STUDY REPORT

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The Engineering Basis of NZS 3604

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Preface

This report is intended as a source document for those who need to know the basis for the engineering decisions underpinning NZS 3604 *Timber framed buildings*.

Acknowledgments

This work was funded by the Building Research Levy.

Note

This report is intended for standards committee members, manufacturers, building officials, consultants, researchers.

The Engineering Basis of NZS 3604

BRANZ Study Report SR168 (2007) RH Shelton

Reference

Shelton RH. 'The Engineering Basis of NZS 3604'. BRANZ *Study Report 168 (2007)*. BRANZ Ltd, Judgeford, New Zealand.

Abstract

NZS 3604 *Timber framed buildings* sets out the construction requirements for light timber framed buildings in New Zealand which do not require specific structural engineering design. By limiting the size of the building and scope of application, a series of solutions are presented, enabling a designer to select an element or detail appropriate to the situation without the necessity to engage a structural engineer.

For those users of NZS 3604 whose projects, designs or systems fall outside its scope, it is important to know the technical basis for the standard. This report documents the engineering basis used to derive the technical provisions of NZS 3604, including the member selection tables and connections. Ambiguities and problem areas are highlighted for attention by future standards drafting committees.

Keywords

Timber framing, NZS 3604, wind loading, earthquake loading, structural engineering, design solutions, timber properties, timber buildings.

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1. INTRODUCTION

1.1 Objective of this report

The objective of this report is to provide users of NZS 3604 with the design basis, the assumptions, and other criteria which were used to derive the technical provisions and member selection tables. The criteria may therefore be used as a basis for preparation of equivalent design solutions where the limits of NZS 3604 do not apply, or for products and methods not already covered by NZS 3604.

The section headings of the report refer to the section and clause numbers of NZS 3604 for easy cross referencing. Where requirements or criteria have changed, or are different between revisions of the standard, generally the explanation in this report relates to the current (1999) version, as subsequently revised by Amendment 2.

1.2 Background

NZS 3604, the New Zealand *Timber framed buildings standard* (SNZ 1999), sets out the construction requirements for light timber framed buildings which do not require specific design by structural engineers. By placing limits on the size and scope of buildings covered by the standard, safe but not unduly conservative solutions are provided for a wide range of buildings without the need for the involvement of structural engineer designers. Thus the cost of the inherent conservatism is balanced against the savings in structural design fees, where the simplicity of the design does not warrant such close attention. Those limits (as set out in the scope provisions of Clause 1.1.2) govern the building size, height and roof slope, floor loadings, and snow loadings. Wind and earthquake loads are also limited by restricting the zone or area in which the building may be situated. For buildings outside these limits, a specific structural design is required for each building.

NZS 3604 (SNZ 1978) was first published by the Standards Council in November 1978 to replace NZS 1900: Chapter 6.1:1964 *Construction* requirements *for timber buildings not requiring specific design* (SANZ 1964a). The first revision was made in 1981, and it was considerably amended in 1984 when it was reprinted as a complete document. The next revision was in 1990 and the current version was published in June 1999. Amendment 1, which included revisions to snow loading, was published in December 2000, and Amendment 2 dealing with timber properties was published in May 2006.

While the standard was originally written as a means of compliance with NZS 1900 *Model building bylaw* and intended for adoption by Council building authorities, the New Zealand building control environment changed dramatically with the advent of the Building Act in 1990. NZS 3604:1999 is now cited by the New Zealand Building Code (NZBC) as an “Approved Document” for Clauses B1, B2, and E2, meaning that it is deemed to be an Acceptable Solution for the structural, durability and external moisture aspects of timber framed buildings.

Although the emphasis of the standard has changed over the years, its intent has remained unchanged, and is best described in the words of the foreword to the original standard:

This standard is a revision in means-of-compliance format of the technical requirements of NZS 1990: Chapter 6.1: 1964 (other than requirements for timber and wood-based products as materials, which are contained in NZS 3602).*

There is a change of emphasis in that this standard adds to and modifies the 'traditional' practices that were codified in the previous Chapter 6.1 by taking account on a rational engineering basis of the actual loads, which the building is expected to withstand. This has led to differing requirements for different seismic zones and for different wind exposure areas as well as for different floor loadings.

This change of emphasis, together with the desire to allow a wide range of choices between alternative building practices, has led to this standard being much longer than the previous Chapter 6.1, with many more tables and diagrams.

The words in the title "not requiring specific design" have always been something of a misnomer, because there never was any intention that buildings covered by it were not to be actually "designed". Rather, its purpose was to avoid specific structural engineering design. As a result, the title was changed with the 1999 revision to become *Timber framed buildings*.

A rational engineering approach was used as the basis for NZS 3604, although advantage was taken of the redundancies, additional strength, and other favourable factors known to be present in light timber frame buildings complying with this standard, even though such factors cannot normally be taken into account in specific structural design. Accordingly, it must be recognised that this standard will frequently give different solutions from those using the loadings standard (NZS 4203) and the timber design standard (NZS 3603).

The engineering basis of the document has become more wide ranging, and perhaps a little more sophisticated, over the years as results of research and practical experience have become available.

In particular, continued development of timber framing and the need to consider an increasing variety of materials and methods have led to the continuing evolution of the design of light timber framed buildings. These developments include:

- recognition of a wider range of wind zones
- the introduction of sheet materials, engineered wood products and proprietary systems (particularly framing timber sizes and stresses)
- the introduction of proprietary bracing systems, specifically developed to "fit into" the NZS 3604 framework, and the companion document referred to as the BRANZ P21 test method
- the change to a limit state design approach, in step with the evolution of NZS 4203
- the need to cater for more "open plan" living, with larger openings, greater spans, and more complex geometries which necessitate a more rational approach to design, and the need to push the boundaries.

This report covers the 1999 version of NZS 3604, and includes amendment 2, which is the current version at the time of writing.

1.3 Technical basis

Derivation of the tables in NZS 3604 was generally based on the following Standards:

Loadings	NZS 4203:1992, "The loading standard" (SNZ 1992)
Members and connections	NZS 3603:1993 "Timber structures standard" (SNZ 1993)
	NZS 3101:1995 "Concrete structures standard" (SNZ 1995)
	NZS 4230:1990 "Masonry structures" (SNZ 1990)

The symbols used in this report are consistent with the symbols used in those standards, unless noted otherwise.

Where the NZS 3604 committee chose an alternative approach as being more appropriate for light timber framed construction, this is detailed in this report, along with some background information on why the change was made.

1.4 Timber dimensions

When NZS 3604 was first published in 1978, the New Zealand building industry was undergoing a transition from "rough sawn" to "planer gauged" framing timber. Today the transition is virtually complete, and the industry now uses almost exclusively "dry dressed" and proprietary "dryframe" framing timber. Until the 1999 version, NZS 3604 referred to "call" dimensions, with the reader expected to be aware that actual timber sizes differed depending on condition (that is, sawn, dried, gauged or dressed). A table was provided to aid the conversion. For Amendment 2, a change was made to refer to timber in "actual minimum dried" sizes in an effort to make things clearer.

The original calculations for the change from Chapter 6.1 used "green gauged" sizes from NZS 3601 (SANZ 1973). For Amendment 2, the actual dimensions were used to derive the selection tables.

Table 1 below gives a comparison of the various sizes used during the period covered by NZS 3604.

Table 1. Framing timber sizes (all in mm).

Call dimensions	Green gauged (NZS 3601)	Dry dressed (NZS 3601)	Actual dried dimensions (NZS 3604 Amdt 2)
25	-	19	19
30	-	25	-
40	37	35	35
50	47	45	45
75	69	65	70
100	94	90	90
125	119	115	- *
150	144	140	140
200	194	180	190
225	219	205	- *
250	244	230	240
300	294	280	290

* The 125 mm and 225 mm sizes were removed from the standard with Amendment 2 because of advice that they were no longer commercially available in significant quantities.

1.5 Timber strength and stiffness

In recent years there has been increasing awareness that changes in the properties of Radiata pine sawn timber have resulted in a decrease in strength and stiffness of structural members. This is largely a result of the increasing proportion of juvenile (core) wood in the timber being produced from New Zealand's plantation forests.

Amendment 4 to NZS 3603

In March 2005, Standards NZ issued Amendment 4 to NZS 3603:1993 *Timber structures standard* (SNZ 1993). This acknowledged the general consensus within the timber and building industries that structural and framing timbers were not reliably achieving the engineering properties (strength and stiffness) as previously specified in NZS 3603.

Amendment 4 introduced several new grades of timber for both Radiata pine and Douglas Fir and distinguishes between timber that has (or has not) had its engineering properties verified by the procedures set out in NZS 3622 (SNZ 2004). Lower design stresses were provided for timber that has not been verified after grading (whether visually or machine graded). Amendment 4 retained the No 1 framing grade, but its properties were reduced to reflect the general reduction in timber strength and stiffness, and recognising the deficiency of sole reliance on visual grading. Amendment 4 also introduced a "Lower bound modulus of elasticity" (E_{lb}), to be used for isolated members, while the normal E value was intended to be used for members acting together and constrained to similar deformations.

Grades now provided for in NZS 3603 are as follows:

Visual grades

<table>
<tr><td>No 1 framing</td><td>(dry and green)</td><td>Timber visually graded to NZS 3631 (as before), but not verified</td></tr>
<tr><td>VSG8</td><td>(dry and green)</td><td rowspan="3">Timber visually graded to NZS 3631, then verified to NZS 3622</td></tr>
<tr><td>VSG10</td><td>(dry and green)</td></tr>
<tr><td>G8</td><td>(green)</td></tr>
</table>

Machine grades

<table>
<tr><td>MSG6</td><td>(dry)</td><td rowspan="5">All machine graded to AS/NZS 1748, then verified to NZS 3622</td></tr>
<tr><td>MSG8</td><td>(dry)</td></tr>
<tr><td>MSG10</td><td>(dry)</td></tr>
<tr><td>MSG12</td><td>(dry)</td></tr>
<tr><td>MSG15</td><td>(dry)</td></tr>
</table>

Standards referred to above:

AS/NZS 1748:1997 *Timber-stress graded-product requirements for mechanically stress graded timber*

NZS 3631:1988 *New Zealand timber grading rules*

NZS 3622:2004 *Verification of timber properties*.

Amendment 2 to NZS 3604

Amendment 2 to NZS 3604:1999, published in May 2006, provides for the "flow on effects" of these grading changes.

Following telephone and on-line user surveys, Standards NZ decided to include all the above grades in Amendment 2 to NZS 3604 except for MSG12 and MSG15. In addition, use of No 2 framing timber is retained for internal non-loadbearing walls.

Moisture content

The majority of selection tables cover situations where the timber members are anticipated to remain dry in-service and so the "dry" timber properties of NZS 3603 (less than 18% moisture content) are appropriate. However, there are some applications where this restriction is not appropriate. These are itemised in Clause 2.3.4 as: piles, stringers, bearers and joists in decks, posts, and exposed ends of rafters, purlins and battens. For these members, the selection tables are based on "green" properties of NZS 3603. The specific properties used are covered in the individual sections following.

The standard allows timber members to be installed green and allowed to dry in-service, provided they are propped appropriately.

For completeness, timber stresses used to derive the selection tables are shown in Table 2 below.

Table 2. Timber characteristic stresses.

Grade	Moisture condition	Bending strength, f_b (MPa)	Compression strength, f_c (MPa)	Tension strength, f_t (MPa)	Modulus of elasticity, E (GPa)	Lower bound MOE, E_{lb} (GPa)
MSG10, VSG10	Dry	20.0	20.0	8.0	10.0	7.5
	Wet in-service	11.7	12.0	4.0	6.5	4.4
MSG8, VSG8	Dry	14.0	18.0	6.0	8.0	5.4
	Wet in-service	11.7	12.0	4.0	6.5	4.4
MSG6, No1 framing	Dry	10.0	15.0	4.0	6.0	4.0
G8	Wet	11.7	12.0	4.0	6.5	4.4
No1 framing	Wet in-service	7.5	11.0	3.0	4.8	3.2

Shear strength, f_s, = 3.8 MPa (dry), 2.4 MPa (green).

Compression perpendicular to the grain, f_p, = 8.9 MPa (dry), 5.3 MPa (green).

Similarities of engineering properties allowed the selection tables to be consolidated into three groups, which are colour coded in the standard for easy identification.

MSG10, VSG10	Green
MSG8, VSG8, (G8)	Yellow
MSG6, No 1	Blue

It should be noted that MGP (Machine Graded Pine) is an Australian originated timber branding system which is not provided for in the New Zealand building control system.

1.6 Revisions

NZS 3604 is based on loadings from the Loading Standard, NZS 4203:1992. This is currently cited as an Acceptable Solution in NZBC Clause B1. However a suite of loadings standards (AS/NZS 1170) has recently been published and will supersede NZS 4203 in the near future. At the time of writing, the loadings for derivation of the selection tables in NZS 3604 had not been updated to reflect this change.

2. SECTION 5. BRACING DESIGN

The concept of a rational design basis for bracing the building structure against applied horizontal forces was new with the publication of the original NZS 3604 in 1978. Prior to that, wall bracing was prescribed at certain locations in the building, irrespective of wind or earthquake loading. Also there were no provisions for the bracing of piles. This practice codified traditional carpentry "rules of thumb", and had the advantage that it was easy for everyone to understand.

With the advent of the new NZS 3604, designers for the first time were required to consider, separately, resistance to horizontal loads as well as vertical loads. A companion document, BRANZ *Technical Paper P21* (Cooney and Collins 1979), provided the means to evaluate the lateral strength of wall bracing elements by test, so that the designer could now match the bracing demand to the capacity provided. The concept of matching the bracing demand of the building with the capacity provided by the chosen bracing elements was new with this standard, and is one which few other comparable standards in other countries have yet adopted (for example, SA 1992).

The other concept introduced in 1978 was the even distribution of bracing elements along notional "Bracing Lines" in each orthogonal direction, resulting in an "egg-crate" structural form. This was originally introduced to sub-floor bracing to ensure that the principles of even distribution of bracing elements could be followed, and was later extended to walls above ground floor level. Bracing lines were originally spaced at 5 m, but this was later increased to 6 m (or even more with the use of dragon ties or diaphragms). The concept encapsulates an important attribute of timber stick framed construction: evenly distributing loads between numerous, closely spaced resisting elements, rather than fewer, widely spaced stronger elements, which is more common in steel or concrete engineered structures.

The term Bracing Unit (BU) was introduced in 1978 to quantify the amount of bracing, and to measure wall bracing element performance. The value of a BU was established by test at the NZ Forest Research Institute. 100 BUs is the racking capacity value assigned to a 2.4 m square, plasterboard lined and timber weatherboard clad, diagonally braced, timber framed wall when tested under conditions defined in BRANZ P21. It was approximately 5 kN for both wind and earthquake under the working stress design system. Thus 1 kN is equivalent to 20 BUs.

Over the years, as buildings designed using NZS 3604 became more ambitious, the rules were developed and extended and became quite confusing, to the extent that it is common practice today among architects to get structural engineer consultants to carry out the bracing calculations. This obviates one of the original intentions of the document, that of reducing the involvement of structural engineers.

At the request of several comments received on the draft, the 1999 revision of NZS 3604 collected together all of the provisions for the design of lateral bracing, which were scattered throughout the document, and placed them altogether in one new section. A secondary aim was to encourage designers to deal with bracing as a conscious step in the design process, and thus create more awareness of its intent and purpose.

Clauses 5.2 and 5.3 deal with the bracing demand side of the equation (wind and earthquake). Clauses 5.4, 5.5, 5.6 cover the distribution of bracing elements. The resistance of the various elements and their construction details remain in the applicable sections of the standard.

2.1 Wind bracing demand (Clause 5.2)

The wind bracing demand was derived following essentially the "static analysis procedure" as set out in NZS 4203, Clauses 5.3 to 5.8 (Eq. 5.4.1).

2.1.1 Site wind speed

The governing equation is:

$$V_{(z)} = V.M_{ls}.M_{(z,cat)}.M_s.M_t.M_r$$

where:

$V_{(z)}$ is the site wind speed

V is the basic directional wind speed

M_{ls} is the limit state multiplier

$M_{(z,cat)}$ is the site terrain/height multiplier

M_s is the shielding multiplier

M_t is the topographic multiplier

M_r is the structure risk multiplier.

Values used for deriving the site wind speed were:

V = 45 metres/second for regions II, III, IV, and VI

= 48 metres/second for regions I, V, and VII

M_{ls} = 0.93

$M_{(z,cat)}$; The factors were defined as follows:

Ground roughness (NZS 3604)	**Terrain category (NZS 4203)**	**Height (z) (NZS 4203)**	**$M_{(z,cat)}$**
Urban	3.0	7.5m	0.79
Rural	2.5	7.5m	0.87
Open	2.0	7.5m	0.96

M_s; The factors were defined as follows:

Site exposure (NZS 3604)	Ground roughness (NZS 3604)	M_s
Sheltered	Urban	0.80
	Rural & open	0.90
Exposed	Urban	0.90
	Rural & open	1.0

M_t; The factors were defined as follows:

Topographic class	M_t
T1	1.0
T2	1.15
T3	1.25
T4	1.4
T5	1.55

M_r = 1.0 (for class 4 buildings as defined in NZS 4203).

The resulting wind speeds calculated by equation 5.4.1 were then grouped into broad band "Wind zones" for simplicity by users, and the dynamic wind pressure calculated using equation 5.5.1 of NZS 4203 ($q_z = 0.6 \times V_{dz}^2$):

Wind zone	Wind speed	Dynamic pressure q_z, (kPa)
Low	Up to 32 m/s	0.62
Medium	33 to 37 m/s	0.82
High	38 to 44 m/s	1.16
Very High	45 to 50 m/s	1.50

2.1.2 Wind bracing demand

Demand tables are a summation of external wind pressures on building elements (resolved in the horizontal direction in the case of roofs) based on the procedures of NZS 4203.

The overall building height in NZS 3604 from the lowest point of the ground to the roof apex is limited to 10 m. Practical foundation and storey heights and building widths will limit roof heights and pitches for buildings that are at their maximum permitted height. The coverage of the demand tables takes these permutations into account.

The building width on which to base the roof pitch (and therefore its C_{pe} value for wind across the ridge) has been taken as 7.2 m. Thus the bracing demand for buildings wider than 7.2 m will be greater than is technically required.

A summary of the C_{pe} factors used is as follows:

Roofs	Height above eave (m)	Net C_{pe} (allowing for upwind and downwind slopes)
	1	0
	2	0.4
	3	0.7
	4	0.85
	5	1.2
Walls	**(including gable ends)**	**Net C_{pe} (windward and leeward walls)**
		1.2

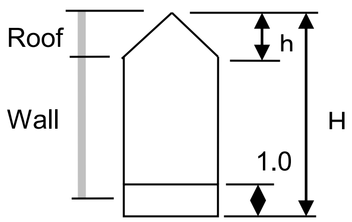
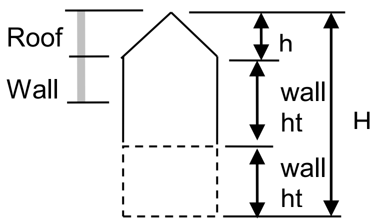
k_a, k_l, k_p were all taken as 1.0.

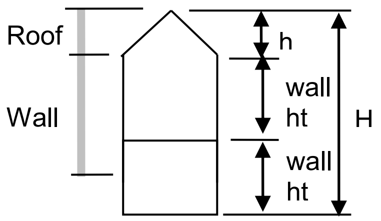
Demand was calculated on a 1 m length of building to give BUs per metre. For wind along the ridge, the height of the gable was taken as ½ roof height "h", and for wind across the total projected height "h" was used.

For Table 5.5, the foundation height was assumed to be 1 m.

For Tables 5.6 and 5.7, the wind load on the wall under consideration was assumed to be equally distributed to the floor level above and below, so that the demand was based on half the wall height.

The equations used are tabulated below, where D = wind load demand in BU.

NZS3604 Table Ref:		Equations
5.5		**Across:** $D = q_z\left[C_{pw}(H - h - 0.5) + C_{pr}(h)\right]x20$ **Along:** $D = q_z\left[C_{pw}\left(H - \frac{h}{2} - 0.5\right)\right]x20$
5.6		**Across:** $D = q_z\left[C_{pw}\left(\frac{wall\ height}{2}\right) + C_{pr}(h)\right]x20$ **Along:** $D = q_z\left[C_{pw}\left(\frac{wall\ height}{2} + \frac{h}{2}\right)\right]x20$

5.7		**Across:** $D = q_z\left[C_{pw}\left(H - h - \dfrac{wall\ height}{2}\right) + C_{pr}(h)\right]x20$ **Along:** $D = q_z\left[C_{pw}\left(H - \dfrac{wall\ height}{2} - \dfrac{h}{2}\right)\right]x20$

Note that "H" in Figure 5.3 of NZS 3604 is not clearly defined, but is intended to be measured from lowest adjacent ground level.

2.2 Earthquake bracing demand (Clause 5.3)

The earthquake bracing demand was derived following essentially the "equivalent static method" as set out in NZS 4203, Clause 4.8 (Eq. 4.8.1) (SNZ 1992).

The governing equation:

$$V = C \,.\, W_t$$

was adopted for NZS 3604 use into:

$$D = C \,.\, w \,.\, A \,.\, 20$$

where:

D = demand (bracing units)

C = lateral force coefficient as defined in NZS 4203

w = seismic weight (kPa)

A = gross floor area (m^2).

Advice was sought from the NZS 4203 technical drafting committee as to the application of Clause 4.8 to low rise timber buildings coming within the scope of NZS 3604, in particular the parameters making up the lateral force coefficient C, and the effective live load contributing to the seismic weight w. The outcome is discussed below.

2.2.1 Lateral force coefficient C

The essential problem in adopting NZS 4203 is the wide disparity between the bi-linear elasto-plastic models used to develop the NZS 4203 response spectra and the degrading "slack system" response of sheathed timber framed bracing panels (see Figure 1 below).

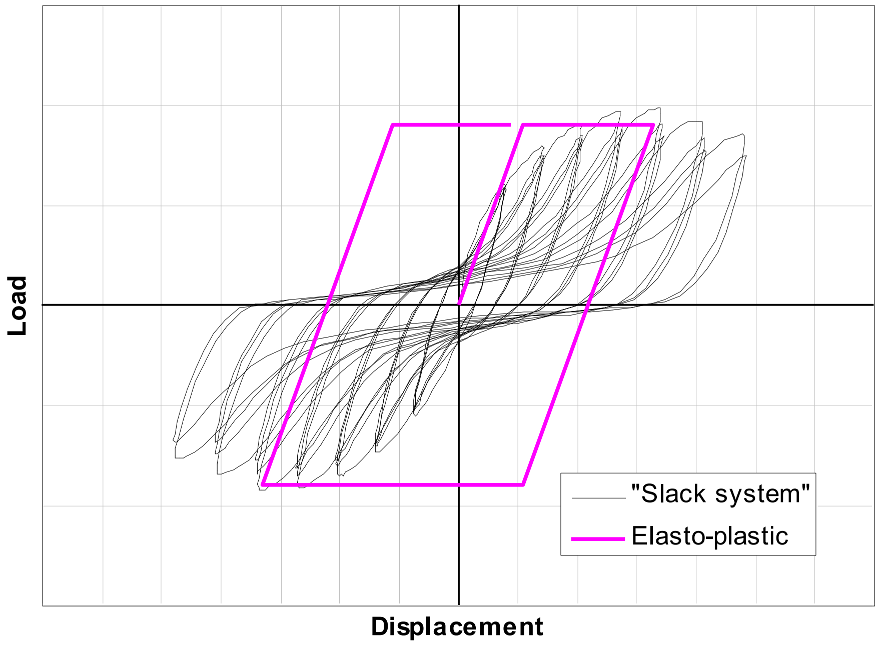


Figure 1. Comparison between elasto-plastic and slack system response.

The approach used was to transpose the NZS 4203 elastic response spectra back into the time domain, and use the resulting synthetic ground motion as the input into an integrated time history analysis. The resisting element to which this motion is applied was calibrated to match experimental results.

The process is described in detail in Deam 1997, but it can be summarised here as:

1. Load/displacement characteristics are obtained from racking test specimens of similar construction to typical timber framed houses.
2. Maximum reliable displacement (MRD) is defined from the load/displacement plot as the point where load at the third displacement cycle begins to fall from the previous third cycle.
3. A single degree of freedom model with the same load/displacement characteristics is then subjected to several time history analyses using representative accelerograms, scaled to the appropriate NZS 4203 design spectra.
4. Displacement and acceleration response spectra are then generated from the outputs from the analyses.
5. The greatest mass the bracing element is able to restrain is calculated from the equation of motion for a single degree of freedom oscillator, using the period obtained from the above displacement spectrum at the MRD.

The method was used for three typical examples of braced wall elements (plasterboard, plywood, and fibre-cement) and the resulting acceleration coefficients (obtained from the acceleration spectra) plotted on Figure 4.6.1 of NZS 4203 *Basic seismic coefficient for intermediate soil sites*.

On this basis the NZS 4203 committee recommended a base shear coefficient of 0.3 as appropriate for determining earthquake demand on typical timber framed constructions likely to be used with NZS 3604.

This is equivalent to the basic seismic hazard coefficient, $C_h(T_1,\mu)$, for a ductility factor, μ, of 3.5, period, T_1, of less than 0.45 sec, on an intermediate soil site taken from Table 4.6.1 of NZS 4203.

Using a seismic performance factor, S_p, of 0.67, a risk factor, R, of 1.0, and a limit state factor, L_u, of 1.0, the lateral force coefficient, C, for the relevant seismic zones may be tabulated below:

Zone	Zone factor Z	Lateral force coefficient C
A	1.2	0.241
B	0.9	0.181
C	0.6	0.121

2.2.2 Building parameters

The parameters used in the calculation model used to derive the bracing demand tables were as follows:

Floor area	100 m^2
Length/breadth	2
Soffit width	600 mm

Single storey on timber sub-floor (Table 5.8)

Storey	Height (m)	% openings
Top storey	2.4	30
Foundation	1.0	-

Two-storey on timber sub-floor (Table 5.9)

Storey	Height (m)	% openings
Top storey	2.4	30
Lower storey	2.7	30
Foundation	1.3	-

One or two storeys on concrete slab (Table 5.10)

Storey	Height (m)	% openings
Top storey	2.4	30
Single storey	2.4	30
Lower of two storeys	2.7	30

2.2.3 Assumed loads

Dead loads

Application		Dead load (kPa)
Roof	Light	0.2
	Heavy	0.6
Ceiling		0.24
Walls	Light	0.3
	Medium	0.8
	Heavy	2.2
Partitions (based on floor area)		0.2
Floor		0.6

Effective seismic live load

The two issues addressed by the NZS 4203 committee were:

- the probable live load likely to be present at the time of the earthquake
- the extent to which the live load participates in the dynamic response of the building (degree of participation is influenced by lack of connection with the structure, for example loose objects able to slide or rock).

For the purpose of deriving the bracing demand, the arbitrary point-in-time live load likely to be present was considered to be an average, rather than the 90% value, thus giving a value for the live load combination factor ψ_u = 0.2. Based on the floor area of 100 m^2, the tributary area factor ψ_a was taken to be = 0.67, giving an effective live load contributing to the seismic weight of:

$$\psi\ (= \psi_a \times \psi_u) = 0.134$$

Basic floor live loads considered were 2.0 kPa for Section 5 of the Standard, and 3.0 kPa for Section 14, giving 0.134 x 2 = 0.268 kPa, and 0.4 kPa respectively.

2.2.4 Load distribution

Vertical

Seismic weights of outside walls were distributed to the adjacent floor and roof levels as follows:

33% to roof (or floor above)

66% to floor below.

The vertical distribution between levels used the provisions of NZS 4203, Clause 4.8:

$$F_1 = 0.92 \cdot Vx \left(\frac{W_i \cdot h_i}{\sum (W_i \cdot h_i)} \right)$$

Horizontal

No allowance was made for eccentricity of load in plan. Some resistance to accidental eccentricity and torsion of the building as a whole is provided by the limits on distribution of bracing lines and bracing elements in Clauses 5.4 and 5.5.

2.3 Sub-floor and wall bracing design (Clauses 5.4 and 5.5)

2.3.1 Distribution

The distribution of bracing was initially devised to satisfy the broad engineering principles of:

- Symmetry of distribution of lateral force resisting elements to reduce torsion loading under lateral forces of both wind and earthquake (see Section 2.2 of this report).
- A relatively even distribution of lateral force resisting elements to avoid concentrations of loading on individual elements and their connections to the rest of the structure (Clause 5.5.3).
- Ensuring that bracing elements were spread out to the building extremities (for example, corners of external walls) where they are more effective in resisting torsion loads (Clause 5.5.3).

As architectural styles moved towards open planning, larger exterior openings and larger rooms, steps had to be taken to ensure that the original structural concepts were not undermined. At the same time it was desirable to accommodate modern architectural styles without resorting too often to specific engineering design.

2.3.2 Walls at angles

Braced walls at angles to the principal orthogonal directions were simply handled by factoring the bracing ratings by the cosine of the relative angle to fit into the orthogonal loading concept.

2.3.3 Internal walls

The 6 m spacing is a metric conversion of the 20 foot wall spacing permitted in NZS 1900: Chapter 6.1. This was modified to take account of lateral support for top plates from ceilings or dragon ties.

A minimum amount of bracing of 70 BU for each internal brace line was chosen to be equivalent to about 2 m of conventional plasterboard lined wall frame under the earlier working stress design method. With the new assessment methods and limit states design, these values could have been increased by some 30% to 50% to maintain the same level of distribution. However, it was decided that the intent (that bracing should be distributed evenly) is still clear, so it was left as is. The pairs of bracing elements rule Clause 5.5.5.4(b) was introduced to allow additional planning flexibility without compromising the minimum bracing intent.

2.3.4 External walls

A minimum value of 10 BU/m for each external bracing wall was chosen to ensure that a minimum amount of bracing was at least considered in each external wall. Short lengths of walls, offset less than 2 m from each other, could be added together for bracing assessment purposes under this rule.

2.4 Bracing capacity ratings of sub-floor bracing elements (Table 5.11)

The options for sub-floor bracing in the 1978 standard were concrete foundation walls, cantilever piles and diagonal braces. Since then, sub-floor bracing elements have evolved considerably to the following types:

Anchor piles

Anchor piles began as short, stubby concrete piles (maximum 150 mm above ground level) used as a base anchorage to resist horizontal loads from what were effectively braced jack studs. In 1984 the permitted height of attachment was increased to 500 mm above ground level, so anchor piles were now required to act as cantilevers to a significant degree. The option of extending them up so they could be directly connected to the bearer was first introduced in 1984, and this is now the only configuration for an anchor pile in the current version of the standard. The maximum cantilever height above ground has been increased to 600 mm to give more flexibility with floor and ground levels. The resulting bending moment demand on an anchor pile has meant that they are loaded beyond the capacity of visually graded timber, and are required to be proof tested. This is addressed in the standard for timber piles and poles, NZS 3605 (SNZ 2001).

Braced piles

Braced piles started out as diagonal braces, which could be attached to a wide variety of elements at each end (a pile, a bearer, a joist or blocking between joists, a foundation wall, and even cut between the studs and plates of a jack stud wall). The necessity to describe details more and more precisely for a wider variety of applications increased the demands of this section, until the complexity of its requirements exceeded the ability of most users to understand them. Coincidentally, the market share of this type of sub-floor structure dwindled, and so the braced pile has now been reduced in application and simplified to what is now only a braced pile system consisting of two piles, a brace and a connection to the floor structure.

Cantilever piles

Cantilever piles were initially intended to be either shallow piles bedded in a concrete footing, or timber piles driven to a specified set to provide adequate vertical loadbearing through softer ground. The cantilever strength of the shallow founded option is limited because of the minimal size of the footing, therefore making it a less attractive proposition compared with an anchor pile. Thus the driven pile option is the only one remaining in the current version of the standard.

2.4.1 Sub-floor bracing ratings (Clause 5.4.4)

Bracing ratings for anchor and braced piles were originally derived from the capacity of the diagonal braces given as an option in the 1978 version. Tests on typical brace assemblies under cyclic loading (Wood et al 1976) assessed the axial capacity of this element as 17 kN. At the maximum permitted inclination (45^0), vectorial resolution gave a basic horizontal load of 12 kN.

The rating for braced piles was then derived from that basic 12 kN load, using the following equation:

$$\text{Rating} = 12 \times F_1 \times F_2 \times 20 \qquad \text{..................(1)}$$

where:

F_1 was a factor (based on Dean 1987) to allow for the lower performance of "pinched" hysteresis loop systems, compared with the equivalent elasto-plastic loops that the design loads were based on ($F_1 = 0.5$)

F_2 was a factor to allow for the limited ductility of braced piles ($\mu = 2$ for bolted structures with bolts less than 18 mm diameter in which the bolts are likely to yield in bending without causing a brittle fracture in the timber) compared with $\mu = 4$ that the design loads were based upon. F_2 was based on the spectral coefficient from Table 4.6.1 of NZS 4203 at a period of 0.45 sec for intermediate soil sites ($F_2 = 0.27/0.49 = 0.55$)

and 20 is the conversion factor from kN to BUs.

This gives:

Rating = 12 x 0.5 x 0.55 x 20 = 66 BUs,

which was then rounded up by the committee to 70 BUs.

Elastically responding structures, such as cantilevered driven piles, have a recommended ductility factor of 1.0.

For the 1999 revision of NZS 3604, the results of BRANZ experimental studies of piled foundations were available (Thurston 1996). Lateral load tests were undertaken on anchor piles, braced piles and cantilever piles. From the load/displacement characteristics the analysis procedure, described above under Section 2.2 of this report, was used to derive a bracing value of 120 BUs for anchor and braced piles. The committee increased the ratings for these elements accordingly, but all other bracing values were unchanged from NZS 3604:1990. Ultimate pile top deflections of between 30 mm and 50 mm were recorded in these tests.

The resulting parameters are tabulated as follows:

2.4.2 Design for safety (Ultimate Limit State – ULS)

Element	**Earthquake**			**Wind**		
	rating		**Deflection**	**rating**		**Deflection**
	(BU)	**(kN)**	**(mm)**	**(BU)**	**(kN)**	**(mm)**
Anchor pile	120	6	30	160	80	30
Braced pile	120	6	50	160	80	50
Cantilever pile	30	1.5	25	70	3.5	45

2.4.3 Design for serviceability (Serviceability Limit State – SLS)

Element	**Earthquake**			**Wind**		
	Rating		**Deflection**	**Rating**		**Deflection**
	(BU)	**(kN)**	**(mm)**	**(BU)**	**(kN)**	**(mm)**
Anchor pile	20	1	3	120	60	10
Braced pile	20	1	3	120	60	13
Cantilever pile	5	0.4	1	45	2.25	4

2.4.4 Foundation walls

The bracing values for reinforced concrete and masonry foundation walls set out in Table 5.11 were determined on the basis of static overturning of the wall, and the capacity of the fixings prescribed in Clauses 6.11 and 6.13 to secure the framing to the wall. The longer the wall becomes, the higher the resistance to overturning and hence the increase in BUs per metre of wall length.

While dynamic considerations would indicate that overturning stability of foundation walls will never be an issue, BRANZ research (Thurston 1995) indicates that there may be shortcomings in the current provisions for timber plate fixings.

2.4.5 Bracing capacity ratings for wall elements (Clause 8.3)

The earlier versions of the standard included bracing ratings for a number of generic wall bracing elements. These were based on estimated strengths until the advent of the BRANZ P21 test method in 1979 (Cooney and Collins 1979). The method has stood the test of time well, although it is currently being revised to incorporate experience gained over the years. The P21 test has enabled the rating of many proprietary bracing systems, and also revealed the inefficiency of many of the generic systems, especially those based on diagonal braces. As a result, the table of generic systems has been gradually phased out in favour of ratings based on tests. A death blow was struck in 1992 when NZS 4203 moved New Zealand structural design into limit states format, and with the 1999 version of the standard, it was dropped altogether. Manufacturers have responded well to the incentive of achieving reliable ratings for their systems based on tests, and there are rarely any requests for generic bracing information.

3. SECTION 6. FOUNDATIONS AND SUB-FLOOR FRAMING

The provisions of this section of the Standard (and Section 14) apply to the parts of the structure supporting suspended timber floors. The complete sub-floor structure is specifically required to resist vertical (gravity) and horizontal (from wind and earthquake actions) loads.

Details are provided in this section for a variety of vertical support elements (piles, foundation walls etc), which may be used in any combination, except that for three storey buildings, perimeter walls must be supported by foundation walls. In practice, use of some of the elements is limited by their maximum allowable height above ground level. This ranges from 600 mm for anchor piles to a maximum of 3 m in the case of timber ordinary piles. Beyond these heights specific engineering design is required to properly allow for slenderness and lateral flexibility.

Several of the foundation elements have also been allocated a bracing resistance, which is set out in Section 5 Bracing. Proprietary (tested) sub-floor bracing elements are also provided for. Section 5 also covers the arrangement and disposition of bracing elements so as to match the applied loads, which may affect the choice of some of the foundation elements in this section.

The provisions for sub-floor ventilation and access crawl space encapsulate typical good practice and have remained essentially unchanged since 1978.

3.1 Setting out (Clause 6.3)

The restriction of 200 mm offset between loadbearing wall and bearer or pile is intended to limit high bending moments in the floor joists (or bearer) caused by concentrated loads not allowed for in their derivation.

3.2 Piles (Clause 6.4)

3.2.1 General

The introduction of NZS 3604 in 1978 increased the options for piles to include cantilever piles, braced piles, and anchor piles, as well as the traditional ordinary piles. This was done to provide alternatives for lateral bracing of the sub-floor structure.

The details provided generally encapsulated standard practice of the day, except where more robust provisions were required for bracing – connections to the floor framing in particular.

The lower height limits for piles, Clause 6.4.1.1 (c), are intended to prevent transmission of moisture from the ground to bearers and other framing timbers which are not required to be as heavily treated as the piles. The upper limit of 3 m for timber piles is to prevent excessive slenderness. The height limits for anchor and cantilever piles match the proof loading provisions of NZS 3605 (SNZ 2001).

3.3 Pile footings (Clause 6.4.5)

3.3.1 Design for safety (ULS)

General

The provisions for pile footings for light and heavy roofs in L, M, H, and VH wind zones are the same as those used in NZS 3604:1978. The table is a carry over from NZS 1900: Chapter 6.1. The table values are based on Working Stress Design (WSD) methods assuming that the soil has a safe bearing capacity of 100 kPa. This equates to an ultimate bearing capacity of 300 kPa using a safety factor of 3. This in turn equates to a cohesion shear strength of clay or clayey soils for shallow continuous or square footings of 60 kPa, derived from the work done by AW Skempton (Teng 1962).

It was assumed that approximately half the live load in a house is furniture and therefore should be considered as a permanent load with a safety factor of 3. Because the remaining portion of the live load is considered to be of short duration, for the combination of dead + live load, a safety factor of 2 was considered acceptable. An independent check by ultimate strength methods on the foundation sizes did not justify changing the table.

Loads

	Dead load (G)	Live load (Q)
Light roof cladding	0.287 kPa	0.25 kPa
Heavy roof cladding	0.766 kPa	0.25 kPa
Floors	0.58 kPa	1.5, 2, 3 kPa
Partitions	0.58 kPa	-

The area reduction factor for live load (ψ_a) was taken as 1.0.

The loads were applied as a uniformly distributed load, over the tributary area given by the product of the span of bearers and joists in Table 6.1.

Capacity

Allowable bearing loads per pile were calculated as follows:

Load combination	Safety factor	Bearing load (kPa)
G + Q/2	3	100
G + Q	2	150

3.4 Driven timber piles (Clause 6.6)

3.4.1 General description

During the drafting of NZS 3604 in 1978, it was recognised that there was a place for the use of driven, natural round timber, treated piles as foundations in light timber framed construction.

3.4.2 Design for strength

General

Design for strength was based on a series of tests carried out on 32 piles at 10 sites in South Auckland in 1972 (Cocks et al 1974). The piles were 1.8 m long, 140 mm diameter, and were driven to 1.2 m depth. The piles were loaded vertically, laterally and then withdrawn.

Loads

Loads used to obtain pile spacings in Table 6.2 of NZS 3604 were:

	Dead load (G)	Live load (Q)
Light roof cladding	0.287 kPa	0.25 kPa
Heavy roof cladding	0.766 kPa	0.25 kPa
Floors	0.58 kPa	1.5, 2, 3 kPa
Partitions	0.58 kPa	-

The area reduction factor for live load (ψ_a) was taken as 1.0.

Member capacity

The ultimate loads resisted by all the piles were plotted against set per blow, and a “best fit” line drawn, giving the following ultimate loads for the given sets per blow:

Set per blow (mm)	Ultimate load (tonnes)
25	6.16
50	4.08
100	2.0

Allowable bearing loads per pile were calculated using the following safety factors,

Load combination	Safety factor
G + Q/2	3
G + Q	2

and the spacings tabulated to suit practical joist and bearer spans.

It should be noted that the Hiley Dynamic Pile Driving Formula provided ultimate loads well in excess of those measured. The variation ranged between 50% to 100% above those actually measured. A safety factor of 5 is therefore recommended to obtain a safe working load when using the Hiley Formula for pile sizes covered by NZS 3604.

3.4.3 Design for serviceability

General

Design for serviceability included consideration of vibration due to wind loading. The details may be found in Cocks et al (1974).

3.5 Jack studs (Clause 6.10.2)

3.5.1 General description

Jack studs are vertical members extending up from piles to the underside of floor or deck framing, and which support only gravity loads. Because there is no lining attached to a jack stud, there can be no lateral bending load applied.

The selection table for jack studs (Table 6.3) was initially taken from NZS 1900: Chapter 6.1:1964 (SANZ 1964), and re-analysed using working stress design methods from NZS 1900: Chapter 9.1:1964 Timber (SANZ 1964b). For Amendment 2, tables for 1.5 and 2.0 kPa floor loading were consolidated into one covering both. 3 kPa floor loads were included in Section 14.

3.5.2 Design for safety (ULS)

General

The maximum jack stud height given in Table 6.3 of NZS 3604 was determined from the axial strength of the jack studs calculated using Section 3.3 of NZS 3603.

Loads

	Dead load (G)	Live load (Q)
Roof	0.84 kPa (along slope)	-
Ground floor	0.35 kPa	2.0 kPa 3.0 kPa (Section 14)
Upper floor	0.4 kPa	
Wall	1.44 kN/m	-

Floor loads were applied as a uniformly distributed load over the tributary area given by the product of the span of bearer and its loaded dimension. Roof loads were based on an eaves width of 0.6 m, and roof span of 12 m (giving a loaded dimension of 6 m), times span of the bearer.

Load cases considered:

1	1.4G
2	1.2 G + 1.6 Q

Structural model used for strength

The model used for strength was a pin ended column subjected to axial loading, as shown in Figure 2 below.

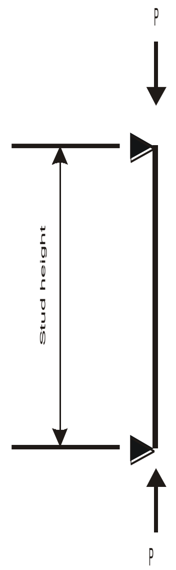


Figure 2. Structural model used for jack studs.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors for strength, k_1, = 0.6 for load case 1, and 0.8 for load case 2.

No restraint was assumed to each end of the member, with L_{ay} taken as the height of the stud.

Dry timber properties were used, as set out in Section 1.5 of this report.

The maximum length of the jack stud was calculated by an iterative process controlled by a macro.

3.6 Bearers (Clause 6.12)

3.6.1 General description

Bearers are beams supporting floor joists and are themselves supported by piles, jack-studs or foundation walls.

Bearers supporting floor joists over two or three spans were taken from NZS 1900: Chapter 6.1:1964, and re-analysed using working stress design methods from NZS 1900: Chapter 9.1:1964 Timber. New tables were developed for Amendment 2.

The bearer tables include loading from the ground floor only. Loadbearing walls above (either parallel or perpendicular to the bearer) were not taken into account, hence the provision in Clause 6.3 for limiting the location of the supporting pile to within 200 mm of the wall above running perpendicular to the bearer.

The stiffening effect of the wall above, running parallel to the bearer and acting as a deep beam, was considered to be sufficient for satisfactory performance of the bearer, and the lack of problems in practice has borne this out. However, while the 1990 version of the standard limited the use of pile/bearer support to single storey buildings, it is now permitted for two storey buildings. The reason for this change is unknown.

3.6.2 Design for strength

General

Design for safety included a strength check of bending, shear and bearing.

Loads

Dead load of floor (G)	0.35 kPa
Live load on floor (Q)	1.5 and 2.0 kPa (Section 6) 3.0 kPa (Section 14)

Load cases considered:

1	1.4G
2	1.2G + 1.6Q

Structural model used for strength

The model used for strength was a two span, simply supported beam as shown in Figure 3 below.

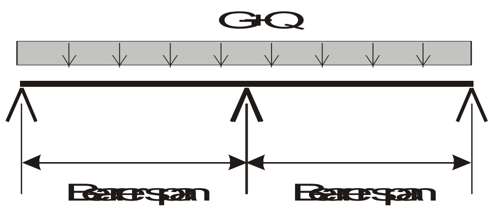


Figure 3. Structural model used for bearers.

Load was applied to both spans as a uniformly distributed load perpendicular to the member, with no load sharing.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors for strength, k_1, = 0.6 for load case 1, and 0.8 for load case 2.

Dry timber properties from NZS 3603 were used for 1.5 kPa and 3.0 kPa floor live loading, as these were considered internal sub-floor structures and protected from the weather. Floors designed for 2 kPa loading are most likely to be used for decks, and exposed to the weather, so "green" properties were used. Timber properties are set out in Section 1.5 of this report.

3.6.3 Design for serviceability

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

Loads were as given above for strength design.

Load cases considered were:

Load case	Combination
1	G + ψ_s Q_s (short-term loading)
2	G + ψ_l Q_s (long-term loading)

The short and long-term live load factors were:

ψ_s = 0.7, ψ_l = 0.0,

Structural model used for serviceability

The model used for serviceability is shown in Figure 3 of this report. Load was applied to both spans together as a uniformly distributed load. No allowance was made for load sharing.

Deflection calculation

The average modulus of elasticity, E, was used to calculate bearer deflection, as set out in Section 1.5 of this report. The load duration factor, k_2, was taken as 2.5 for load case 2.

Deflection criteria

The deflection limit was span/300, with no absolute limit.

3.7 Stringers, spacing of fixing bolts (Clause 6.13)

3.7.1 General

A stringer is a horizontal timber member fixed to the side of a concrete or masonry foundation wall, and supporting the ends of floor joists.

3.7.2 Loads

Dead load of floor	0.35 kPa
Live load on floor	2.0 kPa (Section 6) 3.0 kPa (Section 14)

Area reduction factor for live load (ψ_a) was taken as 1.0.

Load cases considered:

1	1.2G + 1.6Q
2	G + Q

3.7.3 FRI tests to establish bolt capacity

The capacity of a bolt in this situation was not defined in NZS 3603. Accordingly, the NZS 3604 committee requested the NZ Forest Research Institute to carry out tests on a range of bolt diameters, to establish suitable bolt loads.

Test specimens

Bolts of the range of diameters suitable for stringers were cast into ready-mixed concrete of nominal strength 17.5 MPa. The range of specimens tested is given in Table 3 of this report. Green Radiata pine stringer sections were attached to the concrete via the bolts using 50 mm square washers and the nuts tightened finger tight. The specimens were then set aside and allowed to dry out. Ten specimens were tested for each bolt size and direction of load in relation to the grain direction.

Test method

The concrete blocks were attached to the platen of a universal testing machine and the stringers were loaded at two points away from the bolt, simulating the joists that the bearer is supporting. The load was applied at a constant cross-head speed of 5 mm/minute until failure. A load/deflection chart was plotted during loading. Moisture content and density were measured on the timber components of the test specimens.

Results

A summary of the test results is given in Table 3 below.

Table 3. Summary of results of stringer tests.

Series	Bolt diam.	Nominal timber thickness	Load direction	Assembly	Test	Average load at 2 mm deflection		Average load at 5 mm deflection	
No.	mm	mm	To grain	mc	mc	kN	CoV%	kN	CoV%
1.1	9.5	50	Parallel	Green	Dry	3.99	38	7.91	17
1.2	12.5	50	Parallel	Green	Dry	4.70	26	9.61	14
1.3	16	75	Parallel	Green	Dry	4.91	36	12.54	15
1.4	19	75	Parallel	Green	Dry	6.04	57	15.01	27
1.5	9.5	50	Perpendicular	Green	Dry	4.24	21	7.62	14
1.6	12.5	50	Perpendicular	Green	Dry	4.43	28	8.96	22
1.7	16	75	Perpendicular	Green	Dry	4.00	3 tests	8.42	3 tests
1.8	19	75	Perpendicular	Green	Dry	3.93	40	11.22	20

At 2 mm deflection no permanent deformation of the bolts was observed, deformation being a combination of elastic bending of the bolt and crushing of the timber. At 5 mm, crushing of the timber under the bolt and some permanent bending of the bolts took place. None of the concrete blocks failed at these deformations.

Derivation of allowable loads

It was considered that an average short-term deflection of 2 mm under a load of G + Q was a reasonable design limit for the situation of a bearer in light frame construction. This is less than the shrinkage of a 100 mm deep joist or bearer drying from green to 15% moisture content, so is well within the capacity of a light frame system to accommodate. The density of the timber components is of the order of the density to be expected of structural timber for light framed construction.

Hence it was recommended that the allowable loads be the average of the test loads at 2 mm deflection. Table 6.7 of NZS 3604 is based on these values. A check of timber bending under the ULS was done for Amendment 2 using the timber properties of Section 1.5 of this report.

4. SECTION 7. FLOORS

Floors may be at ground level, or suspended above a useable space below. A ground floor may be a concrete slab-on-grade, or of timber joist construction supported on a sub-floor structure. The sub-floor structure, from bearers on down, is covered in Section 6 of NZS 3604 Foundations and sub-floor framing.

Timber ground floors may be up to 3 m above the ground, the actual height depending on the shape of the ground. However limits are placed on this height by the need to ventilate the floor and maintain access to the sub-floor space, and also by the nature of the bracing required of the sub-floor structure. These aspects are discussed in detail in Section 3 of this report. As far as the floor plate itself is concerned (that is, the flooring and the joists and blocking), there is no difference in the requirements or detailed criteria between one that is supported on the walls of the storey below and one supported by a sub-floor structure.

The main structural function of a suspended floor is to resist gravity loads perpendicular to its own plane, although there is also a need to provide resistance to loading in the plane of the floor by diaphragm action. Where bracing lines below the floor are spaced within 6 m centres, this requirement for diaphragm action is modest, but for longer spanning floors it is more demanding.

4.1 Timber floors

The two major elements are the flooring (sheet or strip) and floor joists. Secondary members such as dwangs are required to provide lateral stability to the joists and support edges of floor sheeting. Bearers and below are considered part of the sub-floor structure and are covered in Section 6 of this report.

Floor live loads are set out as part of the overall scope of the document in Table 1.2 of NZS 3604. The categories and occupancies are generally based on NZS 4203 (SNZ 1992). However, a number of occupancies covered by NZS 4203 have been excluded, either because the loads specified were considered too high for realistic timber construction, or because the loads in those occupancies are required to be calculated, which was not considered appropriate for users of NZS 3604. These exclusions are as follows:

Category	Spatial occupancy	Load	Reason for exclusion
Domestic	1.4 Garages	9 kN concentrated	too high
Residential	2.1 Balconies	4 kN distributed	too high
Educational	3.2 Laboratories	3 kN distributed	requires calculation
	3.4 Library stacks	4 kN distributed	too high
Institutional	4.2 Operating theatres 4.4 Heavy equipment rooms	3 kN distributed	requires calculation
Office	6.1 Banking chambers	4 kN distributed	too high

It is clear that buildings coming within several of the occupancies included in NZS 3604 (for example, schools or offices) would have considerable input from a structural engineer designer. With ready-made solutions in NZS 3604 available for floors (and other building elements), this has the effect of widening the scope of NZS 3604 well beyond the original intention, thus increasing its usefulness to the overall benefit of the industry.

On the other hand, if these buildings have no structural engineer input there is a risk that the level of loading goes beyond the expertise of the designer (for example, builder or draughtsman) and the risk of unintentionally omitting or overlooking a critical element in the load path is increased.

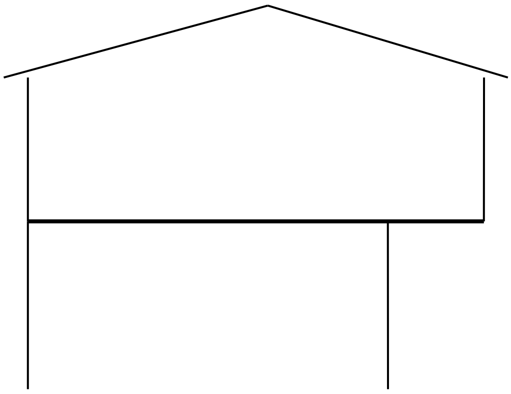
4.2 Floor joists (Clause 7.1)

4.2.1 General description

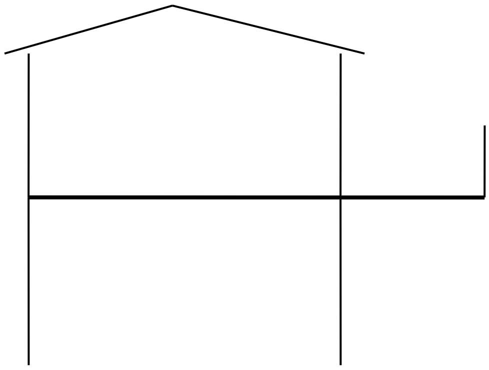
Floor joists are closely spaced parallel beams supporting flooring directly attached to their top edges. Floor joists may also be required to support ceiling linings (either directly or via ceiling battens) and interior walls running either parallel or perpendicular to the joist direction.

Initially the floor joist tables (Tables 7.1 and 7.2) were prepared by extrapolation on the basis of stiffness from the existing tables in Chapter 6.1 of NZS 1900. Plots were made of [(spacing) $\times$ (span)^3 / (E.I)] versus span, and the trends of relative deflection versus span were used in the extrapolation. The current 450 mm spacing is an approximate equivalent of the original 18 inch spacing.

Cantilever floor joists were introduced in the 1978 version of the standard. Two alternative configurations were considered as shown in Figure 4 below



Fi Cantilever joists supporting wall and roof



loor jois Cantilever joists supporting external deck

Table 7.2 of NZS 3604 is applicable to both.

4.2.2 Design for strength (ULS)

General

Design for safety considered bending and shear strength.

Loads

The loads used in the derivation of the tables were:

	Internal floors	External decks and balconies	Superstructure
Dead load	0.40 kPa	Decking: 0.40 kPa * Balustrade: 0.54 kPa +	Light roof & ceiling: 0.45 kPa (0.2 kPa with wind load) Heavy roof & ceiling: 0.85 kPa (0.4 kPa with wind load) Wall: 0.8 kPa (0.2 kPa with wind load)
Live load	1.5 kPa (Section 7) 3.0 kPa (Section 14)	2.0 kPa	0.25 kPa (on roof)

Notes: * This loading is based on the mass limit of 25 kg/m^2 stated in Clause 7.5.1 of NZS 3604. Timber decking or plywood with a fibre-cement soffit comes within this limit, but tiles on a fibre-cement substrate would generally be heavier and therefore would require specific design.

+ This is based on a mass limit of 5.5 kg/m^2 as stated in Clause 7.5.1. Few realistic balcony constructions complying with NZBC Section F4 come within this limit.

Load cases considered:

Case	Simply supported joists	Cantilevered joists

1	1.4G	1.4G
2	1.2G + 1.6Q	1.2G + 1.6Q
3	-	$1.2G + Q_u + W_u$ (wind acting down, on roof)
4	-	$0.9G \pm W_u$ (wind on underside of deck)

The live load combination factor (ψ_u) for case 3 was taken as 0.0.

Structural models used for strength

The structural models used were a simply supported, uniformly loaded beam for the internal joists, and a cantilever with a uniformly distributed load and a concentrated load at the free end, as shown in Figure 5 below Loads on the back span were taken as zero for ULS considerations.

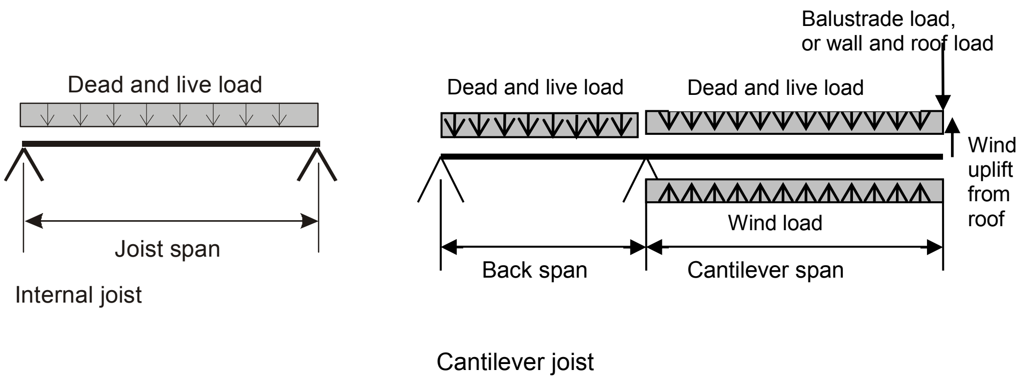


Figure 5. Structural models used for floor joists

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0
4	1.0

Stability factor, k_8, was taken as 1.0. For the deeper joists (290 x 45) at maximum span this is slightly unconservative based on the stability calculations using the formulas in NZS 3603.

Characteristic stresses

Dry stresses from NZS 3603 were used for 1.5 kPa and 3.0 kPa floor loads. However, joists used with 2.0 kPa loads are likely to be used for external decks and will be subject to weather in-service. So the selection tables for 2.0 kPa floor loads were derived using the "green" stresses from NZS 3603. Thus for Table 7.2 of NZS 3604, the spans in the right hand column (2 kPa floor loads) were derived using "green" stresses, and all the other spans were based on "dry" stresses.

Stresses as used are summarised in Table 4 below. Note that both MSG8 and MSG10 were considered to revert to G8 stresses once wet in-service.

Table 4. Timber strength properties used for floor joists.

Grade	Loading	Bending strength, f_b (MPa)	Compression strength, f_c (MPa)	Tension strength, f_t (MPa)
MSG10, VSG10	1.5 and 3.0 kPa	20.0	20.0	8.0
	2.0 kPa	11.7	12.0	4.0
MSG8, VSG8	1.5 and 3.0 kPa	14.0	18.0	6.0
	2.0 kPa	11.7	12.0	4.0
MSG6, No 1 framing	1.5 and 3.0 kPa	10.0	15.0	4.0
G8	2.0 kPa	11.7	12.0	4.0
No 1 framing	2.0 kPa	7.5	11.0	3.0

Lateral support of floor joists (Clause 7.1.2)

Lateral support of floor joists is required in two situations:

1. To transfer lateral loads from the floor acting as a diaphragm, into bracing elements below the floor, thus preventing joist roll-over (Clause 7.1.2.1). If the floor is specifically designated as a diaphragm, this requirement is more onerous.

2. To provide lateral stability to the floor joists acting as beams (Clause 7.1.2.3). The provisions are a simplified version of those contained in NZS 1900: Chapter 9.1 (SNZ 1964), which was current at the time that NZS 3604 was first written. For beams continuously restrained by flooring, this requirement is merely to keep slender floor joists stable during construction.

Floor joists supporting walls (Clause 7.1.3)

- *Parallel*

  The use of pairs of floor joists under loadbearing walls codifies industry standard practice (Clause 7.1.3.1). Under some loading situations, this can be shown by calculation to be inadequate. However, the stiffening effect of the wall is ignored in such simple calculations, and the lack of reported problems indicates that the provision is adequate. Large concentrated loads, for example from studs trimming large openings in the wall above, are not provided for and need to be specifically designed, although this is not actually stipulated in NZS 3604.

- *Right angles*

  The basis of the 200 mm positional restriction for bearing walls at right angles to the joists (Clause 7.1.3.2) is not known. It is expected to be very conservative and restrictive for joists over 200 x 50 in size. However, there is no such restriction for bracing walls, which seems to be a deficiency. For example, a bracing wall located in the centre of a 5 m span of floor joists will behave very differently to the wall as tested. This is currently the subject of a BRANZ research project.

4.2.3 Design for serviceability (SLS)

General

The joist selection tables are based on static deflection and vibration considerations.

Loads

Loads are the same as used for strength design.

Short and long-term load factors:

$\psi_s = 0.7$,

$\psi_l = 0.4$.

Structural models used for serviceability

The structural models used were the same as for strength, and described above. A load of G + 0.4 x Q was used on the back spans for the cantilever joists. This has the effect of reducing the free end deflection, thus allowing slightly longer spans. The back span joists were assumed to be the same timber section as the cantilevered portion.

Static deflection calculation

Floor joists were considered to be constrained to similar deflections, as noted in commentary Clause C2.4.2.2 of NZS 3603, thus allowing the "average" modulus of elasticity E to be used, rather than the lower bound value E_{lb} (see Table 5 below).

The duration of load factor, k_2, was taken as 2.0 for all floor joists except for cantilevered decks which were considered to be wet in-service, and for which k_2 = 3.0 was used.

Static deflection limits were span/300 for simply supported floor joists, and cantilever length/200 for cantilevered floor joists. An absolute value was not used.

Table 5. Elastic properties of floor joists.

Grade	Loading	Modulus of elasticity, E (GPa)
MSG10, VSG10	1.5 and 3.0 kPa	10.0
	2.0 kPa	6.5
MSG8, VSG8	1.5 and 3.0 kPa	8.0
	2.0 kPa	6.5
MSG6, No 1 framing	1.5 and 3.0 kPa	6.0
G8	2.0 kPa	6.5
No 1 framing	2.0 kPa	4.8

The 10% increase in allowable spans for joists continuous over two spans recognises the reduction in static deflection for beams with continuity. This increase was removed in the 1990 revision because it was not applicable to vibration actions. However, it was reinstated for the 1999 revision because it was recognised that very few floor vibration problems had been reported as being attributable to excessive spans of floor joists, and because only a few of the span options were governed by vibration.

Vibration considerations

Vibration criteria were introduced with the 1990 version of the NZS 3604 as a result of submissions by Forest Research (Bier 1989), which concluded with a set of amended spans. These were adopted for joist spacings of 600 mm, and sizes 150 x 50 and smaller.

Forest Research analysed a simply supported, uniformly loaded timber joist floor system with a width of 4.8 m. Formulae for estimating natural frequency and RMS acceleration were based on equations from Chui and Smith (1987). Criteria chosen were that natural frequency should be above 12 Hz and RMS acceleration should be less than 0.45 m/s^2.

This work was based on No 1 framing timber, so to adjust the joist spans for the other timber grades for Amendment 2, equation 2 of Chui and Smith was used to keep acceleration A_r constant:

$$A_r \alpha \frac{1}{m_1 \cdot f_0^2} \tag{4.1}$$

where

$$m \, \alpha \, span \tag{4.2}$$

and

$$f_0 \alpha \frac{\sqrt{E}}{span^2} \tag{4.3}$$

So to keep A_r constant, for two similar floors, differing only in span and E:

$$A_r = \frac{1}{m_1 \cdot f_1^2} = \frac{1}{m_2 \cdot f_2^2} \tag{4.2}$$

Giving:

$$\frac{span_1}{span_2} = \sqrt[3]{\frac{E_1}{E_2}} \tag{4.3}$$

This expression was then used to adjust all spans where vibration was the limiting case.

4.3 Structural floor diaphragms (Clause 7.3)

Where bracing lines or walls below the floor are spaced at greater than 6 m, the floor is specifically designated a structural floor diaphragm, and must comply with the details of Clause 7.3. This situation may occur, for example, where an upper floor spans a large open space or where a ground floor is supported on unbraced piles but surrounded by perimeter foundation walls.

The additional requirements are fairly modest, and in practical terms merely restrict the diaphragm span and aspect ratio. Also there are slightly more onerous requirements for lateral support of the floor joists, and bracing around the perimeter in the storey below.

However, there are no specific provisions to resist the diaphragm chord forces. Joints between joists, as provided for in Clause 7.1.1.7, have a capacity of about 6 kN, but where the perimeter consists of blocking between joists the capacity of the chord element would be very much lower (for example, the circled joint in Figure 6 below reproduced from Figure 7.9 of NZS 3604), Alternative load paths utilising other structural members (for example, wall plates) are tenuous.

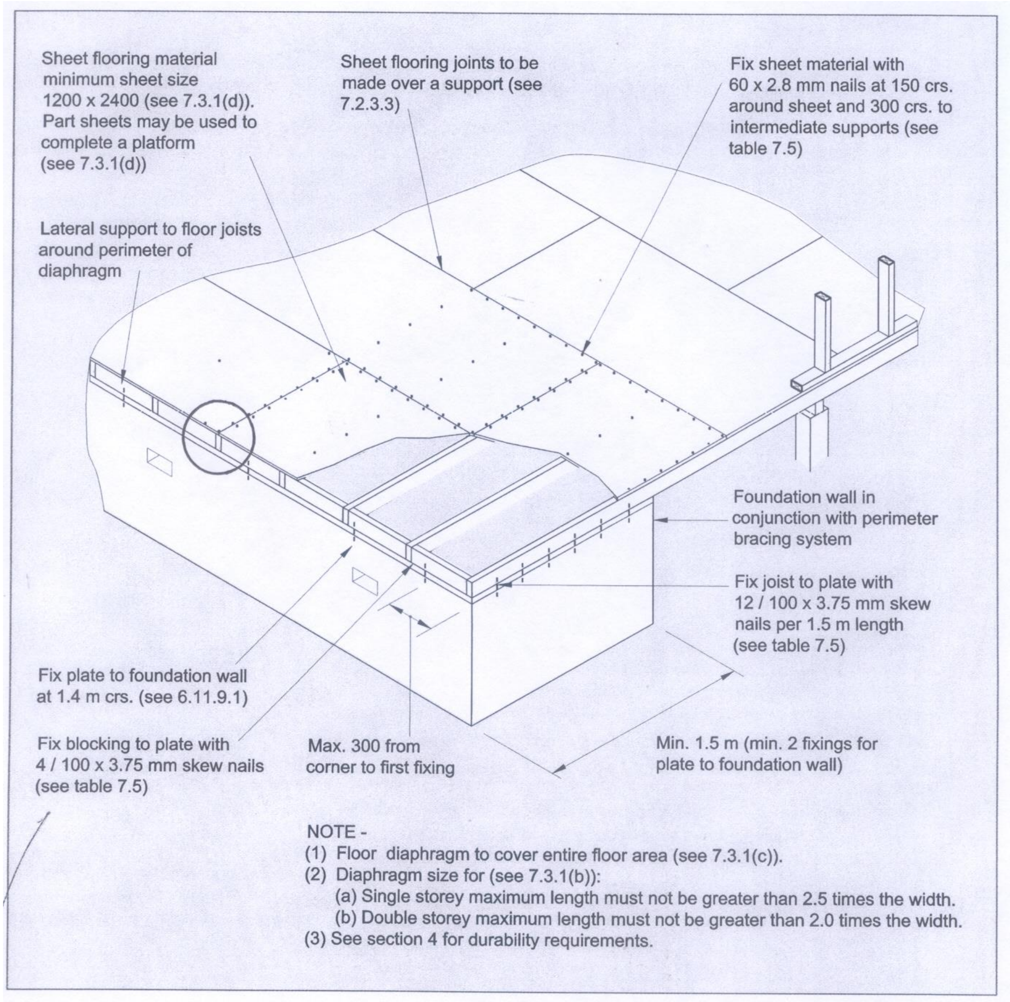


Figure 6. Continuity of diaphragm chords (reproduced from NZS 3604).

4.4 Concrete slab on ground floors (Clause 7.5)

4.4.1 General

The provisions for concrete slab-on-ground floors contained in Appendix E of NZS 3604:1978 were based on a bulletin published by the NZ Portland Cement Association (NZPCA 1975). The details were a mixture of standard practice and common sense, and the scope and contents of Appendix E remained largely unchanged until 1990.

Details of the slab thickening under loadbearing internal walls were taken from NZPCA 1975 for wall loads of up to 2,200 kg/m. The basis is not known, but probably is based on a "rule-of-thumb".

For the 1999 revision of NZS 3604, the provisions for concrete floors were incorporated into the floor Section 7, and two additional matters were included, shrinkage control joints and fixing of timber framing.

4.4.2 Shrinkage control joints (Clause 7.5.8.6)

Whilst shrinkage control joints were required in unreinforced slabs in the 1990 revision, these are now also required in mesh reinforced slabs. Control joint spacing is required at 3 m centres for unreinforced slabs, 4 m for slabs incorporating polypropylene fibre, and 6 m for reinforced slabs. This is consistent with the allowance for shrinkage control in industrial floor slab design, and the universal use of control joints minimises the risk of random cracking from drying shrinkage of concrete slabs. The importance of avoiding random cracking in a residential situation is more critical when the slab is to be exposed, or covered with vinyl or ceramic tiles.

In a typical residential floor slab there is a high degree of restraint to shrinkage imposed by the perimeter concrete footings. The resulting strain is relieved by cracking induced at the control joints. The use of steel reinforcement will help to limit the crack widths at the joints, and distribute the cracking evenly between joints. For unreinforced slabs or slabs with polypropylene fibre, individual joint widths up to 5 mm are probable. Slabs with polypropylene fibre are essentially unreinforced. However, the wider control joint spacing allowed recognises the role of the fibre in reducing plastic cracking, and hence the risk of random shrinkage cracking.

In the 1990 revision of NZS 3604, slabs cast in one operation up to 25 m were allowed. Slabs longer than 24 m are now allowed, provided they are separated by free joints. A free joint is defined as "*a construction joint where no reinforcement passes through the joint linking both sides of the concrete slab and the vertical faces of the joint are not in bonded contact with each other".*

4.4.3 Fixing of timber to concrete floor slabs (Clause 7.5.12)

Details for fixing framing timber to concrete floor slabs include M12 bolts, R10 dowels, and shot fired fasteners (for internal walls only). These methods codify building practice which has been in common use in New Zealand for a long time, and are presumably satisfactory.

At the request of several commentators, the 1999 revision included target strengths for proprietary fasteners, as these were generally preferred by the industry to the generic types already provided for. To establish a benchmark strength for a minimum level of general robustness, BRANZ *Evaluation Method EM1* was used assess the capacity of the bent dowel, as provided for in NZS 3604 Clause 2.4.7.

A series of tests was undertaken on dowels installed as described in NZS 3604, and used to fasten a timber plate to concrete. They were tested under earthquake loading in pull-out and shear, both parallel and perpendicular to the grain of the plate. The characteristic residual strengths derived in accordance with EM1 were:

Shear in-plane	4.4 kN
Shear out-of-plane	3.4 kN
Tension	2.1 kN

The committee rounded the values to give the numbers in Clause 7.5.12, in particular matching the tension demand so as to satisfy the lintel hold-down provisions of 7.5 kN in Clause 8.6.1.8.

Bracing applications are specifically excluded from these values because the demand on the anchor in that application can only be quantified by specific testing on the proprietary bracing element being evaluated. Thus no target values were provided for bracing applications. There is still a difficulty in that bracing tests use a through bolt to firmly hold down the bottom plate, and there is no recognised procedure to determine what is a substitute post fixed anchor.

5. SECTION 8. WALLS

Walls are required to resist either vertical or horizontal loads or both.

Walls resisting vertical gravity loads are defined as “loadbearing walls”, and all others are therefore deemed to be “non-loadbearing walls”, even though they may be required to resist racking loads and/or face loads. This nomenclature follows Australian practice.

Walls which are specifically designed to contribute resistance to lateral racking loads are defined as “wall bracing elements” in Clause 1.3.

All walls will be required to resist face loads at some time in their design life. External walls will be subjected to wind pressure loading, and most internal walls will also be subjected to some differential wind pressures if door and window openings are unfavourably disposed. In addition, all walls require a certain level of robustness to withstand face loads from shelving and attached items, human impact, doors closing, and so on. Also, under earthquake actions, all walls are required to resist face loads caused by the inertial effects of their own self-weight and anything attached to them (for example, masonry veneer).

These loads are all provided for in the selection tables and detailed provisions for individual wall members.

Generic details for wall bracing elements, and their associated ratings, were initially included in the wall section of NZS 3604 (SNZ 1978). However, the rationale for the ratings given was not technically robust and it was recognised that testing of specific, proprietary bracing systems produced more reliable ratings. Accordingly, as proprietors of bracing systems gained experience with testing, the details and ratings of generic bracing elements were moved to an appendix in the 1990 version of the standard, and were dropped altogether in the 1999 version, as by then there were an adequate number of tested systems readily available on the market. In the rare situation where a proprietary bracing system may not be available or desired, the generic details are still available from old versions of the standard, although this will be a conservative option.

5.1 Timber properties

Dry timber properties from NZS 3603 were used for all wall applications (strength and serviceability), and are summarised in Table 6 below.

Table 6. Timber properties used for walls.

Grade	Bending strength, f_b (MPa)	Compression strength, f_c (MPa)	Modulus of elasticity, E (GPa)	Lower bound MOE, E_{lb} (GPa)
MSG10, VSG10	20.0	20.0	10.0	6.7
MSG8, VSG8	14.0	18.0	8.0	5.4
MSG6, No1 framing	10.0	15.0	6.0	4.0

Shear strength was taken as 3.8 MPa, and bearing strength perpendicular to grain as 8.9 MPa.

5.2 Studs (Clause 8.5)

5.2.1 General description

Studs are the vertical members of wall framing, running between top and bottom plates, and supporting the cladding or lining. Where studs are less than full wall height (for example, beneath sill trimmers) they are called jack studs (first meaning of the definition in Clause 1.3). Studs each side of an opening supporting a lintel or sill trimmer are referred to as trimmer studs, and are of greater thickness to provide enhanced strength and stiffness. Additional studs provided for the purpose of supporting the ends of a lintel are called doubling studs (more simply called prop studs in Australia), and are considered to contribute to the strength or stiffness of the trimmer stud if they extend up to within 400 mm of the full wall height (Clause 8.5.2.4).

Studs are required to carry vertical gravity loads from the supported roof and floor(s), and also to transfer wall face loading to the top and bottom plates. Under wind loading there may also be a need to transfer uplift loads from the roof to the foundations, although this was not considered a critical load case in the calculations for NZS 3604. Thus studs were designed for axial compression and bending. Trimmer studs at the sides of openings are also required to resist the concentrated bending loads resulting from the reactions at the ends of lintels and sill trimmers under face loading.

5.2.2 Design for safety (ULS)

General

Wall stud tables were originally prepared in 1976 using a Fortran computer program and working stress design methods. Tables 8.2, 8.3, 8.4 in NZS 3604:1999 are based on those tables. For Amendment 2, the stud tables were re-calculated on a spreadsheet using a macro to carry out an iterative trial and error process.

The tables were originally developed on the basis of providing the most economical timber size that met both the strength and serviceability criteria for the specific loading conditions established by the table. Both interior and exterior studs were considered.

Loads

Gravity (axial loading on studs)

Loads were calculated per metre of wall length and then multiplied by the required stud spacing to arrive at load per stud, assuming an even distribution of loading across all studs in the wall.

The floor dead and live loads were based on contributory loading from an assumed joist span of 5.2 m, the longest span in the 2 kPa joist tables, and an eaves overhang of 600 mm was used in the calculation for roof loads.

Component		Dead load (G)	Live load (Q)	Snow load (S)
Suspended floor		1.1 kN/m of wall	2.0 kPa – Section 8 3.0 kPa – Section 14	–
Upper wall		1.44 kN/m of wall	–	–
Wall under consideration		0.8 kPa	–	–
Roof (including framing)	Light	0.4 kPa	0.25 kPa	0.5 and 1.0 kPa (Section 15)
	Heavy	0.85 kPa (incl. ceiling)		

Wind (face loading on studs)

Wind zone	**Wind speed V_{dz} (m/s)**	**q_z (kPa)**
Low	32	0.61
Medium	37	0.82
High	44	1.16
Very High	50	1.50

The differential pressure coefficients across external walls, $\sum(C_{pe,}\ C_{pi})$, was taken as 1.1. A face load of 0.61 kPa was also considered appropriate for internal wall studs, to give a minimum level of general robustness against uneven wind loads and domestic scale impacts. Wind loading on the roof was not considered in conjunction with face loading on the wall, so did not contribute to axial loading on the studs.

Load cases considered were:

1	1.4G
2	1.2G + 1.6Q
3	$1.2G + Q_u + W_u$

Snow loading was initially allowed for in the stud tables, although it was removed in the 1999 revision of the standard as the effect of the additional axial loading was negligible.

The live load combination factor, ψ_u, for load case 3 was taken as 0.0 for roofs and 0.7 for floor loads in combinations with wind face loading on the studs.

Structural model used for strength

The models used for strength are shown in Figure 7 below.

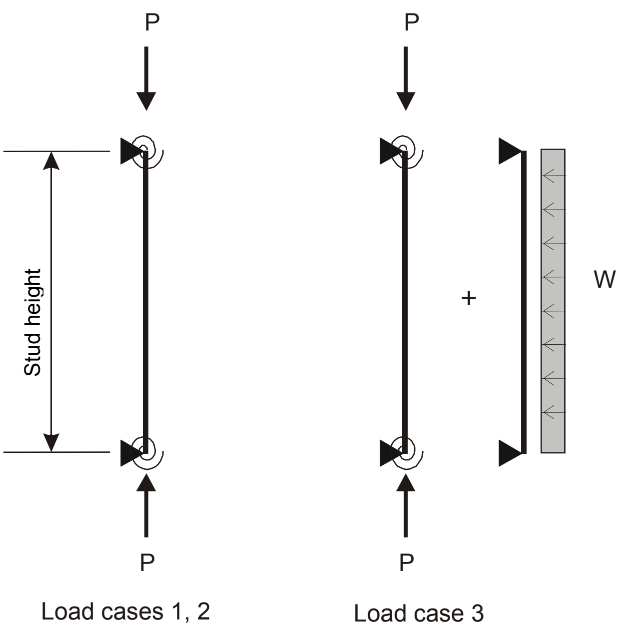


Figure 7. Structural models used for strength design of studs.

The model is a beam column with partial end fixity due to square ends, end fixings and a contribution from the wall linings. Partial fixity was accounted for by using an effective length factor (k_{10} from NZS 3603) of 0.75 x length for axial compressive loads. For uniformly distributed bending caused by wind face loading, a simply supported member was assumed.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0

No restraint was assumed for out-of-plane action. Restraints at 600 mm were assumed for in-plane action. This restraint is provided by either dwangs or lining fixings.

The parallel support system factor for load sharing in bending (k_4) was taken as 1.1, recognising the load sharing provided by dwangs, lining and cladding.

Under dead load only and dead plus live load cases, the bearing strength perpendicular to the grain on the bottom plate was also considered. This may be critical for short studs.

Bearing area factor, k_3, was taken as 1.0.

5.2.3 Design for serviceability (SLS)

General

The deflection of the stud under wind face loading was the critical load case for serviceability.

Loads

Wind (face loading on studs)

Zone	Wind speed (m/s)	Q (kPa)
Low	26	0.40
Medium	30	0.53
High	37	0.82
Very High	42	1.06

The differential pressure coefficients across external walls, $\sum(C_{pe,}\ C_{pi})$, was taken as 1.1. A face load of 0.40 kPa was also considered appropriate for internal wall studs, to give a minimum level of general robustness against uneven wind loads and domestic scale impacts.

Live and earthquake loads were not considered for serviceability.

Structural model used for serviceability

The model used for serviceability is shown in Figure 8 below.

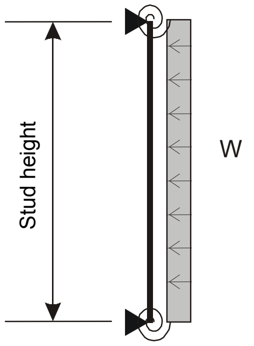


Figure 8. Structural model used for serviceability of studs.

The systems effect of the linings and claddings resulting in an increased stiffness of the studs in the wall was allowed for by effectively increasing the stud E value by a factor of 1.69. This factor was derived from full size face loading tests conducted on lined and clad 2.4 m high walls in the 1970s, and has been applied to the E_{lb} value from NZS 3603. An MSG8/VSG8 stud thus has an effective E for serviceability of 9.13 GPa.

Deflection criteria

Maximum deflection = stud height/180, with an upper limit of 15 mm.

5.3 Stud spacing adjustment factor (Clause 8.5.5)

5.3.1 General description

The stud spacing adjustment factor allows studs of the required cross-section to be substituted for by studs of a smaller cross-section spaced more closely together. This is particularly relevant to the situation of studs in raking walls, which are desirably all the same cross-section width, although they are different heights.

The basis of the recommendations is that:

1. The stud size is determined by bending stiffness requirements only.
2. The reduced stiffness of a smaller stud size may be compensated for by placing these studs closer together in proportion to the ratio of the smaller cross-section moment of inertia to the larger (using equation 5.3).

To ensure that the first condition is met, only studs 3 m and higher may be substituted for, as the sizes of these studs are determined by bending stiffness considerations. However, this condition was omitted in the published standard.

Hence, the following formula applies.

$$\frac{\text{Spacing of smaller stud}}{\text{Spacing of original larger stud}} = \frac{\text{MoI smaller stud}}{\text{MoI original stud}} \qquad (5.1)$$

or

$$\frac{S_2}{S_1} = \frac{I_2}{I_1} \qquad (5.2)$$

where S = stud spacing.

Whence

$$S_2 = S_1 \times (I_2/I_1) \qquad (5.3)$$

Thus the stud spacing factor = (I_2/I_1).

Example: Table 8.1 requires a 150 x 50 stud at 600 centres. What spacing must be used with a 100 x 75 stud?

From the "Original larger stud size" row labelled 150 x 50, run along to the "Desired smaller stud size" column headed 100 x 75. The spacing adjustment factor is 0.38. Hence the maximum spacing of the 100 x 75 stud is 0.38 x 600 = 230 mm.

Alternatively, a 100 x 100 stud may be used at 0.53 x 600 = 320 mm spacing.

5.4 Lintels (Clause 8.6)

5.4.1 General description

Lintels are horizontal framing members spanning across openings in loadbearing walls. They are supported by either a doubling stud or a check out in the trimmer stud. Selection tables are provided for solid timber lintels only in Section 8 of the Standard. Composite lintels are covered in Section 16, and for proprietary lintels the manufacturer's literature has to be consulted.

Lintel loads were assumed to be uniformly distributed along the lintel length. Thus any lintel supporting a concentrated load, such as from a girder truss, or a trimmer stud within an upper wall, falls outside the scope of the standard and must be specifically designed.

The concept of "loaded dimension" was introduced with the 1999 revision to quantify the weight of construction supported by the lintel. It is similar in concept to "tributary area", used by structural engineers in design for many years.

5.4.2 Design for safety (ULS)

General

The lintel span tables were derived using dead loads and wind pressure coefficients for roof pitches up to 45^0 only. Above this, the lintel span multipliers (Table 8.7) must be used. Design for safety included consideration of the ULS in bending, shear, bearing, and the ultimate capacity of connections.

Loads

Gravity loads

Component (light, medium, heavy cladding defined in cl 1.3)		Dead load (G) (kPa)	Live load (Q) (kPa)	Snow load (S) (kPa)
Roof (including framing and ceiling)	Light	0.45 (0.2 under wind uplift)	0.25	0 (Section 8) 0.5 & 1.0 (Section 15)
	Heavy	0.85 (0.4 under wind uplift)		
Wall	Light	0.2 (0.1 under wind uplift)	-	-
	Medium	0.8 (0.2 under wind uplift)	-	-
Floor		0.4	2.0 (Section 8) 3.0 (Section 14)	-

Eaves overhang was taken as 600 mm, wall height as 2.4 m, and floor span as 5.75 m.

Wind loads

Only the Very High wind zone was considered, with q_z = 1.5 kPa.

External pressure coefficients C_{pe}:

Roof pitch	Upwind slope	Downwind slope
15^0	- 0.7	- 0.5
30^0	- 0.3	- 0.6
45^0	+ 0.6	- 0.6

Internal pressure coefficient: (C_{pi}) = + 0.3, 0, -0.3.

Modifying factors: (K_a, K_l, K_p) = 1.0

Roof slopes of 15^0, 30^0, 45^0 were considered with the above coefficients and factors, giving a maximum uplift of:

0.946 x q_z x Loaded Dimension

And maximum down load of:

0.3 x q_z x Loaded Dimension.

Load cases considered were:

1	1.4G
2	1.2G + 1.6Q
3	1.2G + Q_u + W_u
4	0.9G - W_u
5	1.2G + Q_u + 1.2S_u

Snow loading (load case 5) was removed to Section 15 with the 1999 revision of the standard, so the lintel tables in Section 8 are for zero snow load.

The load combination factor for ULS, ψ_u, was taken as 0.0 for roof and 0.4 for floors.

Structural model used for strength

The model used for strength is shown in Figure 9 below.

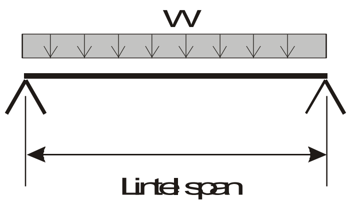


Figure 9. Structural model used for strength of lintels

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0
4	1.0
5	0.8

Restraint to the top of the members was assumed to be provided by the wall framing at 600 mm centres, and full rotational restraint at the ends provided by the trimmer studs. Thus k_8 was taken as 1.0.

Load sharing by other members such as roof and wall framing was not considered, so k_4 = 1.0.

Dry stresses from NZS 3603 were used for all applications, as set out in Table 6.

The bearing area factor, k_3, was taken as 1.36 for a single supporting stud and 1.1 for a doubled stud.

Connection capacity

Provisions for securing of lintels against uplift were provided for two levels of connection capacity:

1. Standard fixings for lintel to trimmer stud as provided in the nail schedule in Table 8.19 of NZS 3604. Load transfer is provided either by the check into the trimmer stud or by the prescribed end nailing to the lintel through the trimmer stud. There is no specific attachment between trimmer stud and floor – a load path through lining or cladding was assumed, with some acknowledgment of the spreading of the concentrated load through the wall and down to the floor. The capacity of this system was assessed at 2 kN.

2. A more robust nail strap connection between lintel and trimmer stud, and between bottom of trimmer stud and floor. The capacity of this system was initially

assessed at 5 kN in WSD values, but this was increased to 7.5 kN for the 1999 revision (at ULS).

Little experimental confirmation of the performance of these connections has been undertaken and the capacities quoted are a mixture of calculation from NZS 3603, and "engineering judgement".

5.4.3 Design for serviceability (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

Gravity and wind loads are as given above for strength design.

The following load cases were considered:

1	G
2	$G + Q_s + W_s$
3	$G + Q_s + S_s$

The short and long-term load factors were:

Load	Short-term factor (ψ_s)	Long-term factor (ψ_l)
Live (floor)	0.7	0.4
Live (roof)	0.7	0.0
Snow	0.5	0.0

Structural model used for serviceability

Models used for serviceability were as for strength, Figure 9. Load was applied to the member as a uniformly distributed load. No allowance was made for load sharing.

Deflection calculation

Lintel deflections were calculated using the lower bound modulus of elasticity, E_{lb}, from NZS 3603, and are set out in Table 6. Only timber in the dry condition was considered.

Deflection criteria

A deflection limit of span/300 was used, with an overall limit of 25 mm.

5.5 Steep roof multipliers for lintels (Table 8.7)

5.5.1 General description

The lintel tables were prepared on the basis of loads calculated for roof pitches up to 45^0. However, the scope of the standard includes roof pitches up to 60^0. The steep roof multipliers of Clause 8.6.1.3 provide for adjustments to the loaded dimension to allow for the additional loads on lintels caused by wind loads acting on roof pitches between 45^0 and 60^0.

5.5.2 Design for safety (ULS)

General

Roof trusses within the scope of NZS 3604 are supported at two points on the same level. Because the lintel loaded dimension is measured horizontally between these points, an increase of roof pitch (for constant loaded dimension) has two effects on the load applied to the lintel:

- dead load is increased because the rafter length increases with increasing pitch
- the wind load increases the overturning moment on truss as a whole, which in turn increases the load on the lintel.

The second effect is illustrated in the diagram of Figure 10 below

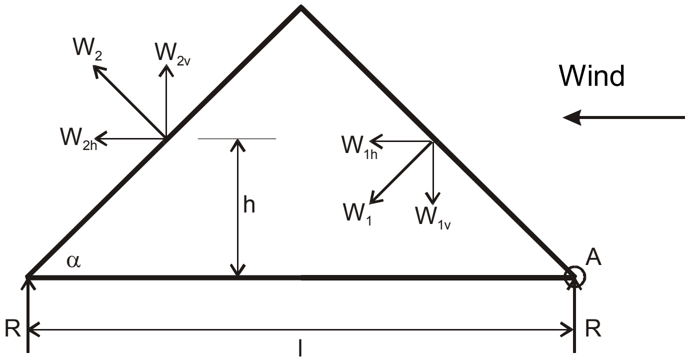


Figure 10. Overturning forces on a steep roof.

Equating moments about A:

$$(W_{1h} + W_{2h}) \cdot h = R \cdot l \qquad (5.1)$$

where:

$$h = \frac{l}{4} \cdot \tan\alpha \qquad (5.2)$$

Total load on the lintel is given by the following equation:

$$R = \frac{G}{\cos\alpha} \times LD + Q \times LD + (W_1 + 3W_2) \times \frac{LD}{4} + (W_1 + W_2) \times \tan^2\alpha \times \frac{LD}{4} \qquad (5.3)$$

The steep roof multipliers were calculated using this equation and the following parameters:

Loads

Construction	Load (kPa)
Light roof (including framing and ceiling)	0.46
Heavy roof (including framing and ceiling)	0.84
Roof live load	0.25
Design wind pressure (VH zone – 50 m/s)	1.50

External pressure coefficients:

Roof pitch	Upwind slope	Downwind slope
45^0	+ 0.50	- 0.60
50^0	+ 0.50	- 0.60
55^0	+ 0.55	- 0.60
60^0	+ 0.60	- 0.60

Internal pressure coefficient on the ceiling was taken as 0.30.

5.6 Plate (Clause 8.7)

5.6.1 General description

Plates are horizontal or raking members forming the top and bottom boundary members for walls. They support and distribute loads from walls, floors, roofs and ceilings, particularly when roof and floor members do not align vertically with the studs.

Top and bottom plates are primarily designed to transfer and distribute **vertical** loads between rafters or joists and the supporting studs. Although linings or claddings are usually directly fixed to plates and can transfer loads directly to or from them, the contribution to plate stiffness and strength from these supporting components was not taken into account. There are no provisions for the support of trimmer joists or roof girder trusses, which may introduce high concentrated loads. This issue is briefly referred to in the commentary to Clause 10.2.2.2, but should be included in the plate provisions as well. In practice the problem is usually solved by adding an additional stud at that location for direct load support to floor level.

Plates are also required to transfer **horizontal** loads between the wall studs and roof or floor framing members. Face loads on the wall originating from wind pressure and earthquake inertial forces are applied to top and bottom plates as a series of reactions from the ends of the studs. It was assumed that these loads are transferred to the orthogonal bracing walls by ceiling linings acting as diaphragms, or by framing members spaced at 2.5 m maximum (Clause 8.7.4.1), although it is not specifically stated in NZS 3604 that these members themselves are attached to linings. Where low density ceiling linings (less than 600 kg/m^3) are used such that diaphragm action is doubtful, the top plate must be reinforced (Clause 8.7.4.2) where bracing walls are spaced at between 5 and 6 m.

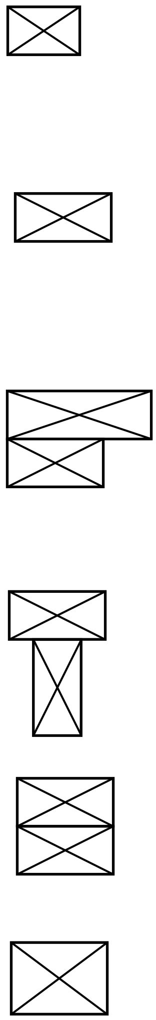
5.6.2 Design for safety (ULS)

General

Under vertical loading, the general approach was to calculate the flexural and shear capacities of the various top plate configurations. The next step was to derive the concentrated load to reach that capacity for each stud spacing and structural model, and then for each wall application (top or lower storey), the matching loaded dimension was determined.

Member capacity

Member capacities in bending and shear were calculated for the plate configurations shown in Figure 11 below.



70 x 45

90 x 45

90 x 45 plus 140 x 35

90 x 45 plate plus 90 x 45 dwang (only used for top storey walls and allows for post-installation where rafter does not land within 150 mm of stud)

2/90 x 45

90 x 70

Figure 11. Configurations of top plates.

Strength reduction factor, ϕ, = 0.8.

Duration of load factors:

Load type	**k_1**
Dead	0.6
Dead plus live	0.8
Wind combinations	1.0

Characteristic stresses

Dry stresses from NZS 3603 were used for all applications, and are summarised in Table 6.

Shear strength, f_s, was taken as 3.8 MPa for all timber grades as per NZS 3603. According to Keenan (Keenan 1974), for small shear spans a size factor of 2.0 may be applied. This was used in earlier versions of the standard and was retained for Amendment 2. However, in recognition that the same shear strength of 3.8 MPa was used for Douglas Fir as well as Radiata pine, this was reduced to 1.5.

The resulting shear capacity was calculated from:

$$\phi V_n = 0.8 \times k_1 \times k_4 \times (1.5 \times f_s) \times (2/3bd).$$

Bending strength was checked using a flexural enhancement factor of 1.25 to allow for the short spans.

Capacities of the combined top plate member, in the case of double member configurations, were obtained by summing the capacities of the individual sticks so that no composite action was assumed. However, to allow for the lower timber variability in doubled members, the parallel support factor, k_4, from NZS 3603 was taken as 1.14.

Compression perpendicular to the grain was not considered.

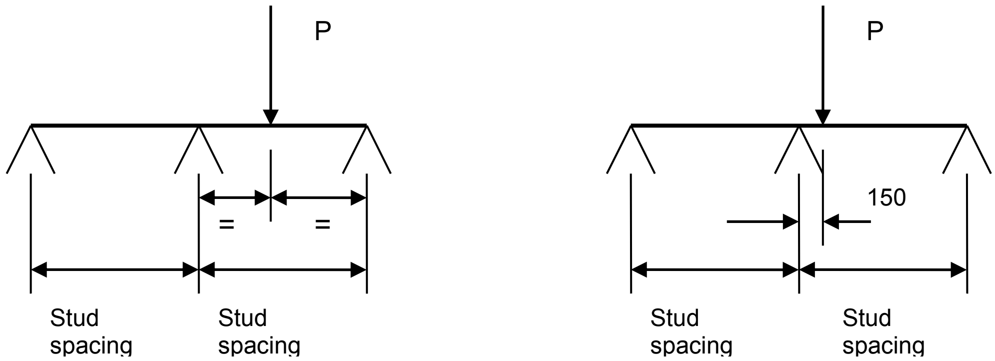
Structural model used for strength

The structural model used for the strength check was a two span continuous beam with a concentrated load in two alternative positions within one span as shown in Figure 12 below. Bending moments, shears and deflections were calculated by elastic analysis for each stud spacing option, resulting in the following formulae:

M_{max} = 0.0203 P x *stud spacing*

V_{max} = 0.844 P

δ_{max} = 0.015 P x *stud spacing*^{3}/EI + 4.5 P x *stud spacing*/EA.



Load at midspan

models used for plates.

Load within 150 mm of support

Loads

The concentrated load, P, was derived from roof rafter loads or floor joist loads for top plates, and stud loads for bottom plates, each calculated for an appropriate spacing. No load sharing between rafters or floor joists was assumed.

The values of gravity loads used are given in Table 7 below.

Table 7. Dead and live loads used for plates.

Item	Loads	
	Dead	**Live**
Light roof & ceiling	0.45 kPa (along slope) (0.2 kPa with wind uplift)	0.25 kPa
Heavy roof & ceiling	0.85 kPa (0.4 kPa with wind uplift)	0.25 kPa
Wall	0.4 kPa (0.2 kPa with wind uplift)	–
Floor	0.4 kPa	2.0 kPa (Section 8) 3.0 kPa (Section 14)

Wall height used to calculate wall loads was 2.4 m, and an eaves overhang of 600 mm was assumed.

Wind loads were based on a design wind pressure, q_z, of 1.5 kPa for the Very High wind zone, and pressure coefficients used were:

Net $C_{pi} + C_{pe}$ = -0.946 under uplift, and + 0.3 under downwards pressure

(these were derived using Figure 10, with a roof slope of 15^0 for uplift, and 45^0 for downwards pressure)

k_a, k_l, k_p = 1.0

Load cases considered:

Load case	Combination
1	1.4G
2	1.2G + 1.6Q
3	1.2G + Q_u + W_u
4	0.9G ± W_u

The live load combination factor (ψ_u) for load case 3 was taken as 0.0 for the roof, and 0.4 for floors where appropriate.

The plate loaded dimension was then determined by subtracting the constant load components (floor, wall, eaves) from the limiting load on the plate and calculating the applicable loaded dimension using the following formulae:

Single or top storey

$$LD = \frac{P - (\text{wall weight} \times \text{height} + \text{eave load} \times \text{overhang})}{\text{roof load}}$$

Lower storey

$$LD = \frac{P - (\text{wall wt} \times \text{ht} + \text{eave load} \times \text{overhang} + \text{floor load} \times \text{LD})}{\text{roof load}}$$

where P = maximum calculated concentrated load.

5.6.3 Design for serviceability (SLS)

General

Design for serviceability included consideration of deflection only. The same approach was used as for strength design.

Loads

Dead and live loads used for the serviceability check were the same as for strength and are set out in Table 7.

Structural model used for serviceability

The structural model used for serviceability was the same as used for strength (Figure 12).

Deflection calculation

Limiting plate loads were calculated using the lower bound modulus of elasticity, E_{lb}, from NZS 3603, and are set out in Table 6. Only timber in the dry condition was considered.

A duration of load factor for serviceability k_2 = 2.0 was used for the long-term deflection calculation. Deflection was calculated (including both bending and shear effects).

Deflection criteria

The maximum allowable total mid-span deflection (including bending and shear) was limited to 5 mm under long or short-term loading.

5.6.4 Joints in plates (Clause 8.7.3)

The provisions for joints in top plates are based on good practice rather than any formal analysis. They are designed to tie the walls together and provide rational load paths to bracing elements.

Where a top plate bounds a diaphragm the provisions for joints are unconservative because the prescribed connections are not sufficient to carry the chord forces (Shelton 2004), and need consideration by future committees.

5.6.5 Lateral support of top plates (Clause 8.7.4)

Top plates basically span horizontally between orthogonal support walls. If a plate is connected to diaphragm complying with Clause 5.6, no further provisions are required. Otherwise lateral support by framing members is required at 2.5 m centres. If the adjacent ceiling is low density (softboard) a doubled top plate is required where the support walls are spaced at greater than 5 m. The situation is further confused by the lateral support provisions for heavy hip roofs (Clause 10.3.3.3) which also require connection back to a wall brace element.

These provisions are based on good practice, rather than rational engineering analysis, and require further investigation and rationalisation in the light of more modern building practices and materials.

5.7 Connection of plates to studs (Clause 8.7.6, Table 8.18)

5.7.1 General description

Provisions for fixing roof framing members to walls are covered in Section 10 Roof framing. To ensure a continuous load path down the wall, fixings for the wall top plate to supporting members such as studs and lintels are provided for in this Clause. Any contribution from the lining fixings was ignored.

The stud to plate strap fixing shown in Figure 8.12 of the standard is not referenced from the text, is in conflict with the provisions of Clauses 8.7.6 and Table 8.18 *Connection of plates to studs* and appears to be spurious. Advice from the BRANZ Helpline is that this is confusing the users of the standard and should be deleted.

5.7.2 Design for safety (ULS)

Loads

Construction	Load
Heavy roof cladding + framing	0.84 kPa
Light roof cladding + framing	0.46 kPa

Wind loads:

Wind zone	Wind speed (m/s)	q_z (kPa)
Low	26	0.4
Medium	30	0.53
High	35	0.75
Very High	40	0.98

Pressure coefficients:

$C_{pe} + C_{pi} = -1.1$,

K_a, K_l, $K_p = 1.0$.

Eaves overhang: 600 mm.

Load case considered:

0.9G & W_u

Member capacity

The capacity of a single wire dog was assessed at 2.0 kN. It was considered that the configuration of the plate and stud in the wall was sufficient to cope with any eccentricity associated with a non-symmetrical arrangement of wire dogs.

6. SECTION 9. POSTS

NZS 3604:1978 included the first provisions for securing posts against uplift due to wind forces. Prior to that, only downwards gravity loads on posts were considered.

6.1 Posts, footings and connections (Clauses 9.1, 9.2 and 9.3)

6.1.1 General description

Posts are isolated vertical members supporting a portion of a roof. They are subject to axial loads only, arising from gravity loading (compression) and wind uplift (tension).

Posts are not intended to resist lateral loads from wind or earthquake, so their use in freestanding structures such as pergolas or carports is outside the scope of the standard, as stated in Clause 1.1.2 (b).

Section 9 of the standard also includes example connections at the top and bottom of the post, and concrete post footings.

6.1.2 Design for safety (ULS)

General

Design for safety includes consideration of the actions causing instability and those resisting instability, as provided for in Clause 2.5.3.4 of NZS 4203.

There are limits on the size and length of posts to limit the slenderness ratio to 30.

Loads

i) Gravity

Cladding	Dead load (G)
Light (including framing)	0.2kPa
Heavy (including framing)	0.6 kPa

ii) Wind

Wind zone	ULS (W_u)	
	Site wind speed (V_z)	Design wind pressure (q_z)
Low	32 m/sec	0.62 kPa
Medium	37 m/sec	0.82 kPa
High	44 m/sec	1.16 kPa
Very High	50 m/sec	1.50 kPa

Design load on the post (kN) is given by:

$$W_u = q_z.p_n.A$$

where: A = tributary area, determined from Figure 9.1 of NZS 3604,

other symbols are defined by NZS 4203, Clause 5.7.

Pressure coefficients:

C_{pn} = 1.0 for light roofs,

= 0.8 for heavy roofs (where the roof slope will be greater than 12.5 degrees),

k_a, k_l = 1.0.

Load cases considered:

Load case	Combination
1	$0.9G \pm W_u$

Structural model used for strength

The model used for the strength of posts is shown in Figure 13 below

Figure 13. Structural model used for posts.

Member capacity

The capacity of the bolted joints at each end of the posts was derived by:

$$N^* = \phi.k_1.k_{12}.k_{13}.(Q_{skl} \text{ or } Q_{skp}),$$

with the factors as tabulated below.

Strength reduction factor (ϕ)	0.7
Duration of load factor (k_1)	1.0
Modification factor for green timber (k_{12})	1.0
Modification factor for multiple fasteners (k_{13})	1.0
Characteristic strength of M12 bolt in single shear (b_e = 90 mm for all options)	6.97 kN perpendicular to grain
	10.4 kN parallel to grain

For resistance to wind uplift, a concrete mass of 2,340 kg/m^3 was used (23 kN/m^3).

7. SECTION 10. ROOF FRAMING

This section contains provisions for roofs constructed of proprietary nail plate roof trusses, and for roofs framed up on-site from individual sticks of timber. The complete roof structure is required to resist vertical loads (gravity originated dead, live and snow loads) and horizontal loads (wind and earthquake). To limit loading on other components of the building (such as top plates), roof trusses are limited in dimensions to 12 m in span and 750 mm overhang at the eaves. There are additional limits on cantilevered rafters in the provisions for eaves (see Section 7.1.1 of this report).

The provisions for roofs exclude “flat roofs”, defined in Clause 1.3 as less than 10^0 slope (1 in 6). This is done to limit live loading to 0.25 kPa. Roof slopes less than this must be designed using the provisions for decks, which allow for the appropriate live load.

Clause 10.2.2 states that roof trusses are to be specifically designed (usually by the truss supplier or the proprietor of the system) in accordance with NZS 3603, and site-specific information provided by the builder. Detailed procedures for this are outlined in the commentary to that clause. In practice, the interface of responsibilities between supplier of the design software, truss designer and overall project designer/builder is fraught with difficulties. It is common for house plans to be submitted for Building Consent, and then approved by the Territorial Authority (TA), without any specific truss design or details appearing on the working drawings. During the construction process, the trussed roof is treated rather as a commodity, with choice of supplier based principally on price. Each truss package is supported by a design Producer Statement, but the specific details required for gravity support, tie-down, bracing, and how they fit into the already completed building structure are thus outside the realistic control of the TA.

Generally, trusses are assumed to be supported only on the external walls. However, where the truss designer wants to utilise an internal wall for support, additional details (including foundations) will need to be specifically designed.

Bracing provisions for horizontal loading are required to be applied to both trussed and framed roofs. This requirement has really only been made clear in the latest revision. It is likely that additional bracing will be required for the stability of the slender chord members of proprietary trusses. This is usually identified as part of the truss design package.

There are provisions for tie down/anchorage for roof members spaced at 900 and 1200 mm centres. It is assumed that these also apply to members spaced at 600 mm or other closer spacings, although presumably this would be conservative.

Wind loads

To avoid repetition in the individual member sections that follow, the wind loads that were used for the design of all roof members were based on the following:

Wind zone	**Ultimate Limit State (W_u)**		**Serviceability Limit State (W_s)**	
	Site wind speed (V_z)	**Design wind pressure (q_z)**	**Site wind speed (V_z)**	**Design wind pressure (q_z)**
Low	32 m/sec	0.62 kPa	26 m/sec	0.40 kPa
Medium	37 m/sec	0.82 kPa	30 m/sec	0.53 kPa
High	44 m/sec	1.16 kPa	35 m/sec	0.76 kPa
Very High	50 m/sec	1.50 kPa	40 m/sec	0.98 kPa

Derivation of these values is covered in the Section 3 *Bracing*. Pressure coefficients are covered in the individual roof member sections.

Timber properties

Dry characteristic stresses from NZS 3603, Amendment 4 were used for all roof members and are summarised in Table 8 below.

Table 8. Timber properties used for roof framing.

Grade	Bending strength, f_b (MPa)	Compression strength, f_c (MPa)	Tension strength, f_t (MPa)	Modulus of elasticity, E, (GPa)	Lower bound MoE, E_{lb}, (GPa)
MSG10, VSG10	20.0	20.0	8.0	10.0	6.7
MSG8, VSG8	14.0	18.0	6.0	8.0	5.4
MSG6, No 1 framing	10.0	15.0	4.0	6.0	4.0

Shear strength, f_s, was taken as 3.8 MPa for all timber grades, as per NZS 3603.

Connections of roof framing members

Nail schedules were introduced into the original standard in 1978, thus providing a standard of nailing that was considered good practice in the industry at the time. Since then nail sizes have changed and gun-driven nails introduced, along with a variety of more specialised fixings. Also most timbers today are nailed “dry” rather than “green”.

Little experimental confirmation of the basic generic nail performance has been undertaken to date, and the capacities given in this section are a mixture of calculation from NZS 3603, and “engineering judgement”. The fixing capacities quoted for alternatives to the given examples (see Table 9 below) allow suppliers and manufacturers a target to develop their own solution for many applications.

Table 9. Uplift resistance of various roof framing connections.

Fixing type	Example of fixing	Capacity □(ϕQ_n)
A	2/100 x 3.75 skewed nails	0.7 kN
B	2/100 x 3.75 skewed nails + 1 wire dog	2.7 kN
C	2/100 x 3.75 skewed nails + 2 wire dog	4.7 kN
D	2/100 x 3.75 skewed nails + 3 wire dog	6.7 kN
E	2/100 x 3.75 skewed nails + 4 wire dog	8.7 kN

7.1 Rafters – including valley rafters (Clause 10.2.1.3)

7.1.1 General description

Rafters are roof members that run parallel to the slope or fall of the roof and provide support to purlins, tile battens or sarking. They may be single or multiple span, and frequently terminate at the lower end as a cantilevered eave overhang. The cantilever length is limited to 750 mm, or ¼ of the permitted rafter span, whichever is less. This is based on standard formulas for distributed loading on simply supported and cantilever beams.

Hip rafters (and also ridge boards) have no specific structural function, as they merely resist equal and opposite thrusts of opposing jack or ordinary rafters tied at their lower ends by ceiling framing (couple close roof structure).

There are no specific provisions for rafters trimming the sides of openings, such as dormer windows. Thus each application requires specific structural engineering design.

7.1.2 Design for safety (ULS)

General

Design for safety included consideration of the ULS in bending, shear, bearing, and the ultimate capacity of connections.

Loads

i) Gravity

Cladding	Dead load (G)	Live load (Q)	Snow load (S)
Light (including cladding framing)	0.2 kPa (0.1 kPa with wind)	0.25 kPa (distributed)	0 kPa 0.5 kPa, 1.0 kPa (Section 15)
Heavy (including cladding framing)	0.6 kPa (0.6 kPa with wind)	0.25 kPa (distributed)	0 kPa 0.5 kPa, 1.0 kPa (Section 15)

The 1 kN live load provided for in NZS 4203 dominates rafter spans, especially for light roofs in the lower wind zones. The committee considered that 0.7 kN concentrated load (when the 1.6 load factor is applied, this equates to 112 kg) was more appropriate, and also that a 50% load sharing by virtue of the purlins or battens was realistic.

The area reduction factor for live load (ψ_a) was taken as 1.0.

For the 1999 version of the standard, no allowance was made for snow loads in the body of the standard. Snow loads of 0.5 kPa and 1.0 kPa were removed to a separate Section 15 of the Standard. For snow loading, roof slope was assumed to be less than 30^0, giving the maximum value of roof slope coefficient, C_r of 1.0.

ii) Wind

Design load on the rafter (kN/m) is given by:

$$W_u = q_z.\Box\Sigma p_z.s$$

where: s = rafter spacing,

other symbols are defined in NZS 4203.

Pressure coefficients and factors:

$C_{pe} + C_{pi} = -1.1$ (for wind uplift) or $+ 0.3$ (for wind down),

$k_a, k_l, k_p = 1.0$.

The pressure coefficients used are a compromise from the range of values in NZS 4203 in the interests of simplicity, considering the most likely options from the infinite variety of roof shapes and pitches possible for buildings within the scope of NZS 3604. They will be conservative in many cases, especially moderately sloped hip roofs.

Load cases considered:

1	1.4G
2	1.2G + 1.6Q
3	$1.2G + Q_u + W_u$
4	$0.9G \pm W_u$
5	$1.2G + Q_u + 1.2S_u$

The load combination factor (ψ_u) for cases 3 and 5 was taken as 0.0.

Structural model used for strength

The models used for strength are shown in Figure 14 below.

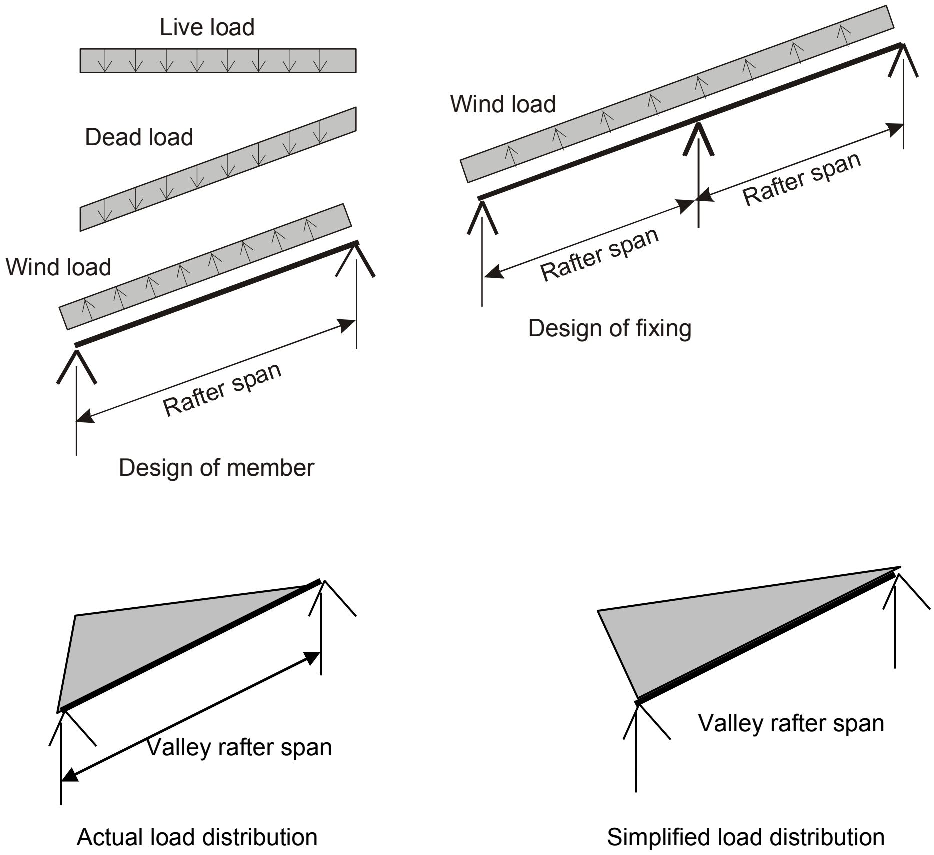


Figure 14. Structural models used for rafters.

Load was applied to the rafter as a uniformly distributed load perpendicular to the member rather than as a series of point loads. The maximum error for realistic rafter spans and purlin spacings is approximately 11%. No allowance was made for load sharing between adjacent rafters.

For valley rafters, the diagram on the left shows the actual load distribution from the purlins but, for simplicity, the triangular load on the right was used.

The bending moment for this model is given by:

$M = 0.064 \times w \times span^3$,

where:

w = applied load (in kPa).

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0
4	1.0
5	0.8

Restraint to the top edge of the rafters was assumed to be provided by the purlins spaced at a maximum of 1.2 m centres. No restraint was assumed to the bottom of the rafters. Slenderness factor, k_8, was calculated by an iterative process controlled by a macro.

Dry timber stresses, as outlined in Table 8 of this report, were used to derive the rafter span tables.

7.1.3 Design for deflection (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

i) Gravity loads were as given above for strength design.

ii) Wind

Pressure coefficients and factors:

$C_{pe} + C_{pi} = -1.1$ or $+0.3$,

$k_a, k_l, k_p = 1.0$.

Load cases considered were:

1	$G + Q_s$ (short and long-term)
2	$G + Q_s + W_s$
3	$G + Q_s + S_s$

The short and long-term load factors were:

Load	Short-term factor (ψ_s)	Long-term factor (ψ_l)
Live	0.7	0
Snow	0.5	0

Structural model used for serviceability

Models used for serviceability are the same as those for strength.

Deflection is given by:

$$\delta = \frac{0.0065 \text{ x } w \text{ x } span^5}{EI} \text{ x } k_2$$

Deflection calculation

The duration of load factor, k_2, was taken as 2.0, and the lower bound modulus of elasticity, E_{lb}, was used for the deflection calculation.

Deflection criteria

Limits on deflection for the load cases considered:

Load case	Deflection limit
1	$\frac{\text{Span}}{300}$
2	$\frac{\text{Span}}{300}$
3	$\frac{\text{Span}}{250}$

with an upper limit of 25 mm for each.

7.2 Ridge beams and underpurlins (Clauses 10.2.1.5 and 10.2.1.9)

7.2.1 General description

Ridge beams support the upper ends of rafters at a ridge line. They are required in situations where the lower ends of the rafters are not tied together by ceiling joists (collar ties were not considered adequate to perform this function because of their unfavourable location). Ridge beams are usually supported by walls or internal posts. Support of ridge beams by hip rafters introduces potential instability of the roof structure because of the horizontal component of the reactions from the hips, and is therefore outside the scope of NZS 3604.

Underpurlins are beams supporting rafters at intermediate points along their length. They are usually supported by struts supported on internal walls, or strutting beams running between internal walls.

7.2.2 Design for safety (ULS)

General

Design for safety included consideration of the ULS in bending, shear, bearing, and the ultimate capacity of connections.

Loads

i) Gravity

Cladding	Dead load (G)	Live load (Q)	Snow load (S)
Light (including framing)	0.3 kPa (0.15 kPa with wind)	0.25 kPa	0 kPa 0.5 kPa (Section 15) 1.0 kPa (Section 15)
Heavy (including framing)	0.7 kPa (0.45 kPa with wind)	0.25 kPa	0 kPa 0.5 kPa (Section 15) 1.0 kPa (Section 15)

The area reduction factor for live load (ψ_a) was taken as 1.0.

For the 1999 revision of the standard, no allowance was made for snow loads in the main roof section, and snow loads of 0.5 kPa and 1.0 kPa were introduced to a separate Section 15 of the Standard. Roof slopes of 30^0 and 45^0 were considered when calculating the worst case for dead and snow loads.

ii) Wind

Design load on the member (kN/m) is given by:

$$w_u = q_z.\ \sum p_z.LD$$

where: LD = loaded dimension of the beam as defined in Figure 1.3 of NZS 3604,

other symbols are defined by NZS 4203.

Pressure coefficients:

$C_{pe} + C_{pi} = -1.1$, (for wind uplift) or + 0.3 (for wind down),

K_a, K_l, K_p = 1.0.

Load cases considered:

1	1.4G
2	1.2G + 1.6Q
3	$1.2G + Q_u + W_u$
4	$0.9G - W_u$
5	$1.2G + Q_u + 1.2S_u$

The live load combination factor, ψ_u, for wind and snow loading (cases 3 and 5) was taken as 0.0.

Structural model used for strength

The model used for strength was a single span simply supported beam as shown in Figure 15 below

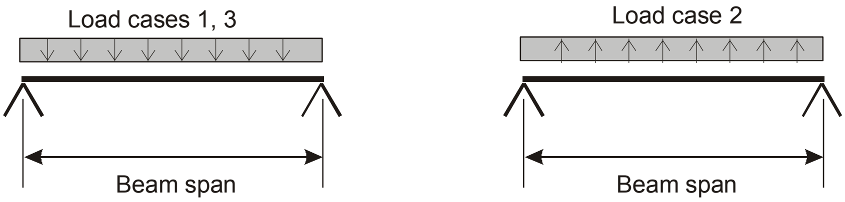


Figure 15. Structural model used for ridge beams and underpurlins.

Load was applied to the member as a uniformly distributed load perpendicular to the span. To allow for the continuity of rafters applying reaction to the underpurlin, a "reaction enhancement factor" of 1.25 was used. No allowance was made for load sharing.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0
4	0.8
5	0.8

Restraint to the top of the members was assumed to be provided by the rafters spaced at 1.2 m centres. No restraint was assumed to the bottom of the members. Slenderness factor, k_8, was calculated by an iterative process controlled by a macro.

Dry timber stresses, as outlined in Table 8 of this report, were used to derive the ridge beam span tables.

Connection capacity (ridge beams only)

The additional ridge beam uplift fixings in Table 10.3 of NZS 3604 were derived using parameters from NZS 3603.

7.2.3 Design for deflection (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

i) Gravity loads are as given above for strength design.

ii) Wind

Pressure coefficients

$C_{pe} + C_{pi} = -1.1$,

K_a, K_l, K_p = 1.0.

Load cases considered:

1	G
2	$G + Q_s + W_s$
3	$G + Q_s + S_s$

The short and long-term load factors were:

Load	Short-term factor (ψ_s)	Long-term factor (ψ_l)
Live	0.7	0.0
Snow	0.5	0.0

Structural model used for serviceability

Models used for serviceability are shown in Figure 15 of this report. Load was applied to the member as a uniformly distributed load. No allowance was made for load sharing.

Deflection calculation

The duration of load factor, k, was taken as 2.0, and the lower bound modulus of elasticity, E_{lb}, was used for the deflection calculation.

Deflection criteria

Limits on deflection for the load cases considered:

Load case	Deflection limit
1	$\frac{\text{Span}}{300}$
2	$\frac{\text{Span}}{300}$
3	$\frac{\text{Span}}{250}$

7.3 Ceiling joists and ceiling runners (Clauses 10.2.1.6 and 10.2.1.7)

7.3.1 General description

Ceiling joists are closely spaced framing members supporting ceiling linings, which are either attached directly, or to ceiling battens spanning between the joists. They also connect the bottom ends of the rafters to form a couple close roof. Ceiling runners are beams supporting ceiling joists at intermediate points along their length, introduced to reduce the joist span. Both span between walls.

Spans of both were taken from NZS 1900: Chapter 6.1 and re-analysed using NZS 1900: Chapter 9.1 (in WSD). New USD tables were developed for Amendment 2, using a spreadsheet.

7.3.2 Design for safety (ULS)

General

Design for safety included consideration of the ULS in bending and shear.

Loads

Dead load (G)	Live load (Q)
0.175 kPa	0.5 kPa (distributed) 1.0 kN (concentrated)

Load cases considered:

1	1.4G
2	1.2G + 1.6Q (distributed)
3	1.2G + 1.0Q (concentrated)*

Note: *A load factor of 1.0 under concentrated loading was chosen as it was considered that 1.0 kN was more than adequate to simulate the weight of a typical tradesman crawling in the ceiling space.

Structural model used for strength

The models used for strength are shown in Figure 16 below.

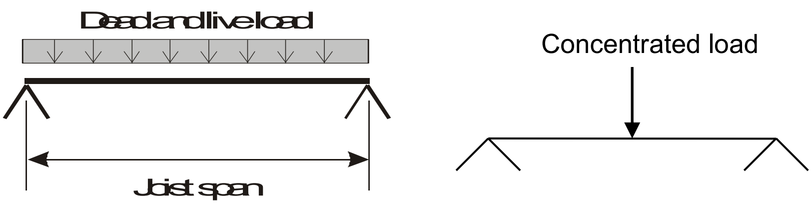


Figure 16. Structural model for ceiling joists and ceiling runners.

No allowance was made for load sharing for distributed loading on ceiling joists or ceiling runners, nor for concentrated loads on runners. However, 40% of the concentrated load on ceiling joists was distributed to adjoining joists.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1.4G	0.6
1.2G + 1.6Q	0.8

Restraint to the bottom (tension) edge of the joists was assumed to be provided by the ceiling lining or closely spaced battens. The slenderness factor, k_8, was calculated using the slenderness coefficient S_1 = 3d/b from Clause 3.2.5.3 of NZS 3603. The same reasoning was used for the ceiling runners, with restraint provided by the connections to the ceiling joists.

Dry timber stresses, as outlined in Table 8 of this report, were used to derive the rafter span tables.

7.3.3 Design for deflection (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

Gravity loads were as given above for strength design.

Structural model used for serviceability

The model used for serviceability is shown in Figure 16 of this report. Load was applied to the member as a uniformly distributed load. No allowance was made for load sharing.

Deflection calculation

The duration of load factor, k, was taken as 2.0 for both ceiling joists and runners. Standard modulus of elasticity, E, was used for ceiling joists, and the lower bound modulus of elasticity, E_{lb}, was used for ceiling runners in the deflection calculations.

Deflection criteria

Maximum deflection span/300.

7.4 Verandah beams (Clause 10.2.1.12)

7.4.1 General description

Verandah beams span between posts (or walls) and support the outer ends of the verandah rafters. They differ from lintels in being subject to wind pressures on the underside of the rafters. Thus, they are subjected to high uplift loads requiring substantial fixings and anchorages. However, the deflection criteria are not as stringent as for lintels.

7.4.2 Design for safety (ULS)

General

Design for safety included consideration of the ULS in bending, shear, bearing, and the ultimate capacity of connections. Unfortunately an error in the spreadsheet used to derive the span selection tables means that the spans as published are up to 20% shorter than they should be.

Loads

i) Gravity

Cladding	Dead load (G)	Live load (Q)	Snow load (S)
Light (including framing)	0.3 kPa (0.15 under wind uplift)	0.25 kPa (distributed) 1 kN (concentrated)	0 kPa 0.5 kPa (Section 15) 1.0 kPa (Section 15)
Heavy (including framing)	0.7 kPa (0.45 under wind uplift)	0.25 kPa (distributed) 1 kN (concentrated)	0 kPa 0.5 kPa (Section 15) 1.0 kPa (Section 15)

The area reduction factor for live load (ψ_a) was taken as 1.0.

For the 1999 version of the standard, no allowance was made for snow loads in the body of the standard. Snow loads of 0.5 kPa and 1.0 kPa were removed to a separate Section 15.

ii) Wind

Pressure coefficients:

$C_{pe} + C_{pi}$ = -1.2 (up), 0.5 (down)

K_a, K_l, K_p = 1.0.

Load cases considered were:

Load case	Combination
1	1.4G
2	1.2G + 1.6Q (distributed and concentrated)
3	0.9G $-W_u$ (up)
4	1.2G + 1.0W_u (down)
5	1.2G + Q_u + 1.2S_u

The load combination factor (ψ_u) for snow loading was taken as 0. Snow loading (case 5) was considered for roof slopes of both 30^0 and 45^0.

Structural model used for strength

The models used for strength are shown in Figure 17 of this report.

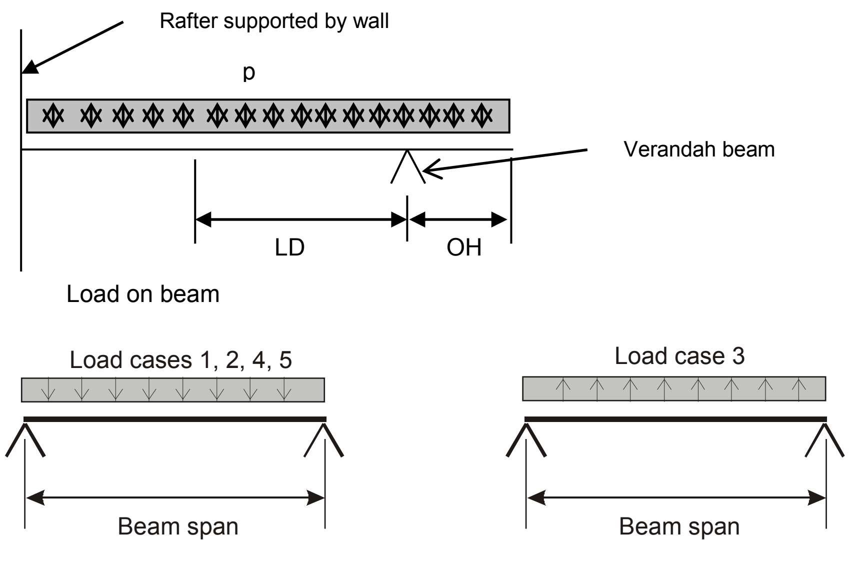


Figure 17. Structural model for verandah beams.

Design load on the beam (kN/m) was a uniformly distributed load perpendicular to the span, given by:

$$w = p \times (LD + OH + OH^2/4LD)$$

where: LD = loaded dimension,

OH = rafter overhang beyond the beam – taken as 750 mm.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0
4	1.0
4	0.8

Restraint to the top of the members was assumed to be provided by the rafters spaced at 1.2 m centres. No restraint was assumed to the bottom of the members, with L_{ay} taken as the length of the beam.

Dry timber stresses, as outlined in Table 8 of this report were used to derive the verandah beam span tables.

Connection capacity

The uplift fixings in Table 10.8 were derived using parameters from NZS 3603.

7.4.3 Design for deflection (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

i) Gravity and wind loads are as given above for strength design.

Load cases considered:

Load case	Combination
1	G + Q_s (long and short-term)
2	G + Q_s + W_s
3	G + Q_s + S_s

Snow loading was considered for both 30^0 and 45^0 roof slopes.

The short and long-term load factors were:

Load	Short-term factor (ψ_s)	Long-term factor (ψ_l)
Live	0.7	0
Snow	0.5	0

Structural model used for serviceability

The model used for serviceability is shown in Figure 17 of this report. Load was applied to the member as a uniformly distributed load.

Deflection calculation

The duration of load factor, k_2 was taken as 2.0. The lower bound modulus of elasticity, E_{lb}, was used for the deflection calculation, except for double member beam options, where $E = (E + E_{lb})/2$.

Deflection criteria

A deflection limit of span/300 was used for all load cases.

7.5 Purlins and tile battens (Clause 10.2.1.16)

7.5.1 General description

Purlins are horizontal members spanning across rafters or trusses, and to which roof cladding is directly attached. Tile battens for heavy and light roofs are also included in the section.

There are several subtle differences between purlins and tile battens:

- purlins are usually fixed by the builder before the roofing installation begins, while tile battens are installed by the roofer
- tile batten spacings are determined by tile sizes (maximum 400 mm spacing), while purlin spacings are dependent on the span of the cladding and available rafter length
- tile battens are generally supplied rough sawn.

7.5.2 Design for safety (ULS)

General

Design for safety included consideration of the ULS in bending, shear, and the ultimate capacity of connections.

Loads

i) Gravity

Cladding	Dead load (G)	Live load (Q)	Snow load (S)
Light	0.1 kPa	0.25 kPa (distributed) 1.0 kN* (concentrated)	0 kPa 0.5 kPa (section 15) 1.0 kPa (section 15)
Heavy	0.6 kPa	0.25 kPa (distributed) 1.0 kN* (concentrated)	0 kPa 0.5 kPa (section 15) 1.0 kPa (section 15)

* Concentrated load was not used for cantilevered purlins.

The area reduction factor for live load (ψ_a) was taken as 1.0.

ii) Wind

For “impervious” claddings (all roofing types except for interlocking concrete tiles), the design load on the member (kN/m) is given by:

$$w_u = q_z.\square\Sigma p_z.s$$

where: s = purlin or batten spacing,

other symbols are defined by NZS 4203.

Pressure coefficients:

C_{pe} = -0.8

C_{pi} = 0.0 (enclosed roof space)

K_l = 1.0 for the main body of the roof, and 1.5 for periphery areas

K_a, K_p = 1.0.

The periphery areas of the roof are defined as 0.2 times the building width. Because hips and ridges are included (for all roof slopes in contradiction to NZS 4203) this results in virtually all the roof being considered as in the periphery area.

For concrete tile roofs which are permeable (especially when the fronts of the tiles tilt up under wind suction), it is not possible to accurately estimate wind forces analytically. Instead, to determine the batten fixing requirements, tests were carried out to ascertain the load required to remove a tile from the batten when fixed in accordance with NZS 4206. The average pull-off resistance was 0.15 kN. At the maximum permitted rafter/truss spacing, alternate tiles fixed, and maximum tile cover width, this gives a maximum applied load at each batten/rafter fixing point of 0.23 kN.

Load cases considered:

Load case	Combination
1	1.4G
2	1.2G + 1.6Q (distributed)
3	1.2G + 1.0Q (concentrated)*
4	0.9G ± W_u

Note: *A load factor of 1.0 under concentrated loading was chosen as it was considered that 1.0 kN was more than adequate to simulate the weight of a typical roofing installer. Once the roof is installed and maintenance loading is applied, the roofing material is able to distribute the concentrated load to adjacent members, which was not allowed for in the derivation.

Structural model used for strength

The models used for strength are shown in Figure 18 below.

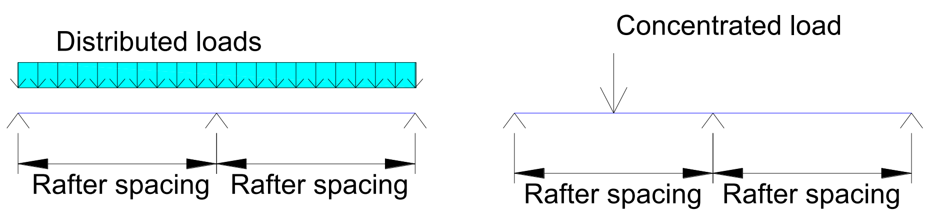


Figure 18. Structural models used for purlins and tile battens (strength).

Load was applied to the member as a uniformly distributed load perpendicular to the span. No allowance was made for load sharing under wind loading because all members will receive the same load. The same assumption was made for concentrated loading because during roof installation, individual members can receive the full load of a person.

Bending moments under concentrated loading were derived using the following formula:

$$M = 0.20 \times P \times \text{span}.$$

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors for strength:

Load case	k_1
1	0.6
2	0.8
3	0.8
4	1.0

Timber properties for purlins are as set out in Table 8 of this report. Note that purlins are supplied in dressed sizes and tile battens in rough sawn sizes, and this is as noted in the tables.

Timber properties for tile battens received special consideration by the committee. Using the loads and assumptions discussed above, tile battens will be subject to stresses ranging up to 20 MPa, which is well in excess of design stress given in NZS 3603 Amendment 4 for No 1 framing as usually supplied for tile battens. However, enquiries with OSH, ACC and tiling manufacturers revealed negligible incidents of failure for either No 1 framing or “cut of log” battens. The committee recognised that during tile roofing installation a selection process is used by the installers, which effectively ensures defects are cut out or located in non-critical positions. Clause 10.2.1.16.5 was introduced to encapsulate this process.

Purlins and battens are installed so that bending is about the weak axis, so the stability factor, k_8, was taken as 1.0. Bending about the strong axis (parallel to the slope of the roof) was not considered.

Connection capacity

Fixing capacity	Example of fixing
0.4 kN	1/100x3.75 hand-driven nail or 1/90x3.15 power-driven nail
0.7 kN	2/100x3.75 hand-driven skew nails or 2/90x3.15 power-driven nails
2.7 kN	2/100x3.75 skew nails + 1 wire dog or 2/100x3.75 skew nails + 1/12g Type 17 screw
4.7 kN	2/100x3.75 skew nails + 2 wire dogs or 2/100x3.75 skew nails + 2/12g Type 17 screws

7.5.3 Design for serviceability (SLS)

General

Design for serviceability included consideration of bending deflection, including the effects of creep where appropriate.

Loads

Gravity and wind loads are as given above for strength design.

Load cases considered were:

Load case	Combination
1	$G + \psi_s Q_s$ (short-term loading)
2	$G + \psi_l Q_s$ (long-term loading)
3	$G + Q_s + W_s$

The short and long-term live load factors were:

$\psi_s = 0.7$, $\psi_l = 0.0$,

and combination factor, $\psi_u = 0.0$.

Structural model used for serviceability

Models used for serviceability are shown in Figure 19 below. Load was applied to the member as a uniformly distributed load. No allowance was made for load sharing.

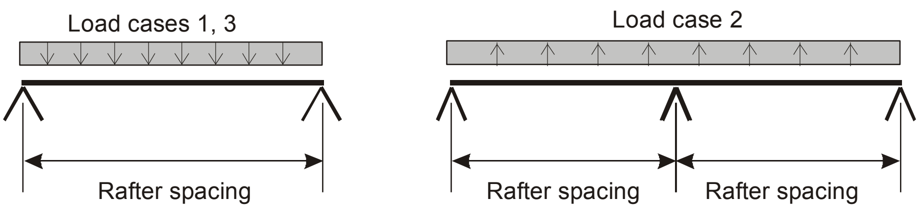


Figure 19. Structural models used for purlins and tile battens (deflection).

Deflection calculation

The lower bound modulus of elasticity, E_{lb}, was used for purlin and batten deflection, as set out in Table 8 of this report. The load duration factor, k_2, was taken as 2.0 for load case 2.

Deflection criteria

Limits on deflection for the load cases considered:

Load case	Deflection limit
1	$\frac{\text{Span}}{300}$
2	$\frac{\text{Span}}{300}$
3	$\frac{\text{Span}}{250}$

with no upper limit.

7.6 Roof trusses (Clause 10.2)

Roof trusses are a proprietary supplied items, with each one specifically designed to suit the application using NZS 3603 (typically using specialist software developed for the purpose by the nail plate suppliers). Thus it is important that the truss designer (via the supplier) is given all the relevant site-specific information to allow the design to be properly carried out.

To keep truss reactions within the capacity of wall members and the rest of the structure to support them, limits are placed on truss spans and spacing in Clause 10.2.2. In particular, girder trusses (trusses that carry loads from other trusses) may require additional support. The requirements for this extra support can only be determined by the truss system designer, and thus is outside the scope of NZS 3604.

Lateral bracing requirements of individual truss members is usually provided by the purlins and ceiling framing. Any special needs must be communicated between the designer and builder. Lateral bracing of the roof as a whole is no different whether the roof is a proprietary truss system or stick framed, and is provided for in Clauses 10.3 and 10.4.

7.7 Roof truss anchorage (Clause 10.2.2.6)

7.7.1 General description

Requirements for the anchorage of roof trusses against uplift forces can best be determined by the truss designer, and suitable fixing details provided with the truss system. As a default option, the provisions of 10.2.2.6 may be used. However this Clause does not provide for special situations such as girder trusses, hip trusses and other complex truss roof systems.

7.7.2 Design for safety (ULS)

General

Design for safety included consideration of the actions causing instability and those resisting instability, as provided for in Clause 2.5.3.4 of NZS 4203.

Loads

i) Gravity

Cladding	Dead load (G)
Light (including framing and ceiling)	0.46 kPa
Heavy (including framing and ceiling)	0.84 kPa

ii) Wind

Design load on the anchorage (kN) is given by:

$$W_u = q_z . \square \sum p_z . A$$

where: A = tributary area, determined as the product of truss spacing x loaded dimension,

other symbols are defined by NZS 4203.

Pressure coefficients:

$C_{pe} + C_{pi} = -1.1$,

$K_a, K_l, K_p = 1.0$.

Load cases considered:

Load case	Combination
1	$0.9G \pm W_u$

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factor (k_1) = 1.0.

Connection capacity

Fixing type	Example of fixing	Capacity □(ϕQ_n)
A	2/100 x 3.75 skewed nails	0.7 kN
B	2/100 x 3.75 skewed nails + 1 wire dog	2.7 kN
C	2/100 x 3.75 skewed nails + 2 wire dog	4.7 kN
D	2/100 x 3.75 skewed nails + 3 wire dog	6.7 kN
E	2/100 x 3.75 skewed nails + 4 wire dog	8.7 kN
F	2/100 x 3.75 skewed nails + U strap 27x1.2mm, 10/30x3.15 nails each end	16 kN

7.8 Roof bracing (Clause 10.3)

The provisions for roof bracing encapsulated what was considered "good building practice", and no formal engineering calculations were undertaken to verify them. Unfortunately the ambitious roof shapes frequently being used today are well beyond the scope envisaged in the section of NZS 3604, resulting in the provisions being very difficult to interpret. This topic needs revision.

8. SECTION 11. BUILDING ENVELOPE

The provisions in this section for the building envelope cover roof and wall claddings, and their appropriate underlays. The emphasis is on providing solutions complying with sections B2 (Durability) and E2 (External Moisture) of the NZBC. However, the provisions for masonry veneer cladding do have an engineering component and therefore comply with B1.

8.1 Masonry veneer wall cladding (Clause 11.7)

Most of the provisions of this clause date back to the original standard in 1978, although items such as spacing of ties which are already covered by NZS 4102 (SNZ 1989) were deleted from the 1999 revision of NZS 3604. Provisions for the performance of the ties themselves are in turn called up by reference to AS/NZS 2699 (SA/SNZ 2000).

8.1.1 Design for safety (ULS)

General

NZS 1900: Chapter 6.1 *Timber* sets a maximum height for veneers of 12 feet (3.67 m) above the foundation, with a maximum height above ground level of 18 feet (5.5 m). These limits were based on engineering judgement, and observations of the performance of veneers in past earthquakes.

NZS 3604:1978 relaxed these heights with the knowledge then available on the in-plane load deflection performance of braced walls and wall ties. The limits were changed to 4 m veneer height, and 7 m above ground respectively. The 4 m height limit effectively prevents the use of masonry veneer in two storey construction without specific engineering design to stiffen and strengthen the building structure.

It is known that 2.4 m high sheet lined timber framed bracing panels under cyclic testing deflect between 20 and 40 mm in-plane at ultimate load (Cooney and Collins 1979). The test method for ties prescribed in AS/NZS 2699 provides for an in-plane cyclic displacement of ±20 mm, so the deflection capacities of the ties and bracing panels to some degree coincide. This was done to ensure that the frame, ties and veneer can sustain the differential movements expected under earthquake actions.

The maximum height of veneers in gable walls is limited to 5.5 m on the assumption that the triangular frame of a gable does not deflect laterally, and therefore will not increase the differential deflection significantly further above that for the single storey frame. Studs at close spacing are required to secure the wall ties to the masonry at the appropriate centres.

The maximum allowable mass of 220 kg/m^2 dates back to the use of 110 mm veneers. The bracing demand tables of Section 5 of the Standard are based on this value, which means some conservatism is built in when used with 70 mm veneers with a mass of approximately 130 kg/m^2.

8.2 Veneer lintels (Table 11.4)

8.2.1 General description

Lintel bars are required to support masonry veneer over wall openings while the bricks are being laid, and until the mortar joints are cured. After full curing of the mortar, the veneer will generally be self-supporting without the need for a permanent lintel. However. under earthquake actions or other distress which may compromise the stability of the veneer some durable long-term support is required, and this is provided for in Clause 11.7.6 and Table 11.4.

The table was based on a simple structural model and generally followed the procedures of Structural Note 17a published by the Australian Brick Development Research Institute (BDRI 1984).

8.2.2 Design for safety (ULS)

General

Design for safety included consideration of the bending strength of steel lintel sections. No contribution from the masonry was assumed. Length of landing on the supporting masonry at each end was set at 200 mm in the original 1978 version of NZS 3604. This was relaxed to 100 mm for shorter openings in the 1999 revision, because of the lack of any evidence that 200 mm was necessary.

Loads

Weight of brick masonry was taken as:

70 mm bricks 1.35 kPa

90 mm bricks 1.70 kPa.

The only strength load case considered was 1.4 x G.

Structural model used for strength

The model used for the strength is shown in Figure 20 below.

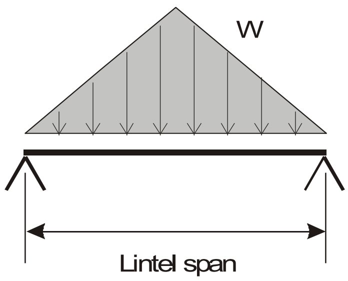


Figure 20. Structural model used for veneer lintels.

Total load W was calculated assuming that only the masonry within a triangle of included angle 45^0 from the ends of the opening would contribute load to the lintel. The remainder is carried by arching directly to the supports. For the lower heights of veneer supported, and longer lintel spans, this triangle is truncated by the top of the masonry.

Member capacity

For steel sections of less than 12 mm thickness, steel yield strength, F_y, was taken as 250 MPa, and the masonry was assumed to provide full lateral support.

8.2.3 Design for serviceability (SLS)

General

Design for serviceability included consideration of bending deflection.

Load

Loading and structural model was the same as for strength, except that the load factor was taken as 1.0 G.

Deflection calculation

Modulus of elasticity for steel was taken as 210 x 10^3 Pa. The stiffness of the masonry itself was not considered to contribute to the lintel as a whole.

Deflection criteria

To avoid brick cracking, the deflection limit was set at span/360.

9. SECTION 13. CEILINGS

Although NZS 3604:1999 states that *ceiling linings are not a general requirement of this standard*, they are assumed to provide lateral restraint to roof framing members (Section 10 of the Standard) and are also required to distribute lateral loads to bracing elements by diaphragm action. The concept of bracing lines depends on this function. The weight limit of 17.5 kg/m^2 (which is equivalent to about 19 mm total thickness of plasterboard) was introduced to avoid heavy fire-rated ceiling systems from overloading the ceiling framing members.

The provisions for diaphragms were introduced in the 1978 version of the standard, and have changed little since then. Their engineering basis is not known, but is probably based on practice of the day and sound engineering judgement.

10. SECTION 15. 0.5 AND 1.0 KPA SNOW LOADING

For the 1999 revision of NZS 3604 it was decided to produce all member selection tables in the body of the standard for zero snow loading, and to introduce a new section covering snow loading on only those members affected (that is, lintels and some roof members). The initial publication covered only snow loads up to 0.5 kPa, but this was soon corrected by Amendment 1 to include up to 1.0 kPa loading.

The provisions of Section 15 of the Standard are based on Part 6 of NZS 4203:1992, with the 0.5 and 1.0 kPa values corresponding to the "open ground snow load", s_g. The table in Figure 15.1 of NZS 3604 is a simplified version of Figure 6.3.1 of NZS 4203. Because a live load of 0.25 kPa was already included as a load case in all the member selection tables, the 0.2 kPa plateau in Figure 6.3.1 could be disregarded and snow loading ignored for sites below 200 m altitude in zones 1 to 3. Thus the majority of new buildings constructed in New Zealand every year are not affected by snow loading. Note that the entry "NA" in the table for zones 4 and 5 is misleading and should be "0".

For Amendment 2 the following parameters were used in the derivation of the selection tables:

Loading:	$C_c = 1.0$	(for category IV buildings)
	$C_e = 1.0$	(for sheltered sites)
	$C_r = 1.0$	(for roof slopes less than 30^0)
	$= 0.625$	(for roof slopes at 45^0)

Deflection limit was span/300, with an upper limit of 25 mm for the lintels.

Other parameters were as for the corresponding entry in the main body of the standard.

11. SECTION 16. COMPOSITE LINTELS

Provisions for composite lintels were introduced with the 1999 revision of NZS 3604 in response to the growing trend in New Zealand construction away from solid timber lintels. The intention was that if selection tables and construction details were provided for lintels constructed of generic materials, then builders would have additional “off-the-shelf” options to the solid timber lintels of Section 8 of the Standard. It was expected that suppliers of proprietary lintel, such as folded metal or nail-plate laminated lintels, would provide similar material for their systems. Section 20 *Industry information* was provided for this purpose.

11.1 Plywood box beam lintels (Clause 16.1)

11.1.1 General description

Plywood box beam lintels are members which may be readily constructed on-site or off-site with standard carpentry techniques, and using materials that are covered by existing New Zealand standards.

The scope of this section covers lintels supporting roof loads only. Loads were assumed to be uniformly distributed along the lintel length. Thus any lintel supporting a concentrated load, such as from a girder truss, falls outside the scope of this section and must be specifically designed.

11.1.2 Design for safety (ULS)

General

The lintel span tables were derived using dead and live loads, and wind pressure coefficients for roof pitches up to 45^0 only. Above this, the lintel span multipliers (Table 8.7) must be used.

Design for safety included consideration of the ULS in bending, shear, bearing, and the ultimate capacity of connections. Design for bending took account of the contribution of the plywood sheets (even though the plywood sheets are of finite length and there are nailed joints between the ends of the sheets) as well as the chord members. The method used is described in Sandie (1988).

Loads

i) Gravity loads

	Dead load (G) (kPa)	Live load (Q) (kPa)
Light roof (including framing and ceiling)	0.46	0.25
Heavy roof (including framing and ceiling)	0.84	0.25

Eaves overhang was taken as 600 mm.

ii) Wind loads

Wind zone	Wind speed, V_{dz} (m/s)	Design wind pressure, q_z (kPa)
Low	32	0.61
Medium	37	0.82
High	44	1.16
Very High	50	1.50

External pressure coefficients:

Wind loads were checked for pitches of 15^0, 30^0, 45^0 with C_{pe}s as follows:

Roof pitch	Upwind slope	Downwind slope
15^0	- 0.7	- 0.5
30^0	- 0.3	- 0.6
45^0	+ 0.6	- 0.6

Internal pressure coefficient: C_{pi} = + 0.3,

Modifying factors: (K_a, K_l, K_p) = 1.0

Load cases considered were:

1	1.4G
2	1.2G + 1.6Q
3	0.9G - W_u

Structural model used for strength

The model used for strength is shown in Figure 21 below.

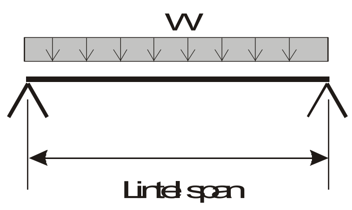


Figure 21. Structural model used for strength of composite lintels.

Member capacity

Strength reduction factor (ϕ) = 0.8.

Duration of load factors:

Load case	k_1
1	0.6
2	0.8
3	1.0

Restraint to the top of the members was assumed to be provided by the wall framing at 600 mm centres. Thus, k_8 was taken as 1.0. Load sharing by other members such as roof and wall framing was not considered, and k_4 = 1.0.

Characteristic stresses

Radiata pine, No 1 framing (moisture content <16 %), was used for the derivation of Table 16.1. For Amendment 2 no re-calculation was done, but timber grades were limited to VSG8/MSG8 and above.

Parameter	Value
Bending strength (f_b)	17.7 MPa
Shear strength (f_s)	3.8 MPa
Compression parallel (f_c)	20.9 MPa
Compression perpendicular (f_p)	8.9 MPa
Modulus of elasticity (E)	8.0 GPa

Construction plywood, Grade F11.

Parameter	Value
Modulus of elasticity (E)	10.5 GPa
Modulus of rigidity (G)	0.525 GPa
Bending strength (f_{pb})	28.8 MPa
Tension parallel (f_{pt})	17.3 MPa
Panel shear (f_{ps})	4.7 MPa
Compression in the plane of the sheet (f_{pc})	21.6 MPa

Bearing width on the supporting doubling stud = 47 mm.

Plywood design factors:

Factor	Value
k_{14}, k_{15}, k_{16}, k_{18}, k_{19}	1.0
Characteristic nail strength (Q_k)	526 N
Secant stiffness of nail joint (k)	1.4
Stiffness factor (h_{32})	590
Duration factor (j_{12})	4

11.1.3 Design for serviceability (SLS)

General

Design for serviceability included consideration of bending and shear deflection, including the effects of creep where appropriate. The contribution of the ply webs to the stiffness of the beam followed the method given by Sandie (1988).

Loads

i) Gravity loads are as given above for strength design.

ii) Wind

Wind zone	Wind speed V_{dz} (m/s)	Design wind pressure, q_z (kPa)
Low	26	0.4
Medium	30	0.53
High	35	0.75
Very High	40	0.98

Pressure coefficients:

$C_{pe} + C_{pi} = -1.1$,

$K_a, K_l, K_p = 1.0$.

Load cases considered:

1	G
2	$G + Q_s$
3	$G + W_s$

The short and long-term load factors were:

Load	Short-term factor ($\square_s$)	Long-term factor ($\square_l$)
Live load	0.7	0.4
Wind load	0.7	0.0

Structural model used for serviceability

The model used for serviceability was the same as for strength, as shown in Figure 21 of this report. Load was applied to the member as a uniformly distributed load. No allowance was made for load sharing.

Deflection calculation

Modulus of elasticity (E)	8 GPa
Duration of load factor (k_2)	2.0

Deflection criteria

Limits on deflection for the load cases considered:

Load case	Deflection limit
1	$\frac{\text{Span}}{300}$
2	$\frac{\text{Span}}{300}$
3	$\frac{\text{span}}{300}$

11.2 Glue laminated timber lintels (Clause 16.2)

11.2.1 General description

A table of glue laminated timber lintels, manufactured in accordance with AS/NZS 1328 Parts 1 and 2 (SA/SNZ 1998), was provided as an alternative to ply box lintels on the basis of equivalent performance and the same width. This gave additional lintel options of lesser depth than the ply box lintels. No changes were made for Amendment 2.

11.2.2 Design for safety and serviceability

Member capacity

Depths of glulam sections of 90 mm width were calculated to give equivalent bending strengths and stiffnesses to the ply box beams. Table 16.2 gives the minimum depths satisfying both criteria. The equations used were:

$$f_{t(bb)} \cdot Z_{(bb)} = f_{b(gl)} \cdot \left(\frac{b_{(gl)} \cdot d_{(gl)}^{\ 2}}{6} \right) \qquad \text{(strength)}$$

$$E_{(bb)} \cdot I_{(bb)} = E_{(gl)} \cdot \left(\frac{b_{(gl)} \cdot d_{(gl)}^{\ 3}}{12} \right) \qquad \text{(stiffness)}$$

where: subscript (bb) refers to box beam

subscript (gl) refers to glulam

and other symbols are as in NZS 3603.

The relevant parameters are set out below:

	Radiata pine No 1 framing (ply box beam)	GL8	GL10	GL12
Modulus of elasticity (E)	8 GPa	8 GPa	10 GPa	11.5 GPa
Bending strength (f_b)	17.7 MPa	19 MPa	22 MPa	25 MPa
Tension strength (f_t)	11 MPa	10 MPa	11 MPa	12 MPa

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