathing board, and assuming that all nails extend into the top and bottom plates, the probable strength of the shear wall is given by:

$$V_{\text{prob}} = \frac{F_n N B}{b} \quad \dots\text{C9C.8}$$

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

$$V_{\text{prob}} = F_c B t \quad \dots\text{C9C.9}$$

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

$$V_{\text{prob}} = \frac{F_c B A}{H} \quad \dots\text{C9C.10}$$

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

$$V_{\text{prob}} = F_c A_p \quad \dots\text{C9C.11}$$

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 in Sections C9C.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 Deflections

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

$$\Delta_w = \Delta_4 + \Delta_5 + \Delta_6 + \Delta_7 \quad \dots \text{C9D.1}$$

where:

$$\begin{aligned} \Delta_4 &= \text{deflection due to support connection relaxation (mm)} \\ \Delta_5 &= \text{wall shear deflection (mm)} \\ \Delta_6 &= \text{deflection due to nail slip (mm)} \\ \Delta_7 &= \text{deflection due to flexure as a cantilever (mm) (may be ignored for single storey shear walls).} \end{aligned}$$

For transverse sheathing:

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

$$\Delta_5 = 0 \quad \dots \text{C9D.3}$$

$$\Delta_6 = 2 \frac{H}{s} e_n \quad \dots \text{C9D.4}$$

$$\Delta_7 = H\theta \quad \dots \text{C9D.5}$$

For single diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t) \frac{H}{B} \quad \dots \text{C9D.6}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \dots \text{C9D.7}$$

$$\Delta_6 = 2\sqrt{2}e_n \quad \text{for the case where } H \leq B, \text{ OR} \quad \dots \text{C9D.8}$$

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

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \dots \text{C9D.9}$$

For double diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t) \frac{H}{B} \quad \dots \text{C9D.10}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \dots \text{C9D.11}$$

$$\Delta_6 = \sqrt{2}e_n \quad \text{for the case where } H \leq B, \text{ OR} \quad \dots \text{C9D.12}$$

$$= \sqrt{2} \frac{H}{B} e_n \quad \text{for the case where } H > B \quad \dots \text{C9D.13}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \dots\text{C9D.14}$$

For panel sheathing:

$$\Delta_4 = (\delta_c + \delta_t) \frac{H}{B} \quad \dots\text{C9D.15}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \dots\text{C9D.16}$$

$$\Delta_6 = 2(1 + a)me_n \quad \dots\text{C9D.17}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \dots\text{C9D.18}$$

where:

- a = aspect ratio of each sheathing panel:
 - = 0 when relative movement along sheet edges is prevented
 - = 1 when transverse 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) (mm²)
- 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 Transverse 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 $M = F_n \cdot s$, then the shear force in Newtons per metre length at the support, v , that can be resisted is:

$$v_{\text{prob}} = \frac{F_n}{l} \cdot \frac{s}{b} \quad \dots\text{C9E.1}$$

and the total probable strength capacity of the diaphragm in Newtons based on nail capacity is:

$$F_{\text{prob}} = \frac{2F_n s B}{bl} \quad \dots\text{C9E.2}$$

Friction between the board edges can increase the probable capacity of the diaphragm by the addition of a term, $2Bv'$, where:

$$\begin{aligned} v' &= 74 \text{ N/m for 25 mm sawn boards} \\ &= 148 \text{ N/m for 50 mm sawn boards} \\ &= 222 \text{ N/m for tongue and groove boards.} \end{aligned}$$

The probable in-plane strength capacity of the diaphragm based on the strength of the sheathing is given by the expression:

$$F_{\text{prob}} = \frac{2F_b z B}{bl} \quad \dots\text{C9E.3}$$

where:

$$\begin{aligned} F_n &= \text{nominal nail strength (N)} \\ F_b &= \text{the probable bending stress of the sheathing board, N/mm}^2 \\ s &= \text{nail spacing (mm)} \\ l &= \text{spacing between joists (m)} \\ b &= \text{width of sheathing board (mm)} \\ B &= \text{depth of diaphragm (m)} \\ z &= \text{section modulus of the sheathing board} = \frac{b^2 t}{6}, \text{ where } t \text{ is the thickness of the board (mm}^3\text{).} \end{aligned}$$

C9E.2 Single Diagonal Sheathing

The probable strength of the diaphragm depends on the probable shear capacity of the total number of nails into the edge member across the width of the diaphragm. The probable shear capacity of the diaphragm in kilonewtons is:

$$V_{\text{prob}} = \frac{F_n N B}{b} \quad \dots\text{C9E.4}$$

where:

$$N = \text{total number of nails into the edge member}$$

The probable in-plane shear strength of the diaphragm based on the strength of the sheathing in kilonewtons is given by the expression:

$$V_{\text{prob}} = F_c B t \quad \dots \text{C9E.5}$$

where:

- F_c = characteristic stress in the sheathing board in compression parallel to the grain (N/mm²)
 t = thickness of the sheathing board (mm).

Other symbols are as defined in Section C9E.1.

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

C9E.3 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 Deflections

The mid span deflection of a horizontal diaphragm, Δ_h , can be calculated from:

$$\Delta_h = \Delta_1 + \Delta_2 + \Delta_3 \quad \dots\text{C9F.1}$$

where:

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

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

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

For transverse sheathing:

$$\Delta_1 = 0 \quad \dots\text{C9F.2}$$

$$\Delta_2 = 0 \quad \dots\text{C9F.3}$$

$$\Delta_3 = \frac{Le_n}{2s} \quad \dots\text{C9F.4}$$

For single diagonal sheathing:

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

$$\Delta_2 = \frac{WL}{4EBt} \quad \dots\text{C9F.6}$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad \dots\text{C9F.7}$$

For panel sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \dots\text{C9F.8}$$

$$\Delta_2 = \frac{WL}{8GBt} \quad \dots\text{C9F.9}$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad \dots\text{C9F.10}$$

where:

a = aspect ratio of each sheathing panel:
 = 0 when relative movement along sheet edges is prevented
 = 1 when transverse 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 (mm²)

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).

Note:

The assumption made in the equations above is that the diaphragm remains essentially elastic. The deformation estimate is therefore the nominal yield displacement.