NATIONAL ASSOCIATION OF STEEL-FRAMED HOUSING INC. (NASH)

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NASH’s key objectives are to:
- Support the long-term growth and sustainability of the steel framing industry.
- Maximise awareness of the steel-framing industry in the market place.
- Promote the advantages of steel-framing to the building industry and homeowners.

Acknowledgment

The Committee would like to acknowledge the assistance supplied by the committee in the production of this Standard and also the help and support of NASH Australia as parts of this Standard are based on a similar Australian document.

Committee

This NASH Standard was prepared by representatives of the following organisations:
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- Heavy Engineering Research Association (HERA)
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- LGSC Ltd.
- National Association of Steel-Framed Housing Inc. (NASH)
- New Zealand Steel
- Redco Consulting Professional Engineers Ltd.
- Rollforming Services
- University of Auckland
- Winstone Wallboards Ltd.

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Foreword

This Standard is intended to be an alternative solution verification method to the New Zealand Building Code (NZBC) clause B1 *Structure*. It sets out design criteria for the steel-framing of low-rise buildings including houses and low-rise commercial buildings.

The 2016 edition of the NASH Standard Part 1: 2016 is a revision of the 2010 edition that is currently an acceptable solution within the NZ Building code. This edition is currently with MBIE and in the process to becoming an acceptable solution as a replacement of the 2010 edition.

The major developments of this edition of the NASH standard includes:

- Limit state design to align with the AS/NZS 1170 series of Standards.
- The introduction of serviceability criteria.
- Tolerances for the manufacture of steel components and guidance on their installation to comply with the provisions given in this standard.
- Guidance for calculating the self-weight of materials.

This Standard is part of a suite of NASH Standards alternative solutions which including the following:

CONTENTS

GENERAL 7
  1.1 Scope 7
  1.2 Interpretation 9
  1.3 Referenced documents 9
  1.4 Basis for design 10
    1.4.1 General 10
    1.4.2 System-based assumption 10
    1.4.3 Durability 10
  1.5 Design actions 11
    1.5.1 General 11
    1.5.2 Determination of imposed actions 11
    1.5.3 Determination of wind actions and reference pressures 12
    1.5.4 Determination of Earthquake actions 12
    1.5.5 Evaluation of Overstrength Forces. 14
    1.5.6 Determination of snow loads 14
  1.6 Design properties 14
    1.6.1 Material properties 14
    1.6.2 Section properties 15
    1.6.3 Tolerances 15
  1.7 Design criteria 16
    1.7.1 Stability 16
    1.7.2 Strength 16
    1.7.3 Serviceability 16
  1.8 Methods of assessment 16
    1.8.1 General 16
    1.8.2 Calculation 16
    1.8.3 Testing 17
    1.8.4 Combination of calculation and testing 17

2. ROOF MEMBERS 18
  2.1 General 18
  2.2 Roof battens 18
    2.2.1 Design for strength 18
    2.2.2 Design for serviceability 19
  2.3 Roof trusses or rafters 20
    2.3.1 Design for strength 20
    2.3.2 Design for serviceability 21
  2.4 Ceiling battens 22
    2.4.1 Design for strength 22
    2.4.2 Design for serviceability 23
  2.5 Roof connections and bracing 23

3. WALL MEMBERS 24
  3.1 General 24
3.2 Load bearing wall studs
3.2.1 Load paths
3.2.2 External load bearing wall studs for single storey or upper storey of a two storey construction
3.2.3 External load bearing wall studs for lower storey of two storey construction
3.2.4 Internal load bearing wall studs

3.3 Non load bearing studs
3.3.1 Load path
3.3.2 Design for strength
3.3.3 Design for serviceability

3.4 Nogging

3.5 Wall plates for load bearing walls
3.5.1 Load path
3.5.2 Design model
3.5.3 Design for strength
3.5.4 Design for serviceability (for upper storey and lower storey wall plates)

3.6 Lintels
3.6.1 Load path
3.6.2 Design for strength
3.6.3 Design for serviceability (for upper storey and lower storey lintels)

3.7 Wall bracing

4. FLOOR MEMBERS
4.1 General

4.2 Floor joists and bearers
4.2.1 Load paths
4.2.2 Design for strength
4.2.3 Design for serviceability

4.3 Floor and sub-floor bracing

5. CONNECTIONS
5.1 General
5.2 Design criteria

6. BRACING
6.1 General
6.2 Roof bracing
6.2.1 General
6.2.2 Truss bracing

6.3 Ceiling diaphragm bracing
6.3.1 Ceiling lining material

6.4 Wall bracing
6.4.1 Load path
6.4.2 Design for strength

6.5 Floor and sub-floor bracing
6.5.1 Floor joists or bearers
6.5.2 Sub-floor
6.5.3 Design to meet loads
6.6  Floor diaphragm bracing  
   6.6.1  Floor diaphragms  
   6.6.2  Subdivide floor  
   6.6.3  Ground floor diaphragms  
   6.6.4  Upper floor diaphragms

7.  TESTING  
   7.1  General  
   7.2  Additional requirements for prototype testing  
   7.3  Design assisted by testing  
      7.3.1  General  
      7.3.2  Interpolation of values obtained by prototype testing

APPENDIX A. CONSTRUCTION  
APPENDIX B. SYSTEM EFFECT  
APPENDIX C. DYNAMIC PERFORMANCE OF FLOOR SYSTEM  
APPENDIX D. TOLERANCES  
APPENDIX E. GUIDE FOR DETERMINATION OF SELF-WEIGHTS  
APPENDIX F. SERVICE HOLES IN FRAME MEMBERS  
APPENDIX G. MEMBER CLASSIFICATIONS
GENERAL

1.1 SCOPE

This Standard sets out the structural design criteria, at both serviceability and ultimate limit states, for the design of low-rise steel-framed buildings. These include residential and commercial low-rise buildings using New Zealand cold-formed framing components and methods (see Figure 1.1).

The design criteria shall be applicable for the steel framing of buildings that comply with the geometric limitations given in Figure 1.2.

For buildings outside the geometric limits given in Figure 1.2, but not exceeding 10m in height, the design actions shall be determined from the AS/NZS 1170 suite of standards.

Figure 1.1 Framing components

The ground floor, subfloor and foundations shall comply with NZS 3604 or NZS 4229 as appropriate. Alternatively they shall be a steel frame system to AS 4600 or NZS 3404 as appropriate subject to Specific Engineering Design that shall also consider bracing, connection strength and durability requirements.
Figure 1.2 Geometric limitations

NOTE: Requirements to support the compliance with other NZBC clauses can be found in the NASH handbook best practice for the design and construction of residential and low rise steel framing.
1.2 INTERPRETATION

The word “shall” denotes mandatory requirements for compliance with this Standard. The word “should” denotes requirements that are practices that are recommendations only.

In this Standard, notes provide guidance only and do not provide mandatory requirements.

Where other documents, that are themselves referenced or cited in regulations, legislation, or provide a legal means of demonstrating compliance with legislation are referred to by this Standard, they shall be considered along with any modifications made in their statutory incorporation by reference or citing.

Appendices may be either informative guidance or normative requirements as indicated.

Further guidance material on steel-framed housing is available from www.nashnz.org.nz.

1.3 REFERENCED DOCUMENTS

The following are referred to in this document:
• AS/NZS 1163: 2016 Cold-formed structural steel hollow sections
• AS/NZS 1170 Structural design actions
  Part 0: 2002 General principles, amendments 1,2,3,4,5.
  Part 1: 2002 Permanent, imposed and other actions, amendments 1,2.
  Part 2: 2011 Wind actions, amendments 1,2,3.
  Part 3: 2003 Snow and ice actions, amendment 1
• NZS 1170 Part 5: 2005 Earthquake Actions – New Zealand
• AS/NZS 1365: 1996 Tolerances for flat-rolled steel products
• AS 1397: 2011 Continuous hot-dip metallic coated steel sheet and strip - Coatings of zinc and zinc alloyed with aluminium and magnesium
• AS 3566.2: 2002 Self-drilling screws for the building and construction industries – Corrosion resistance requirements
• AS/NZS 3404: Part 1:2009 Steel structures Standard
• AS/NZS 3679.1: 2016 Structural steel – Hot-rolled bars and sections
• NZS 3604: 2011 Timber framed buildings
• AS/NZS 4600: 2005 Cold-formed steel structures
• The New Zealand Building Code including acceptable solutions and verification methods
• The NASH Standard Part 2: 2016 Non-specific Light Steel-framed Buildings
• The NASH handbook best practice for the design and construction of residential and low rise steel framing.
1.4 BASIS FOR DESIGN

1.4.1 General

The design criteria contained in this Standard is based on the AS/NZS 1170 series and AS/NZS 4600 specially formulated for low rise buildings using cold-formed steel-framing methods.

1.4.2 System-based assumption

The design criteria shall recognise the interactions between structural elements and other elements of the construction system.

When provision is made for the redistribution of loads, the load redistribution shall be accounted for by one of the following:

- calculation of the load redistribution factor $k_s$ (see Appendix B for calculation examples for concentrated loads in a grid system); or
- appropriate rational analysis of the system or the subsystem (such as finite element analysis), in such case $k_s = 1.0$; or
- prototype testing of the subsystem in accordance with Section 7.

NOTE: In other sections of this Standard, notes are used to indicate areas where there is potential for the application of system-based assumptions.

1.4.3 Durability

The design criteria assumes that the materials used their installation, and their maintenance will ensure that components will fulfil their intended structural function and will comply with the requirements of NZBC B2 for the intended design life of the structure.

They shall comply with AS 1397-2001: Steel sheet and strip – Hot-dipped zinc-coated or aluminium/zinc-coated and AS 3566.2–2002: Self-drilling screws for the building and construction industries – Corrosion resistance requirements.

The minimum requirements for framing in dry internal environments that should be applied are as follows:

- Galvanised 275g/m² (Z 275)
- Aluminium/zinc 150g/m² (AZ 150)

NOTE: For determination of the corrosivity for other environments, refer to Section 5 of NZS 3404.1:2009. Further guidance may also be found in the NASH Handbook best practice for the design and construction of residential and low rise steel framing.
1.5 DESIGN ACTIONS

1.5.1 General

Structural design actions shall be in accordance with AS/NZS 1170.0. Permanent, imposed and other actions, in general, shall be in accordance with AS/NZS 1170.1. Wind actions shall be in accordance with AS/NZS 1170.2.

Any other actions and combinations of actions shall also be considered using AS/NZS 1170 series.

In each situation, the combination of actions that produce the most severe action effect shall be used as the governing criteria. Where appropriate, different combinations of actions shall be considered for different action effects.

NOTES:
1. Construction loads may also become critical on certain components of an unfinished building. Guidance on appropriate load combinations for construction may be found in Appendix A.
2. Appendix E provides guidance for the determination of self-weight for some systems.

1.5.2 Determination of imposed actions

For the design of houses the following imposed actions shall apply:

1. For roofs not accessible except for normal maintenance:
   a. Uniformly distributed action – 0.25 kPa (Q1); and
   b. Concentrated action – 1.1 kN applied anywhere (Q2).

2. For general floor areas:
   a. Uniformly distributed action – 1.5 kPa (Q1); and
   b. Concentrated action – 1.8 kN (Q2).

3. Balconies and roofs used for floor type activities more than 1m above ground:
   a. Uniformly distributed action – 2.0 kPa (Q1); and
   b. Concentrated action – 1.8 kN (Q2); and
   c. Balcony edge action as follows:
      i. Residential – 0.35 kN/m run along top edge horizontal and vertical; or
      ii. Other than residential – 1.5 kN/m run along top edge horizontal and vertical.

4. For ceiling joists and supports:
   a. Concentrated action – 1.4kN where the member is required to support the force imposed by a person for any purpose; or
   b. Concentrated action – 0.9kN where the structural element is not required to support a person before the cladding is in place, and there is headroom of less than 1.2m after installation of the cladding.
For floors of other occupancy, the actions shall be determined in accordance with AS/NZS 1170.1.

1.5.3 Determination of wind actions and reference pressures

The wind actions and reference pressures shall be determined from 1.5.3.1 and 1.5.3.2.

1.5.3.1 Design wind speed and pressure for ultimate limit state

The design wind speed $V_u$ (in m/s) shall be determined as follows:

$$V_u = V_{des}, \theta$$

as defined in AS/NZS 1170.2.

Where:

$V_{des}, \theta$ is determined from regional wind speed ($V_R$) for the annual probability of exceedance as given in AS/NZS 1170.0 Section 3.

The reference pressure for the ultimate limit state shall be determined as follows:

$$q_u = 0.6(V_u)^2/1000 \text{ kPa}.$$

1.5.3.2 Design wind speed and pressure for serviceability limit state

The designed wind speed $V_s$ (in m/s) shall be determined as follows:

$$V_s = V_{des}, \theta$$

as defined in AS/NZS 1170.2.

Where:

$V_{des}, \theta$ is determined from regional wind speed ($V_R$) for the annual probability of exceedance as given in AS/NZS 1170.0 Section 3.

The reference pressure for the serviceability limit state shall be determined as follows:

$$q_s = 0.6(V_s)^2/1000 \text{ kPa}.$$

1.5.4 Determination of Earthquake actions

Earthquake actions shall be determined in accordance with NZS1170:5 as modified by B1/VM1 or 1.5.4.1;

1.5.4.1 Determination of earthquake design action coefficient

The earthquake design coefficient shall be determined as follows:

$$C_d = Z.C_h(T)S_p/k_\mu$$

Where:

$Z$ is the hazard factor and shall be as specified in NZS1170:5 as modified by B1/VM1.

$C_h(T)$ is the spectral shape factor and shall be as specified in NZS1170:5 or for site.

Subsoil class $C_h(0.4)$, may be taken as listed in Table 1.1.

<table>
<thead>
<tr>
<th>Site Subsoil</th>
<th>A (strong)</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
</table>

Table 1.1 Spectral shape factor
The Site Subsoil Classes shall be determined in accordance with NZS1170:5.

The structural ductility factor, $\mu$, the structural performance factor, $S_p$, and $k_\mu$ shall be determined in accordance with NZS1170:5 and the appropriate material standard or testing.

Alternatively, the values listed in Table 1.2 may be adopted for the applicable bracing system, provided the bracing system is designed and detailed in accordance with the capacity design principles of NZS1170.5.

### Table 1.2 Structural performance and ductility bracing system

<table>
<thead>
<tr>
<th>Bracing system</th>
<th>$\mu$</th>
<th>$S_p$</th>
<th>$k_\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>K – Brace</td>
<td>3</td>
<td>0.7</td>
<td>1.47</td>
</tr>
<tr>
<td>X - Brace *</td>
<td>3</td>
<td>0.7</td>
<td>1.47</td>
</tr>
<tr>
<td>Gypsum or Fibre Cement Board Panels</td>
<td>3</td>
<td>0.7</td>
<td>2.14</td>
</tr>
<tr>
<td>Plywood or OSB Wood Panels</td>
<td>4</td>
<td>0.7</td>
<td>2.71</td>
</tr>
<tr>
<td>Steel Sheet Panels</td>
<td>4</td>
<td>0.7</td>
<td>2.71</td>
</tr>
</tbody>
</table>

**NOTES:**
The $k_\mu$ factor for X- braced systems is an effective value taking into account the inelastic cyclic behaviour of this form of bracing system in accordance with NZS 3404.

Values obtained from P21 testing commissioned by NASH at BRANZ and Winstone wall boards.

The bracing systems shall be designed and detailed in accordance with Section 6.

#### 1.5.4.2 Determination of earthquake design base shear.

The earthquake design base shear shall be determined as follows:

$$V = C_d W_l$$

Where:

$W_l$ is the seismic weight of the structure defined as;

$$W_l = G + 0.3 Q_{floor}$$

$$W_l = G + 0.6 Q_{floor}$$ (for storage applications)

#### 1.5.4.3 Determination of earthquake design force at each level.

The earthquake design force shall be determined from NZS 1170.5 or as follows:

$$F_i = 1.2 \times W_i h_i \Sigma (W_i h_i)$$

Where:

$W_i$ = seismic mass at level $i$
\[ h_i = \text{height of level } i \]

**NOTE:**
The base shear force at each level has been magnified by a factor of 1.2 in lieu of specifically accounting for accidental eccentricity.

### 1.5.4.4 Application of design actions

Design forces determined in accordance with this document may act through the centre of mass at each level and to act separately along two orthogonal principal bracing directions.

### 1.5.5 Evaluation of Overstrength Forces.

Components and connections intended to remain elastic during an earthquake shall be designed for the forces determined based on the overstrength capacities of the principal ductile components, but need not be taken as greater than the actions evaluated for a nominally ductile system \((\mu = 1.25)\).

Overstrength actions on connections and components shall be determined as specified in AS/NZS 1170 and the appropriate material standards, or may be taken as the actions evaluated for the design earthquake actions, magnified by the overstrength factor, \(\Omega\), in Table 1.3.

#### Table 1.3 Structural over strength factor bracing system

<table>
<thead>
<tr>
<th>Bracing system</th>
<th>Overstrength factor ((\Omega))</th>
</tr>
</thead>
<tbody>
<tr>
<td>K – Brace</td>
<td>1.5</td>
</tr>
<tr>
<td>X – Brace</td>
<td>1.5</td>
</tr>
<tr>
<td>Gypsum or Fibre Cement Board Panels</td>
<td>2.0</td>
</tr>
<tr>
<td>Plywood or OSB Wood Panels</td>
<td>2.0</td>
</tr>
<tr>
<td>Steel Sheet Panels</td>
<td>2.0</td>
</tr>
</tbody>
</table>

### 1.5.6 Determination of snow loads

Snow shall be determined in accordance with AS/NZS1170.3 for Subalpine and Alpine regions.

### 1.6 DESIGN PROPERTIES

#### 1.6.1 Material properties

Material properties used in design shall be in accordance with 1.6.1.1 or 1.6.1.2.

#### 1.6.1.1 Steels compliant with AS/NZS 4600

Steels that comply with AS/NZS 4600 shall also meet the following:

For steels conforming to AS 1397 Grade G550, the design yield stress \((f_y)\) and tensile strength \((f_u)\) shall be:

1. 90% of the specified values or 495 MPa, whichever is the lesser for a steel base metal thickness (BMT) of less than 0.9 mm; or
2. 75% of the specified values or 410 MPa, whichever is the lesser for a steel BMT of less than 0.6 mm.

For standard gauges in use, the following design values shall be applicable for grade G500 and G550:

- 0.55 BMT \( f_y = 410, f_u = 410 \)
- 0.75 BMT \( f_y = 495, f_u = 495 \)
- 0.95 BMT \( f_y = 550, f_u = 550 \)
- 1.15 BMT \( f_y = 500, f_u = 520 \)

1.6.1.2 Steels that do not comply with AS/NZS 4600

Steels that do not comply with the standards listed in AS/NZS 4600 shall be permitted to be used for the design and construction of cold-formed steel provided that they comply with the following requirements:

1. The ratio of tensile strength to yield stress shall be not less than 1.08.
2. The total elongation shall be not less than 10% for a 50 mm gauge length or 7% for a 200 mm gauge length standard specimen tested in accordance with AS 1391.

Unidentified steel shall be permitted provided that:

1. It shall be free from surface imperfections;
2. It shall be used only where the particular physical properties of the steel and its weldability will not adversely affect the design capacities and serviceability of the structure; and
3. The yield stress of the steel used in design \( f_y \) shall be 170 MPa or less, and the tensile strength used in design \( f_u \) shall be 300 MPa or less unless a full test in accordance with AS 1391 is made. Sufficient evidence of compliance with the standards referred to in this standard shall be required. This may include certified mill or test certification issued by the mill and/or third party verification tests.

1.6.2 Section properties

Section properties used in design shall be obtained in accordance with AS/NZS 4600 or evaluated from tests according to Section 7.

Service holes in members shall be taken into consideration in design (see Appendix F).

1.6.3 Tolerances

Manufacturing tolerances of components shall be in accordance with Appendix D of this Standard.

Construction tolerances shall be in accordance with Appendix D of this Standard.
1.7 DESIGN CRITERIA

1.7.1 Stability

The building, as a whole and as its parts, shall be designed to prevent instability due to overturning, uplift and sliding in accordance with AS/NZS 1170.0.

1.7.2 Strength

The design action for the strength limit state shall be the combination of (factored) actions which produces the most adverse effect on the building, as determined from, but not limited to, the combinations given in Section 2, 3 and 4 of this Standard.

NOTE: Only combinations of actions usually deemed as potentially critical have been included in the design criteria in Section 2, 3 and 4. AS/NZS 1170.0 provides further information for other situations.

1.7.3 Serviceability

The design criteria for serviceability shall be taken from, but not limited to, the criteria given in Section 2, 3 and 4 of this Standard.

NOTE: The design criteria have been determined on the basis of experience. The serviceability limits are intended to provide satisfactory service for the typical situations. AS/NZS 1170.0 provides further advice for other situations.

1.8 METHODS OF ASSESSMENT

1.8.1 General

The assessment shall be carried out by one of the following methods:

1. Calculation; or
2. Testing; or a
3. Combination of calculation and testing.

1.8.2 Calculation

Calculations shall be based on appropriate structural models for the strength or serviceability limit states under consideration. Allowance for the system effects may be considered when appropriate.

The method of structural analysis shall take into account equilibrium, general stability and geometric compatibility.

The combinations of actions shall include all appropriate combinations outlined in this Standard.

The design properties shall be in accordance with Clause 1.6. The design capacities shall be determined in accordance with AS/NZS 4600.
1.8.3 Testing

Only prototype testing on full size members or sub-assemblies in accordance with Section 7 shall be used in assessment.

1.8.4 Combination of calculation and testing

A combination of testing and calculation based on appropriate structural model may be used in assessment.
2. **ROOF MEMBERS**

2.1 **GENERAL**

All roof members including roof battens, roof trusses or rafters, ceiling battens and bracing shall be designed to act together as a structural unit to transfer all the actions imposed on the roof to appropriate supports (see Figure 2.1).

![Figure 2.1 Typical roof assembly](image)

2.2 **ROOF BATTENS**

2.2.1 **Design for strength**

The combinations used for the determination of the design action effects for strength shall be as follows:

1. $1.35G$
2. $1.2G + 1.5Q_2$
3. $0.9G + W_u \text{ (up)}$
4. $1.2G + W_u \text{ (down)}$
5. $1.2G + 1.0F_{sn} \text{ (snow)}$

Where:

- $G =$ permanent actions including the weight of roofing, battens and insulation
- $Q =$ imposed actions (due to occupancy and use)
- $Q_2 =$ 1.1 kN
- $q_u =$ reference pressure, in kPa, for the ultimate limit state
- $F_{sn} =$ snow action calculation determined in accordance with AS/NZS 1170.3
NOTES:
1. Guidance for the determination of roof weight may be found in Appendix E.
2. \( Q_2 \) may be shared with adjacent battens due to system effect (see Appendix B).
3. For the overhang portion of roofs, \( Q_2 \) is to be applied 100mm from end.

The ultimate wind action, in kN/m, shall be determined as follows:

\[
W_u = q_u C_{pt} S
\]

Where:
\( Q_u \) = reference pressure, in kPa
\( C_{pt} \) = net pressure coefficient determined in accordance with AS/NZS 1170.2
\( S \) = spacing of roof battens, in metres

The referenced wind pressure, in kPa, shall be determined as follows:

\[
Q_u = 0.6(V_u)^2/1000
\]

Where
\( V_u \) = as defined in 1.5.3.1

2.2.2 Design for serviceability

Design for serviceability under the issue of concern, the calculated value of the serviceability parameter under the nominated actions shall be kept within the limiting value of the response given in Table 2.1.

**Table 2.1 Serviceability response limits – roof battens**

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Nominated Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/300</td>
<td>Batten deflection</td>
</tr>
<tr>
<td></td>
<td>Cantilever Deflection (Δ)</td>
<td></td>
<td>L/150</td>
<td></td>
</tr>
<tr>
<td>Comfort</td>
<td>Mid-span Deflection (Δ)</td>
<td>( Q_2 )</td>
<td>L/150</td>
<td>Batten deflection</td>
</tr>
<tr>
<td></td>
<td>Cantilever Deflection (Δ)</td>
<td></td>
<td>L/75</td>
<td></td>
</tr>
<tr>
<td>Comfort</td>
<td>Mid-span Deflection (Δ)</td>
<td>( W_s )</td>
<td>L/150</td>
<td>Batten deflection</td>
</tr>
<tr>
<td></td>
<td>Cantilever Deflection (Δ)</td>
<td></td>
<td>L/75</td>
<td></td>
</tr>
</tbody>
</table>

NOTES:
For flat or near flat roofs, effects of ponding should be considered.
\( L \) = span of batten (mm)
\( G \) = permanent actions including weight of roofing, battens and insulation
\( Q_2 \) = 1.1 kN concentrated roof imposed action

The serviceability wind action, in kN/m, shall be determined as follows:

\[
W_s = q_s C_{pt} S
\]

Where
\( q_s \) = The reference pressure, in kPa, for the serviceability limit state. This shall be determined as follows:

\[
q_s = 0.6\ (V_s)^2/1000
\]
Where

\[ V_s = \text{the designed wind speed and shall be as defined in 1.5.3.2} \]

\[ C_{pt} = \text{net pressure coefficient determined in accordance with AS/NZS 1170.2} \]
\[ S = \text{spacing of roof battens, in metres} \]

NOTES:
Guidance for the determination of roof weight may be found in Appendix E.
\( Q_2 \) may be shared due to system effect (see Appendix B).

### 2.3 ROOF TRUSSES OR RAFTERS

#### 2.3.1 Design for strength

The combinations used for the determination of the design action effects for strength shall be as follows:

1.35\( G \)
1.2 \( G + 1.5 \, Q_1 \)
1.2 \( G + 1.5 \, k_s \, Q_2 \)
0.9 \( G + W_u \, \text{(up)} \)
1.2 \( G + W_u \, \text{(down)} \)
1.2 \( G + 1.0 \, F_{sn} \, \text{(snow)} \)

Where:

\( G = \text{permanent actions of the complete roofing system including the weight of roofing, battens, insulation, ceiling, ceiling battens, trusses or rafters and services as appropriate} \)

\( Q_1 = 0.25 \, \text{kPa} \) for general housing applications. For other design conditions, AS/NZS 1170.1 shall apply

\( Q_2 = 1.4 \, \text{kN} \) applied to any point on the top or bottom chord, wherever it will have the worst effect. A load distribution factor \((k_s)\) shall be applied to \( Q_2 \) due to the system effect as determined by the calculation method for \( k_s \) in Appendix B. Alternatively a \( k_s \) factor of 0.5 may be used where continuous purlins or battens are fixed to the chord. For dwellings, a 1.1 \, \text{kN} \) applied load shall be used

The ultimate wind action, in \( \text{kN/m} \), shall be determined as follows:

\[ W_u = q_u \, C_{pt} \, S \]

Where:

\( q_u = \text{reference pressure, in \text{kPa} for the ultimate limit state (see 1.5.3.2)} \)

\( C_{pt} = \text{net pressure coefficient as given in Table 2.2} \)

\( S = \text{spacing of roof trusses or rafters, in metres} \)

\( F_{sn} = \text{Snow action calculation determined in accordance with AS/NZS 1170.3.} \)
Table 2.2 Net pressure coefficient (C<sub>pt</sub>) for strength

<table>
<thead>
<tr>
<th>Members</th>
<th>Net Pressure Coefficient (C&lt;sub&gt;pt&lt;/sub&gt;)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trusses</td>
<td>-1.0, + 0.36</td>
</tr>
<tr>
<td>Rafters</td>
<td>-1.0, + 0.63</td>
</tr>
</tbody>
</table>

NOTES:
1. The values of C<sub>pt</sub> are based on internal pressure coefficients of ±0.2, -0.3.
2. Specific identifiable concentrated loads such as hot water systems placed in the roof space or on the roof should be allowed for where required.
3. For permeability conditions different from those assumed, internal pressure coefficients should be obtained from AS/NZS 1170.2. The above pressure coefficients are applicable for buildings without dominant openings and with equally permeable walls or, 2 or 3 walls equally permeable, other walls impermeable.
4. For the design of the bottom chord, consideration should be given to the effect of internal pressure on the bottom chord in terms of bending action between nodal points.
5. The combined action of bending and axial load on the truss chords needs to be considered.

2.3.2 Design for serviceability

Design for serviceability under the issue of concern shall be the calculated value of the parameter under the nominated action and be kept within the limiting value of the response, as shown in Table 2.3.

Table 2.3 Serviceability response limits – trusses & rafters

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Nominated Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual sagging</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/300 (max 20mm)</td>
<td>Truss top chord or rafter</td>
</tr>
<tr>
<td>Visual sagging</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/300 (max 12mm)</td>
<td>Truss bottom chord or ceiling joist</td>
</tr>
<tr>
<td>Cracking of</td>
<td>Mid-span Deflection (Δ)</td>
<td>Q&lt;sub&gt;2&lt;/sub&gt;</td>
<td>d/250</td>
<td>Truss bottom chord</td>
</tr>
<tr>
<td>ceiling comfort</td>
<td></td>
<td>Q&lt;sub&gt;1&lt;/sub&gt; or Q&lt;sub&gt;2&lt;/sub&gt;</td>
<td>d/200 L/250</td>
<td>Truss top chord or Rafter</td>
</tr>
<tr>
<td>Comfort</td>
<td>Mid-span Deflection (Δ)</td>
<td>W&lt;sub&gt;s&lt;/sub&gt;</td>
<td>L/150</td>
<td>Truss or rafter deflection</td>
</tr>
<tr>
<td>Visual</td>
<td>Differential Mid-span</td>
<td>G</td>
<td>S/150 (&lt;4 mm)</td>
<td>Vertical deflection between adjacent trusses or rafters or other ceiling supports</td>
</tr>
</tbody>
</table>

NOTES:
For cantilever, the limit of response may be taken as twice that of mid-span deflection.
L = span of the truss or rafter (mm).
S = spacing of trusses or rafters.
G = permanent actions of the complete roofing system including the weight of roofing, battens, insulation, ceiling, ceiling battens, trusses or rafters and services (where appropriate).
d = distance between nodal/support points, millimetres.
The $Q_1$ and $Q_2$ actions for the design for serviceability response limits for trusses and rafters shall be determined as follows:

$$Q_1 = 0.25 \text{ kPa}$$ for general housing applications. For other design conditions, AS/NZS 1170.1 shall apply.

$$Q_2 = 1.1 \text{ kN}$$ applied to any point on the top or bottom chord, wherever it will have the worst effect. A load distribution factor ($k_s$) shall be applied to $Q_2$ due to the system effect as determined by the calculation method for $k_s$ in Appendix B. Alternatively a $k_s$ factor of 0.5 may be used where continuous purlins or battens are fixed to the chord. For dwellings, a 1.1 kN applied load shall be used.

The serviceability wind action, in kN/m, shall be determined as follows:

$$W_s = q_s C_{pt} S$$

Where:

$q_s$ = reference pressure, in kPa, for the serviceability limit state (see 2.2.2)

$C_{pt}$ = net pressure coefficient as given in Table 2.4

$S$ = spacing of roof battens (m)

**Table 2.4 Net pressure coefficient ($C_{pt}$) for serviceability**

<table>
<thead>
<tr>
<th>Members</th>
<th>Net Pressure Coefficient ($C_{pt}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trusses</td>
<td>-1.0, + 0.36</td>
</tr>
<tr>
<td>Rafters</td>
<td>-1.0, + 0.63</td>
</tr>
</tbody>
</table>

**NOTES:**
1. These values of $C_{pt}$ are based on internal pressure coefficients of +0.2,-0.3.
2. Specific identifiable concentrated loads such as hot water systems placed in the roof space or on the roof should be allowed for where required.
3. For permeability conditions different from those assumed, internal pressure coefficients should be obtained from AS/NZS 1170.2. The above pressure coefficients are applicable for buildings without dominant openings and with equally permeable walls or 2 or 3 walls equally permeable and other walls impermeable.

### 2.4 CEILING BATTENS

#### 2.4.1 Design for strength

The load combinations used for the determination of the design action effects for strength are as follows:

$$0.9 \ G + W_u \ (up)$$

$$1.2 \ G + W_u \ (down)$$

Where:

$G$ = permanent actions including weight of ceiling and insulation (if applicable)

**NOTE:** Guidance for the determination of roof weight may be found in Appendix E.
The ultimate wind action, in KN/m, shall be determined as follows:

\[ W_u = q_u C_{pt} S \]

Where:
\[ q_u = \text{reference pressure, in kPa, for the ultimate limit state (see 1.5.3.1).} \]
\[ S = \text{spacing of ceiling battens, in metres} \]
\[ C_{pt} = +0.2 \text{ or } -0.3 \]

2.4.2 Design for serviceability

Design for serviceability under the issue of concern shall be the calculated value of the parameter under the nominated action within the limiting value given in Table 2.5.

NOTE: For plasterboard ceilings, these limits correspond to a Level 4 finish to AS/NZS 2589.

Table 2.5 Serviceability response limits – ceiling battens

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ripple</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/500</td>
<td>Ceiling with matt or gloss paint finish</td>
</tr>
<tr>
<td>Ripple</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/300</td>
<td>Ceiling with textured finish</td>
</tr>
<tr>
<td>Ripple</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/360</td>
<td>Suspended Ceiling</td>
</tr>
<tr>
<td>Sag</td>
<td>Mid-span Deflection (Δ)</td>
<td>G</td>
<td>L/360</td>
<td>Ceiling support framing</td>
</tr>
<tr>
<td>Cracking</td>
<td>Mid-span Deflection (Δ)</td>
<td>G + W_s</td>
<td>L/200</td>
<td>Ceiling with plaster finish</td>
</tr>
</tbody>
</table>

NOTES:
For cantilever, the limit of response may be taken as twice that of mid-span deflection.
\[ L = \text{span of the ceiling batten, in millimetres.} \]
\[ G = \text{permanent actions of the complete roofing system including the weight of roofing, battens, insulation, ceiling, ceiling battens, trusses or rafters and services (where appropriate).} \]

The serviceability wind action, in kN/m, shall be determined as follows:

\[ W_s = q_s C_{pt} S \]

Where:
\[ q_s = \text{reference pressure, in kPa, for the serviceability limit state (see 2.2.2)} \]
\[ C_{pt} = \text{net pressure coefficient, given as } +0.2 \text{ or } -0.3 \text{ for serviceability} \]
\[ S = \text{spacing of roof battens (m)} \]

2.5 ROOF CONNECTIONS AND BRACING

Roof connections shall be designed in accordance with Section 5.

Roof bracing shall be designed in accordance with Section 6.
3. WALL MEMBERS

3.1 GENERAL

All wall members including load bearing wall studs, wall plates, posts, lintels and bracing shall be designed to act together as a structural unit to transfer all the actions imposed on the roof and walls to appropriate supports (see Figure. 3.1).

*Figure 3.1 Components of typical wall assembly*

For foundations refer to Section 1.1

Noggings, if required to provide lateral supports for the studs or for fixing of external cladding or internal lining, shall be designed to suit their intended purposes.
3.2 LOAD BEARING WALL STUDS

3.2.1 Load paths

Load bearing wall studs include:

- Common studs: These studs support the vertical loads applied to the top wall plate by rafters or trusses, ceiling joists and horizontal loads due to wind.
- Jamb studs: These studs are provided on each side of an opening. They support loads from lintel over the opening and the horizontal wind load across the width of the opening.
- Studs supporting concentrated loads: These studs are installed in addition to common studs and/or jamb studs to carry concentrated vertical loads arising from support for principal roof or floor supporting members.
- Load bearing wall studs shall be designed to transfer tension or compression loads from supported floors or roofs and to transfer horizontal wall loads in bending to top and bottom wall supports.

Wind action effects for studs shall include the combination of axial loads from wind pressure on roofs \( W_{ur} \) and uniformly distributed lateral loads from wind pressure on walls \( W_{uw} \).

3.2.2 External load bearing wall studs for single storey or upper storey of a two storey construction

3.2.2.1 Design for strength

The load combinations used for the determination of the design action effects for the strength of wall studs shall be as follows:

1.35G
1.2 G + 1.5 Q₁
1.2 G + 1.5 Q₂
1.2 G + \( W_{uw} + W_{ur} \, \text{(down)} \)
0.9 G + \( W_{uw} + W_{ur} \, \text{(up)} \)
1.2 G + 1.0 F_{sn} \, (snow)

Where:

\( G = \) dead load of roof structure, includes roof structure, roof cladding, roof battens, ceiling battens, ceiling, services and roof insulation if appropriate.

\( Q₁ = \) roof live load
\( = 0.25 \) kPa

\( Q₂ = 1.1 \) kN

\( W_{uw} = \) wind load normal to wall
\( = q_u \, C_{pw} \, A_w \)
Where:

\[ q_u = \text{reference pressure, in kPa, for the ultimate limit state (refer 1.4.3.1)} \]

\[ C_{pw} = \text{net pressure coefficient as given in Table 3.1.} \]

\[ A_w = \text{wall area for wind action supported by the stud} \]
\[ = L_w S_s \]

Where:

\[ L_w = \text{length of the stud} \]

\[ S_s = \text{spacing between studs for common studs} \]

\[ F_{sn} = \text{snow action calculation determined in accordance with AS/NZS 1170.3} \]

**NOTE:**

1. Guidance for the determination of roof mass may be found in Appendix E.
2. Where wind pressures acting on two or more surfaces of an enclosed building contribute simultaneously to a structural action effect in a member, action combination factors \( K_{c,e} \) and \( K_{c,i} \) (AS/NZS1170.2) may be applicable to external and internal pressure coefficients, respectively.
3. Wall studs may also be subject to additional compression due to racking forces.
4. Windows and doors, including garage roller doors, should be designed to withstand the same ultimate wind actions as the wall in which they are installed otherwise a dominant opening shall be designed for. Manufacturers should provide product installation details to achieve appropriate performance.

**Table 3.1 Net pressure coefficient \( (C_{pw}) \) for strength**

<table>
<thead>
<tr>
<th>Net Pressure Coefficient ( (C_{pw}) )</th>
<th>1.0</th>
</tr>
</thead>
</table>

**NOTES:**

The values of \( C_{pw} \) are based on internal pressure coefficients of -0.3.

The value for \( A_w \) may have to be modified for studs beside openings or other studs of non-standard spacing.

Check AS/NZS 1170.2 to confirm this applies – Otherwise use the coefficients appropriate to the application

\[ W_{ur} = \text{wind load on roof} \]
\[ = q_u C_{pr} A_r \]

Where:

\[ q_u = \text{reference pressure, in kPa, for the ultimate limit state} \]

\[ C_{pr} = \text{net pressure coefficient as given in Table 3.2.} \]

\[ A_r = \text{area of roof supported by stud in square metres} \]
\[ = L_r S_r / 2 \]

\[ L_r = \text{span of roof trusses supported by stud in metres} \]

\[ S_r = \text{the greater of the truss spacing or the wall stud spacing in metres} \]
Table 3.2 Net pressure coefficient ($C_{pr}$) for strength

<table>
<thead>
<tr>
<th>Net Pressure Coefficient ($C_{pr}$)</th>
<th>+0.7, -1.1</th>
</tr>
</thead>
</table>

**NOTES:**
1. The values of $C_{pr}$ are based on internal pressure coefficients of +0.2, -0.3.
2. For permeability conditions different from those assumed, internal pressure coefficients should be obtained from AS/NZS 1170.2. These are applicable for buildings without dominant and with equally permeable or 2 or 3 walls equally permeable. Other walls impermeable.

### 3.2.2.2 Design for serviceability

Design for serviceability under the issue of concern shall be the calculated value of the parameter under the nominated action, within the limiting value of the response given in Table 3.3.

Table 3.3 – Serviceability response limits – external walls, single/upper storey

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discerned movement</td>
<td>Mid-height Deflection ($\Delta$)</td>
<td>$W_s$</td>
<td>$H/150$ ($&lt;20$ mm)</td>
<td>Face loading Non brittle claddings</td>
</tr>
<tr>
<td>Discerned movement</td>
<td>Mid-height Deflection ($\Delta$)</td>
<td>$W_s$</td>
<td>$H/400$</td>
<td>Face loading Masonry or brittle cladding</td>
</tr>
<tr>
<td>Impact</td>
<td>Mid-height Deflection ($\Delta$)</td>
<td>$Q$</td>
<td>$H/200$ ($&lt;12$ mm)</td>
<td>Soft body impact on wall</td>
</tr>
</tbody>
</table>

**NOTE:** These limits have been used satisfactorily with brick veneer and ceramic tiled walls.

Where:

$$W_s = \text{wind load normal to the wall}$$

$$= q_s C_{pw} A_w$$

Where:

$$q_s = \text{reference pressure, in kPa for serviceability limit state}$$

$$C_{pw} = +1.0$$

$$A_w = H S_s$$

$$H = \text{height of wall in metres}$$

$$S_s = \text{spacing of studs in metres}$$

$$Q = 0.7 \text{ kN}$$

**NOTE:** Deflection limit is applicable where deflection of walls is evaluated taking consideration of the stiffness of masonry or cladding H/400 or H/150
3.2.3 External load bearing wall studs for lower storey of two storey construction

3.2.3.1 Design for strength

The load combinations used for the determination of the design action effects for the strength of wall studs shall be as follows:

1.35G  
1.2 G + 1.5 Q  
1.2 G + 0.4 Q + (W_{uw} + W_{ur \ (down)})  
0.9 G + (W_{uw} + W_{ur \ (up)})  
1.2 G + 1.0 F_{sn \ (snow)}

Where:

G = dead load, includes roof structure, roof cladding, roof battens, ceiling battens, ceiling, upper storey walls, upper storey floor, services and roof insulation if appropriate.

Q = floor live load = 1.5 kPa for residential or in accordance with NZS 1170.1 for other occupancies

W_{uw} = wind load normal to wall (kN)  
= q_u \ C_{pw} A_w

Where

q_u = reference pressure, in kPa, for the ultimate limit state
C_{pw} = net pressure coefficient as given in Table 3.4.
A_w = wall area for wind action supported by the stud = L S_s

L = length of the stud
S_s = spacing between studs for common studs
F_{sn} = snow action calculation determined in accordance with AS/NZS1170.3

NOTES:
1. Where wind pressures acting on two or more surfaces of an enclosed building contribute simultaneously to a structural action effect in a member, action combination factors K_{c,e} and K_{c,i} (AS/NZS1170.2 Clause 5.4.3 and Table 5.5) may be applicable to external and internal pressure coefficients, respectively.
2. Wall studs may also be subject to additional compression due to racking forces.
3. Guidance for the determination of roof mass may be found in Appendix E.

Table 3.4 Net pressure coefficient (C_{pw}) for strength

<table>
<thead>
<tr>
<th>Net Pressure Coefficient (C_{pw})</th>
</tr>
</thead>
<tbody>
<tr>
<td>+1.0</td>
</tr>
</tbody>
</table>

NOTES:
1. The values of C_{pw} are based on internal pressure coefficients of -0.3.
2. The value for A_w may have to be modified for studs beside openings or other studs of non-standard spacing.

W_{ur} = wind load on roof
\[ q_u \, C_{pr} \, A_r \]

Where:
\[ q_u = \text{reference pressure, in kPa, for the ultimate limit state} \]
\[ C_{pr} = \text{net pressure coefficient as given in Table 3.5.} \]
\[ A_r = \text{area of roof supported by stud in square metres} = \frac{L_r \, S_r}{2} \]

\[ L_r = \text{span of roof trusses supported by stud, in metres} \]
\[ S_r = \text{the greater of the truss spacing or the wall stud spacing, in metres} \]

### Table 3.5 Net pressure coefficient \((C_{pr})\) for strength

<table>
<thead>
<tr>
<th>Net Pressure Coefficient ((C_{pr}))</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>+0.7, -1.1</td>
<td>NOTE: The values of (C_{pr}) are based on internal pressure coefficients of +0.2, -0.3.</td>
</tr>
</tbody>
</table>

### 3.2.3.2 Design for serviceability

Design for serviceability under the issue of concern shall be the calculated value of the parameter under the nominated action kept within the limiting value of the response as given in Table 3.6.

### Table 3.6 Serviceability response limits – external walls, lower of 2 storey

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discerned movement</td>
<td>Mid-height Deflection ((\Delta))</td>
<td>(W_s)</td>
<td>(H/150) (&lt;20 mm)</td>
<td>Face loading Non brittle claddings</td>
</tr>
<tr>
<td>Discerned movement</td>
<td>Mid-height Deflection ((\Delta))</td>
<td>(W_s)</td>
<td>(H/400)</td>
<td>Face loading Masonry or brittle cladding</td>
</tr>
<tr>
<td>Impact</td>
<td>Mid-height Deflection ((\Delta))</td>
<td>(Q)</td>
<td>(H/200) (&lt; 12 mm)</td>
<td>Soft body impact on wall</td>
</tr>
</tbody>
</table>

NOTE: These limits have been used satisfactorily with brick veneer and ceramic tiled walls.

Where:
\[ W_s = \text{wind load normal to the wall} \]
\[ = q_s \, C_{pw} \, A_w \]

Where:
\[ q_s = \text{reference pressure, for the serviceability limit state} \]
\[ C_{pw} = +1.0 \text{ for both non-cyclonic and cyclonic regions} \]
\[ A_w = \text{height of wall, in metres} \]
\[ S_s = \text{spacing of studs, in metres} \]
\[ Q = 0.7 \text{ kN} \]
3.2.4 Internal load bearing wall studs

Design criteria for internal load bearing wall studs shall be similar in principle to external load bearing wall studs. Wind action normal to the wall is limited to differential pressure between the wall faces. That is, an assumed $C_{pw}$ of 0.5

NOTE: External wind action effects from the roof may be ignored.

3.3 NON LOAD BEARING STUDS

3.3.1 Load path

Non load bearing studs are defined here as wall studs that are not required to carry gravity loads, other than their own self-weight. These studs are, however, expected to carry any lateral loads such as wind loads, impact loads or internal pressures and shall be designed accordingly.

3.3.2 Design for strength

Non load bearing studs shall be designed as follows:
1. External non load bearing studs shall be designed for the full wind load normal to wall ignoring the external wind action effects arising from the roof.
2. Internal non load bearing studs shall be designed for the differential pressure between the wall faces ignoring the external wind action effects arising from the roof but internal pressure shall be accounted for if relevant.

3.3.3 Design for serviceability

The serviceability requirements for a non load bearing stud shall be the same as those for a load-bearing stud (see Section 3.2)

3.4 NOGGING

Nogging shall be designed to provide lateral and torsional restraints to the studs.

Nogging shall be designed to support an imposed concentrated load of 1.1 kN placed anywhere on its span to produce the maximum action effects during construction (See Appendix A).

3.5 WALL PLATES FOR LOAD BEARING WALLS

3.5.1 Load path

Load bearing wall plates shall be designed to transfer vertical loads. Wall plates may also need to be designed to transfer horizontal loads laterally to brace walls. Ceiling and floor diaphragms are assumed to transfer any horizontal loads when used in the design.

NOTES:
1. The reaction due to roof or floor loads may be ignored in the design of the plates if the system is such that the loads are transferred directly into the studs.
2. Where wall studs are aligned with roof trusses or floor joists, care should be taken to ensure that local crushing does not occur at bearing locations.
3. Loads due to self-weight of the wall system may result in out-of-plane loads on wall panels during the fabrication and construction processes. These loads may be critical in some plates.

4. While the plates may be required to carry horizontal loads such as wind loads, these loads will be transferred into other members such as the floor or roof trusses which will limit the spans and corresponding loads in most cases.

### 3.5.2 Design model

Wall plates shall be designed as continuous beams of three equal spans (L) to support a series of concentrated loads (P) with load spacing (S) as given in Table 3.7.

#### Table 3.7 Load spacing (S) and span (L) for wall plates

<table>
<thead>
<tr>
<th>Applications</th>
<th>Load Spacing (S)</th>
<th>Span (L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper storey or single</td>
<td>Top plate</td>
<td>Rafter or truss spacing</td>
</tr>
<tr>
<td></td>
<td>Bottom plate</td>
<td>Stud spacing</td>
</tr>
<tr>
<td>Lower storey of two</td>
<td>Top plate</td>
<td>Upper floor joist spacing</td>
</tr>
<tr>
<td></td>
<td>Bottom plate</td>
<td>Stud spacing in lower wall</td>
</tr>
</tbody>
</table>

#### 3.5.3 Design for strength

The magnitude of the load P is the maximum reactions (up and down) obtained from the members that determined the load spacing.

This load shall be placed at mid-span for the determination of the bending action effects and at 1.5 x depth of the plate from the support for the determination of the shear action effects (see Figure 3.2).

#### Structural model

For determination of design action effect in bending

For determination of design action in shear

![Structural models for wall plates](Figure 3.2)

**LEGEND:**

<table>
<thead>
<tr>
<th>S</th>
<th>L</th>
<th>D</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td>load spacing</td>
<td>span</td>
<td>depth of plate</td>
<td>concentrated load</td>
</tr>
</tbody>
</table>

**Figure 3.2 Structural models for wall plates**

The value of P for a single or upper storey shall be determined from the following load combinations:
ALTERNATIVE SOLUTION: NASH STANDARD PART 1: 2016

1.35G
1.2 G₁ + 1.5 Q₁
1.2 G₁ + 1.5 Q₂
0.9 G₁ + Wₜₐₜ (up)
1.2 G₁ + Wₜₐₜ (down)
1.2 G₁ + Wₜₜ + Wₜₐₜ (down)
1.2 G + 1.0 Fₛₙ (snow)

Where

G₁ = weight of complete roof
Q₁ = 0.25 kPa
Q₂ = 1.1 kN
Wₜₚ = wind load on roof
Wₜₜ = wind load normal to wall
Fₛₙ = Snow action calculation determined in accordance with AS/NZS1170.3

The value of P for a lower storey shall be determined from the following load combinations:

1.35G
1.2 G₁ + 1.5 Q₁
1.2 G₁ + 1.5 Q₂
0.9 G₁ + Wₜₚ (up)
1.2 G₁ + Wₜₚ (down)
1.2 G₁ + Wₜₜ + Wₜₚ (down)
1.2 G + 1.0 Fₛₙ (snow)

Where

G₁ = weight of complete roof and upper floor
Q₁ = 0.25 kPa roof imposed action
Q₂ = 1.5 kPa floor imposed action
Wₜₚ = wind load on roof
Wₜₜ = wind load normal to wall
Fₛₙ = Snow action calculation determined in accordance with AS/NZS1170.3

3.5.4 Design for serviceability (for upper storey and lower storey wall plates)

For design for serviceability under the issue of concern the calculated value of the parameter under the nominated action shall be kept within the limiting value of the response as given in Table 3.8.

Table 3.8 Serviceability response limits for upper and lower storey wall plates

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sagging or uplift at mid span</td>
<td>Maximum Deflection (Δ)</td>
<td>Pₛ</td>
<td>L/200 (&lt;3 mm)</td>
<td>For single storey or upper storey top plate Pₛ arising from G₁ or 0.9 G₁ + Wₜₚ (up)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>For lower storey top plate Pₛ arising from G₁ + Q₂</td>
</tr>
</tbody>
</table>
3.6 LINTELS

3.6.1 Load path

Lintels shall be designed to transfer the vertical loads applied over the opening to the jamb studs on the sides of the opening.

Lintels in single or upper storey walls are designed to support rafters, trusses or any other load carrying members that are located over the opening. (see Figure 3.3)

![Figure 3.3 Single or upper storey lintel](image)

Lintels in lower storey walls of two-storey construction are designed to support the loads from the wall above including the roof loads and the floor loads from the storey above. (see Figure. 3.4)

Lintels should be designed as part of a system that includes top wall plates and other structural components located directly above and connected to the lintel.
3.6.2 Design for strength

3.6.2.1 Single storey or upper storey lintels

Lintels in single storey or upper storey walls shall be designed to support a series of equally spaced concentrated loads $P_1$ from the roof trusses or rafters via the studs.

The magnitudes of the loads $P_1$ are the maximum reactions of the members that the lintel has to support across the opening.

These loads $P_1$ shall be placed at mid-span for the determination of the bending action effects and at $1.5 \times$ depth of the plate from the support for the determination of the shear action effects (see Figure 3.5).

NOTE: Lintels may also be required to support additional concentrated roof loads. ($P_2$)

<table>
<thead>
<tr>
<th>Structural model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design action effect</td>
</tr>
<tr>
<td>Common lintels</td>
</tr>
</tbody>
</table>

*Figure 3.4 Lower storey lintel*
3.6.2.2 Lower storey lintels

For lintels in lower storey walls, the loads from the roof, wall and floor above should be considered as a uniformly distributed load (W).

Its magnitude is determined from the maximum reactions of the members that the lintel has to support across the opening.

NOTE: Lintels are not normally designed to carry the wind load normal to the wall arising from the opening. These loads are normally transferred to the jamb studs on both sides of the opening and from there to the floor and ceiling diaphragms.

3.6.2.3 Load combinations

The magnitudes of P₁, P₂ and W for a single or upper storey shall be obtained from the following combinations of actions:

1.35G
1.2 G₁ + 1.5 Q₁
1.2 G₁ + 1.5 Q₂
0.9 G₁ + Wₜᵣ (up)
1.2 G₁ + Wₜᵣ (down)
1.2 G₁ + Wᵤᵤ + Wₜᵣ (down)
1.2 G₁ + 1.0 Fₛₙ (snow)

Where:

G₁ = weight of complete roof
Q₁ = 0.25 kPa
Q₂ = 1.1 kN
Wₜₙᵩ = wind load on roof
Wᵤₜₙᵩ = wind load normal to wall
Fₛₙᵦ = snow action calculation determined in accordance with AS/NZS1170.3

The magnitudes of P₁, P₂ and W for the lower of two storeys shall be obtained from the following combinations of actions:

\[
\begin{align*}
1.35G \\
1.2 G₁ + 1.5 Q₁ \\
1.2 G₁ + 1.5 Q₂ \\
0.9 G₁ + Wₜₙᵩ (up) \\
1.2 G₁ + Wₜₙᵩ (down) \\
1.2 G₁ + Wᵤₜₙᵩ + Wₜₙᵩ (down) \\
1.2 G₁ + 1.0 Fₛₙᵦ (snow)
\end{align*}
\]

where
G₁ = weight of complete roof, upper storey walls and floors
Q₁ = 0.25 kPa roof imposed action
Q₂ = 1.5 kPa floor imposed action
Wₜₙᵩ = wind load on roof
Wᵤₜₙᵩ = wind load normal to wall
Fₛₙᵦ = snow action calculation determined in accordance with AS/NZS1170.3

3.6.3 Design for serviceability (for upper storey and lower storey lintels)

Design for serviceability under the issue concerned shall calculate the value of the parameter under the nominated action, kept within the limiting value of the response given in Table 3.9.

### Table 3.9 Serviceability response limits for lintels

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
</table>
| Sagging at mid span   | Mid-span Deflection (Δ)  | Pₛ or wₛ | L/300 (<10 mm)    | For single storey or upper storey lintel Pₛ or wₛ arising from G₁  
|                       |                          |        |                   | For lower storey lintel Pₛ or wₛ arising from G₁ + Q₂ |
| Wind uplift           | Mid-span Deflection (Δ)  | Pₛ or wₛ | L/200             | For upper storey lintel Pₛ or wₛ arising from 0.9 G₁ + Wₜₙᵩ(up) |

3.7 WALL BRACING

Wall bracing shall be designed in accordance with Section 6.
4. FLOOR MEMBERS

4.1 GENERAL

All floor members including floor joists, bearers and flooring shall be designed to act together as a structural unit to transfer all the actions imposed on the roof, walls and floors to appropriate supports.

For foundations refer to Section 1.1

The floor assembly is expected to act as a diaphragm to transmit the horizontal shear action effects arising from wind and earthquake actions (see Figure 4.1).

![Figure 4.1 Components of typical floor frame.](image)

**Figure 4.1 Components of typical floor frame.**

4.2 FLOOR JOISTS AND BEARERS

4.2.1 Load paths

Floor joists are designed to support floor loads.

Floor bearers shall be designed to support the floor joists.

**NOTES:**

1. Floor joists or bearers may also be required to support ceilings (of storey below), load bearing and non-load bearing walls which may run either parallel or perpendicular to the direction of the joists or bearers (see Figure 4.2).
4.2.2 Design for strength

The combinations of actions used for the determination of the design action effects for floor joists or bearers shall be as follows:

1.2 G + 1.5 Q₁
1.2 G + 1.5 Q₂
1.2 G + 1.5 Q₃ (housing balcony only)

The action effects of concentrated loads shall be considered where appropriate.

Where:

G = weight of flooring and non load bearing walls for the flexural design of joist (plus weights of joists for the design of bearers).

Q₁ = floor uniformly distributed live load
    = 1.5 kPa over the appropriate tributary area except for balconies where a 2.0 kPa shall apply.

Q₂ = floor concentrated live load
    = 1.8 kN (use also to check punching shear or crushing by applying it over an area of 350mm² of flooring)

Q₃ = balcony line load
Internal = 0.35 kN/m run along top edge horizontal and vertical or 0.5 kPa on infill.
External = 0.75 kN/m run along top edge horizontal and vertical or 1.0 kPa on infill.
NOTES:
1. A load of 0.5 kPa may be used for non load bearing walls as per AS/NZS 1170.1.
2. Q1 applies for all areas except balconies.
3. Q2 may be shared with adjacent member due to system effect (see Appendix B).

For other types of occupancy, refer to AS/NZS 1170.1.

### 4.2.3 Design for serviceability

Design for serviceability under the issue of concern the calculated value of the parameter under the nominated action shall be kept within the limiting value of the response, as given in Table 4.1.

**Table 4.1 Serviceability response limits – floors**

<table>
<thead>
<tr>
<th>Issue of concern</th>
<th>Serviceability Parameter</th>
<th>Action</th>
<th>Limit of Response</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Noticeable Sag</td>
<td>Mid-span Deflection (Δ)</td>
<td>G+Ψ₁ Q₁</td>
<td>L/400</td>
<td>Normal floor system</td>
</tr>
<tr>
<td>Masonry wall cracking</td>
<td>Mid-span Deflection (Δ)</td>
<td>G+Ψ₁ Q₁</td>
<td>L/500</td>
<td>Floor supporting masonry walls</td>
</tr>
<tr>
<td>Vibration</td>
<td>Mid-span Deflection (Δ)</td>
<td>G + Q₁</td>
<td>L/500 (&lt; 12 mm)</td>
<td>Dynamic performance of floor ¹</td>
</tr>
<tr>
<td>Vibration</td>
<td>Mid-span Deflection (Δ)</td>
<td>1.0 kN</td>
<td>Less than 2.0 mm deflection</td>
<td>Dynamic performance of floor ¹</td>
</tr>
<tr>
<td>Noticeable sag</td>
<td>Mid-span Deflection (Δ)</td>
<td>G+Ψ₂ Q₂</td>
<td>L/400</td>
<td>Normal floor system</td>
</tr>
<tr>
<td>Noticeable Horz. Movement</td>
<td>Mid-span horz. Deflection (Δ₃)</td>
<td>G+Ψ₂ Q₃</td>
<td>H/60 + L/240</td>
<td>Horizontal movement of barrier</td>
</tr>
</tbody>
</table>

**NOTES:**
1. Appendix C provides further guidance on dynamic performance of floors.
2. Mid span deflection refers to the total floor system deflection.
3. Limit of response for cantilever may be taken as half of the values given above.
4. Ψ Long term combination factor (0.4)
5. Ψ₂ = short-term combination factor (1.0)

### 4.3 FLOOR AND SUB-FLOOR BRACING

Floor and sub-floor bracing and their connections shall be designed in accordance with Section 5 and 6.

**NOTE:** Access shall be provided to permit visual inspection of all sub-floor framing members. A crawl space for this purpose shall be not less than 450mm high to the underside of the floor joists.
5. CONNECTIONS

5.1 GENERAL

Connection elements include connection components (framing anchors, brackets, straps, plates, parts of members to be connected) and connectors (welds, bolts, screws, rivets, clinches, nails, structural adhesives).

Connections shall be capable of carrying the design action effects resulting from the forces in the connected members including the uplift forces due to the wind and earthquake action and transferring these forces to appropriate supports.

NOTE: Further guidance on connections can be found in the NASH handbook best practice for the design and construction of residential and low rise steel framing.

5.2 DESIGN CRITERIA

Connection components and connectors shall be designed to satisfy the following:
1. Connection elements are capable of resisting design action effects arising in the connection as the result of the design action effects in the connecting members and their supports.
2. Deformations at the connection are within the acceptable limits.
3. Appropriate allowance shall be made for any eccentricity at the connection.
4. Appropriate allowance shall be made for any local effects at the connections (such as stress concentration and local buckling).
5. The uplift forces due to wind action shall be assessed in accordance with AS/NZS 1170.2 as appropriate and tie-down shall be provided to resist these forces.
6. The strength and serviceability of the connection shall be assessed by computation using AS/NZS 4600 or by prototype testing in accordance with Section 7.
6. BRACING

6.1 GENERAL

This section describes the requirements for the design of bracing. These include roof bracing, wall bracing, floor, and sub-floor bracing.

NOTE:
1. Temporary bracing may be required during construction (see Appendix A).
2. Where wind pressures acting on two or more surfaces of an enclosed building contribute simultaneously to a structural action effect in a member, action combination factors $K_{c,e}$ and $K_{c,i}$ (AS/NZS1170.2) may be applicable to external and internal pressure coefficients, respectively.

6.2 ROOF BRACING

6.2.1 General

All roof members including roof battens, roof trusses or rafters, ceiling battens and bracing shall be designed to act together as a structural unit to transfer all the actions imposed on the roof to appropriate supports (see figure 6.1).

For lateral restraints, the roof battens are intended to provide the lateral support for the top chords of the trusses, and the ceiling battens provide the lateral support for the bottom chords of the trusses. These assumptions shall require engineering verification including:

1. Provision of additional bracing such as cross braces to ensure that the assumptions are valid; and
2. Computation to verify the adequacy of the roof and ceiling battens and their connections to the trusses to act as lateral restraint members.

Roof bracing for both truss and framed roofs shall be provided in accordance with this section, provided that roof plane braces and roof space braces may be omitted where there is a structural ceiling diaphragm complying with 6.3 and directly attached to the rafters.

Small roof planes, of less than 6 m², such as dormers or porches shall not require bracing.

NOTE: The adequacy of the bracing system is particularly important if the trusses are loaded on the bottom chords (for example to support other girder trusses).
**6.2.2 Truss bracing**

**6.2.2.1 Top chord bracing**

A top chord bracing system design shall transfer the forces generated in the top chord restraints (usually by battens or purlins) back to the supporting structures.

The actions to be considered are those required to restrain the top chord against buckling wind action perpendicular to the span of the trusses and earthquake loads which may govern with a heavy roof.

Battens or purlins acting as top chord restraints shall be continuous or fixed together to perform as a continuous top chord restraint.

Diagonal bracing angle shall be between 30 and 60 degrees to the truss top chord or rafter and shall not sag more than 1/500 of the distance between supports. Where tension devices are used to remove excessive sag.

**NOTE:** Care should be taken not to over-tension the braces.

**Top chord bracing shall be as below**

1. Light pitched roofs be braced by one brace having a capacity of 4.0 kN for each 50 m², or part thereof, of plan area, with a minimum of 2 braces for each ridge line.

2. Heavy pitched roofs be braced by one brace having a capacity of 4.0 kN for each 25 m², or part thereof, of plan area including verges, with a minimum of 2 braces for each ridge line.
6.2.2.2 Bottom chord bracing

Bottom chord bracing is required to restrain bottom chords against lateral buckling under wind uplift. Bracing shall be fixed to each truss and to the wall in the same manner as for top chord brace fixing.

Where ceiling battens do not provide restraint to bottom chords, appropriate ties shall be provided.

Ties shall be fixed to supporting elements to transfer the bracing loads to appropriate supports.

For trusses with ceiling directly fixed to the bottom chords by glue or nails, ties shall be required as temporary bracing for the bottom chords. The bottom chord ties shall not replace the binders required to support the end walls.

6.2.2.3 Web bracing

Where truss design requires bracing of the web members, it shall be provided with longitudinal ties or other supplementary members to provide the appropriate restraints.

6.3 CEILING DIAPHRAGM BRACING

Ceiling diaphragms shall be constructed as follows

1. The length of the diaphragm shall not exceed twice the width of the diaphragm, with both length and width being measured between the supporting walls;
2. The ceiling lining shall consist of a sheet material complying with 6.3.1 over the entire area of the diaphragm; and
3. Complete sheets with a minimum size of 1800 x 900 shall be used, except where building dimensions require sheets to be cut.

6.3.1 Ceiling lining material

Ceiling lining material shall comply with the following:

(a) For diaphragms not steeper than 25° to the horizontal, and not exceeding 7.5m long under light or heavy roofs; a gypsum-based sheet material not be less than 8mm thick or ceiling lining material given in 6.3.1(b) shall be used.
(b) For diaphragms not steeper than 25° to the horizontal, and not exceeding 15m long under light or heavy roof, ceiling lining material shall be:
   i. Structural Plywood complying with AS/NZS 2269;
   ii. Any other wood or fibre-cement based product not less than 880 kg/m³; or
   iii. other wood or fibre-cement based product not less than 6mm thick and having a density of not less than 600 kg/m³ (for example particleboard).
(c) For diaphragms not steeper than 45° to the horizontal and not exceeding 7.5m long, for light or heavy roofs, ceiling lining material shall be as given by 6.3.1(b).

6.4 WALL BRACING

6.4.1 Load path

Wall bracing shall be designed to transfer all horizontal forces from roof, walls and floors to the appropriate ceiling and floor diaphragms or foundation walls. These forces arise from wind or earthquake actions. Typical wall bracing is shown in Figure 6.2.

![Wall Bracing Diagram]

Figure 6.2 Typical wall bracing systems

6.4.2 Design for strength

The design of the wall bracing shall conform to the following criteria:

1. The magnitudes of the forces shall be determined in accordance with AS/NZS 1170.2 and NZS 1170.5;
2. Bracing shall be provided in two orthogonal directions and shall be distributed evenly. (see Figure 6.3);
3. The angle between a metal strap in a wall bracing element and the horizontal shall be between 30 and 60 degrees to the horizontal;
4. Sheet bracing elements shall not have an aspect ratio (height/width) greater than 3;
5. The structure shall be designed to carry the design level of uplift force from each bracing element, and to hold down anchor to the foundation;
6. A combination of systems for a line of wall bracing should only be used if it can be established that the systems can both meet the strength and
serviceability criteria of AS/NZS 1170 at the design displacements. Otherwise, the strength of the bracing shall be taken as that of only one of the systems;

7. The racking strength of the system shall be established by either full size prototype testing or in the case of discrete bracing systems the capacity may be calculated by rational analysis. Connection details shall be designed to resist the forces specified in AS/NZS 1170.2, and NZS 1170.5; and

8. The anchor resistance shall be greater than that required to achieve the bracing element capacity.

Figure 6.3 Typical distribution of bracing walls

6.5 FLOOR AND SUB-FLOOR BRACING

6.5.1 Floor joists or bearers

Floor joists shall rely on the floor decking to provide lateral restraint.

Bearers shall rely on floor joists to provide lateral restraint.

NOTE: Blocking may be required at supports to transfer horizontal shear forces from the floor deck to the bearers or walls; and along the spans for lateral and torsional stability particularly for long span members.

6.5.2 Sub-floor

All lateral and vertical actions are eventually transmitted to the foundation of the building. The foundation shall be designed to resist all these forces.

Roof and wall bracing shall be designed to transfer the lateral forces (from wind, earthquake and other actions) to the floor plane.

The sub-floor support structure shall be designed to transfer these forces to the footings.
Slab-on-ground construction shall be in accordance with NZS 3604 as modified by B1/AS1.

For suspended ground floor, appropriate sub-floor bracing shall be provided depending on the arrangement of vertical support systems (such as piles/posts/block or reinforced concrete ring walls).

6.5.3 Design to meet loads

Floor and fixings shall be designed to meet the appropriate design loads.

NOTE: Guidance for floor connections is provided in NASH Standard Part 2: 2016

6.6 FLOOR DIAPHRAGM BRACING

6.6.1 Floor diaphragms

Diaphragms shall have a maximum length of 15m and comply with the following limitations:

1. The length and width of a diaphragm shall be between supporting bracing lines at right angles to each other;
2. Any diaphragm or part of a diaphragm shall have a length not exceeding 2.5 times its width for single storey buildings, and a length not exceeding 2.0 times its width for 2 storey buildings;
3. The flooring shall consist of a sheet material complying with 6.3.1(b) over the entire area of the diaphragm;
4. The minimum sheet size shall be 2400mm x 1200mm except where the building dimensions prevent the use of a complete sheet; and
5. Floor joists in a structural floor diaphragm shall be laterally supported around the entire perimeter of the diaphragm with a perimeter joist.

6.6.2 Subdivide floor

Where it is necessary to subdivide a floor into more than one diaphragm so as to comply with 6.6.1, one wall should be used to support the edges of 2 diaphragms.

6.6.3 Ground floor diaphragms

The entire perimeter of the ground floor diaphragm which is separated from the ground for single storey buildings shall be supported by either;

1. A continuous foundation wall, or an evenly distributed perimeter bracing system; or
2. The entire perimeter of the ground floor diaphragm which is separated from the ground for two storey buildings shall be directly supported by a continuous foundation wall.
6.6.4 Upper floor diaphragms

The entire perimeter of an upper floor diaphragm shall be located over, and connected to walls containing the number of required bracing units.
7. TESTING

7.1 GENERAL

Testing in accordance with Section 8 of AS/NZS 4600 may be used in conjunction with assessment methods in the Standards listed in Section 1 of this Standard. Where testing in accordance with AS/NZS 4600 Section 8 is used, the provisions of Sections 7.2 and 7.3 of this Standard apply.

7.2 ADDITIONAL REQUIREMENTS FOR PROTOTYPE TESTING

The coefficient of variation of structural characteristics ($V_{sc}$) refers to the variability of the total population of the production units. This includes the total population variation due to fabrication ($k_f$) and material ($k_m$), which may be approximated to be:

$$V_{sc} = \sqrt{(k_f)^2 + (k_m)^2}$$

Unless a comprehensive test program shows otherwise, the value of $V_{sc}$ shall not be taken to be less than the following:

- Member or connector strength: 10%
- Connection strength: 20%
- Assembly strength: 20%
- Member stiffness: 5%
- Assembly stiffness: 10%

NOTES:
1. See Section 5.1 for definitions of connector and connection.
2. Where materials are sourced from multiple suppliers, the coefficient of variation of material properties (such as thickness, strength, diameter, etc.) should be assessed to ensure the above assumptions are applicable.

7.3 DESIGN ASSISTED BY TESTING

7.3.1 General

When the design value $R_d$ for a specific product is established by prototype testing, the failure mode shall be the same across all test specimens. The design value $R_d$ shall satisfy either:

$$R_d = \left(\frac{R_{min}}{k_{t-min}}\right); \text{ or } R_d = \left(\frac{R_{ave}}{k_{t-ave}}\right)$$

Where
1. $R_{min}$ is the minimum value of the test results and $k_{t-min}$ is the sampling factor as given in Table 7.1.
2. $R_{ave}$ is the average value of the test results and $k_{t-ave}$ is the sampling factor as given in Table 7.2.

NOTE: Wherever possible, the minimum number of tests should be 3 for members and assemblies and 10 for connectors.
Table 7.1 Sampling factor $k_{t\text{-min}}$ for use with the minimum value of the test results

<table>
<thead>
<tr>
<th>Number of test units</th>
<th>Coefficient of variation of structural characteristics ($V_{sc}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
</tr>
<tr>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1.17</td>
</tr>
<tr>
<td>3</td>
<td>1.15</td>
</tr>
<tr>
<td>4</td>
<td>1.15</td>
</tr>
<tr>
<td>5</td>
<td>1.13</td>
</tr>
<tr>
<td>7</td>
<td>1.11</td>
</tr>
<tr>
<td>10</td>
<td>1.10</td>
</tr>
<tr>
<td>20</td>
<td>1.06</td>
</tr>
<tr>
<td>100</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 7.2 Sampling factor $k_{t\text{-ave}}$ for use with the average value of the test results

<table>
<thead>
<tr>
<th>Number of test units</th>
<th>Coefficient of variation of structural characteristics ($V_{sc}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
</tr>
<tr>
<td>1</td>
<td>1.20</td>
</tr>
<tr>
<td>2</td>
<td>1.18</td>
</tr>
<tr>
<td>3</td>
<td>1.17</td>
</tr>
<tr>
<td>4</td>
<td>1.16</td>
</tr>
<tr>
<td>5</td>
<td>1.15</td>
</tr>
<tr>
<td>7</td>
<td>1.15</td>
</tr>
<tr>
<td>10</td>
<td>1.14</td>
</tr>
<tr>
<td>20</td>
<td>1.13</td>
</tr>
<tr>
<td>100</td>
<td>1.11</td>
</tr>
</tbody>
</table>

7.3.2 Interpolation of values obtained by prototype testing

When prototype testing is conducted for a range of a specific parameter (such as span) to establish design values for a specific product in accordance with Clause 7.3.1, it is permissible to interpolate the obtained results for that parameter provided that there is no change in structural behaviour such as no change in collapse mode within the interpolating range.

No extrapolation of test values shall be permitted.
APPENDIX A. CONSTRUCTION
(Informative)

A1. INTRODUCTION
Buildings are most vulnerable during construction. An incomplete building is still required to be safe for the people on site. The actions that are to be taken depend on the method of construction. The following is a list of factors that need to be considered.

A2. FACTORS TO BE CONSIDERED DURING CONSTRUCTION

A2.1 Actions and combinations of actions
Critical actions and combinations of actions during construction may be different from those for the complete structure. These include:

1. Imposed action arising from the stacking of construction materials.
2. Imposed action arising from people working on the incomplete frame.
3. Wind action during construction:
   - Wind speed: To maintain the same risk of exposure for the completed structure (50 years) during construction (1 year), the wind load during construction should be based on a design wind speed with annual probability of exceedance of 1:100 for structures of Importance Level 2 - this figure is available from AS/NZS 1170.2.
   - Wind action effects: The wind action effects on the incomplete structure may be different from that on the complete structure e.g. supported walls may become free standing walls during construction and therefore need temporary bracing.

   NOTE: The wind load for construction thus derived is about 80% of the ultimate wind load on the complete structure.

4. Unbalanced actions arising during construction.

A2.2 Other Considerations
Other factors that need to be considered include:

1. Regulatory safety requirements for workers.
2. Provision of scaffolding and barriers particularly those that rely on the building frame for support.
3. Temporary bracing and tie-down during the installation of permanent bracing and tie-down. Particular care should be taken to provide adequate temporary bracing for the lower storey of multi-storey construction where there are significantly higher racking loads than those in single storey buildings.
APPENDIX B. SYSTEM EFFECT
(Informative)

B1. INTRODUCTION

The design criteria recognises the interaction between structural elements and other elements of the construction system. This is known as the system effect. Some of the elements of the system effect may be established by calculation; others may be assessed by testing.

Once a particular system effect is quantified either by calculation or testing, it may be incorporated into the design calculation. It is important to recognize that the system effect may change with changes in materials and method of construction particularly those effects that are established by testing.

The following sections are examples of system effect and how to incorporate it in design.

B2. LOAD REDISTRIBUTION FACTOR FOR CONCENTRATED LOADS

For a beam in a grid system subjected to a concentrated load P, the beam will have to be designed to carry only a proportion of P because the load will have to be shared with adjacent beams on the grid. The load effect on the beam should be taken to be equal to that of an isolated beam loaded by a concentrated load $P_e$:

$$P_e = k_s P$$

(B1)

Where:

$k_s$ = load redistribution factor.

This $k_s$ value is valid only when:
1. the concentrated loads lie within the middle half of the beam; and
2. the loaded beam is at least two beams in from the edges.

$k_s$ may be established for any particular beam grid system by calculation (such as computer analysis of the grid) or may be approximated by the following:

$$k_s = 0.2 \log_{10} \left( \frac{k_b}{n k_c} \right) + 0.95 \quad (0.2 \leq k_s \leq 1.0)$$

(B2)

Where

$k_b$ = flexural rigidity of the member

$$= E_b I_b / L^3$$

Where:

$E_b$ = modulus of elasticity of the member

$I_b$ = second moment of area of the member

$L$ = span of the beam
\[
n = \text{number of crossing members}
\]
\[
k_c = \text{flexural rigidity of the crossing members} = \frac{E_c I_c}{s^3}
\]

Where:
\[
E_c = \text{modulus of elasticity of the crossing member}
I_c = \text{second moment of area of the crossing member}
\]
\[
s = \text{span of the crossing members}
\]

**B3. LOAD REDISTRIBUTION FACTOR FOR PARTIAL AREA LOADS**

For a beam in a grid system subjected to a load of intensity \( "w" \) distributed over an area of width \( "b" \), the beam will have to be designed to carry only a proportion of \( "w" \) because the load will have to be shared with adjacent beams on the grid.

The load effect on the beam should be taken to be equal to that of an isolated beam loaded by a load of intensity \( "w_e" \):

\[
w_e = k_s w
\]

Where:
\[
k_s = \text{load redistribution factor.}
\]

This \( k_s \) value is valid only when:
1. distributed loads lie within the middle half of the beam; and
2. loaded beam is at least two beams in from the edges.

\( k_s \) may be established for any particular beam grid system by calculation (such as computer analysis of the grid) or may be approximated by the following:

\[
k_s = k_1 \log_{10} \left( \frac{k_b n}{n k_c} \right) + k_2 \quad (0.2 \leq k_s \leq 1.0)
\]

Where:
\[
k_b = \text{flexural rigidity of the member} = \frac{E_b I_b}{L^3}
\]

Where:
\[
E_b = \text{modulus of elasticity of the member.}
I_b = \text{second moment of area of the member.}
L = \text{span of the beam}
\]
\[
n = \text{number of crossing members}
\]
\[
k_c = \text{flexural rigidity of the crossing members} = \frac{E_c I_c}{s^3}
\]

Where:
\[
E_c = \text{modulus of elasticity of the crossing member.}
\]
\[ l_c = \text{second moment of area of the crossing member.} \]
\[ s = \text{span of the crossing members} \]

**Table B1 Load distribution factor coefficients**

<table>
<thead>
<tr>
<th>'b'</th>
<th>( k_1 )</th>
<th>( k_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.20</td>
<td>0.95</td>
</tr>
<tr>
<td>s</td>
<td>0.15</td>
<td>0.75</td>
</tr>
</tbody>
</table>

**B4. Crossing members**

When the crossing member is the floor slab or deck, its effect can be accounted for in load sharing as follows:

1. The flexural rigidity of the slab or deck is calculated on a metre width basis; and
2. \( n = 0.75L \), where \( L \) is the span of the beam.
APPENDIX C. DYNAMIC PERFORMANCE OF FLOOR SYSTEM
(Informative)

C1. INTRODUCTION
For user comfort, floors should be sufficiently stiff such that the floor vibration response is due to impulsive excitation. Suitable end connections and flooring materials will improve the overall performance in service.

In general, providing that the requirements given in Section C.2 and C.3 are achieved, floors will be acceptable for normal design purposes. However, when an assessment of the multiplying factor according to ISO 10137 is required, or the floor is used for dance-type activities, the methodology presented in reference [C1] may be adopted.

C2. MINIMUM STIFFNESS OF FLOOR SYSTEM
The deflection of the floor system $\Delta$ under a 1.0 kN static load may be obtained using a computer analysis of the grid system.

Alternatively the following expression may be used to obtain an approximate estimate for a floor joist and decking system:

$$\Delta = k_d \left( \frac{L^3}{48 E_b I_b} \right) . \text{(C1)}$$

Where:

$$k_d = 0.883 - 0.34 \log_{10} \left( \frac{k_c}{k_b} + 0.44 \right)$$

Where:

$$k_c = \frac{E_c t^3 L}{12 s^3} \quad k_b = \frac{E_b I_b}{L^3}$$

Where:

$E_c, E_b$ = modulus of elasticity of the decking and the joist respectively
$t_f$ = thickness of the decking
$s$ = joist spacing
$I_b$ = moment of inertia of a joist
$L$ = span of the joist

The deflection of the floor system under a 1.0 kN static load should not exceed 2mm to ensure satisfactory floor dynamic performance.

C3. NATURAL FREQUENCY OF FLOOR SYSTEM

The lowest natural frequency (otherwise known as the “first” or “fundamental mode”) of a floor system $f_1$ may be obtained using a computer analysis or approximated by the following for a joist (main member) and decking (crossing member) system:
\[ f_1 = \frac{\pi}{2} \sqrt{\frac{K_x}{wL^4}} \sqrt{1 + \left[ 2 \left( \frac{L}{B} \right)^2 + \left( \frac{L}{B} \right)^4 \right] \frac{K_y}{K_x}} \] (C2)

Where

- \( K_x \) = flexural stiffness of the main members.
  \[ = E_b I_b / s \]

- \( K_y \) = flexural stiffness of the crossing members - joist only systems.
  \[ = E_f t_f^3 / 12 \]

Where:

- \( E_b, E_f \) = modulus of elasticity of the joist and the decking respectively
- \( I_b \) = moment of inertia of a joist
- \( t_f \) = thickness of the decking
- \( s \) = joist spacing

- \( L \) = span of the joist.

- \( B \) = width of the floor.

- \( w \) = mass of the floor in kg/m² including allowance for live load of 0.3 kPa.

The lowest natural frequency of the floor system should be kept above 8Hz for a satisfactory dynamic performance.

NOTE: Walls on floor system may affect the dynamic performance of the system.

APPENDIX D. TOLERANCES
(Normative)

D1. MANUFACTURING AND ASSEMBLY TOLERANCES

D1.1 SECTIONS
The manufacturing and assembly tolerances for the following types of sections shall comply with the following:

1. Cold-formed sections

   Cold-formed sections shall comply with the following:
   a. Material thickness shall conform to AS/NZS 1365;
   b. Tolerances of sections, assuming design thickness, shall be determined such that the relevant actual sectional properties are not more than ±5% from the design section properties; and
   c. Tolerances appropriate for particular sections shall be specified to comply with the above.

2. Structural steel hollow sections

   Tolerances of hollow sections shall comply with the requirements of AS/NZS 1163.

3. Hot-rolled sections

   Tolerances of hot-rolled sections shall comply with the requirements of AS/NZS 3679.1.

D1.2 LENGTH
The length of a component shall not deviate from its specified length by more than ± 2 mm

D1.3 STRAIGHTNESS
A component, specified as straight, shall not deviate about any axis from a straight line drawn between the end points by an amount exceeding l/1000 or 1.0 mm whichever is greater.

D1.4 ASSEMBLY
Assembled wall panels shall not deviate from the specified dimension by more than:
Length   +2 / - 4 mm
Height    ± 2 mm

The height of assembled roof trusses shall not deviate by more than ± 4mm from the specified dimension.
D2. INSTALLATION TOLERANCES

D2.1 ATTACHMENT TO SUPPORTING STRUCTURE

For load bearing walls, gaps between the bottom plate and the concrete slab greater than 3 mm shall be packed with load bearing shims or grouted at each stud.

For non-load bearing walls gaps greater than 3 mm shall be packed with load bearing shims or grouted at jamb studs and points where the bottom plate is fastened to the slab.

For the attachments of floor joists, bearers, trusses and rafters to walls, where the gap is over 3 mm, the gap shall be packed with load bearing shims.

D2.2 WALLS

The following tolerances shall be applicable to all vertical members including walls, posts, and piles.

1. Position

   Walls shall be positioned within 5 mm from their specified position.

2. Plumb

   Walls shall not deviate from the vertical by more than height/600 or 3 mm whichever is greater (see Figure D1).

![Figure D1 Plumb of walls](image-url)
3. **Straightness**

Walls, specified as straight, shall not deviate by more than 5 mm over a 3 metre length at shown in Figure D2.

Where wall panels join to form a continuous wall, the critical face or faces of the panel shall not deviate by more than ±2 mm at the joint.

![Figure D2 Straightness of walls](image)

4. **Flatness of walls for installation of linings**

The flatness of an individual wall, that is to be lined, shall be such that when a 1.8 metre straight edge is placed parallel to the wall face, the maximum deviation from the straight edge shall not exceed 3 mm over 90% of the area and not exceed 4 mm over the remaining area.

**D2.3 TRUSSES, RAFTERS, CEILING JOISTS, AND FLOOR MEMBERS**

The following tolerances shall be applicable to trusses, rafters, ceiling joists, and floor members:

1. **Position**
   - Trusses, rafters, ceiling joists and floor members shall be positioned within 5 mm from their specified position.

2. **Straightness**
   - Trusses, rafters, ceiling joists and floor members shall be installed with an overall straightness not greater than L/500 where L is the length of the member (see Fig D3).

   Differential in vertical bows between adjacent members shall not exceed 1/150 of their spacing or 6 mm whichever is less.
Figure D3 Straightness

3. Plumb

Trusses, rafters, ceiling joists and floor members shall not be installed out of plumb at any point along the length of the truss from top to bottom by more than the minimum of h/100 or 20 mm, unless the trusses are specifically designed to be installed out of plumb (see Figure D4).

Figure. D4 Plumbness of trusses

4. Spacing

The spacing of trusses, rafters, ceiling joists and floor joists shall not vary from the specified dimension by more than 20 mm.

5. Flatness

The flatness of the floor surface is to be within ± 10 mm over the entire room, but not exceeding ±5 mm over any 3 metre length.
Abutting floors between rooms shall be aligned unless specifically designed otherwise. e.g. steps, different finishes.

**D2.4 VERTICAL ALIGNMENT OF MEMBERS**

When members such as joist, rafter truss and structural wall stud (above or below) are designed to be vertically aligned, the centre lines of the members shall not be more than 20 mm apart as shown in Figure D5.

*Figure D5 Vertical alignment of members*
APPENDIX E. GUIDE FOR DETERMINATION OF SELF-WEIGHTS
(Informative)

E1. TYPICAL FLOOR CONSTRUCTION

Typical floor thicknesses and self-weights are given in Table E1 for guidance.

Table E1 Floor construction and self-weights

<table>
<thead>
<tr>
<th>Floor and/or ceiling type</th>
<th>Self-weight (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber flooring up to 22 mm thick plus lightweight floor covering*</td>
<td>0.18</td>
</tr>
<tr>
<td>Timber flooring up to 22 mm thick plus lightweight floor covering* and ceilings**</td>
<td>0.28</td>
</tr>
<tr>
<td>Timber flooring up to 22 mm thick plus ceramic or terracotta floor covering***</td>
<td>0.35</td>
</tr>
<tr>
<td>Timber flooring up to 22 mm thick plus ceramic or terracotta floor covering*** and ceilings**</td>
<td>0.45</td>
</tr>
</tbody>
</table>

NOTES:
* light weight floor covering = carpet + underlay
** ceilings = 10 mm plasterboard (10kg/m²)
*** ceramic or terracotta floor covering = 20kg/m²

E2. TYPICAL SELF-WEIGHTS OF FLOOR COMPONENTS

Typical floor components and their self weights are given in Table E2 for guidance.

Table E2 Floor components and self weights

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>Self-weight (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FLOORING</td>
<td></td>
</tr>
<tr>
<td>Timber Strip flooring</td>
<td>- 12 mm softwood 0.06</td>
</tr>
<tr>
<td></td>
<td>- 19 mm softwood 0.10</td>
</tr>
<tr>
<td></td>
<td>- 12 mm hardwood 0.10</td>
</tr>
<tr>
<td></td>
<td>- 19 mm hardwood 0.15</td>
</tr>
<tr>
<td>Particleboard flooring</td>
<td>- 19 mm 0.13</td>
</tr>
<tr>
<td></td>
<td>- 22 mm 0.15</td>
</tr>
<tr>
<td></td>
<td>- 25 mm 0.18</td>
</tr>
<tr>
<td>Plywood flooring</td>
<td>- 15 mm 0.08</td>
</tr>
<tr>
<td></td>
<td>- 17 mm 0.09</td>
</tr>
<tr>
<td></td>
<td>- 19 mm 0.11</td>
</tr>
<tr>
<td>Fibre Cement Sheet</td>
<td>- 18 mm 0.33</td>
</tr>
<tr>
<td></td>
<td>- 24 mm 0.44</td>
</tr>
<tr>
<td>Carpet and underlay</td>
<td>0.01 to 0.06</td>
</tr>
<tr>
<td>Ceramic or terracotta floor tiles</td>
<td>0.10 to 0.40</td>
</tr>
</tbody>
</table>
E3. TYPICAL ROOF CONSTRUCTION

Typical roof types and their self weights are given in Table E3 for guidance.

<table>
<thead>
<tr>
<th>Roof type</th>
<th>Self-weight (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel sheet roofing 0.48 mm thick and 0.55 mm thick roof battens @ 900 mm</td>
<td>0.06</td>
</tr>
<tr>
<td>Steel sheet roofing 0.48 mm thick, 0.55 mm thick steel roof battens @ 900 mm, 10 mm plaster ceiling and 0.55 mm thick steel ceiling battens @ 450 mm, underlay and lightweight insulation</td>
<td>0.15</td>
</tr>
<tr>
<td>Concrete or Terracotta roof tiles and 0.55 mm thick steel/timber roof battens @ 330 mm, underlay and lightweight insulation</td>
<td>0.61</td>
</tr>
<tr>
<td>Concrete or Terracotta roof tiles and 0.55 mm thick steel/timber roof battens @ 330 mm, 10 mm plaster ceiling and 0.55 mm thick steel ceiling battens @ 450 mm, underlay and lightweight insulation</td>
<td>0.70</td>
</tr>
</tbody>
</table>

E4. TYPICAL SELF-WEIGHTS OF ROOF COMPONENTS

Typical roof components and their self-weights are given in Table E4 for guidance.

Table E4 Roof components and self weights
<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>Self-weight (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ROOFING</strong></td>
<td></td>
</tr>
<tr>
<td>Steel Sheet</td>
<td>- 0.42 mm 0.044</td>
</tr>
<tr>
<td></td>
<td>- 0.55 mm 0.060</td>
</tr>
<tr>
<td>Aluminium Sheet</td>
<td>- 1.2 mm 0.050</td>
</tr>
<tr>
<td>Tiles</td>
<td>- Terracotta 0.580</td>
</tr>
<tr>
<td></td>
<td>- Concrete 0.540</td>
</tr>
<tr>
<td></td>
<td>- Metal sheet 0.075</td>
</tr>
<tr>
<td><strong>CEILING LINING</strong></td>
<td></td>
</tr>
<tr>
<td>Plasterboard</td>
<td>- 10 mm 0.075</td>
</tr>
<tr>
<td></td>
<td>- 13 mm 0.100</td>
</tr>
<tr>
<td>Timber lining board</td>
<td>- 12 mm softwood 0.065</td>
</tr>
<tr>
<td></td>
<td>- 19 mm softwood 0.105</td>
</tr>
<tr>
<td>Plywood</td>
<td>- 12 mm softwood 0.065</td>
</tr>
<tr>
<td></td>
<td>- 8 mm hardwood 0.050</td>
</tr>
<tr>
<td>Hardboard</td>
<td>- 4.8 mm 0.050</td>
</tr>
<tr>
<td></td>
<td>- 5.5 mm 0.055</td>
</tr>
<tr>
<td>Fibreboard</td>
<td>- 50 mm low density 0.100</td>
</tr>
<tr>
<td></td>
<td>- 50 mm high density 0.200</td>
</tr>
<tr>
<td>Fibre cement sheet</td>
<td>- 4.5 mm 0.070</td>
</tr>
<tr>
<td></td>
<td>- 6.0 mm 0.090</td>
</tr>
<tr>
<td>Lightweight insulation plus sarking</td>
<td>0.060</td>
</tr>
<tr>
<td>Heavyweight insulation</td>
<td></td>
</tr>
<tr>
<td><strong>BATTENS OR PURLINS</strong></td>
<td></td>
</tr>
<tr>
<td>Z or C section 100mm x 1.5mm</td>
<td>@ 1200 mm 0.021</td>
</tr>
<tr>
<td>Z or C section 150mm x 1.5mm</td>
<td>@ 1200 mm 0.029</td>
</tr>
<tr>
<td>Z or C section 200mm x 1.9mm</td>
<td>@ 1500 mm 0.038</td>
</tr>
<tr>
<td>Z or C section 250mm x 2.4mm</td>
<td>@ 1500 mm 0.053</td>
</tr>
<tr>
<td>Z or C section 300mm x 3.0mm</td>
<td>@ 1800 mm 0.070</td>
</tr>
<tr>
<td>Z or C section 350mm x 3.0mm</td>
<td>@ 1800 mm 0.083</td>
</tr>
<tr>
<td>Ceiling batten 0.55mm</td>
<td>@ 450 mm 0.010</td>
</tr>
<tr>
<td></td>
<td>@ 600 mm 0.007</td>
</tr>
<tr>
<td>Roof batten 0.55 mm</td>
<td>@ 330 mm (for tile roof) 0.020</td>
</tr>
<tr>
<td></td>
<td>@ 900 mm (for sheet roof) 0.007</td>
</tr>
<tr>
<td>Roof batten 0.75 mm</td>
<td>@ 1200 mm 0.010</td>
</tr>
<tr>
<td>Roof batten 0.9 mm</td>
<td>@ 1200 mm 0.022</td>
</tr>
<tr>
<td><strong>MISCELLANEOUS</strong></td>
<td></td>
</tr>
<tr>
<td>Photovoltaic roof panels (plastic covered or glazed up to 3 mm thick)</td>
<td>0.15</td>
</tr>
<tr>
<td>Solar water heating panels – including water (excluding tanks)</td>
<td>0.20</td>
</tr>
</tbody>
</table>
APPENDIX F. SERVICE HOLES IN FRAME MEMBERS
(Normative)

Service holes in frame members shall comply with the following:

1. Holes in frame members intended for electrical, communication or data services shall be:
   a. flared with no sharp angles or projecting edges that would be likely to damage a conductor or the insulation, braiding or sheathing of a cable; or
   b. capable of being fitted with plastic grommets or bushes to protect the cable.

2. Holes in frame members intended for plumbing services shall be capable of being fitted with plastic grommets or other effective means of isolation of the plumbing service from the frame members.

NOTES:
1. Plumbing services may be copper, brass, stainless steel or plastic.
2. Plumbing, electrical, data and communications services may be installed when a building is first built, or when it is renovated, modified or extended.
3. Plumbing services use a variety of fasters, clips, grommets and other accessories to support and isolate pipes and fittings. Where galvanic corrosion is likely to occur, e.g. Copper piping, isolation should be employed.
4. Effective support and isolation assists in preventing corrosion, heat loss, water hammer and physical abrasion of pipes due to thermal movement. These effects can be undesirable for the structural frame as well as the plumbing service.
5. Electrical, data and communications cables are frequently protected by conduit and accessories during installation and operation. In particular, where there is a change of direction, either vertically or horizontally, of electrical, data or communication cable that is not enclosed in a conduit or the holes are flared, plastic grommets or bushes should be fitted to protect the cable.
APPENDIX G. MEMBER CLASSIFICATIONS
(Normative)

Members for use with NASH Standard part 2 shall be classified in accordance with this section.

G.1. STUDS/WEB MEMBER CLASSIFICATION

Stud/web members shall be classified as SA, SB or SC as given by Table G1.

The maximum allowable service hole for stud/web members shall be given by the stud member classification.

Studs/webs of type SA, SB or SC shall have capacities greater than 95% of the values given in Table G.1 and with a minimum slenderness about the strong axis \((l/r_x)\) of 100 and with a minimum section depth \((D)\) of 70mm.

Stud/web capacities shall be established by calculation using the appropriate procedures of AS/NZS 4600 or by prototype testing in accordance with Section 7 of this Standard.

Stud/web capacities shall be established for the conditions given in G1.1 and G1.2.

G.1.1. By Calculation

Stud/web capacities shall consider the following conditions when being determined by calculation:

- End supports are assumed to be pinned
- Lateral restraints
  - In the plane of the frame
  - Out of plane of the frame
- Axial load is assumed to be concentric about effective section.
- Bending load is assumed to be uniformly distributed.
- Service hole location in accordance with Fig G.1
- Effective length may be taken as 80% of the distance between restraints if both flanges are restrained.
- \(C_m = 0.85.\)

G.1.2. By Prototype testing

Stud/web capacities shall consider the following conditions when being determined by prototype testing:

- End of studs/webs are to be supported as in actual construction
- Lateral restraints are as in actual construction (depending on the load case under consideration)
- Axial loads are to be applied without eccentricity
- Bending loads are to be applied uniformly over the length of the stud/web
- Service hole locations in accordance with Fig G.1
Table G.1 Stud/web classification

<table>
<thead>
<tr>
<th>Stud height</th>
<th>2440mm</th>
<th>2740mm</th>
<th>3040mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stud type</td>
<td>ΦcNc (kN)</td>
<td>ΦbMb (kNm)</td>
<td>ΦcNc (kN)</td>
</tr>
<tr>
<td>SA</td>
<td>8</td>
<td>0.6</td>
<td>7</td>
</tr>
<tr>
<td>SB</td>
<td>12</td>
<td>0.8</td>
<td>10</td>
</tr>
<tr>
<td>SC</td>
<td>16</td>
<td>1.0</td>
<td>14</td>
</tr>
<tr>
<td>SD</td>
<td>20</td>
<td>1.4</td>
<td>18</td>
</tr>
</tbody>
</table>

NOTES:
ΦcNc: Design member compression capacity
ΦbMb: Design member moment capacity

G.2. PLATE/CHORD MEMBER CLASSIFICATION
Plate/chord members shall be classified as PA, PB, PC, PD or PE as given by Table G2.

The maximum allowable service hole for plate/chord members shall be given by the plate/chord member classification.

Plates/chords of type PA, PB, PC, PD or PE shall have capacities greater than 95% of the values given in Table G2. Plates/chords may consist of a single section, or a built-up section in compliance with AS/NZS 4600.

Plate/chord capacities shall be established by calculation using the appropriate procedures of AS/NZS 4600 or by prototype testing in accordance with Section 7 of this standard.
Plate/chord capacities shall be established for the following conditions:

G.2.1. By calculation
Plate/chord member capacities shall consider the following conditions when being determined by calculation:

- Bending capacity on the weak axis as applicable for direction of loading
- Shear capacity on the weak axis on the section with cut outs to fit with stud
- Axial Capacity on minimum section with cut out to fit with stud
- Service holes with spacing accordance with Figure G.1

G.2.2. By prototype testing
Plate/chord member capacities shall consider the following conditions when being determined by prototype testing:

- Plates/chords are to be as in actual construction (with cut-outs etc.)
- Service holes of diameter not more than those in studs.
- Frame configuration and loading conditions:
  - Moment capacity – single 600 mm span with mid span load
o Shear capacity – single 600 mm span with load located at ‘d’ from the edge of the supporting stud, where ‘d’ is the flange width of the plate section.
o Axial capacity – 1200 mm high assembly of two plate sections connected by stud sections at mid-height and each end – ends must be reinforced to prevent local buckling at supports; Axial capacity taken as half the factored capacity of the assembly.

Table G.2 Plate/chord member classification

<table>
<thead>
<tr>
<th>Plate type</th>
<th>( \Phi_cV ) (kN)</th>
<th>( \Phi_bMb ) (kNm)</th>
<th>( \Phi_bNc ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PA</td>
<td>3</td>
<td>0.2</td>
<td>12</td>
</tr>
<tr>
<td>PB</td>
<td>3</td>
<td>0.3</td>
<td>16</td>
</tr>
<tr>
<td>PC</td>
<td>4</td>
<td>0.4</td>
<td>16</td>
</tr>
<tr>
<td>PD</td>
<td>5</td>
<td>0.6</td>
<td>24</td>
</tr>
<tr>
<td>PE</td>
<td>6</td>
<td>0.8</td>
<td>24</td>
</tr>
</tbody>
</table>

NOTE: Plate/chord members could be built up sections fabricated from more than one member, such as the addition of a header plate.

Figure G1 Hole layout details – minimum spacing for studs and plates.
G.3. BATTEN SECTION PROPERTIES

Battens used for ceiling or roofing support shall comply with the requirements of Table G3. Examples of battens are shown in Figure G2.

Table G3 Battens

<table>
<thead>
<tr>
<th>Code</th>
<th>Type</th>
<th>BMT mm</th>
<th>D mm</th>
<th>B1 mm</th>
<th>Area mm²</th>
<th>Iₓ mm⁴</th>
<th>J mm⁴</th>
<th>lex mm⁴</th>
<th>lex+ mm⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>22CB42</td>
<td>Ceiling</td>
<td>0.42</td>
<td>22.0</td>
<td>31.5</td>
<td>45</td>
<td>2896</td>
<td>2.4</td>
<td>2418</td>
<td>2301</td>
</tr>
<tr>
<td>20CB55</td>
<td>Ceiling</td>
<td>0.55</td>
<td>19.6</td>
<td>20.1</td>
<td>56.53</td>
<td>3897</td>
<td>5.7</td>
<td>3500</td>
<td>3400</td>
</tr>
<tr>
<td>30CB75</td>
<td>Ceiling &amp; Roof</td>
<td>0.75</td>
<td>30.0</td>
<td>68.1</td>
<td>91.50</td>
<td>14000</td>
<td>18.0</td>
<td>33930</td>
<td>11847</td>
</tr>
<tr>
<td>40RB48</td>
<td>Roof</td>
<td>0.48</td>
<td>40.0</td>
<td>34.0</td>
<td>79.38</td>
<td>18910</td>
<td>5.7</td>
<td>17400</td>
<td>15400</td>
</tr>
<tr>
<td>40RB55</td>
<td>Roof</td>
<td>0.55</td>
<td>40.0</td>
<td>40.0</td>
<td>88.17</td>
<td>20608</td>
<td>8.9</td>
<td>20608</td>
<td>17000</td>
</tr>
</tbody>
</table>

NOTE: Valid for roof batten steel Grade G550: Fu = Fy = 410 MPa.

Figure G2 Batten examples.

G.4. C SECTION PROPERTIES

C sections used for rafters and floor joists shall be comply with the requirements of Table G4. Examples of C section members are shown in figure G3.

Table G4 Dimensions and properties of steel lipped C sections

<table>
<thead>
<tr>
<th>Section designation</th>
<th>Grade AS 1397</th>
<th>Nominal dimensions</th>
<th>Minimum section properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Web depth (d) mm</td>
<td>Flange width (b) mm</td>
</tr>
<tr>
<td>C15012</td>
<td>G500</td>
<td>150</td>
<td>65</td>
</tr>
<tr>
<td>C15015</td>
<td>G450</td>
<td>150</td>
<td>65</td>
</tr>
<tr>
<td>C15018</td>
<td>G450</td>
<td>150</td>
<td>65</td>
</tr>
<tr>
<td>C20012</td>
<td>G500</td>
<td>200</td>
<td>75</td>
</tr>
<tr>
<td>C20015</td>
<td>G450</td>
<td>200</td>
<td>75</td>
</tr>
<tr>
<td>C20018</td>
<td>G450</td>
<td>200</td>
<td>75</td>
</tr>
<tr>
<td>C25015</td>
<td>G450</td>
<td>250</td>
<td>85</td>
</tr>
<tr>
<td>C25018</td>
<td>G450</td>
<td>250</td>
<td>85</td>
</tr>
<tr>
<td>C30015</td>
<td>G450</td>
<td>300</td>
<td>100</td>
</tr>
<tr>
<td>C30018</td>
<td>G450</td>
<td>300</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure G2 C section example