AS 1720.3:2016

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Australian STANDARD

Timber structures

Part 3: Design criteria for timber-framed residential buildings

STANDARDS Australia

This Australian Standard® was prepared by Committee TM-010, Timber Structures and Framing. It was approved on behalf of the Council of Standards Australia on 13 June 2016.
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AS 1720.3:2016

Australian Standard®

Timber structures

Part 3: Design criteria for timber-framed residential buildings

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PREFACE

This Standard was prepared by the Australian members of the Joint Standards Australia/ Standards New Zealand Committee TM-010, Timber Structures and Framing, to supersede AS 1684.1—1999, *Residential timber-framed construction*, Part 1: *Design criteria*.

After consultation with stakeholders in both countries, Standards Australia and Standards New Zealand decided to develop this Standard as an Australian Standard rather than an Australian/New Zealand Standard.

The objective of this Standard is to provide users with the design criteria considered suitable for the design of conventional timber-framed residential buildings, subject to the requirements and limitations described in the Scope. Design criteria for the most commonly used timber members are provided.

This revision continues to recognize the ongoing development of timber framing systems and the need to cater for a widening variety of materials and design conditions. Significant changes include—

(a) closer alignment with the relevant requirements of AS 1720.1;

(b) adoption of the nomenclature and relevant requirements of the AS/NZS 1170 series of loading standards and, in particular, the recently revised edition of AS 4055;

(c) de-linking of this Standard from the timber member span tables components of AS 1684.2, AS 1684.3 and AS 1684.4;

(d) inclusion of design criteria for additional members—wind beams;

(e) limitation to Class 1 and 10 buildings as defined by the National Construction Code—Building Code of Australia;

(f) adjustment of the structural models used for joist and bearer design;

(g) removal of alternative characteristic beam shear strengths for F-grades; and

(h) correcting errors and addressing anomalies and inconsistencies.

Mandatory statements in notes and footnotes to tables are deemed to be requirements of this Standard.

The term 'normative' 'has been used in this Standard to define the application of the appendix to which it applies. A 'normative' appendix is an integral part of a Standard.

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SECTION 1 SCOPE AND GENERAL

1.1 SCOPE AND APPLICATION

1.1.1 Scope

This Standard sets out the design methods, assumptions and other criteria, including uplift forces and racking pressures, suitable for the design of timber-framed buildings constructed within the limitations and parameters of, and using the building practice described in, AS 1684.2, AS 1684.3 and AS 1684.4.

The design criteria apply for the preparation of design data for conventional timber-framed construction where the loading and performance requirements correspond to those for Class 1 and Class 10 buildings as defined by the National Construction Code—Building Code of Australia.

1.1.2 Application

This Standard is to be read in conjunction with AS 1684.2, AS 1684.3 and AS 1684.4, the AS/NZS 1170 series, AS 4055 and AS 1720.1.

NOTE: While this Standard may be applied for Class 10 buildings less conservative levels of design may be permitted, for this Class, by building regulations and other Australian Standards.

The design criteria contained herein provide a basis for the design of timber members and timber-framed building systems and components, including the preparation of Span Tables and design data for structural wood products.

NOTE: The use of the design criteria contained in this Standard may provide evidence of satisfactory safety and serviceability performance.

1.2 REFERENCED DOCUMENTS

The following documents are referred to in this Standard:

AS	
1170	Structural design actions
1170.4	Part 4: Earthquake actions in Australia
1684	Residential timber-framed construction
1684.2	Part 2: Non-cyclonic areas
1684.3	Part 3: Cyclonic areas
1684.4	Part 4: Simplified—Non-cyclonic areas
1720	Timber structures
1720.1	Part 1: Design methods
4055	Wind loads for housing

AS/NZS
1170 Structural design actions
1170.1 Part 1: Permanent, imposed and the other actions
1170.2 Part 2: Wind actions
1170.3 Part 3: Snow and ice actions

CSIRO Low-rise domestic and similar framed structures

CSIRO BCE Report Notched composite beams, Dec. 97/169M, September 1997

1.3 OTHER METHODS

This Standard does not preclude the use of other methods of design, other assumptions or criteria for design or any other means of demonstrating satisfactory safety and serviceability performance in accordance with the requirements of the Building Code of Australia.

1.4 BASIS FOR DESIGN

1.4.1 General

The design criteria contained in this Standard are an interpretation of the AS/NZS 1170 series, AS 4055 and AS 1720.1. The criteria have been formulated for the preparation of generalized design data for residential buildings constructed using the conventional timber framing system described in AS 1684.2, AS 1684.3 and AS 1684.4. The design criteria are based upon the assumptions described in Clauses 1.4.2 to 1.4.11 below.

1.4.2 Geometric limitations

The following geometric limitations have been assumed:

(a) The overall building width at any section, excluding eaves and lean-to verandahs but including verandahs under the main roof, does not exceed 16.0 m.

(b) The roof pitch does not exceed 35°.

(c) Roof shapes may be skillion or gable, hip or gable ended or any combination of these.

(d) The number of trafficable floors supported by timber framing does not exceed two.

(e) Wall height, measured from floor to ceiling, does not exceed 3.0 m.

NOTE: For further details refer to AS 1684.2, AS 1684.3 and AS 1684.4.

1.4.3 Design methods

1.4.3.1 *General*

The design methods used shall be based upon analytical and engineering principles and comply with the requirements of AS 1720.1.

1.4.3.2 *Capacity factor*

Values of capacity factor (φ) shall generally be determined based on Category 1 applications in accordance with AS 1720.1 Table 2.1. Area affected shall typically be computed as area = span × load width × 2 and where this value exceeds 25 m^2, as may be the case for a primary roof support member, the application shall be considered Category 2.

NOTE: Where a member supports a significant point load, such as for a lintel supporting a girder roof truss, special consideration may be required in determining area affected.

1.4.4 System-based assumptions

The design criteria include many system-based assumptions, which recognize the interactions between structural elements and other elements of the overall construction system. These assumptions are based upon the methods of assembly and materials given in AS 1684.2, AS 1684.3 and AS 1684.4.

NOTE: The use of materials (both structural and non-structural) or installation methods other than those given in AS 1684.2, AS 1684.3 and AS 1684.4, may invalidate the system-based assumptions contained in this Standard.

1.4.5 Durability

The structural design criteria have been developed on the assumption that materials used and their installation and maintenance ensure that components will fulfil their intended structural function for the intended life of the structure.

NOTE: In the selection of materials, specific consideration should be given to the risk of and resistance to biological attack and corrosion, long-term durability of adhesives and the long-term strength and rigidity of materials taking into account the short-term and long-term conditions of exposure.

1.4.6 Structural timber

The design criteria are provided primarily for, but are not limited to, design using the generic stress grades and sizes of scantling timber as specified in AS 1684.2, AS 1684.3 and AS 1684.4.

NOTE: For other materials, where the application and performance are claimed to be consistent with AS 1684.2, AS 1684.3 and AS 1684.4, the design procedures and assumptions may require modification in accordance with the requirements of AS 1720.1.

1.4.7 Design properties

The design properties given in AS 1720.1, for generic stress grades and strength groups, have been used as the basis for the development of the design criteria given in this Standard.

1.4.8 Bearing and crushing

A check of bearing strength is indicated, where considered necessary, in the member design Sections of this Standard. Care should be taken to ensure that relevant checks are performed where large concentrated loads may cause crushing of members, such as may occur under the ends of girder trusses, or at the supports for members supporting such high point loads.

1.4.9 Structural design actions

1.4.9.1 *Permanent (dead load)*

Permanent actions are based upon standardized allowances for the mass of roof, wall and floor constructions.

NOTES:

1 Where actions arise as a result of non-standardized imposed dead loads (including permanent proportion of live loads) which may include, but not be limited to, cupboards suspended from walls and or ceilings, roof mounted hot water systems, heavy stone bench tops, safes, tiled waterproof decking systems etc., then specific allowance in design is made for these actions.

2 Where mass or load allowances are different from those referred in this Standard are used, then such variation should be noted in any published data.

1.4.9.2 *Imposed (live load)*

Generally, the imposed actions used for design correspond to those given in AS/NZS 1170.1. The following departures and interpretations have been used:

(a) The partial live load for the serviceability design of continuous span joists and bearers is taken as 0.75 kPa.

(b) The permanent component of floor live load is taken as 0.5 kPa.

(c) Balconies and decks 1 m or more above the ground, are designed for a 1.5 kPa floor live load for the serviceability limit state, as for balconies and decks less than 1 m above the ground.

(d) For generic design, the concentrated load on primary ceiling support members, including ceiling joists, hanging beams and counter beams is taken as 1.4 kN regardless of cladding installation status and available headroom.

NOTES:

1 Live loads specific to construction, e.g. loads resulting from the use of fall protection devices or scaffolding attached to the structure, are not considered.

2 AS/NZS 1170.1 allows this load to be reduced to 0.9 kN for some defined design applications.

1.4.9.3 *Wind*

Wind actions for member design are derived from AS 4055 using wind classifications N1 to N4 and C1 to C3 and associated design wind pressures as specified in Appendix A. Pressure coefficients are determined from AS 4055 unless noted otherwise in the member design information. It is assumed that all of the cladding elements including windows, doors and garage doors are capable of resisting the design winds and comply with the relevant Australian standards. Wind actions for tie-down design are determined from AS 4055 and AS/NZS 1170.2.

1.4.9.4 *Snow*

Snow actions, determined in accordance with AS/NZS 1170.3, up to 0.2 kPa have been considered and determined as not critical. For this reason, snow actions are not included in the action combinations given for member design in this Standard.

1.4.9.5 *Earthquake*

AS 1170.4—2007 introduces a performance parameter K_pZ which is calculated using return period factor determined from the annual probability of exceedance and the hazard factor for the site. Domestic structures designed and detailed for lateral wind forces in conjunction with AS 1684 or AS 1720.1 with a $K_pZ \leq 0.11$ do not require specific earthquake design. For this reason, earthquake actions are not included in the action combinations given for member design or considered necessary for the calculation of racking loads to be resisted by bracing.

1.4.9.6 *Combinations of actions*

Combinations of actions included for the determination of the strength limit states and the serviceability limit states for each member are those considered appropriate in accordance with AS/NZS 1170.1.

1.4.10 Strength limit states

For each member, all strength limit states have been considered; however, only those strength limit states deemed as potentially critical are included in the design criteria and are required to be assessed for the purposes of this Standard.

NOTE: For non-generic timber-based products, design may require consideration of strength limit states other than those included in this Standard.

1.4.11 Serviceability limit states

The serviceability limit states used for the design have been determined on the basis of experience with the known serviceability performance of individual member types in typical applications. Serviceability limits used are intended to provide satisfactory rigidity for average situations.

NOTES:

1 For installations where greater than usual rigidity may be required it is anticipated that larger sizes and or materials with higher or more uniform modulus of elasticity should be used (see AS 1720.1), e.g. granite kitchen benchtops, large ceramic floor tiles, island benches.

2 Consider the relative deformation of joists parallel and close to relatively rigid walls (either above or below the floor). Resultant localized differential deflection can cause cracking of rigid surface finishes and can cause high/narrow furniture to lean noticeably.

3 The serviceability limit states given are for individual elements. Where one element is supported by another, the cumulative deflection of the system of elements should be considered to prevent cracking of linings, cladding or glazing and to prevent result in load transfer to joinery items and consequent damage.

4 The limits on deflection used as part of the definition of the serviceability limit states are limits intended for comparison with calculated deflections only. Actual or measured deflections may differ from calculated deflections due to any or all of the following factors:

(a) Differences between actual loads and design loads used for serviceability calculations.

(b) Differences between the actual modulus of elasticity of components and the average value used for design.

(c) Differences between the structural behaviour of the system and the structural models used for design.

1.5 DEFINITIONS

For the purpose of this Standard, the following definitions apply.

1.5.1 Balcony

An external trafficable floor area of a house.

1.5.2 Birdsmouth

A triangular notch cut into the underside of a sloping beam (e.g. rafter) to permit seating on the supporting member.

1.5.3 Bracing

An assembly intended to resist racking forces including diagonal members, shear panels, diaphragms, cantilevered columns or portal (rigid) frames.

1.5.4 Cladding

Material used for the external surface of walls or roofs.

1.5.5 Deck

An external attached or detached trafficable floor area.

1.5.6 Flooring or decking

Boards or sheets overlying floor joists intended to support floor loads. Flooring is usually tongue and groove jointed along the edges whereas decking comprises individual non-connected boards.

1.5.7 Generic stress grades

Stress grades for which properties are included in AS 1720.1.

1.5.8 Lining

The materials used for the internal faces of walls or ceilings.

1.5.9 Loadbearing walls

Walls required to support vertical loads from roofs and/or floors.

NOTE: This definition differs from that given in the Building Code of Australia.

1.5.10 Nogging

A horizontal member, fitted between studs in a wall frame, which restrains the studs against buckling in the plane of the wall. Noggings may also be used for attachment of cladding or lining or as part of a bracing system.

1.5.11 Non-loadbearing walls

Partition walls not supporting roofs or floors. Non-loadbearing walls may support ceilings and may incorporate posts for the support of roof or floor loads.

NOTE: This definition differs from that given in the Building Code of Australia.

1.5.12 Sheet roofing

Includes sheet metal tile panels and other metal deck roofing of mass up to 10 kg/m^2.

1.5.13 Span

1.5.13.1 *Effective span*

The effective span of flexural members shall be taken as the distance between the centres of areas of bearing or of connections.

For members that extend over bearings longer than is necessary, it is appropriate to measure the span as the distance between centres of imaginary bearings which are chosen in such a way that their lengths are adequate to comply with the requirements of AS 1720.1.

1.5.13.2 *Single span*

The span of a member supported at or near both ends with no immediate supports. This includes the case where members are partially cut through over intermediate supports to remove spring.

1.5.13.3 *Continuous span*

Members supported at or near both ends and at one or more intermediate points such that no span is greater than twice another.

NOTE: Truss spans have traditionally been measured from outside to outside of pitching plates.

1.5.14 Standard roof truss

An engineered, triangulated framework installed at similar centres to rafters and designed to transfer roof and ceiling loads, usually, to external walls.

1.5.15 Tie-down

The connections or fixings designed to resist uplift forces due to wind.

1.5.16 Tiled roofing

Includes slate, terracotta and concrete tiles of mass up to 60 kg/m^2.

1.5.17 Wall/brick tie

A bracket connecting brick cladding to a timber wall frame.

1.6 NOTATION

Generally, the notation used in AS 1720.1, AS 1684 series, AS 4055 and the AS/NZS 1170 series is used also in this Standard. Notation specific to each clause is defined in that clause. Some general notation symbols used in this Standard are as follows:

b = breadth of member

CLW = ceiling load width

d = depth of member

FLW = floor load width

K_c = pressure combination factor (see Section 6)

L = general symbol used for span

L_o = horizontal span for rafter overhang

P = general symbol for concentrated load

RLW = roof load width

S = general symbol used for spacing

w = general symbol for distributed load

SECTION 2 DESIGN OF ROOF MEMBERS

2.1 ROOF BATTENS

2.1.1 Description

A roof batten is a rectangular section used on its flat to provide direct support for sheet or tile roofing. Spans for roof battens are limited to 1200 mm. For tile roofs a standard spacing of 330 mm is considered whereas for sheet roofs, spacings up to 1200 mm are included.

Battens are assumed to span continuously over rafters (or trusses) for at least two spans (see Figure 2.1.1).

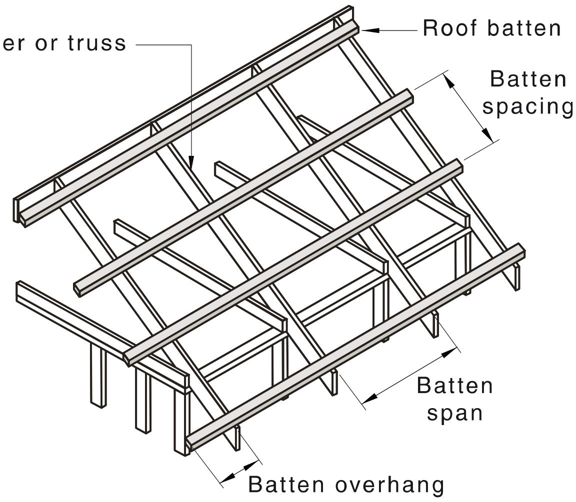


FIGURE 2.1.1 ROOF BATTENS

2.1.2 Design for safety

2.1.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending about the minor axis only and shear.

NOTE: Battens are assumed prevented from bending in the plane of the roof by the attached cladding.

2.1.2.2 *Design actions*

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

(a) *Permanent* Permanent actions, G, corresponding to the typical roof constructions, are determined from Table 2.1.2.2(A).

TABLE 2.1.2.2(A)

PERMANENT ACTIONS FOR ROOF BATTENS

Roof type	Permanent action, (G) kN/m
Sheet roof	$0.1S$ + self weight
Tile roof	$0.6S$ + self weight

LEGEND:
S = spacing of roof battens, in metres
Self weight = roof batten self weight, in kN/m

(b) *Imposed* The uniformly distributed imposed action, Q_1 (in kN/m), and concentrated imposed actions, Q_2 and Q_3 (both in kN), used for design are determined from—

(i) $Q_1 = 0.25S$ . . . 2.1.2.2(1)

(ii) $Q_2 = g_{44} \times 1.1$ . . . 2.1.2.2(2)

(iii) $Q_3 = g_{45} \times 1.1$ . . . 2.1.2.2(3)

where

g_{44} = the lesser of 1.33S and 1.0

S = spacing of roof battens, in metres

and

g_{45} is calculated in accordance with Appendix B3, assuming a bargeboard of rigidity $E_f I_f = 18 \times 10^9$ Nmm^2 is attached to the ends of the parallel overhanging battens, and $g_{47} = 1.0$ (i.e. no birdsmouth notch).

NOTE: The load distribution factor g_{44} is taken from CSIRO, *Low-rise domestic and similar framed structures* (see Clause 1.2). The use of this load distribution factor is based upon construction workers following the traditional practice of not treading at or near mid-span of closely spaced battens prior to the installation of roof claddings.

(c) *Wind* The wind action, W_u (in kN/m), applicable for the strength limit state, is determined from—

$$W_u = q_u C_{pt} S \quad \text{. . . 2.1.2.2(4)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.1.2.2(B)

S = spacing of roof battens, in metres

TABLE 2.1.2.2(B)

NET PRESSURE COEFFICIENTS FOR ROOF BATTENS

Wind classification	C_{pt}		
	General areas	**Areas within 1.2 m of an edge**	**Areas within 1.2 m of an eaves corner**
N1 to N4	+0.63, −1.0	−1.8	−2.61
C1 to C3	+0.95, −1.44	−2.25	−3.06

NOTES:

1 Local pressure effects may be catered for by specifying reductions in batten spacing near edges and eaves corners, as appropriate.

2 The value of C_{pt} for areas within 1.2 m of an eaves corner applies only to roof slopes less than 10°. Where there is no internal pressure, such as in open verandah or eave construction, this value of C_{pt} shall be increased to −2.7.

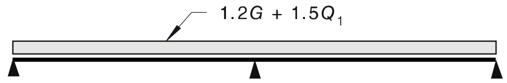
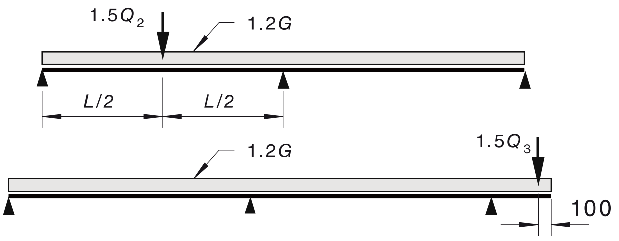
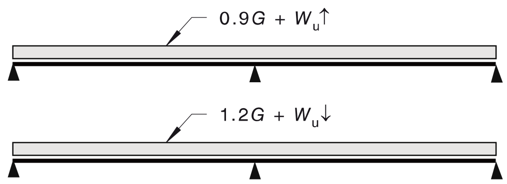
3 Values given in this Table are based on the assumption that a separate ceiling is provided with the maximum internal pressure coefficient (C_{pi}) for roof cavities in cyclonic regions considered as for non-cyclonic regions.

2.1.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.1.2.3. Action combinations shown in Table 2.1.2.3 are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.1.2.4.

TABLE 2.1.2.3

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Action category	Structural model
1	
2	
3	
4	

2.1.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category are given in Table 2.1.2.4.

TABLE 2.1.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.94
3	0.97
4	1.00

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for Action category 4 given in Table 2.1.2.3, use values of k_4 appropriate for thickness as specified in AS 1720.1. For Action categories 1, 2 and 3, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Member restraint* For battens, breadth is greater than or equal to depth and, hence, the lateral stability factor k_{12} = 1.0.

2.1.3 Design for serviceability

2.1.3.1 *Design actions*

The actions used for the serviceability limit states shall be as follows:

(a) *Permanent* Permanent actions, G, corresponding to various typical roof constructions are determined from Table 2.1.2.2(A).

(b) *Wind* The uniformly distributed wind action, W_s (in kN/m), applicable for the serviceability limit state is determined from—

$$W_s = q_s C_{pt} S \qquad \text{. . . 2.1.3.1}$$

where

q_s = free stream dynamic gust pressure, in kPa, for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.1.2.2(B)

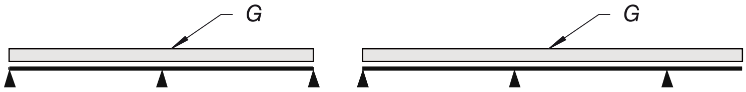
S = spacing of roof battens, in metres

2.1.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.1.3.2. Action cases included in Table 2.1.3.2 are divided into action categories for the purpose of allowing for the effect of duration of load on stiffness as specified in Clause 2.1.3.3.

TABLE 2.1.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

<table>
<tr><th>Action category</th><th>Structural model</th></tr>
<tr><td>1</td><td>
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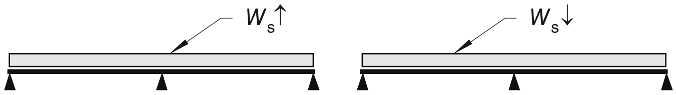
<tr><td>2</td><td>
</td></tr>
</table>

2.1.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deformation shall be applied using the duration of load factor for creep deformation as given in Table 2.1.3.3.

TABLE 2.1.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1 (permanent loads)	Action category 2 (transient loads)
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.1.3.4 *Serviceability limits*

The limits on deflection defining the serviceability limit state are given in Table 2.1.3.4.

TABLE 2.1.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits	
	Mid-span	**End of overhang**
1	Span/300	Overhang/150 or 4 mm whichever is greater(see Note)
2	Span/150	No limitation

NOTE: Limit shall be ignored for upwards deflection.

2.2 RAFTERS

2.2.1 Description

Rafters are roof members that run parallel to the fall of the roof and support roof battens or purlins. They may also support ceilings, either directly or via ceiling battens or joists.

Rafters may be either single span or continuous span and may be cantilevered to form an eaves overhang either with or without a birdsmouth notch at the overhang support. Continuous span rafters are assumed not notched at intermediate supports.

For the determination of the maximum overhang the ends of rafters are assumed rigidly connected to a fascia that acts to share any concentrated loads to adjacent members [see Figure 2.2.1 and Clause 2.2.2.2(b)(iii)].

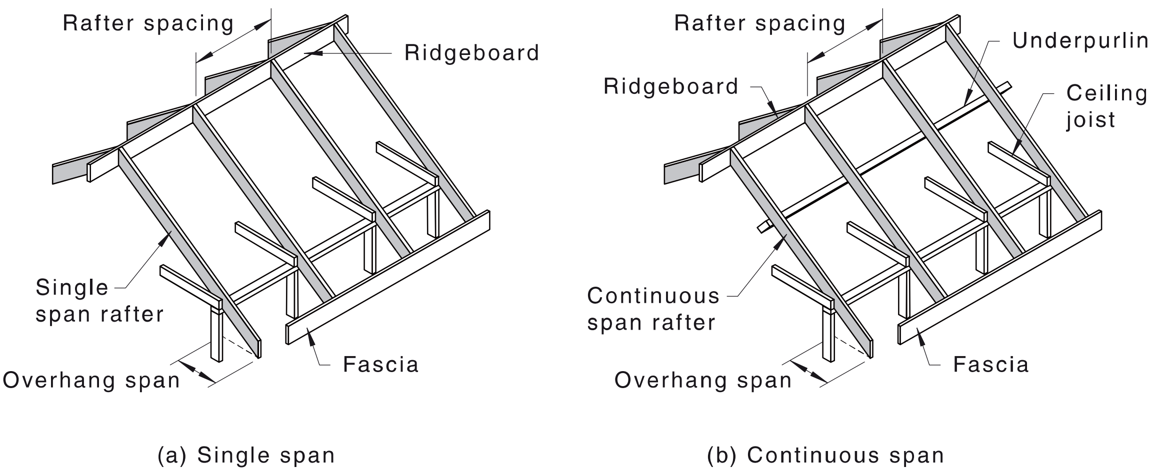


FIGURE 2.2.1 RAFTERS

2.2.2 Design for safety

2.2.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending and shear. In addition, for birdsmouth notches associated with overhangs, the interaction of bending and shear shall also be considered.

2.2.2.2 *Design actions*

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

(a) *Permanent* Permanent actions, G, corresponding to various typical roof constructions, are determined from Table 2.2.2.2(A).

TABLE 2.2.2.2(A)

PERMANENT ACTION

Roof type	Permanent action, (G) kN/m
Sheet roof only	$0.1S$ + self weight
	$0.2S$ + self weight
Sheet roof and ceiling	$0.4S$ + self weight
Tile roof only	$0.6S$ + self weight
Tile roof and ceiling	$0.9S$ + self weight

LEGEND:
S = spacing of rafters, in metres
Self weight = rafter self weight, in kN/m

(b) *Imposed* The distributed imposed action, Q_1 (in kN/m), and concentrated imposed actions Q_2 and Q_3 (in kN), are determined from—

(i) $Q_1 = 0.25S$ . . . 2.2.2.2(1)

(ii) $Q_2 = g_{42} \times 1.1$ . . . 2.2.2.2(2)

(iii) $Q_3 = g_{45} \times 1.1$ . . . 2.2.2.2(3)

where

S = spacing of rafters, in metres

g_{42} = load distribution factor for concentrated load, applied to a grid system, calculated in accordance with AS 1720.1, assuming the crossing members are battens with rigidity and spacing as follows:

(1) *Sheet roofs*: $E_cI_c = 2.3 \times 10^9$ Nmm^2, and spacing = 1200 mm.

(2) *Tile roofs*: $E_cI_c = 380 \times 10^6$ Nmm^2, and spacing = 330 mm.

g_{45} = load distribution factor for parallel rafter overhangs, calculated as detailed in Appendix B for the case where the depth of the birdsmouth notch is no greater than one third of the rafter depth and a structural fascia of minimum rigidity 86×10^9 Nmm^2 is attached to the end of each rafter. Where these conditions are not met, $g_{45} = 1$

(c) *Wind* The wind action, W_u (in kN/m), applicable for the strength limit state is determined from—

$$W_u = q_u C_{pt} S \qquad \text{. . . 2.2.2.2(4)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.2.2.2(B)

S = spacing of rafters, in metres

TABLE 2.2.2.2(B)

NET PRESSURE COEFFICIENTS FOR RAFTERS—STRENGTH

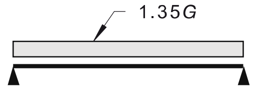
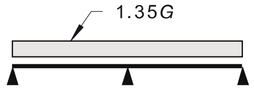
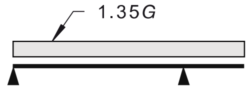
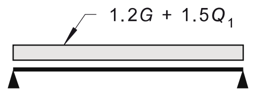
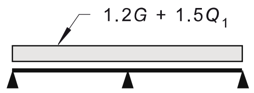
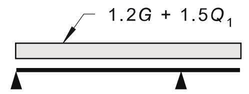
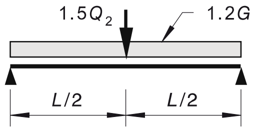
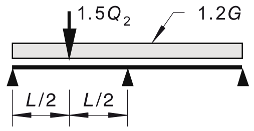
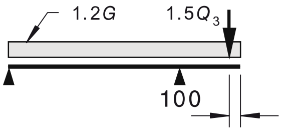
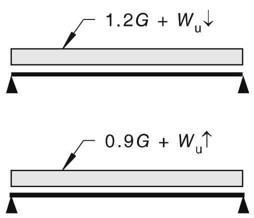
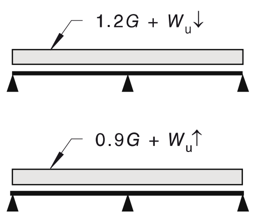
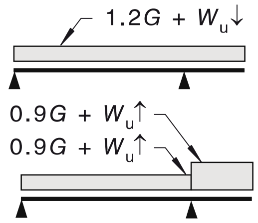
Wind classification	C_{pt}	
	Main spans	Overhang
N1 to N4	+0.63 or −0.99	+0.95 or −1.44
C1 to C3	+0.95 or −1.44	+0.95 or −1.44

2.2.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.2.2.3. Action combinations shown in Table 2.2.2.3 are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.2.2.4.

TABLE 2.2.2.3

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Action category	Structural model		
	Single span	Continuous span	Overhang
1			
2			
3			
4			

2.2.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. In addition, for birdsmouth notches associated with rafter overhangs, the procedures given in Appendix B shall be applied, assuming the notch depth is one third of the rafter depth. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.2.2.3, are given in Table 2.2.2.4.

TABLE 2.2.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.94
3	0.97
4	1.00

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for Action category 4 given in Table 2.2.2.3, use values of k_4 appropriate for thickness as specified in AS 1720.1. For action categories 1, 2 and 3, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* For scantling timber, the strength sharing factor (k_9) is applied as follows:

(i) For calculating maximum main spans, k_9 is determined in accordance with AS 1720.1, assuming n_{mem} = 5 and n_{com} = 1 (for single members).

(ii) For calculating maximum overhangs and for negative moment only (see Appendix B)—

$$k_9 = 1.24 - 0.24\,(S/L_o), \text{ but not less than } 1.0 \quad \ldots 2.2.2.4$$

where

S = spacing of rafters

L_o = horizontal span of the overhang

(d) *Member restraint* For the determination of bending capacity the following assumptions related to lateral restraint are used:

(i) *At supports*—rafters are assumed torsionally restrained at their supports.

(ii) *Between supports*—

(A) the top edges of rafters are assumed laterally restrained by battens, or purlins, at 330 mm centres for tile roofs and 1200 mm centres for sheet roofs; and

(B) *in addition*, continuous span rafters are assumed restrained against torsional buckling at the points of contraflexure taken as one quarter of the span from the intermediate support.

2.2.3 Design for serviceability

2.2.3.1 *Design actions*

The actions used for the purpose of assessing the serviceability limit states shall be as follows:

(a) *Permanent and imposed* Permanent actions and imposed actions are determined as described in Clause 2.2.2.2.

(b) *Wind* The uniformly distributed wind action, W_s (in kN/m), applicable for the serviceability limit state is determined from—

$$W_s = q_s C_{pt} S \quad \ldots 2.2.3.1$$

where

q_s = free stream dynamic gust pressure, in kPa, for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.2.3.1

S = spacing of rafters, in metres

TABLE 2.2.3.1

NET PRESSURE COEFFICIENTS FOR RAFTERS—SERVICEABILITY

Wind classification	C_{pt}	
	Main spans	Overhang
N1 to N4 and C1 to C3	+0.63 or −0.99	+0.95 or −1.44

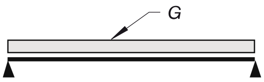
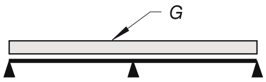
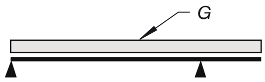
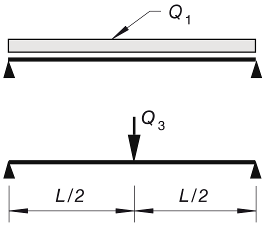
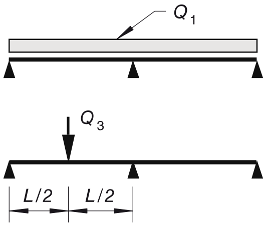
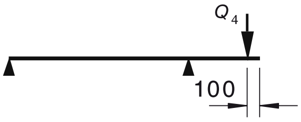
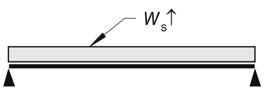
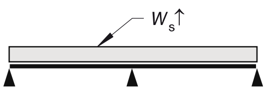
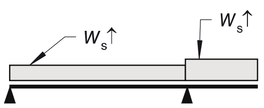
2.2.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.2.3.2.

Action cases included in Table 2.2.3.2 are divided into action categories for the purpose of allowing for the effect of duration of load on stiffness, as specified in Clause 2.2.3.3.

TABLE 2.2.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural model		
	Single span	Continuous span	Overhang (cantilevered)
1			
2			
3			

2.2.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.2.3.4(A). In addition, the deflection at the ends of overhangs for birdsmouth-notched rafters shall be determined using the modified rafter rigidity given in Appendix B.

2.2.3.4 *Serviceability limits*

The limits on deflection, defining the serviceability limit state, are given in Table 2.2.3.4(B).

TABLE 2.2.3.4(A)

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2 or 3
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

TABLE 2.2.3.4(B)

LIMITS ON DEFLECTION

Action category	Deflection limits	
	Mid-span	End of overhang
1	Span/300 or 20 mm max.	10 mm
2	Span/250	10 mm
3	Span/150	20 mm

2.3 ROOF BEAMS—RIDGE OR INTERMEDIATE BEAMS

2.3.1 Description

Ridge or intermediate beams are roof beams that support rafters, which in turn support roof or roof and ceiling loads. Roof beams run perpendicular to the slope of the roof, are either single or continuous span and may cantilever to support a verge overhang. Overhang spans are determined assuming roof beams are not notched at the overhang support.

For the purpose of determining lateral stability, roof beams are assumed to be laterally restrained by rafters fixed to their top edge (see Figure 2.3.1).

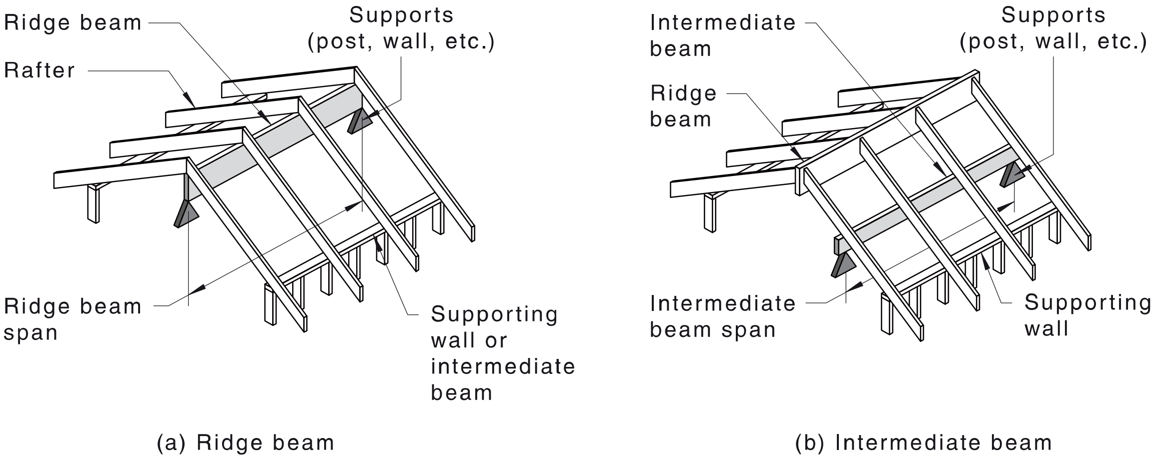


FIGURE 2.3.1 ROOF BEAMS—RIDGE OR INTERMEDIATE BEAM

2.3.2 Design for safety

2.3.2.1 *General consideration*

Roof beam design for safety requires consideration of the strength limit state for bending, shear and bearing.

2.3.2.2 *Design actions*

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

(a) *Permanent* The uniformly distributed permanent action, G (in kN/m), corresponding to various typical roof constructions with additional allowance for the weight of the rafters, is determined from—

$$G = 0.01(RM)(RLW) + 0.02(RLW)^2 + \text{self weight} \quad \ldots 2.3.2.2(1)$$

where

RM = standardized roof mass, i.e. 10, 20, 40, 60 or 90 kg/m^2

RLW = roof load width for the roof beam, in metres

self weight = roof beam self weight, in kN/m

(b) *Imposed* The distributed imposed action, Q_1 (in kN/m), and concentrated imposed action, Q_2 (in kN), are determined from—

(i) $Q_1 = 0.25(RLW)$ . . . 2.3.2.2(2)

(ii) $Q_2 = 1.1$ . . . 2.3.2.2(3)

where

RLW = roof load width for the roof beam, in metres

(c) *Wind* The uniformly distributed wind action, W_u (in kN/m), applicable for the strength limit state is determined from—

$$W_u = q_u C_{pt}(RLW) \quad \ldots 2.3.2.2(4)$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.3.2.2

RLW = roof load width for roof beam, in metres

TABLE 2.3.2.2

NET PRESSURE COEFFICIENTS FOR ROOF BEAMS— STRENGTH

Wind classification	C_{pt}	
	Main spans	**Overhang**
N1 to N4	+0.63 or −0.99	+0.95 or −1.44
C1 to C3	+0.95 or −1.44	+0.95 or −1.44
N1 to N4 and C1 to C3 for alfresco and carports	+0.95 or −1.44	+0.95 or −1.44

2.3.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.3.2.4(A). Action combinations shown in Table 2.3.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.3.2.4.

2.3.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending, shear and bearing. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.3.2.4(A), are given in Table 2.3.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 3 and 4 given in Table 2.3.2.4(A), use values of k_4 appropriate for thickness as given in AS 1720.1. For action categories 1 and 2, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

TABLE 2.3.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

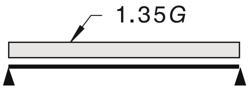
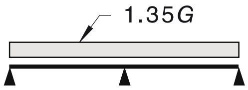
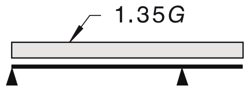
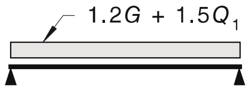
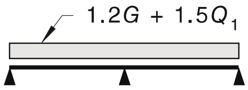
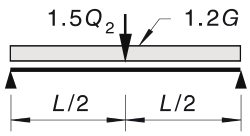
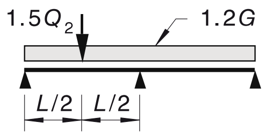
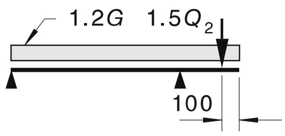
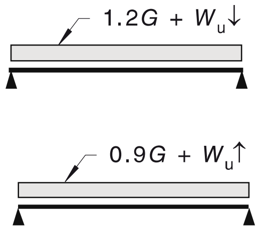
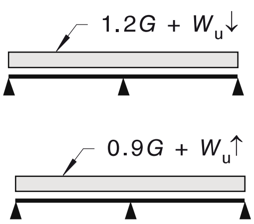
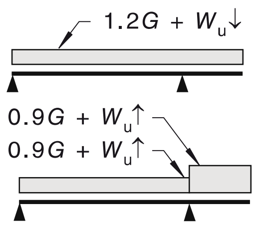
Action category	Structural model		
	Single span	Continuous span	Overhang
1	 1.35G	 1.35G	 1.35G
2	 1.2G + 1.5Q_1	 1.2G + 1.5Q_1	
3	 1.5Q_2; 1.2G; L/2; L/2	 1.5Q_2; 1.2G; L/2; L/2	 1.2G; 1.5Q_2; 100
4	 1.2G + W_u↓; 0.9G + W_u↑	 1.2G + W_u↓; 0.9G + W_u↑	 1.2G + W_u↓; 0.9G + W_u↑; 0.9G + W_u↑

TABLE 2.3.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.94
3	0.97
4	1.00

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1 and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity, the following assumptions relating to lateral restraint are used:

(i) *At supports*—roof beams are assumed torsionally restrained at their supports.

(ii) *Between supports*:

(A) The top edges of roof beams are assumed restrained at 1200 mm centres.

(B) Continuous span roof beams are assumed restrained against buckling at the points of contraflexure.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

2.3.3 Design for serviceability

2.3.3.1 *Design actions*

The actions used for the serviceability limit state shall be as follows:

(a) *Permanent and imposed* Permanent actions and imposed actions are determined as described in Clause 2.3.2.2.

(b) *Wind* The uniformly distributed wind action, W_s (in kN/m), applicable for the serviceability limit state, is determined from—

$$W_s = q_s C_{pt}(RLW) \quad \text{. . . 2.3.3.1}$$

where

q_s = free stream dynamic gust pressure, in kPa, for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.3.3.1

RLW = roof load width for roof beam, in metres

TABLE 2.3.3.1

NET PRESSURE COEFFICIENTS FOR ROOF BEAMS— SERVICEABILITY

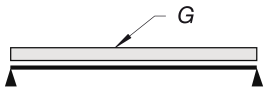
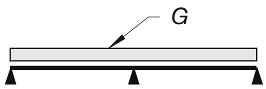
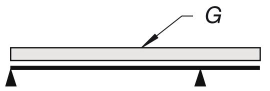
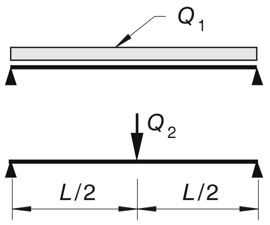
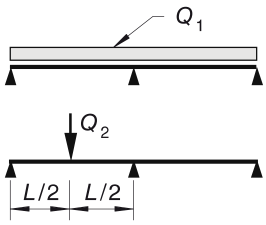
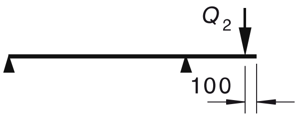
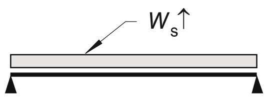
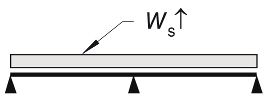
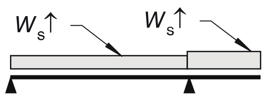
Wind classification	C_{pt}	
	Main spans	Overhangs
N1 to N4 and C1 to C3	+0.63 or −0.99	−0.9

2.3.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.3.3.2. Action cases included in Table 2.3.3.2 are divided into action categories for the purpose of allowing for the effect of duration of load on stiffness, as specified in Clause 2.3.3.3.

TABLE 2.3.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural model		
	Single span	Continuous span	Overhang (cantilevered)
1			
2			
3			

2.3.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.3.3.3.

TABLE 2.3.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2 or 3
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.3.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.3.3.4.

TABLE 2.3.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits	
	Mid-span	End of overhang
1	Span/300 or 20 mm max.	10 mm
2	Span/250	10 mm
3	Span/150	30 mm

2.4 UNDERPURLINS

2.4.1 Description

Underpurlins provide intermediate support for rafters in coupled roof construction. They are orientated as shown in Figure 2.4.1 and primarily support roof loads normal to the plane of the roof over the middle part of the rafter length.

Sections with depth to overall breadth ratios greater than four are not considered for application as underpurlins. Further, where the depth to overall breadth ratio exceeds two, underpurlins are assumed torsionally braced at supports and fly-braced back to rafters at intervals not exceeding 1200 mm along their span. These requirements are intended to minimize weak axis sag that may reduce support to rafters and/or induce buckling, particularly for more steeply pitched roofs.

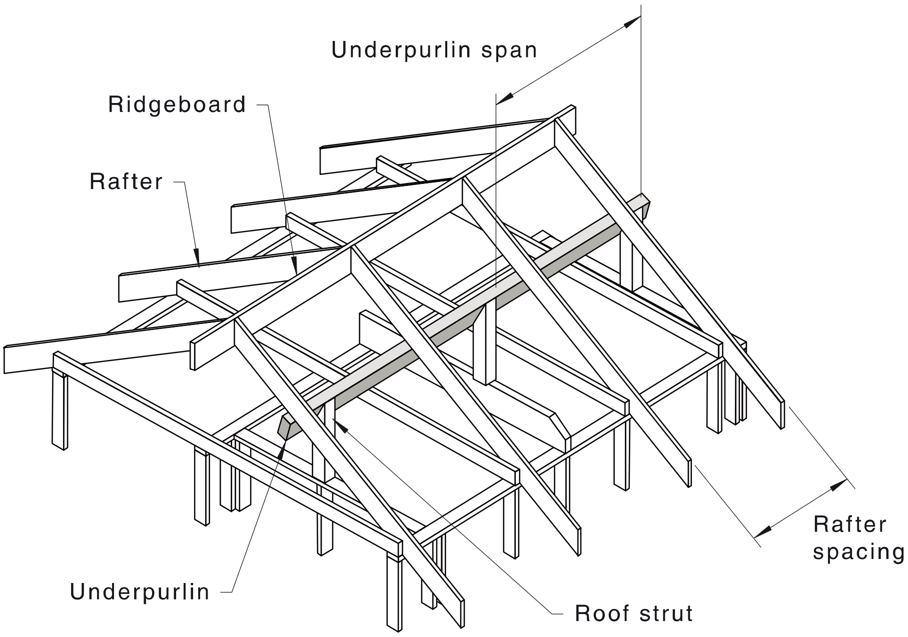


FIGURE 2.4.1 UNDERPURLINS

2.4.2 Design for safety

2.4.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in bending and shear.

2.4.2.2 *Design actions*

The actions used for determination of the design actions effects shall be as follows:

(a) *Permanent* Permanent actions include the self weight of the underpurlin, G_1 (in kN/m), and concentrated action, G_2 (in kN), imposed by the rafters. G_2 is determined from—

$$G_2 = 1.25(0.01RM)S_R(RLW) \quad \text{. . . 2.4.2.2(1)}$$

where

RM = standardized roof mass, i.e. 10, 20 or 60 kg/m^2

S_R = spacing of rafters, i.e. 0.6 m or 1.2 m

RLW = roof load width for underpurlin, in metres

NOTE: The 1.25 factor in Equation 2.4.2.2(1) provides an allowance for the weight of supported rafters and the effect of their continuity.

(b) *Imposed* Imposed actions applied via rafters are considered as concentrated actions, Q_1 (in kN), and are determined as follows:

$$Q_1 = 0.25S_R(RLW) \quad \text{. . . 2.4.2.2(2)}$$

where

S_R = spacing of rafters, i.e. 0.6 m or 1.2 m

RLW = roof load width for underpurlins, in metres

(c) *Wind* Wind actions are considered as concentrated actions (W_u), imposed via the rafters. Concentrated actions, W_u (in kN), are determined as follows:

$$W_u = q_u C_{pt} S_R (RLW) \quad \text{. . . 2.4.2.2(3)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.4.2.2

S_R = spacing of rafters, i.e. 0.6 m or 1.2 m

RLW = roof load width for underpurlin, in metres

TABLE 2.4.2.2

NET PRESSURE COEFFICIENTS FOR UNDERPURLINS

Wind classification	C_{pt}
N1 to N4	+0.63 or −0.99
C1 to C3	+0.95 or −1.44

NOTE: Values given in this Table are based on the assumption that a separate ceiling is provided with the maximum internal pressure coefficient (C_{pi}) for roof cavities in cyclonic regions considered as for non-cyclonic regions.

2.4.2.3 *Structural models and action categories used for strength design*

The structural models used to determine the member design action effects shall be as given in Table 2.4.2.4(A). Action combinations shown in Table 2.4.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.4.2.4.

2.4.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category defined in Table 2.4.2.4(A) are given in Table 2.4.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3, use values of k_4 appropriate for member thickness as given in AS 1720.1. For Action category 1, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1 and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity, the following assumptions related to lateral restraint are used:

(i) *At supports*—underpurlins are considered torsionally restrained at their supports.

(ii) *Between supports*:

(A) The top edges of underpurlins are assumed restrained by rafters at 600 mm or 1200 mm centres, as appropriate.

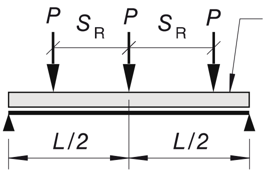
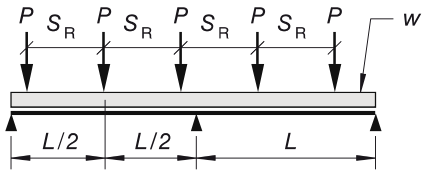
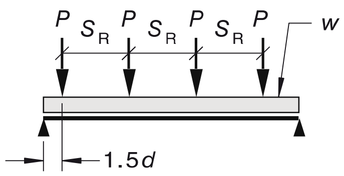
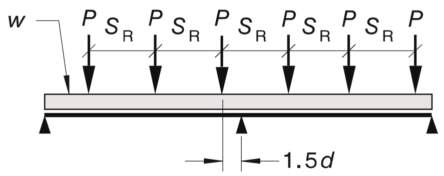
(B) Underpurlins with a depth to overall breadth ratio greater than two are assumed torsionally restrained at 1200 mm centres.

(C) Continuous span underpurlins are assumed restrained against buckling at the points of contraflexure.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination and not the overall breadth.

TABLE 2.4.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Design action effect	Structural models	
	Single span	Continuous span
In bending		
In shear		

Action category	Design actions
1	$w = 1.35G_1$ and $P = 1.35G_2$
2	$w = 1.2G_1$ and $P = (1.2G_2 + 1.5Q_1)$
3	$w = 1.2G_1$ and $P = (1.2G_2 + W_u\downarrow)$ $w = 0.9G_1$ and $P = (0.9G_2 + W_u\uparrow)$

NOTES:

1 S_R is rafter spacing, either 0.6 m or 1.2 m.

2 The number of concentrated actions considered will vary according to span, rafter spacing and locations of concentrated actions.

3 Actions within 1.5d of supports are ignored in the determination of the design action effect in shear.

TABLE 2.4.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

2.4.3 Design for serviceability

2.4.3.1 *Design actions*

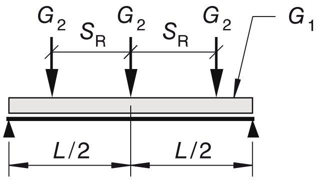
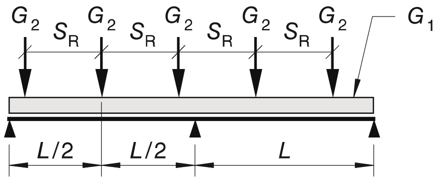
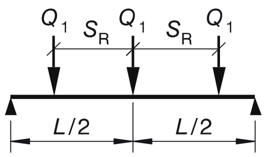
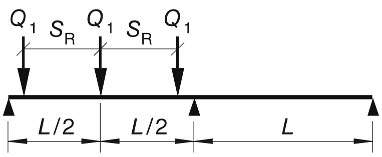
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.4.2.2.

2.4.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.4.3.2. Action cases given in Table 2.4.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.4.3.3.

TABLE 2.4.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models	
	Single span	Continuous span
1		
2		

LEGEND:
S_R = rafter spacing.

2.4.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.4.3.3.

TABLE 2.4.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.4.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.4.3.4.

TABLE 2.4.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300
2	Span/250

2.5 STRUTTING BEAMS

2.5.1 Description

Strutting beams are horizontal or near horizontal single span beams, installed within the roof space clear of ceilings, which provide support to underpurlins via roof struts.

While strutting beams may be loaded by one or more struts located anywhere within the span, the design procedures given conservatively assume all roof load is applied via a single strut.

Strutting beams are assumed torsionally braced at supports and at midspan (see Figure 2.5.1).

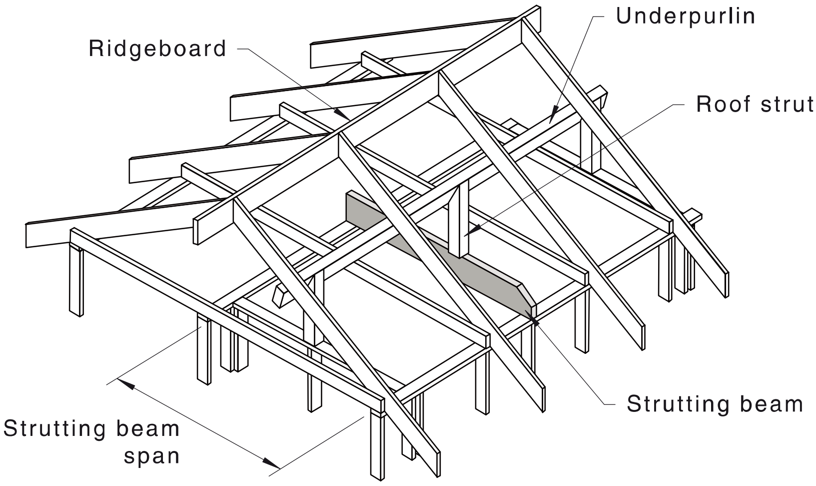


FIGURE 2.5.1 STRUTTING BEAMS

2.5.2 Design for safety

2.5.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending and shear.

2.5.2.2 *Design actions*

Roof actions applied to strutting beams are calculated on the basis of roof area supported.

Design actions shall be as follows:

(a) *Permanent* Permanent actions for strutting beams include the self weight of the strutting beam, G_1 (in kN/m), and the roof mass as a concentrated action, G_2 (in kN), determined as follows:

$$G_2 = 0.01(RM + 10)A \quad \text{. . . 2.5.2.2(1)}$$

where

RM = standardized roof mass allowance, i.e. 20 kg/m^2 for sheet roofs and 60 kg/m^2 for tile roofs

A = area of roof supported by the strutting beam, in square metres

(b) *Imposed* Imposed roof action is considered applied as a concentrated action, Q_1 (in kN), determined as follows:

$$Q_1 = 0.25A \quad \text{. . . 2.5.2.2(2)}$$

where

A = area of roof supported by strutting beam, in square metres

(c) *Wind* Wind action applicable for the strength limit state is considered applied as a concentrated action W_u (in kN), determined as follows:

$$W_u = q_u C_{pt} A \qquad \text{. . . 2.5.2.2(3)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.5.2.2

A = area of roof supported by the strutting beam, in square metres

TABLE 2.5.2.2

NET PRESSURE COEFFICIENTS FOR STRUTTING BEAMS

Wind classification	C_{pt}
N1 to N4	+0.63 or −1.0
C1 to C3	+0.95 or −1.44

NOTE: Values given in this Table are based on the assumption that a separate ceiling is provided with the maximum internal pressure coefficient (C_{pi}) for roof cavities in cyclonic regions considered as for non-cyclonic regions.

2.5.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effect shall be as given in Table 2.5.2.4(A). Action combinations shown in Table 2.5.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.5.2.4.

2.5.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.5.2.4(A), are given in Table 2.5.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3 given in Table 2.5.2.4(A), use values of k_4 appropriate for thickness as given in AS 1720.1. For Action category 1, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming $n_{mem} = 1.0$ and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity the following assumptions relating to lateral restraint are used:

(i) *At supports*—strutting beams are assumed torsionally restrained at their supports.

(ii) *Between supports*—strutting beams having a depth to breadth ratio greater than three are assumed torsionally restrained at midspan (the assumed load point).

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

TABLE 2.5.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

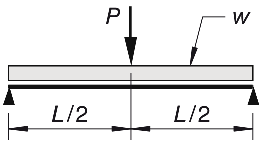
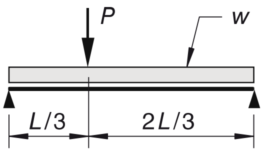
Design action effect	Structural models
In bending	
In shear	
Action category	**Design actions**
1	$w = 1.35G_1$ and $P = 1.35G_2$
2	$w = 1.2G_1$ and $P = (1.2G_2 + 1.5Q_1)$
3	$w = 1.2G_1$ and $P = (1.2G_2 + W_u\downarrow)$ $w = 0.9G_1$ and $P = (0.9G_2 + W_u\uparrow)$

TABLE 2.5.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

2.5.3 Design for serviceability

2.5.3.1 *Design actions*

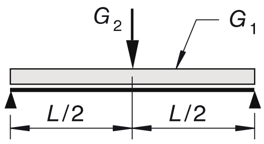
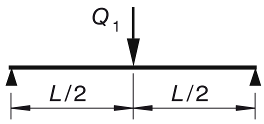
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.5.2.2.

2.5.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.5.3.2. Action cases given in Table 2.5.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.5.3.3.

TABLE 2.5.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models
1	
2	

2.5.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.5.3.3.

TABLE 2.5.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.5.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.5.3.4.

TABLE 2.5.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 20 mm max.
2	Span/250 or 20 mm max.

2.6 COUNTER STRUTTING BEAMS

2.6.1 Description

Counter strutting beams support roof loads from roof struts and ceiling loads from hanging beams.

For design, loading from both roof and ceiling is considered concentrated at midspan.

Counter strutting beams are assumed torsionally braced at their supports and at midspan by the attachment of the hanging beams (see Figure 2.6.1).

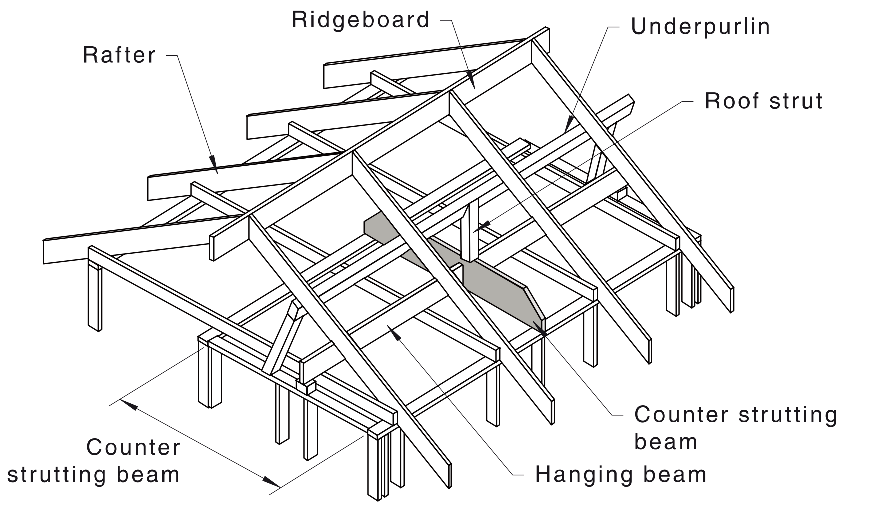


FIGURE 2.6.1 COUNTER STRUTTING BEAM

2.6.2 Design for safety

2.6.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in bending and shear.

2.6.2.2 *Design actions*

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

(a) *Permanent* Permanent actions include the self weight of the counter strutting beam G_1 (in kN/m) and the concentrated action due to the roof and ceiling mass, G_2 (in kN), determined from—

$$G_2 = 0.01(RM + 10)\mathrm{A} + (0.06L + 0.005L^2)(CLW) \qquad \ldots 2.6.2.2(1)$$

where

RM = standardized roof mass allowance, i.e. 20 kg/m^2 for sheet roofs and 60 kg/m^2 for tile roofs

A = area of roof supported by the counter strutting beam, in square metres

L = span of the counter strutting beam, in metres

CLW = ceiling load width for the counter strutting beam, in metres

(b) *Imposed* Imposed roof action is considered as a concentrated action, Q_1 (in kN), applied via a roof strut and determined from—

$$Q_1 = 0.25A \qquad \ldots 2.6.2.2(2)$$

where

A = roof area supported by the counter strutting beam, in square metres

(c) *Wind* Wind action is considered applied as a concentrated action, W_u (in kN), applied via a single roof strut and determined from—

$$W_u = q_u C_{pt} A \qquad \ldots 2.6.2.2(3)$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.6.2.2

A = roof area supported by the counter strutting beam, in square metres

TABLE 2.6.2.2

NET PRESSURE COEFFICIENTS FOR COUNTER STRUTTING BEAM

Wind classification	C_{pt}
N1 to N4	+0.63 or −1.0
C1 to C3	+0.95 or −1.44

2.6.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.6.2.4(A). Action combinations shown in Table 2.6.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.6.2.4.

2.6.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.6.2.2, are given in Table 2.6.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3 given in Table 2.6.2.4(A), use values of k_4 appropriate for thickness as specified in AS 1720.1. For Action category 1, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1.0 and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity the following assumptions relating to lateral restraint are used:

(i) *At supports*—counter strutting beams are assumed torsionally restrained at their supports.

(ii) *Between supports*—counter strutting beams are assumed torsionally restrained at midspan.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

TABLE 2.6.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

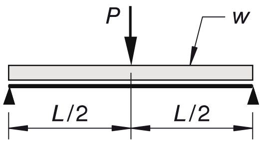
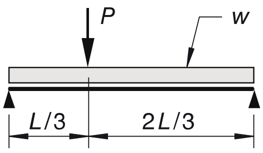
Design action effect	Structural models
In bending	
In shear	
Action category	**Design actions**
1	$w = 1.35G_1$ and $P = 1.35G_2$
2	$w = 1.2G_1$ and $P = (1.2G_2 + 1.5Q_1)$
3	$w = 1.2G_1$ and $P = (1.2G_2 + W_u\downarrow)$ $w = 0.9G_1$ and $P = (0.9G_2 + W_u\uparrow)$

TABLE 2.6.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

2.6.3 Design for serviceability

2.6.3.1 *Design actions*

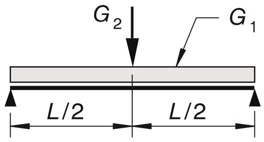
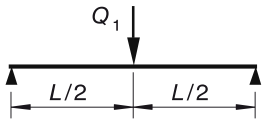
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.6.2.2.

2.6.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.6.3.2. Action cases given in Table 2.6.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.6.3.3.

TABLE 2.6.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models
1	
2	

2.6.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.6.3.3.

TABLE 2.6.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.6.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.6.3.4.

TABLE 2.6.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 12 mm max.
2	Span/300 or 12 mm max.

2.7 COMBINED HANGING STRUTTING BEAMS

2.7.1 Description

Combined hanging strutting beams support roof loads applied via roof struts to the top edge and ceiling loads from ceiling joists along the bottom edge.

For design, roof loads are conservatively assumed applied via a single roof strut and ceiling loads are assumed uniformly distributed (see Figure 2.7.1).

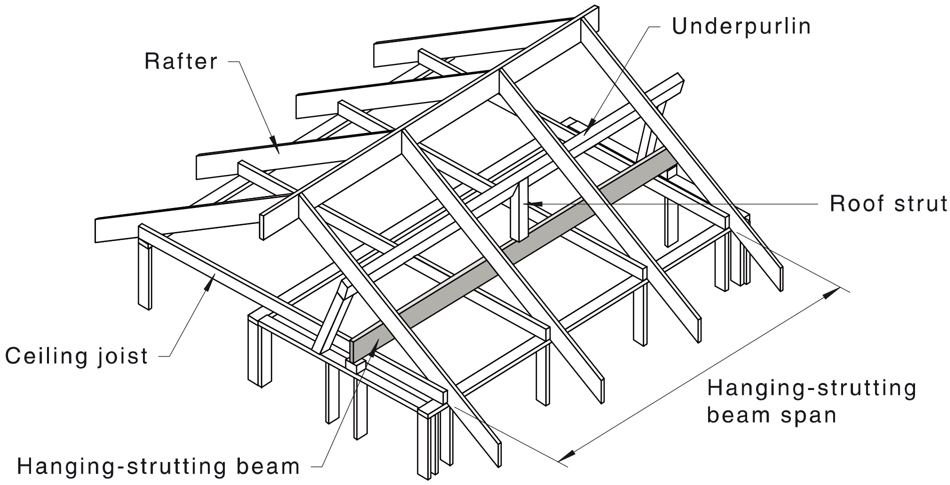


FIGURE 2.7.1 COMBINED HANGING-STRUTTING BEAM

2.7.2 Design for safety

2.7.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending and shear.

2.7.2.2 *Design actions*

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

(a) *Permanent* Permanent actions include the distributed action due to self weight and the weight of the ceiling, G_1 (in kN/m), and the concentrated action due to the weight of the roof, G_2 (in kN), with G_1 and G_2 determined from—

(i) $G_1 = 0.12(CLW) + 0.02(CLW)^2 + \text{self weight}$ . . . 2.7.2.2(1)

(ii) $G_2 = 0.01(RM + 10)A$ . . . 2.7.2.2(2)

where

CLW = ceiling load width for combined hanging strutting beam, in metres

RM = standardized roof mass allowance, i.e. 20 kg/m^2 for sheet roofs and 60 kg/m^2 for tile roofs self weight = hanging-strutting beam self weight in kN/m

(b) *Imposed* Imposed roof action is considered as a concentrated action, Q_1 (in kN), applied via a single roof strut and determined from—

$Q_1 = 0.25A$ . . . 2.7.2.2(3)

where

A = roof area supported by the combined hanging strutting beam, in square metres

(c) *Wind* Wind action is considered as a concentrated action, W_u (in kN) applied via a single roof strut and determined from—

$W_u = q_u C_{pt} A$ . . . 2.7.2.2(4)

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.7.2.2

A = roof area supported by the combined hanging strutting beam, in square metres

TABLE 2.7.2.2

NET PRESSURE COEFFICIENTS FOR COMBINED HANGING STRUTTING BEAM

Wind classification	C_{pt}
N1 to N4	+0.63 or −1.0
C1 to C3	+0.95 or −1.44

2.7.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.7.2.4(A). Action combinations shown in Table 2.7.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.7.2.4.

2.7.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.7.2.4(A) are given in Table 2.7.2.4(B).

(b) *Moisture content of timber*:

  (i) *Unseasoned timber*—for action categories 2 and 3 given in Table 2.7.2.4(A), use values of k_4 appropriate for thickness as specified in AS 1720.1. For Action category 1, k_4 = 1.0.

  (ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1.0 and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity, the following assumptions relating to lateral restraint are used:

  (i) *At supports*—combined hanging strutting beams are assumed torsionally restrained at their supports.

  (ii) *Between supports*—combined hanging strutting beams are assumed laterally restrained by ceiling joists at maximum 600 mm centres along their bottom edge.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

TABLE 2.7.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

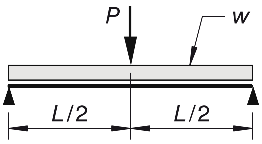
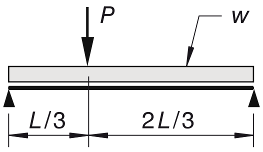
<table>
<tr><th>Design action effect</th><th>Structural models</th></tr>
<tr><td>In bending</td><td>
</td></tr>
<tr><td>In shear</td><td>
</td></tr>
<tr><th>Action category</th><th>Design actions</th></tr>
<tr><td>1</td><td>$w = 1.35G_1$ and $P = 1.35G_2$</td></tr>
<tr><td>2</td><td>$w = 1.2G_1$ and $P = (1.2G_2 + 1.5Q_1)$</td></tr>
<tr><td rowspan="2">3</td><td>$w = 1.2G_1$ and $P = (1.2G_2 + W_u\downarrow)$</td></tr>
<tr><td>$w = 0.9G_1$ and $P = (0.9G_2 + W_u\uparrow)$</td></tr>
</table>

TABLE 2.7.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

2.7.3 Design for serviceability

2.7.3.1 *Design actions*

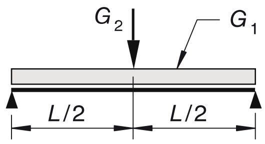
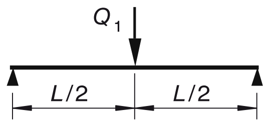
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.7.2.2.

2.7.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.7.3.2. Action cases given in Table 2.7.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.7.3.3.

TABLE 2.7.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models
1	
2	

2.7.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.7.3.3.

TABLE 2.7.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.7.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.7.3.4.

TABLE 2.7.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 12 mm max.
2	Span/300 or 12 mm max.

2.8 CEILING BATTENS

2.8.1 Description

Ceiling battens are closely spaced continuous span members that are attached to the underside of rafters, ceiling joists, floor joists or trusses and provide direct support for ceiling linings (see Figure 2.8.1).

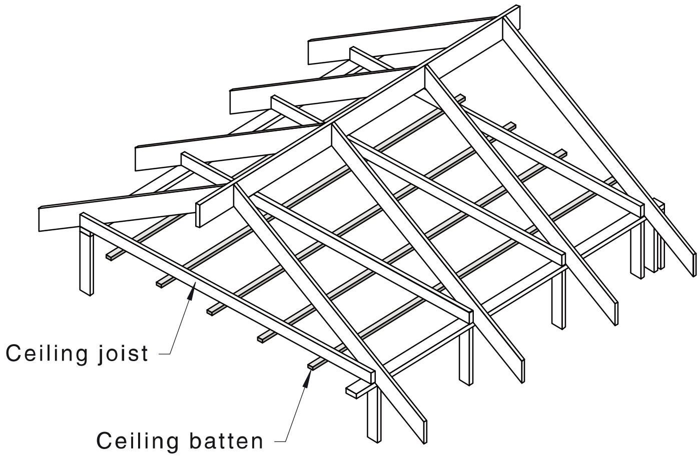


FIGURE 2.8.1 CEILING BATTENS

2.8.2 Design for safety

2.8.2.1 *General consideration*

Design for safety requires consideration of the strength limit state for bending.

2.8.2.2 *Design actions*

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

(a) *Permanent* Permanent action, G (in kN/m), includes self weight and the mass of the supported ceiling lining, with G determined from—

$$G = 0.12S + \text{self weight} \qquad \ldots 2.8.2.2(1)$$

where

S = spacing of the ceiling battens, in metres

self weight = ceiling batten self weight, in kN/m

(b) *Wind* Wind action for the strength limit state is considered applied as a uniformly distributed action, W_u (in kN/m), determined from—

$$W_u = q_u C_{pt} S \qquad \ldots 2.8.2.2(2)$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.8.2.2

S = spacing of ceiling battens, in metres

TABLE 2.8.2.2

NET PRESSURE COEFFICIENTS FOR CEILING BATTENS

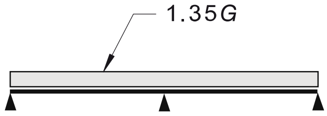
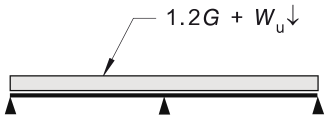
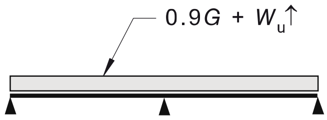
Wind classification	C_{pt}
N1 to N4	+0.2 or −0.3
C1 to C3	+0.7 or −0.65

2.8.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.8.2.3. Action combinations shown in Table 2.8.2.3 are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.8.2.4.

TABLE 2.8.2.3

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Action category	Structural models
1	
2	 

2.8.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.8.2.3, are given in Table 2.8.2.4.

TABLE 2.8.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.94

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for Action category 1, as defined in Table 2.8.2.3, k_4 = 1.0. For Action category 2, use values of k_4 appropriate for thickness as specified in AS 1720.1.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* For ceiling battens, k_9 = 1.0.

(d) *Member restraint* For ceiling battens breadth is greater than or equal to depth and, therefore, k_{12} = 1.0.

2.8.3 Design for serviceability

2.8.3.1 *Design actions*

Only the serviceability limit state for permanent action is considered in design. Permanent action for the serviceability limit state shall be as described in in Clause 2.8.2.2.

2.8.3.2 *Structural model for serviceability design*

The structural model for which deflection is calculated shall be as shown in Figure 2.8.3.2.

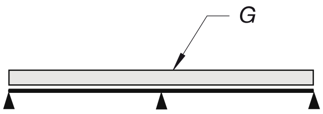


FIGURE 2.8.3.2 STRUCTURAL MODEL FOR SERVICEABILITY DESIGN

2.8.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.8.3.3.

TABLE 2.8.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)
Seasoned	2.0
Unseasoned	3.0

2.8.3.4 *Serviceability limit*

The serviceability limit state is defined by limiting the calculated deflection to span/600.

2.9 CEILING JOISTS

2.9.1 Description

Ceiling joists are closely spaced members primarily intended to support ceiling linings attached to their bottom edge. Ceiling joists also act to tie rafters together in coupled roof construction; however, for design, the axial load is ignored (see Figure 2.9.1).

Permanent and imposed actions are assumed continuously applied along the bottom edge of the joists. Imposed action due to construction or maintenance is considered applied as a concentrated action to the top edge of the joists.

Two installation methods are considered. The first method requires a continuous overbatten to be attached to the top edge of each joist at midspan and is applicable to both single and continuous span joist applications. This over-batten acts to laterally distribute the concentrated imposed action and provide intermediate lateral restraint to the joists. For the alternative method, no over-batten is installed and design does not allow for any load distribution or intermediate lateral restraint.

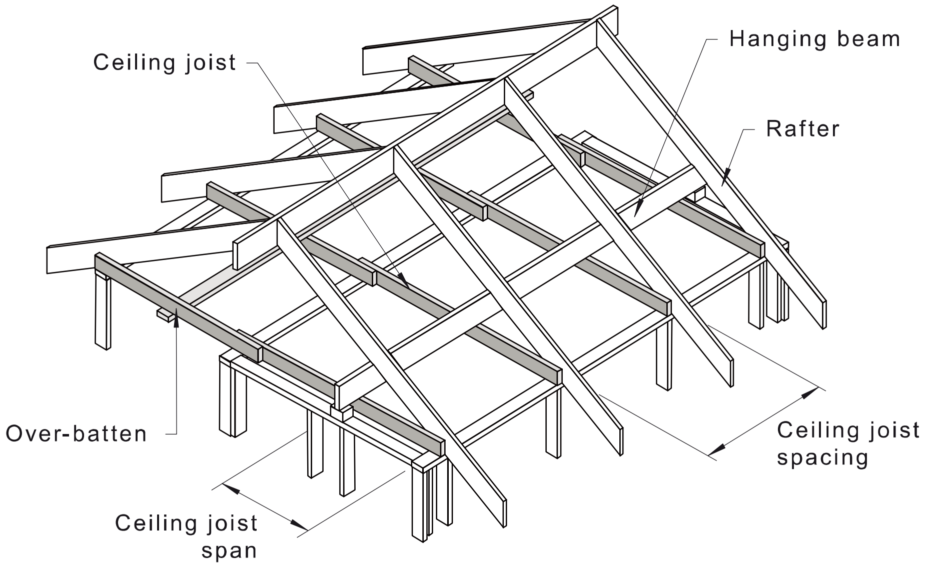


FIGURE 2.9.1 CEILING JOISTS

2.9.2 Design for safety

2.9.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending and shear.

2.9.2.2 *Design actions*

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

(a) *Permanent* Permanent action, G (in kN/m), for ceiling joists supporting ceiling lining (and battens, if appropriate), is determined from—

$$G = 0.12S + \text{self weight} \qquad \ldots 2.9.2.2(1)$$

where

S = spacing of the ceiling joists, in metres

self weight = ceiling joist self weight, in kN/m

(b) *Imposed* Imposed action for ceiling joists is considered as a concentrated action, Q (in kN), determined from—

$$Q = g_{42}\, 1.4 \qquad \ldots 2.9.2.2(2)$$

where g_{42} is a load distribution factor calculated as follows:

(i) For ceiling joists installed without an over-batten, $g_{42} = 1.0$.

(ii) For ceiling joists installed with a midspan over-batten:

(A) *Bending*—g_{42} is determined in accordance with AS 1720.1, for concentrated loads on grid systems, assuming the rigidity of the crossing member (over-batten), E_cI_c is equal to 1.73×10^9 Nmm^2 and the number of crossing members is one.

(B) *Shear*—$g_{42} = 1.0$.

(c) *Wind* The wind action, W_u (in kN/m), applicable for the strength limit state shall be calculated as follows:

$$W_u = q_u C_{pt} S \qquad \ldots 2.9.2.2(3)$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.9.2.2

S = spacing of ceiling joists, in metres

TABLE 2.9.2.2

NET PRESSURE COEFFICIENTS FOR CEILING JOISTS—STRENGTH

Wind classification	C_{pt}
N1 to N4	+0.5 or −0.35
C1 to C3	+0.85 or −1.0

2.9.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.9.2.4(A). Action combinations shown in Table 2.9.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.9.2.3.

2.9.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration, k_1. Values of k_1 appropriate for each action category, as defined in Table 2.9.2.4(A), are given in Table 2.9.2.4(B).

(b) *Moisture content of timber*:

  (i) *Unseasoned timber*—for action categories 2 and 3 given in Table 2.9.2.4(A), use values of k_4 appropriate for thickness as given in AS 1720.1. For Action category 1, k_4 = 1.0.

  (ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* Strength sharing is ignored; k_9 = 1.0.

(d) *Member restraint* For the determination of bending capacity the following assumptions relating to lateral restraint are used:

  (i) *At supports*—ceiling joists are assumed torsionally restrained at supports.

  (ii) *Between supports*:

    (A) For dead load and the dead load plus downward wind load cases the bottom edge is assumed loaded and continuously restrained.

    (B) For net upward wind load plus dead load, the bottom edge is assumed continuously restrained.

    (C) For the dead load plus live load case, with overbattens included, the top edge is assumed loaded and restrained at midspan (L_{ay} = 0.5 × span). For ceiling joists installed without over-battens included, no restraint is assumed (L_{ay} = span).

TABLE 2.9.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

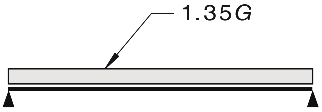
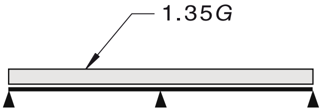
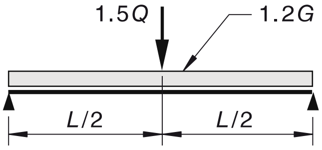
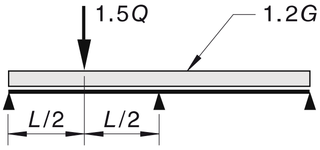
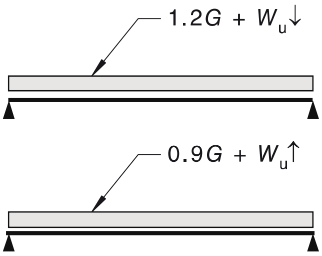
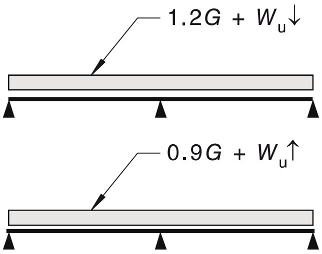
Action category	Structural models	
	Single span	**Continuous span**
1		
2		
3		

TABLE 2.9.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

2.9.3 Design for serviceability

2.9.3.1 *Design actions*

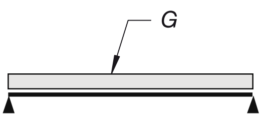
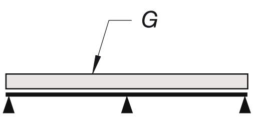
Only the serviceability limit state for permanent action is considered. Permanent actions for calculation of deflection shall be as described in Clause 2.9.2.2.

2.9.3.2 *Structural model for serviceability design*

The structural models for which deflection is calculated shall be as shown in Table 2.9.3.2.

TABLE 2.9.3.2

STRUCTURAL MODEL—SERVICEABILITY

Single span	Continuous span
	

2.9.3.3 *Calculation of deflection*

The requirements of AS 1720.1 shall be applied using the duration of load factor for creep deformation as given in Table 2.9.3.3.

TABLE 2.9.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)
Seasoned	2.0
Unseasoned	3.0

2.9.3.4 *Serviceability limits*

The serviceability limit state is defined by limiting the calculated deflection to span/400 or 12 mm maximum.

2.10 HANGING BEAMS

2.10.1 Description

Hanging beams are used to provide support for ceiling joists where supporting walls are widely spaced. They are installed in the roof cavity above the ceiling joists which are attached to the bottom edges of the hanging beams.

Design assumes that hanging beams are single span beams loaded and continuously restrained by ceiling joists along their bottom edge (see Figure 2.10.1).

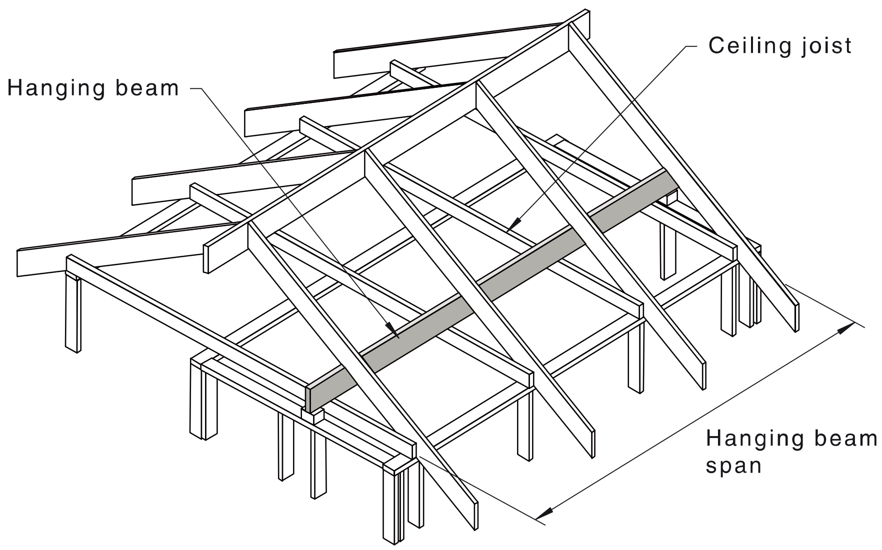


FIGURE 2.10.1 HANGING BEAM

2.10.2 Design for safety

2.10.2.1 *General consideration*

Design for safety requires consideration of the strength limit state for bending.

2.10.2.2 *Design actions*

The actions used to calculate the design action effects shall be as follows:

(a) *Permanent* The uniformly distributed permanent action, G (in kN/m), is determined from—

$$G = 0.12(CLW) + 0.02(CLW)^2 + \text{self weight} \qquad \ldots 2.10.2.2(1)$$

where

CLW = ceiling load width for the hanging beam, in metres

self weight = hanging beam self weight, in kN/m

(b) *Imposed* A concentrated imposed action, Q = 1.4 kN is considered.

(c) *Wind* The uniformly distributed wind action applicable for the strength limit state, W_u (in kN/m), is determined from—

$$W_u = q_u C_{pt} S \quad \text{. . . 2.10.2.2(2)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.10.2.2

CLW = ceiling load width for the hanging beam, in metres

TABLE 2.10.2.2

NET PRESSURE COEFFICIENTS FOR HANGING BEAMS—STRENGTH

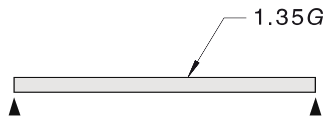
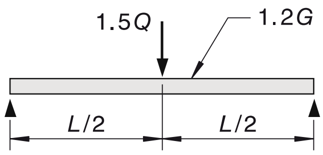
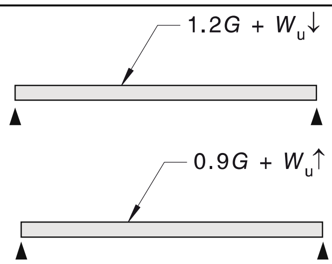
Wind classification	C_{pt}
N1 to N4	+0.2 or −0.3
C1 to C3	+0.7 or −0.65

2.10.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.10.2.3. Action combinations shown in Table 2.10.2.3 are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.10.2.4.

TABLE 2.10.2.3

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Action category	Structural models
1	
2	
3	

2.10.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.10.2.3, are given in Table 2.10.2.4.

TABLE 2.10.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3 given in Table 2.10.2.3, use values of k_4 appropriate for thickness as given in AS 1720.1. For Action category 1, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1.0 and n_{com} = number of combined sections.

(d) *Member restraint* The following assumptions relating to lateral restraint are used:

(i) *At supports*—hanging beams are assumed torsionally restrained at supports.

(ii) *Between supports*—hanging beams are considered loaded and continuously restrained along their bottom edge.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

2.10.3 Design for serviceability

2.10.3.1 *Design actions*

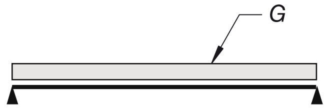
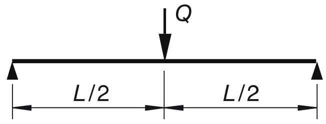
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.10.2.2.

2.10.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.10.3.2. Action cases given in Table 2.10.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.10.3.3.

TABLE 2.10.3.2

STRUCTURAL MODEL—SERVICEABILITY

Action category	Structural models
1	
2	

2.10.3.3 *Calculation of deflection*

The requirements of AS 1720.1 shall be applied using the duration of load factor for creep deformation as given in Table 2.10.3.3.

TABLE 2.10.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.10.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.10.3.4.

TABLE 2.10.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 12 mm max.
2	Span/270

2.11 COUNTER BEAMS

2.11.1 Description

A counter beam is a ceiling member, running parallel to ceiling joists and usually between them, which provides support for hanging beams. The hanging beams are assumed butted to the sides of the counter beam (see Figure 2.11.1).

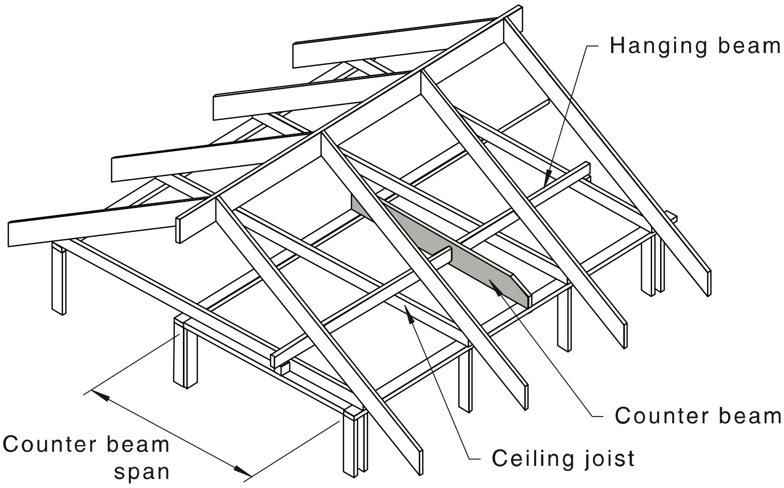


FIGURE 2.11.1 COUNTER BEAM

2.11.2 Design for safety

2.11.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending.

2.11.2.2 *Design actions*

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

(a) *Permanent* Permanent actions include the distributed action due to self weight, G_1 (in kN/m), and a concentrated action imposed by the hanging beams, G_2 (in kN), determined from—

$$G_2 = 0.2(CLW)(L/2) \qquad \text{. . . 2.11.2.2(1)}$$

where

CLW = ceiling load width for the counter beam, in metres

L = span of the counter beam, in metres

(b) *Imposed* A concentrated imposed action, Q = 1.4 kN, is considered.

(c) *Wind* Wind action is considered as a concentrated action, W_u (in kN), determined from—

$$W_u = q_u C_{pt}(CLW)(L/2) \qquad \text{. . . 2.11.2.2(2)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.11.2.2

CLW = ceiling load width for the counter beam, in metres

L = span of the counter beam, in metres

TABLE 2.11.2.2

NET PRESSURE COEFFICIENTS FOR COUNTER BEAMS

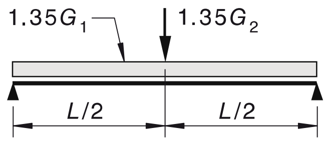
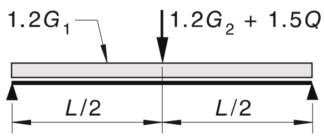
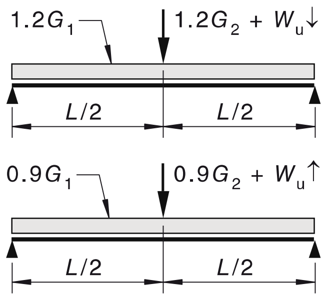
Wind classification	C_{pt}
N1 to N4	+0.2 or −0.3
C1 to C3	+0.7 or −0.65

2.11.2.3 *Structural models and action categories for strength design*

The structural models used to calculate the member design action effects shall be as given in Table 2.11.2.3. Action combinations shown in Table 2.11.2.3 are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.11.2.4.

TABLE 2.11.2.3

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

<table>
<tr><th>Action category</th><th>Structural models</th></tr>
<tr><td>1</td><td>
</td></tr>
<tr><td>2</td><td>
</td></tr>
<tr><td>3</td><td>
</td></tr>
</table>

2.11.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 2.11.2.3 are given in Table 2.11.2.4.

TABLE 2.11.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.97
3	1.00

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3, given in Table 2.11.2.3, use values of k_4 appropriate for thickness as given in AS 1720.1. For Action category 1, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming n_{mem} = 1.0 and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity the following assumptions relating to lateral restraint are used:

(i) *At supports*—counter beams are assumed torsionally restrained at their supports.

(ii) *Between supports*—counter beams are assumed torsionally restrained at midspan by the supported hanging beams.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

2.11.3 Design for serviceability

2.11.3.1 *Design actions*

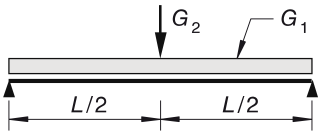
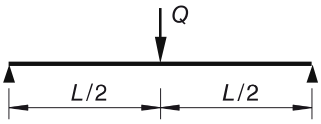
The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 2.11.2.2.

2.11.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.11.3.2. Action cases given in Table 2.11.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.11.3.3.

TABLE 2.11.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models
1	
2	

2.11.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load for creep deformation as given in Table 2.11.3.3.

TABLE 2.11.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Initial moisture content	Load duration factor (j_2)	
	Action category 1	Action category 2
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.11.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.11.3.4.

TABLE 2.11.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 15 mm max.
2	Span/270 or 15 mm max.

2.12 VERANDAH BEAMS

2.12.1 Description

Verandah beams support verandah roof loads imposed by rafters or roof trusses.

Verandah beams for single and continuous span applications are considered. Design assumes the roof load is applied to the top edge of verandah beams as a series of concentrated actions at 600 mm or 1200 mm centres corresponding to rafter (or truss) spacings (see Figure 2.12.1).

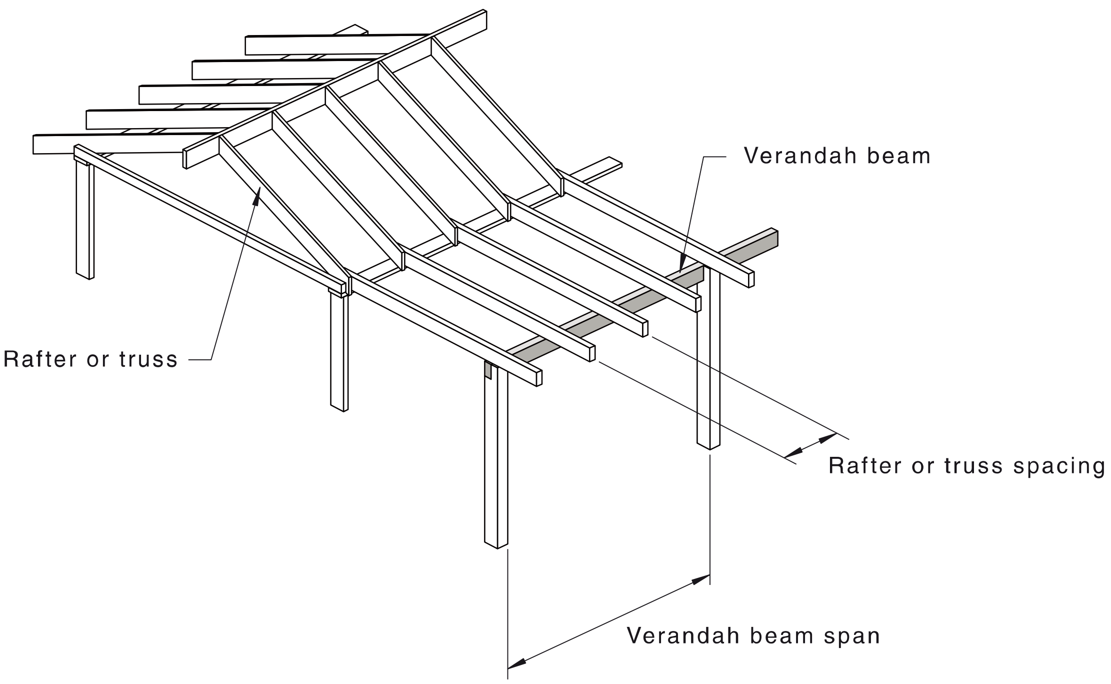


FIGURE 2.12.1 VERANDAH BEAM

2.12.2 Design for safety

2.12.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in bending and shear.

2.12.2.2 *Design actions*

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

(a) *Permanent* Permanent actions include the self weight of the verandah beam, G_1 (in kN/m), and concentrated actions imposed by the rafters, G_2 (in kN), determined from—

$$G_2 = 0.01(RM)(RLW)S_R + 0.02(RLW)^2 S_R \quad \text{. . . 2.12.2.2(1)}$$

where

RLW = roof load width for the verandah beam, in metres

RM = standardized roof mass, i.e. 10, 20, 40, 60 or 90 kg/m^2

S_R = rafter spacing, i.e. 0.6 m or 1.2 m

(b) *Imposed* Imposed actions include a concentrated action Q_1 = 1.1 kN and the actions imposed via rafters, considered as concentrated actions Q_2 (in kN), determined from—

$$Q_2 = 0.25S_R(RLW) \quad \text{. . . 2.12.2.2(2)}$$

where

S_R = rafter spacing, i.e. 0.6 m or 1.2 m

RLW = roof load width for the verandah beams, in metres

(c) *Wind* Wind actions are considered as concentrated actions, W_u (in kN), imposed via the rafters and calculated as follows:

$$W_u = q_u C_{pt} S_R(RLW) \quad \text{. . . 2.12.2.2(3)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 2.12.2.2

S_R = rafter spacing, i.e. 0.6 or 1.2 m

RLW = roof load width for the verandah beam, in metres

NOTE: Horizontal wind pressure on verandah beams is ignored.

TABLE 2.12.2.2

NET PRESSURE COEFFICIENTS FOR VERANDAH BEAMS

Wind classification	C_{pt}
N1 to N4	+0.95 or −1.44
C1 to C3	

2.12.2.3 *Structural models and action categories used for strength design*

The structural models used to determine the member design action effects shall be as given in Table 2.12.2.4(A). Action combinations shown in Table 2.12.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 2.12.2.4.

2.12.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category defined in Table 2.12.2.4(A) are given in Table 2.12.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2 and 3, use values of k_4 appropriate for member thickness as given in AS 1720.1. For Action category 1, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all load categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming $n_{mem} = 1$ and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity, the following assumptions related to lateral restraint are used:

(i) *At supports*—verandah beams are considered torsionally restrained at their supports.

(ii) *Between supports*—

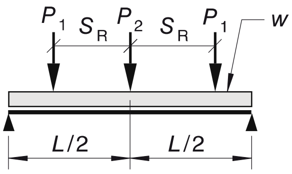
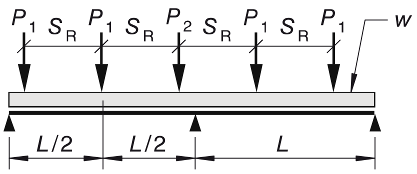
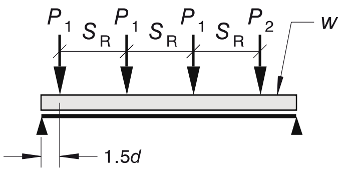
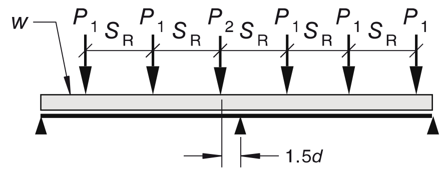
(A) the top edges of verandah beams are assumed restrained by rafters at 600 mm or 1200 mm centres as appropriate; and

(B) continuous span verandah beams are assumed restrained against buckling at the points of contraflexure taken as one quarter of the span from an intermediate support.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination and not the overall breadth.

TABLE 2.12.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Design action effect	Structural models	
	Single span	Continuous span
In bending		
In shear		

Action category	Design actions
1	$w = 1.35G_1$ and $P_1 = P_2 = 1.35G_2$
2	$w = 1.2G_1$ and $P_1 = (1.2G_2 + 1.5Q_2)$ and $P_2 = P_1 + Q_1$
3	$w = 1.2G_1$ and $P_1 = (1.2G_2 + W_u\downarrow)$ $w = 0.9G_1$ and $P_1 = (0.9G_2 + W_u\uparrow)$

NOTES:

1 S_R is rafter spacing, either 0.6 m or 1.2 m.

2 The number of concentrated actions considered will vary according to span, rafter spacing and locations of concentrated actions.

3 Actions within 1.5d of supports are ignored in the determination of the design action effect in shear.

TABLE 2.12.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.94
3	1.00

2.12.3 Design for serviceability

2.12.3.1 *Design actions*

The actions used for the serviceability limit states shall be as follows:

(a) *Permanent and imposed* Permanent actions and imposed actions are determined as described in Clause 2.12.2.2.

(b) *Wind* Wind action is considered applied by the rafters as a series of concentrated actions, W_s (in kN), determined from—

$$W_s = q_s C_{pt} S_R (RLW) \qquad \ldots 2.12.3.1$$

where

q_s = free stream dynamic gust pressure, in kPa, for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients as given in Table 2.12.2.2

S_R = rafter spacing, i.e. 0.6 or 1.2 m

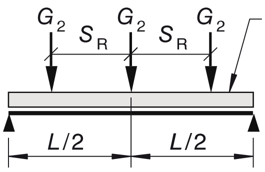
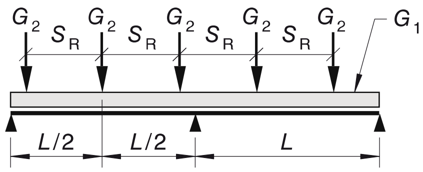
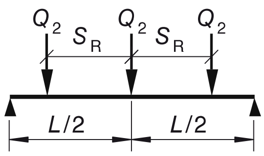
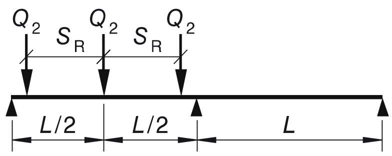
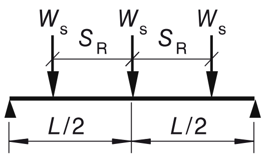
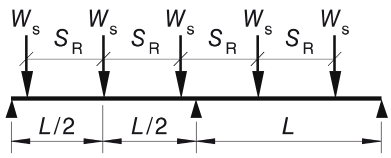
RLW = roof load width for the verandah beam, in metres

2.12.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 2.12.3.2. Action cases given in Table 2.12.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 2.12.3.3.

TABLE 2.12.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Structural models	
	Single span	Continuous span
1		
2		
3		

LEGEND:

S_R = rafter spacing, 0.6 m or 1.2 m

2.12.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 2.12.3.3.

TABLE 2.12.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Initial moisture content	Load duration factor (j_2)	
	Action category 1	Action categories 2 and 3
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

2.12.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 2.12.3.4.

TABLE 2.12.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/400 or 10 mm max.
2	Span/250 or 12 mm max.
3	Span/200

SECTION 3 DESIGN OF WALL MEMBERS

3.1 POSTS

3.1.1 Description

Posts are loadbearing columns designed to support axial loads arising from the vertical support given to roofs and floors.

Posts may be incorporated within or installed separate from walls. Posts are not used to replace common studs in external walls and are, therefore, not designed to support lateral loads.

Posts are assumed laterally supported only at points of attachment to floor and roof members (see Figure 3.1.1).

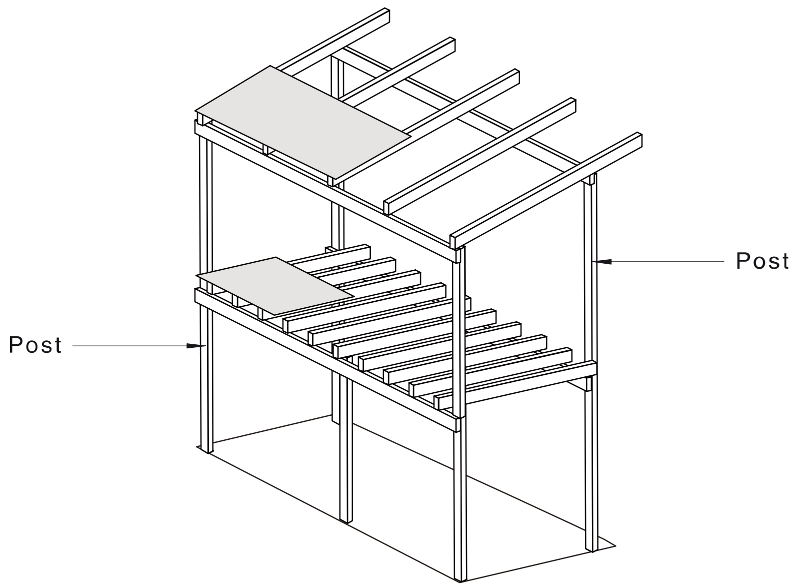


FIGURE 3.1.1 POSTS SUPPORTING ROOF AND/OR FLOOR LOADS

3.1.2 Design for safety

3.1.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in tension and compression.

3.1.2.2 *Design actions*

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

(a) *Permanent* Permanent action, G, is considered as the sum of the concentrated permanent actions from supported roof and floor areas determined from Table 3.1.2.2(A).

TABLE 3.1.2.2(A)

PERMANENT ACTIONS

Source of action	Permanent action, (G) kN
Floor	$0.4A_F$
Roof:	
Tile	$0.9A_R$
Sheet	$0.4A_R$

LEGEND:
A_F = area of floor supported, in square metres
A_R = area of roof supported, in square metres

(b) *Imposed* Concentrated imposed actions, Q_1, Q_2 and Q_3 (in kN), arising from support given to floor and roof areas are determined from:

(i) *For posts supporting floor area* (A_F):

(A) *Permanent live load*—$Q_1 = 0.5A_F$.

(B) *Transient live load*—$Q_2 = 1.5A_F$.

(ii) *For posts supporting roof area* (A_R)—$Q_3 = 0.25A_R$.

NOTES:

1 Imposed actions Q_2 and Q_3 are not considered to act simultaneously.

2 Units for areas A_F and A_R are square metres.

(c) *Wind* The concentrated wind action, W_u (in kN), applicable for the strength limit state arising from support given to roof areas is determined from—

$$W_u = q_u C_{pt} A_R \quad \ldots 3.1.2.2$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients for roof areas supported by posts, as given in Table 3.1.2.2(B)

A_R = roof area supported, in square metres

TABLE 3.1.2.2(B)

NET PRESSURE COEFFICIENTS FOR ROOF AREAS SUPPORTED BY POSTS—STRENGTH

Wind classification	C_{pt}
N1 to N4	+0.95 or −1.44
C1 to C3	

3.1.2.3 *Structural models and action categories for strength design*

Posts are designed as simple columns supporting an axial concentrically applied load. Action combinations used to determine the design action effects in compression (N_c^*) and tension (N_t^*) shall be as given in Table 3.1.2.3. Design action effects given in Table 3.1.2.3 are divided into action categories that are used for the determination of the corresponding member design capacity as specified in Clause 2.8.2.4.

TABLE 3.1.2.3

DESIGN ACTION EFFECTS AND ACTION CATEGORIES—STRENGTH

Action categories	Design action effects
1	$N_c^* = 1.35G$ $N_c^* = 1.2G + 1.5Q_1$
2	$N_c^* = 1.2G + 1.5Q_2$
3	$N_c^* = 1.2G + 1.5Q_3$
4	$N_c^* = 1.2G + W_u\downarrow + Q_1$ $N_t^* = 0.9G + W_u\uparrow$

3.1.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in compression and tension. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 3.1.2.3, shall be as given in Table 3.1.2.4.

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2, 3 and 4, use values of k_4 appropriate for thickness as given in AS 1720.1. For Action category 1, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all action categories.

(c) *Strength sharing* Strength sharing is not considered to apply for posts, i.e. $k_9 = 1.0$.

(d) *Member restraint* For the determination of the compressive capacity of posts the effective length for buckling about either axis is taken as 0.85 times the post height. Post height is the distance between supports and points of attachment to supported floor and roof members, which are assumed to provide lateral restraint for both axes of buckling.

NOTE: Nail-laminated posts are not considered in this Standard.

TABLE 3.1.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.8
3	0.94
4	1.00

3.1.3 Design for serviceability

Axial deformation of posts under the applicable loadings is small and for this reason serviceability design for posts shall be disregarded.

3.2 LOADBEARING WALL STUDS

3.2.1 Description

Loadbearing wall studs are the vertical components of a loadbearing wall required to transfer tension or compression loads from supported floors or roofs and to transfer horizontal wall loads, in bending, to the top and bottom wall supports.

Common studs support the vertical loads applied to the top wall plate by rafters, ceiling joists or floor joists and the horizontal loads due to wind.

Jamb studs are studs at sides of openings that support loads from the lintel over the opening and horizontal wind loads related to the width of the opening.

Studs supporting concentrated loads are studs installed in the wall, in addition to common studs (or jamb studs), that are required to carry concentrations of vertical load arising from support for principal roof or floor supporting members.

Special consideration is given for studs notched for the installation of bracing. For notched studs, notches are assumed in either face of the wall penetrating to a maximum depth of 20 mm into the depth of the studs (see Figure 3.2.1).

NOTE: For maximum allowable stud heights refer AS 1684 series.

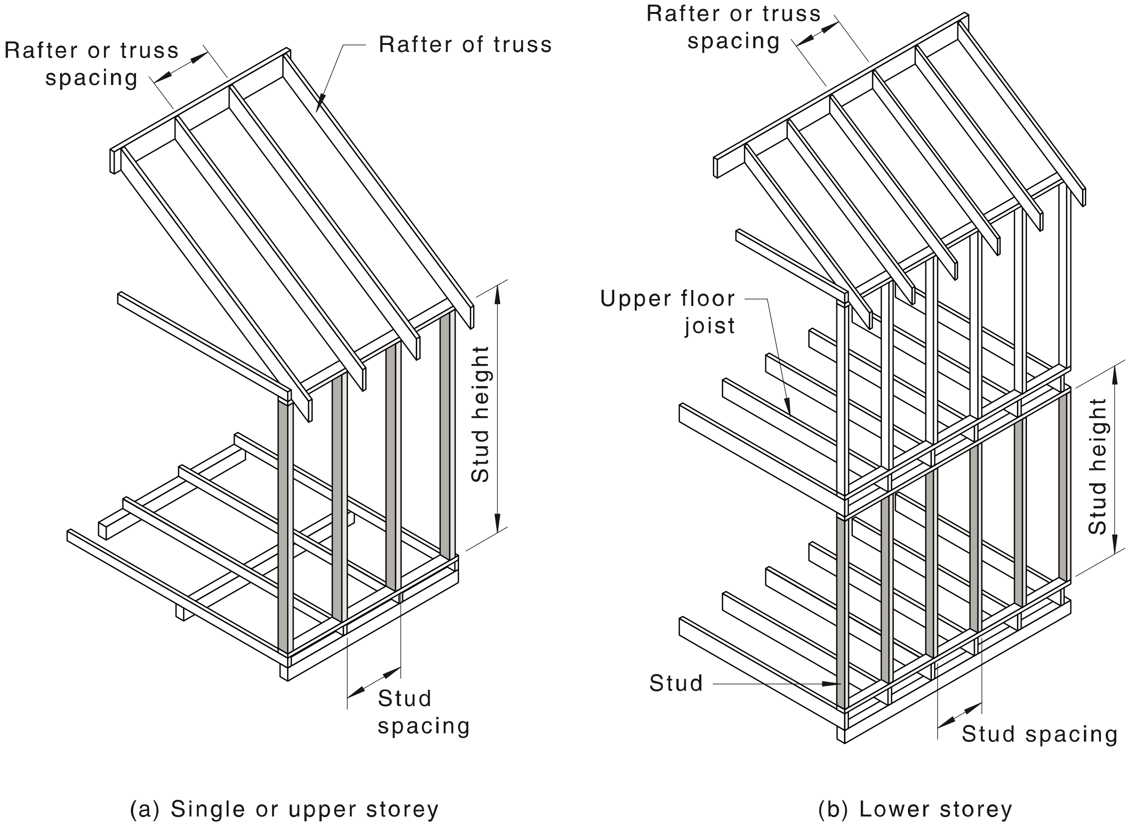


FIGURE 3.2.1 LOADBEARING WALL STUDS

3.2.2 Design for safety

3.2.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in compression, tension, bending, combined bending and compression and combined bending and tension. For notched studs the strength limit state for combined bending and shear at the assumed notch location shall also be considered.

3.2.2.2 *Design actions*

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

(a) *Permanent* The concentrated permanent actions (G) considered axially applied to common studs, jamb studs and studs supporting concentrated loads, in upper or single storey walls or lower storey of two-storey walls, are determined from Table 3.2.2.2(A).

TABLE 3.2.2.2(A)

AXIAL PERMANENT ACTIONS SUPPORTED BY STUDS

Application	**Common studs**	**Jamb studs**	**Studs supporting concentrated loads**
	Axial dead loads, (G) kN		
Upper storey or single storey—			
(a) sheet roof	$0.4(RLW)S_1$	$0.4(RLW)(W_o/2 + 0.3)$	$0.4A_R$
(b) tile roof	$0.9(RLW)S_1$	$0.9(RLW)(W_o/2 + 0.3)$	$0.9A_R$
Lower storey of two-storey:			
(a) Roof, upper wall and floor—			
(i) sheet roof	$[0.4(RLW) + 0.4 + 0.4(FLW) + 0.025(FLW)^2]S_2$	$[0.4(RLW) + 0.4 + 0.4(FLW) + 0.025(FLW)^2](W_o/2 + 0.3)$	—
(ii) tile roof	$[0.9(RLW) + 0.4 + 0.4(FLW) + 0.025(FLW)^2]S_2$	$[0.9(RLW) + 0.4 + 0.4 (FLW) + 0.025(FLW)^2](W_o/2 + 0.3)$	—
(b) Floor only	$[0.4(FLW) + 0.025(FLW)^2]S^2$	$[0.4(FLW) + 0.025(FLW)^2](W_o/2 + 0.3)$	$0.4A_F$

LEGEND:

S_1 = the greater of the rafter (truss) or stud spacing in the wall, in metres
S_2 = the greater of the floor joist or stud spacing in the lower wall, in metres
W_o = width of opening in the wall, in metres
A_R = area of roof supported by the stud, in square metres
A_F = area of floor supported by the stud, in square metres
RLW = roof load width supported by the wall, in metres
FLW = floor load width supported by the wall, in metres

(b) *Imposed* Concentrated imposed actions, Q_1, Q_2 and Q_3 (in kN), considered axially applied to common studs, jamb studs and studs supporting concentrated loads, in upper or single storey walls or the lower storey of two-storey construction, are determined from Table 3.2.2.2(B).

TABLE 3.2.2.2(B)

AXIAL IMPOSED ACTIONS SUPPORTED BY STUDS

Application	Common studs	Jamb studs	Studs supporting concentrated loads
	Axial imposed actions, (Q) kN		
Upper storey or single storey	$Q_1 = 0$	$Q_1 = 0$	$Q_1 = 0$
	$Q_2 = 0.25(RLW)S_1$	$Q_2 = 0.25(RLW)(W_o/2 + 0.3)$	$Q_2 = 0.25A_R$
	$Q_3 = 0$	$Q_3 = 0$	$Q_3 = 0$
Lower storey of two-storey	$Q_1 = 0.5(FLW)S_2$	$Q_1 = 0.5(FLW)(W_o/2 + 0.3)$	$Q_1 = 0.5A_F$
	$Q_2 = 0$	$Q_2 = 0$	$Q_2 = 0$
	$Q_3 = 1.5(FLW)S_2$	$Q_3 = 1.5(FLW)(W_o/2 + 0.3)$	$Q_3 = 1.5A_F$

LEGEND:
RLW = roof load width supported by the wall, in metres
FLW = floor load width supported by the wall, in metres
S_1 = greater of the rafter/truss or stud spacing, in metres
S_2 = greater of the floor joist or stud spacing, in metres
W_o = width of opening in the wall, in metres
A_R = area of roof supported by the stud, in square metres
A_F = area of floor supported by the stud, in square metres
Q_1 = long-term component of floor live load
Q_2 = roof imposed action
Q_3 = short term floor imposed action

(c) *Wind* Wind actions for studs are considered applied as axial concentrated actions (W_{ua}) and uniformly distributed lateral actions (W_{uw}). Values of W_{ua} and W_{uw}, for common studs, jamb studs and studs supporting concentrated loads, are determined from the expressions given in Table 3.2.2.2(C).

TABLE 3.2.2.2(C)

AXIAL AND LATERAL WIND ACTIONS FOR STUDS

Type of load		Common studs	Jamb studs	Studs supporting concentrated loads
Upper storey or single storey	W_{ua} (kN)	$q_u C_{ptr}(RLW)S_1$	$q_u C_{ptr}(RLW)(W_o/2 + 0.3)$	$q_u C_{ptr} A_R$
	W_{uw} (kN/m)	$q_u C_{ptw} S_s$	$q_u C_{ptw}(W_o/3 + 0.3)$	Not applicable
Lower storey of two-storey	W_{ua} (kN)	$q_u C_{ptr}(RLW)S_s$	$q_u C_{ptr}(RLW)(W_o/2 + 0.3)$	Not considered
	W_{uw} (kN/m)	$q_u C_{ptw} S_s$	$q_u C_{ptw}(W_o/3 + 0.3)$	Not applicable

LEGEND:
q_u = free stream dynamic gust pressure for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification
C_{ptr} = net pressure coefficients for roof areas supported by the wall as given in Table 3.2.2.2(D)
C_{ptw} = net pressure coefficients for walls, as given in Table 3.2.2.2(D)
S_1 = for wind down—the rafter spacing, in metres
= for wind up—tie-down spacing, in metres
S_s = stud spacing, in metres
W_o = width of opening between jamb studs, in metres
A_R = roof area supported, in square metres

TABLE 3.2.2.2(D)

PRESSURE COEFFICIENTS FOR ROOF AND WALLS—STRENGTH

Wind classification	C_{ptr}	C_{ptw}
N1 to N4	+0.63 or −0.99	0.9
C1 to C3	+0.95 or −1.44	1.20

NOTE: Positive pressure coefficient indicates an inwards pressure.

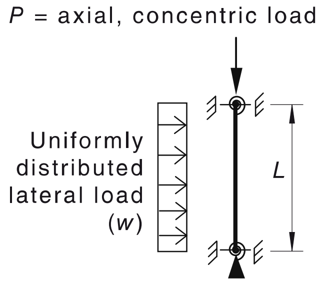
3.2.2.3 *Structural models and action categories for strength design*

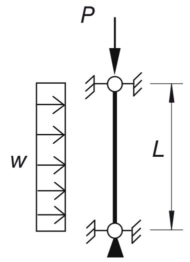
The structural model used to calculate the member design action effects shall be as shown in Table 3.2.5. For the determination of design action effects, axial actions are assumed concentrically applied and maximum bending moments are as given in Table 3.2.2.3.

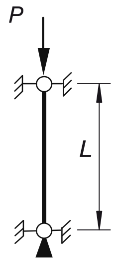
Action combinations given in Table 3.2.2.3 are divided into action categories that are used for the determination of the corresponding member design capacity as specified in Clause 3.2.2.4.

TABLE 3.2.2.3

STRUCTURAL MODEL AND ACTION CATEGORIES—STRENGTH

<table>
<tr><th colspan="3">Structural model</th></tr>
<tr><th>Common stud</th><th>Jamb stud</th><th>Studs supporting concentrated loads</th></tr>
<tr>
<td>

NOTES:
1 For notched studs, the notch is assumed located at mid-height.
2 $M = cwL^2$
where
(a) for $L \leq 2.4$ m, $c = 0.07$;
(b) for $L \geq 4.2$ m, $c = 0.125$; and
(c) for $2.4 < L < 4.2$, $c = (0.0306L - 0.003)$.</td>
<td>

$M = 0.125wL^2$</td>
<td>

$M = 0$</td>
</tr>
</table>

Action category	Design actions
1	$P = 1.35G$ and $w = 0$ $P = 1.2G + 1.5Q_1$ and $w = 0$
2	$P = 1.2G + 1.5Q_3$ and $w = 0$
3	$P = 1.2G + 1.5Q_2$ and $w = 0$
4	$P = 1.2G + W_{ua}\downarrow + Q_1$ and $w = W_{uw}$ $P = 0.9G + W_{ua}\uparrow$ and $w = W_{uw}$ $P = 1.2G + Q_1$ and $w = W_{uw}$

3.2.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in compression, tension and bending and in combined bending and compression and bending and tension. The following assumptions and modifications factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 3.2.2.3, are given in Table 3.2.2.4.

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for Action category 4 in Table 3.2.2.3, use values of k_4 appropriate for thickness as specified in AS 1720.1. For Action categories 1, 2 and 3, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all action categories.

(c) *Strength sharing*:

(i) *For common studs*—the strength sharing factor (k_9) is applied for bending only, assuming $n_{mem} = 5$ and n_{com} = number of sections combined in a stud.

(ii) *For jamb studs*—the strength sharing factor (k_9) is applied for bending only, with $n_{mem} = 1.0$ and n_{com} = number of sections combined in the jamb stud.

(iii) *For studs supporting concentrated loads*—the strength sharing factor (k_9) is not applied.

(d) *Member restraint* For the determination of bending and compressive capacity, the following assumptions relating to lateral restraint are used:

(i) *For bending*:

(A) *At supports*—studs are assumed torsionally restrained.

(B) *Between supports*—studs are assumed torsionally and laterally restrained by noggings; $L_{ay} = 1350$ mm. In addition, the tension edge is assumed laterally restrained at intervals not greater than 600 mm.

(ii) *For compression*:

(A) For buckling about the major axis the effective length of studs is taken as $g_{13}L$, where L is the height of the stud and g_{13} is determined as follows:

(1) *For common studs*:

$L \leq 2.4$ m, $g_{13} = 0.75$.

$L \geq 4.2$ m, $g_{13} = 1.0$.

2.4 m $\leq L \leq 4.2$ m, $g_{13} = (0.139L + 0.417)$.

(2) *For jamb studs*—$g_{13} = 0.9$.

(B) For buckling about the minor axis, L_{ay} is taken as 600 mm.

NOTE: For studs formed by nail laminating one or more sections together, the breadth of section used to determine the slenderness coefficients (S_1 or S_4) is taken as the breadth of an individual lamination.

(e) *Notched studs* For studs up to 125 mm deep, notched to a maximum depth of 20 mm for the installation of diagonal bracing only, the bending capacity is determined as 0.6 times the bending capacity of an un-notched stud. The tensile and compressive capacities are determined using the net cross-section at the notch as the effective cross-sectional area.

NOTE: The method used for studs notched for diagonal bracing is based upon CSIRO BCE Report, *Notched composite beams*, Dec. 97/169M, September 1997.

TABLE 3.2.2.4

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.80
3	0.94
4	1.00

3.2.3 Design for serviceability

3.2.3.1 *General consideration*

Only the serviceability limit state for lateral deformation shall be considered.

NOTE: The application of a serviceability limit state for serviceability wind pressure on the walls is assumed to ensure adequate lateral rigidity for incidental horizontal live loads.

3.2.3.2 *Design actions*

The distributed wind actions (W_{sw}) used for the serviceability limit state, for common studs and for jamb studs, are determined from Table 3.2.3.2(A).

TABLE 3.2.3.2(A)

HORIZONTAL WIND ACTION— SERVICEABILITY

Type of stud	(W_{sw}) kN/m
Common stud	$q_s C_{ptw} S$
Jamb stud	$q_s C_{ptw}(W_o/3 + 0.3)$

LEGEND:

q_s = free stream dynamic gust pressure for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{ptw} = net pressure coefficient for walls given in Table 3.2.3.2(B)

S = spacing of studs, in metres

W_o = width of opening in wall, in metres

TABLE 3.2.3.2(B)

NET PRESSURE COEFFICIENTS FOR WALLS—SERVICEABILITY

Wind classification	Net pressure coefficient for walls (C_{ptw})
N1 to N4	0.9
C1 to C3	

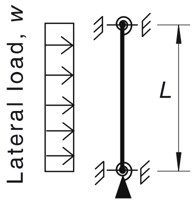
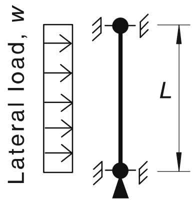
3.2.3.3 *Structural model for serviceability design*

The structural model used to determine deflection under a uniformly distributed lateral action shall be as given in Table 3.2.3.3.

For studs notched for the installation of bracing the presence of notches shall be ignored in the determination of deflection.

TABLE 3.2.3.3

STRUCTURAL MODEL FOR DETERMINATION OF DEFLECTION

Structural model	
For common studs	**For jamb studs**
 	
NOTE: Max. deflection is calculated as follows: Deflection = $cwL^4/(EI)$ where (a) for $L \leq 2.4$ m, $c = 0.0042$; (b) for $L \geq 4.2$ m, $c = 0.013$; and (c) for $2.4 < L < 4.2$, $c = (0.0049L - 0.0076)$ $w = W_{sw}$	NOTE: Max. deflection is calculated as follows: Deflection = $cwL^4/(EI)$ where $c = 0.013$ $w = W_{sw}$

3.2.3.4 *Calculation of deflection*

Deflection of studs under the serviceability wind action specified in Clause 3.2.3.2 shall be calculated assuming the structural model specified in Clause 3.2.3.3. No modification is required for duration of load for this case.

3.2.3.5 *Serviceability limit*

The deflection of common studs and jamb studs under the serviceability wind action, given in Clause 3.2.3.2 and calculated in accordance with Clause 3.2.3.4, is limited to (stud height)/150, but not greater than 20 mm.

NOTE: This limit may not preclude damage to brittle surface finishes.

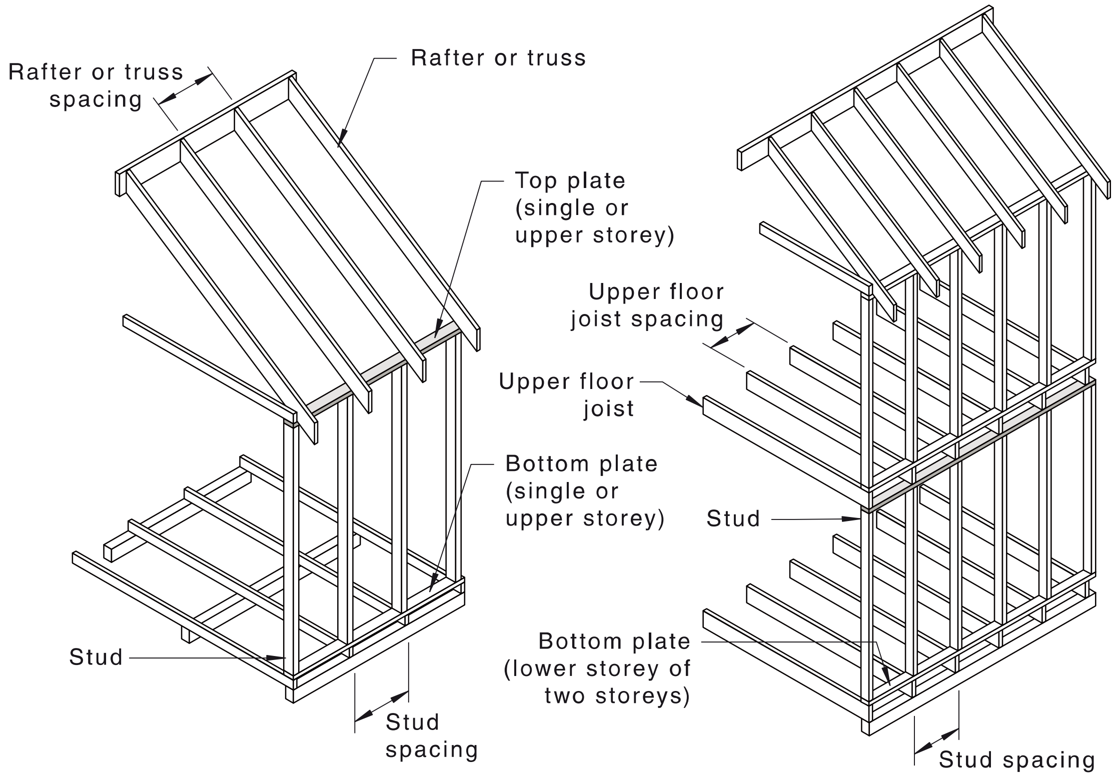
3.3 WALL PLATES FOR LOADBEARING WALLS

3.3.1 Description

Wall plates are the usually horizontal components in a wall frame to which the studs are attached at the top of the wall frame (top plate) and at the bottom of the wall frame (bottom plate).

Where load or support points for a wall frame are not closely aligned with studs, or tiedown supports, then the wall plates in a loadbearing frame are designed to transfer load or support from a rafter/truss or floor joist, as appropriate, to adjacent studs, or tiedown points for top plates in upper storey or single storey walls. Where concentrated loads from girder trusses or other principal roof or floor supporting members occur then special provision for support of such loads (e.g. studs supporting concentrated loads, bridging or blocking) is assumed.

Wall plates are not designed to transfer horizontal loads laterally to braced cross walls; ceiling and floor diaphragms are assumed to perform this function (see Figure 3.3.1).



(a) Single or upper storey

(b) Lower storey

FIGURE 3.3.1 WALL PLATES FOR LOADBEARING WALLS

3.3.2 Design for safety

3.3.2.1 *General consideration*

Wall plate design for safety requires consideration of the strength limit states for minor axis bending and shear.

3.3.2.2 *Design actions*

The vertical permanent actions, imposed actions and wind actions used for the determination of the design action effects for top and bottom plates, in upper storey or single storey walls and for lower storey walls of two-storey construction, shall be as follows:

(a) *Permanent*—the concentrated permanent action, G, is determined from Table 3.3.2.2(A).

TABLE 3.3.2.2(A)

PERMANENT ACTIONS FOR WALL PLATES

Application		Permanent action, (*G*) kN
Upper storey or single storey	Top plates	$G = 0.01RM(RLW)S_R$
	Bottom plates	$G = 0.01RM(RLW)S_S + 0.4S_S$
Lower storey of two storeys	Top plates	$G = 0.01RM(RLW)S_J + 0.4S_J + 0.4(FLW)S_J + 0.025(FLW)^2S_J$
	Bottom plates	$G = 0.01RM(RLW)S_S + 0.8S_S + 0.4(FLW)S_S + 0.025(FLW)^2S_S$

LEGEND:

RM = roof mass allowance—
(a) 40 kg/m^2 for sheet roofs; and
(b) 90 kg/m^2 for tile roofs.

RLW = roof load width supported by wall, in metres

FLW = floor load width supported by wall, in metres

S_R = spacing of rafters/trusses, in metres

S_S = spacing of studs in wall, in metres

S_J = spacing of floor joists, in metres

(b) *Imposed*—the concentrated imposed actions, Q_1, Q_2 and Q_3, are determined from Table 3.3.2.2(B).

TABLE 3.3.2.2(B)

IMPOSED ACTIONS FOR WALL PLATES

Application		Imposed actions, (*Q*) kN		
Upper storey or single storey	Top plates	$Q_1 = 0$	$Q_2 = 0.25(RLW)S_R$	$Q_3 = 0$
	Bottom plates	$Q_1 = 0$	$Q_2 = 0.25(RLW)S_S$	$Q_3 = 0$
Lower storey of two storeys	Top plates	$Q_1 = 0.5(FLW)S_J$	$Q_2 = 0$	$Q_3 = 1.5(FLW)S_J$
	Bottom plates	$Q_1 = 0.5(FLW)S_S$	$Q_2 = 0$	$Q_3 = 1.5(FLW)S_S$

LEGEND:

RLW = roof load width supported by the wall, in metres

FLW = floor load width supported by the wall, in metres

S_R = spacing of rafters/trusses, in metres

S_S = spacing of studs in wall, in metres

S_J = spacing of joists, in metres

Q_1 = long-term component of floor live load

Q_2 = roof imposed action

Q_3 = short term floor imposed action

(c) *Wind*—the concentrated wind load, W_u, considered acting vertically on wall plates is determined from Table 3.3.2.2(C).

TABLE 3.3.2.2(C)

VERTICAL WIND ACTIONS ON WALL PLATES

Application		Wind action, (W_u) kN
Upper storey or single storey	Top plates	$q_u C_{ptr}(RLW)S_1$
	Bottom plates	$q_u C_{ptr}(RLW)S_S$
Lower storey of two storeys	Top plates	$q_u C_{ptr}(RLW)S_J$
	Bottom plates	$q_u C_{ptr}(RLW)S_S$

LEGEND:

q_u = free stream dynamic gust pressure for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{ptr} = net pressure coefficients for roof areas given in Table 3.3.2.2(D)

RLW = roof load width supported by the wall, in metres

S_1 = for wind down—rafter spacing, in metres

= for wind up—tie-down spacing along top plate, in metres

S_S = for wind down—spacing of studs in wall, in metres

for wind up—tie-down spacing along top plate, in metres

S_J = spacing of floor joists supported by wall, in metres

TABLE 3.3.2.2(D)

PRESSURE COEFFICIENTS FOR ROOF

Wind classification	C_{ptr}
N1 to N4	+0.63 or −1.0
C1 to C3	+0.95 or −1.44

NOTE: Positive indicates inwards (downwards) pressure.

3.3.2.3 *Structural models and action categories used for strength design*

The design action effects for the strength limit states shall be determined assuming wall plates are three span beams loaded by equally spaced concentrated actions arranged as shown in Table 3.3.2.3(A). The spacing between actions and the design spans assumed for each type of wall plate are given in Table 3.3.2.3(B).

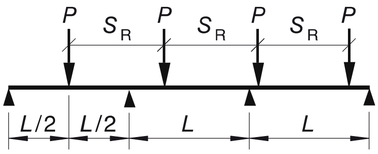
The design actions and the action combinations used for their computation shall be as given in Table 3.3.2.3(C).

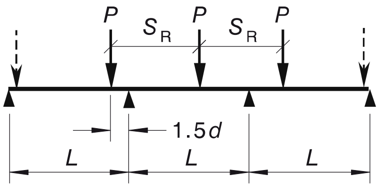
The design actions shown in Table 3.3.2.3(C) are divided into action categories that are used for the determination of the corresponding member design capacities as specified in Clause 3.3.2.4.

TABLE 3.3.2.3(A)

STRUCTURAL MODELS AND ACTION CATEGORIES FOR WALL PLATES—STRENGTH

Structural model	
For determination of design action effect in bending	**For determination of design action effect in shear**





LEGEND:

S_R = load spacing [see Table 3.3.2.3(B)]
L = span [see Table 3.3.2.3(B)]
d = depth of plate
P = concentrated action [see Table 3.3.2.3(C)]

NOTE: For design action effect in shear, actions within 1.5d of supports may be ignored.

TABLE 3.3.2.3(B)

ACTION SPACING AND SPANS FOR WALL PLATES

Application		Action spacing (*S*)	Span (*L*)
Upper storey or single storey	Top plate	Rafter/truss spacing	Stud spacing except for uplift, tie-down spacing
	Bottom plate	Stud spacing in wall	Joist spacing
Lower storey of two storeys	Top plate	Upper floor joist spacing	Stud spacing in lower storey wall
	Bottom plate	Stud spacing in lower storey wall	Ground storey floor joist spacing

TABLE 3.3.2.3(C)

DESIGN ACTIONS FOR WALL PLATES—STRENGTH

Action category	Design actions
1	$P = 1.35G$
	$P = 1.2G + 1.5Q_1$
2	$P = 1.2G + 1.5Q_3$
3	$P = 1.2G + 1.5Q_2$
4	$P = 1.2G + W_u\downarrow + Q_1$
	$P = 0.9G + W_u\uparrow$

3.3.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and shear. The following assumptions and modifications factors shall be used:

(a) *Load duration factor*—the member design capacity includes the modification factor for load duration (k_1). Values of k_1, appropriate for each action category defined in Table 3.3.2.3(C), are given in Table 3.3.2.4(A).

TABLE 3.3.2.4(A)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.80
3	0.94
4	1.00

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for action categories 2, 3 and 4, use values of k_4 appropriate for member thickness as given in AS 1720.1. For Action category 1, k_4 = 1.0.

(ii) *Seasoned timber*—k_4 = 1.0 for all action categories.

(c) *System-based effects*—where multiple plates are used (ribbon plates) the system factor (k_s) given in Table 3.3.2.4(B) is applied.

TABLE 3.3.2.4(B)

SYSTEM FACTOR FOR STRENGTH

Multiple plate no.	Load duration factor (k_1)
2	1.14
3	1.2

(d) *Member restraint*—wall plates are bent about their weak axis and, therefore, k_{12} = 1.

(e) *Composite action*—for nail-laminated multiple plates (ribbon plates) composite action is ignored.

(f) *Trenching*—the effect on strength of trenching up to 3 mm depth is ignored.

3.3.3 Design for serviceability

3.3.3.1 *Design actions*

The permanent actions and imposed actions used for the serviceability limit states shall be as described in Clause 3.3.2.2.

3.3.3.2 *Structural model and action categories for serviceability design*

Deflections shall be calculated assuming wall plates are three span continuous beams supporting uniformly spaced concentrated actions P (in kN) with one action positioned at the centre of an end span.

The design actions corresponding to the selected serviceability action combinations are given in Table 3.3.2.4(B). Design actions given in Table 3.3.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 3.3.3.3.

TABLE 3.3.3.2

DESIGN LOADS AND ACTION CATEGORIES—SERVICEABILITY

Action category	Design actions
1	$P = G + Q_1$
2	$P = Q_2$ $P = Q_3$

3.3.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the duration of load factor for creep deformation, j_2, given in Table 3.3.3.3 for action categories as defined in Table 3.3.3.2. The effect on deflection of trenching up to 3 mm deep and any composite action of nail-laminated ribbon plates shall be ignored.

TABLE 3.3.3.3

LOAD DURATION FACTORS FOR DEFORMATION

Initial moisture content	Action category 1	Action category 2
Seasoned	2.0	1
Unseasoned	3.0	1

3.3.3.4 *Serviceability limits*

The limits on deflection corresponding to the serviceability limit states defined in Clause 3.3.3.3 are given in Table 3.3.3.4.

TABLE 3.3.3.4

LIMITS ON DEFLECTION

Action category	Deflection limits
1 or 2	Span/200 or 3 mm max.

3.4 LINTELS

3.4.1 Description

Lintels are beams, contained within loadbearing walls, over windows or doors. They transfer the vertical loads applied over the opening to the jamb studs on each side.

For single or upper storey walls, common lintels are designed to support regularly spaced rafters or trusses. Design criteria are also included for lintels which, in addition to rafters, support a concentrated load from a roof principal such as a strutting beam or girder truss.

Lintels in lower storey walls of two-storey construction are designed to support uniformly distributed loads from the wall above including the roof loads supported by the upper wall and loads from an upper storey floor.

Lintels are designed as part of a system that includes consideration of the contribution of roof battens, wall plates, jack studs and lintel trimmers (see Clause 3.4.2.4).

For lintels, the limits on design deflections have been determined in order to maintain clearances above the window or door frames contained within the wall (see Figure 3.4.1).

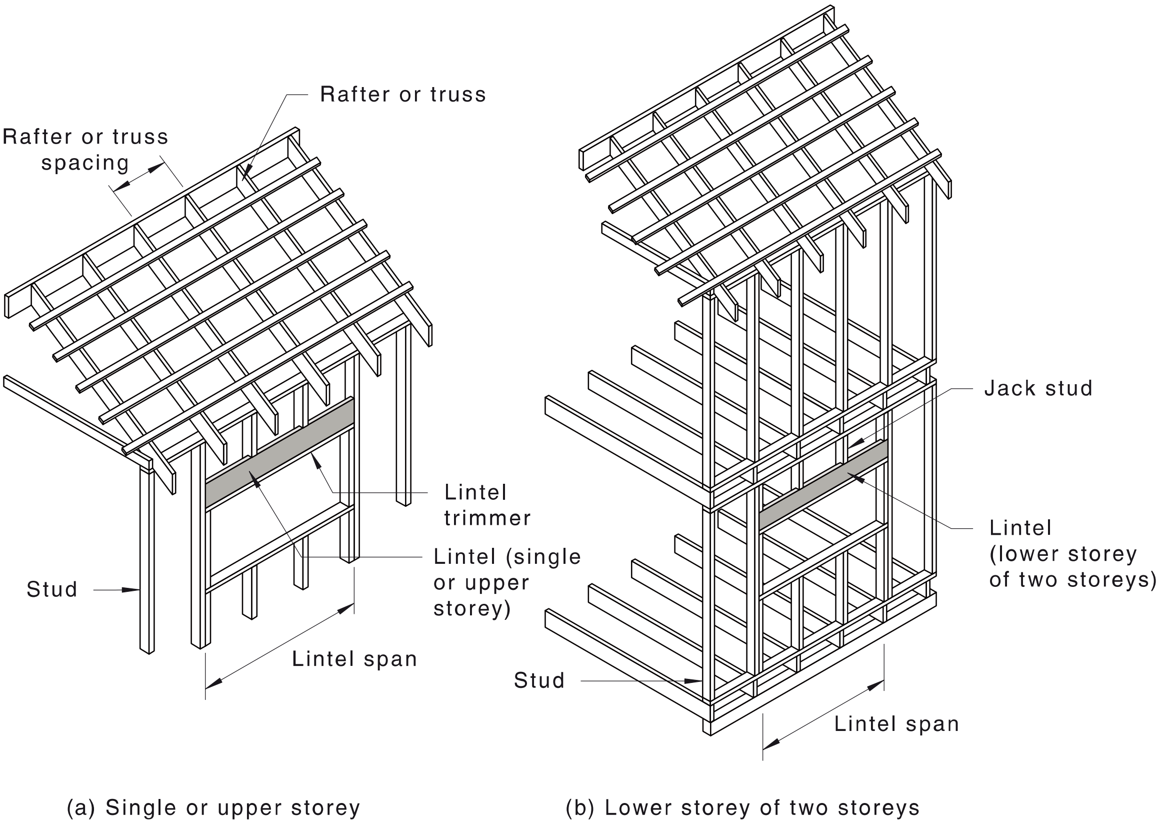


FIGURE 3.4.1 LINTELS

3.4.2 Design for safety

3.4.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in bending, shear and bearing.

3.4.2.2 *Design actions*

For lintels in single or upper storey walls, actions from rafters are considered applied as regularly spaced uniform concentrated actions. Where action from a roof principal is supported, an additional action related to the area of roof supported by the roof principal is considered.

For lintels in lower storey walls, roof, wall and floor actions are considered uniformly distributed.

Permanent actions, imposed actions and wind actions shall be as follows:

(a) *Permanent* The permanent actions considered include a uniformly distributed action G_1, regularly spaced uniform concentrated actions G_2 and, where a roof principal is supported, a single concentrated action G_3. Values of G_1, G_2 and G_3 are determined from Table 3.4.2.2(A).

TABLE 3.4.2.2(A)
PERMANENT ACTIONS

Application	Permanent actions	Unit
Lintels in single or upper storey walls—common lintels	G_1 = self weight $G_2 = 0.01(RM)(RLW)S_R$ $G_3 = 0$	kN/m kN —
Lintels in single or upper storey walls—additional concentrated roof load	G_1 = self weight $G_2 = 0.01(RM)(RLW)S_R$ $G_3 = 0.01(RM)A_R$	kN/m kN kN
Lintels in lower storey of two-storey construction	G_1 = self weight + $0.01(RM)(RLW) + 0.4(FLW) + 0.025(FLW)^2 + 0.4$ $G_2 = G_3 = 0$	kN/m —

LEGEND:
RM = roof mass —40 kg/m^2 for sheet roofs
—90 kg/m^2 for tile roofs
RLW = roof load width supported by wall, in metres
A_R = area of roof in square metres, supported by the lintel via a roof principal
FLW = floor load width supported by the wall, in metres
S_R = rafter spacing, 0.6 m or 1.2 m

(b) *Imposed* The concentrated imposed actions, Q_1 and Q_2, for lintels in single or upper storey walls, and the distributed imposed actions, Q_3, Q_4 and Q_5, for lintels in the lower storey of two storeys are determined from Table 3.4.2.2(B).

TABLE 3.4.2.2(B)
IMPOSED ACTIONS FOR LINTELS

Application	Imposed actions	Unit
Lintels in single or upper storey walls—common lintels	$Q_1 = 0.25S_R(RLW)$	kN
Lintels in single or upper storey walls—additional concentrated roof load	$Q_1 = MS_R(RLW)$ $Q_2 = MA_R$ where M = 0.25 kPa	kN
Lintels in lower storey of two-storey construction	$Q_3 = 0.25(RLW)$ $Q_4 = 0.50(FLW)$ $Q_5 = 1.50(FLW)$	kN/m

LEGEND:
A_R = area of roof supported by the lintel via a roof principal
N = number of equally spaced rafters supported by the lintel
S_R = spacing in metres of the equally spaced rafters, 0.6 m or 1.2 m
RLW = roof load width supported by the wall, in metres
FLW = floor load width for the upper floor supported by the lower storey wall, in metres
Q_1, Q_2 and Q_3 = roof imposed action
Q_4 = permanent floor imposed action
Q_5 = short term floor imposed action

(c) *Wind* The concentrated wind actions for lintels in single or upper storey walls, W_{U1} and W_{U2}, are determined from Table 3.4.2.2(C).

TABLE 3.4.2.2(C)
WIND ACTIONS FOR LINTELS

Application	Wind actions	Unit
Lintels in single or upper storey walls—common lintels	$W_{U1} = q_u C_{pt} S_R (RLW)$	kN
Lintels in single or upper storey walls—additional concentrated roof load	$W_{U1} = q_u C_{pt} S_R (RLW)$ $W_{U2} = q_u C_{pt} A_R$	kN
Lintels in lower storey of two-storey construction	(See Note below)	—

LEGEND:

q_u = free stream dynamic gust pressure for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficient given in Table 3.4.2.2(D)

A_R = area of roof supported by the lintel via a roof principal

S_R = spacing in metres of the equally spaced rafters, 0.6 m or 1.2 m

RLW = roof load width supported by the wall, in metres

NOTE: Horizontal wind action is not considered. Vertical wind action is only considered for lintels in single or upper storey walls.

TABLE 3.4.2.2(D)
NET PRESSURE COEFFICIENTS FOR LINTELS STRENGTH

Wind classification	C_{ptr}
N1 to N4	+0.63 or −1.0
C1 to C3	+0.95 or −1.44

3.4.2.3 *Structural models, design actions and action categories*

The structural models and design actions used to determine the member design action effects shall be as given in Table 3.4.2.4. Action combinations shown in Table 3.4.2.4 are divided into action categories that are used in the determination of corresponding member design capacities as specified in Clause 3.4.2.5.

3.4.2.4 *Design action effects in bending and shear*

The design action effects applied to the lintel in bending and shear, M^* (in kNm) and V^* (in kN) respectively, shall be determined from—

$$M^* = M - 0.55k_1 \quad \text{. . . 3.4.2.4(1)}$$

$$V^* = V - 7.0k_1 \quad \text{. . . 3.4.2.4(2)}$$

where

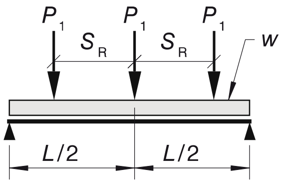
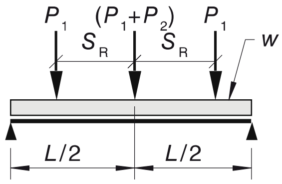
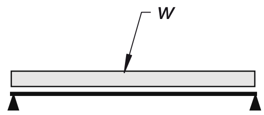
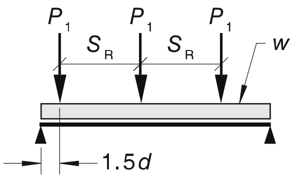
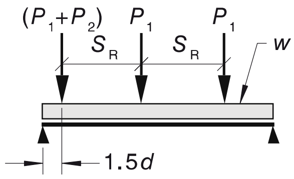
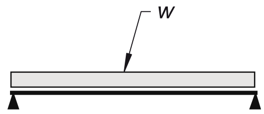
M = maximum bending moment, in kNm, determined using the design actions and structural models given in Table 3.4.2.4(A)

V = maximum shear force, in kN, determined using the design actions and structural models given in Table 3.4.2.4(A)

k_1 = duration of load factor for strength given in Table 3.4.2.4(B) for the corresponding action category given in Table 3.4.2.4(A)

NOTE: The above expressions include an allowance for the contribution made by parallel members, such as roof battens and wall plates, in the support of the loads assumed applied to the lintel. Where these contributions are unlikely to occur, such as where no wall plates are used or the span is in excess of that used in conventional construction, use un-modified versions of M^* and V^*.

TABLE 3.4.2.4(A)
STRUCTURAL MODELS FOR LINTELS—STRENGTH

Design action effect	Structural model: Lintels in single or upper storey walls — Common lintels	Structural model: Lintels in single or upper storey walls — Lintels supporting additional concentrated roof load	Structural model: Lintels in lower storey of two storey walls
For bending			
For shear and bearing			
Action category	**Design actions**		
1	$w = 1.35G_1$ $P_1 = 1.35G_2$	$w = 1.35G_1$ $P_1 = 1.35G_2$ $P_2 = 1.35G_3$	$w = 1.35G_1$ $w = 1.2G_1 + 1.5Q_4$
2	—	—	$w = 1.2G_1 + 1.5Q_5$
3	—	—	$w = 1.2G_1 + 1.5Q_3$
4	$w = 1.2G_1$ $P_1 = 1.2G_2 + 1.5Q_1$	$w = 1.2G_1$ $P_1 = 1.2G_2 + 1.5Q_1$ $P_2 = 1.2G_3 + 1.5Q_2$	—
5	$w = 1.2G_1$ $P_1 = 1.2G_2 + W_{U1}\downarrow$	$w = 1.2G_1$ $P_1 = 1.2G_2 + W_{U1}\downarrow$ $P_2 = 1.2G_3 + W_{U2}\downarrow$	—
	$w = 0.9G_1$ $P_1 = 0.9G_2 + W_{U1}\uparrow$	$w = 0.9G_1$ $P_1 = 0.9G_2 + W_{U1}\uparrow$ $P_2 = 0.9G_3 + W_{U2}\uparrow$	—

NOTE: S_R is rafter spacing, either 0.6 m or 1.2 m.

TABLE 3.4.2.4(B)
LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.80
3	0.94
4	0.97
5	1.00

3.4.2.5 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending, shear and bearing. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category defined in Table 3.4.2.4(A) are given in Table 3.4.2.4(B).

(b) *Moisture content of timber*:

(i) *For unseasoned timber* For action categories 4 and 5, use values of k_4 appropriate for member thickness as given in AS 1720.1. For action categories 1, 2 and 3, $k_4 = 1.0$.

(ii) *For seasoned timber* $k_4 = 1.0$ for all action categories.

(c) *Strength sharing* Where multiple sections of scantling timber are nail-laminated, the strength sharing factor (k_9) is applied for the combined member, assuming $n_{\text{mem}} = 1$ and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity, the following assumptions relating to lateral restraint are used:

(i) *At supports*—lintels are considered torsionally restrained at their supports.

(ii) *Between supports*—lintels are assumed torsionally restrained at 600 mm centres.

NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination and not the overall breadth.

3.4.3 Design for serviceability

3.4.3.1 *Design actions*

The actions used for the serviceability limit states shall be as follows:

(a) *Permanent and imposed* The concentrated and uniformly distributed permanent and imposed actions applied to lintels are determined as described in Clause 3.4.2.2.

(b) *Wind* For lintels in single or upper storey walls, wind action is considered applied as a series of regularly spaced uniform concentrated actions, W_{S1} (in kN), and, where a roof principal is supported, an additional concentrated action, W_{S2} (in kN). W_{S1} and W_{S2} are determined from—

$$W_{S1} = q_s C_{pt} S_R (RLW) \quad \text{. . . 3.4.3.1(1)}$$

$$W_{S2} = q_s C_{pt} A_R \quad \text{. . . 3.4.3.1(2)}$$

where

q_s = free stream dynamic gust pressure, in kPa, for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients given in Table 3.4.3.1

S_R = rafter spacing, 0.6 m or 1.2 m

RLW = roof load width for lintel, in metres

A_R = area of roof supported by the lintel via the roof principal

TABLE 3.4.3.1

NET PRESSURE COEFFICIENTS FOR LINTELS (SERVICEABILITY)

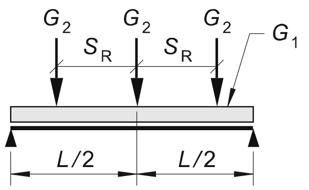
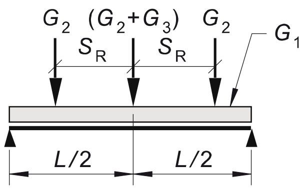
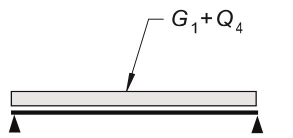
Wind classification	C_{pt}
N1 to N4	+0.63, −1.0
C1 to C3	

3.4.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 3.4.3.2. Load cases given in Table 3.4.3.2 are divided into action categories for the purpose of allowing for duration of load on stiffness as specified in Clause 3.4.3.3.

TABLE 3.4.3.2

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

<table>
<tr><th rowspan="3">Action category</th><th colspan="3">Structural model</th></tr>
<tr><th colspan="2">Lintels in single or upper storey walls</th><th rowspan="2">Lintels in lower storey walls of two storeys</th></tr>
<tr><th>Common lintels</th><th>Lintels supporting concentrated roof loads</th></tr>
<tr><td>1</td><td>
</td><td>
</td><td>
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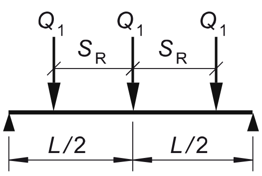
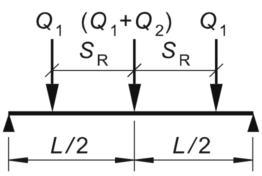
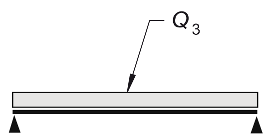
<tr><td>2</td><td>
</td><td>
</td><td>
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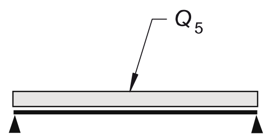
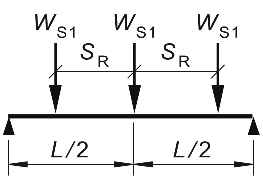
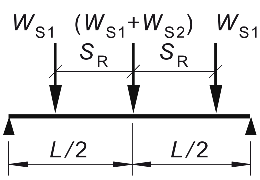
<tr><td>3</td><td></td><td></td><td>
</td></tr>
<tr><td>4</td><td>
</td><td>
</td><td></td></tr>
</table>

NOTE: S_R is rafter spacing, either 0.6 m or 1.2 m.

3.4.3.3 *Calculation of deflection*

The deflection of lintels is calculated taking into account the contribution of parallel members by adding an allowance for their rigidity, $EI = 15.9 \times 10^9$ Nmm^2, to the rigidity of the lintel.

The requirements of AS 1720 for the calculation of deflection shall be applied using the duration of load factor for creep deformation as given in Table 3.4.3.4(A).

3.4.3.4 *Serviceability limits*

The limits on deflection used to define the serviceability limit states are given in Table 3.4.3.4(B).

TABLE 3.4.3.4(A)

LOAD DURATION FACTORS FOR DEFORMATION

Initial moisture content	Load duration factor (j_2)	
	Action category 1	Action categories 2, 3 and 4
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

TABLE 3.4.3.4(B)

LIMITS ON DEFLECTION

Action category	Deflection limits
1	Span/300 or 10 mm max.
2	Span/250 or 15 mm max.
3	Span/360 or 10 mm max.
4	Span/200

3.5 WIND BEAMS

3.5.1 Description

Wind beams are horizontal members, located between upper and lower wall frames, which transfer horizontal wall loads, in bending, to bracing walls.

Wind beams are used to transfer wind loads where there is no floor system located between the upper and lower wall frames, such as may occur in stairwells. Wind beams transfer these loads directly, or indirectly via diaphragm action, to suitable bracing walls (see Figure 3.5.1).

Vertical roof and wall loads above the wind beam are assumed to be transferred directly to the wall frame below the wind beam and are thus not considered in design.

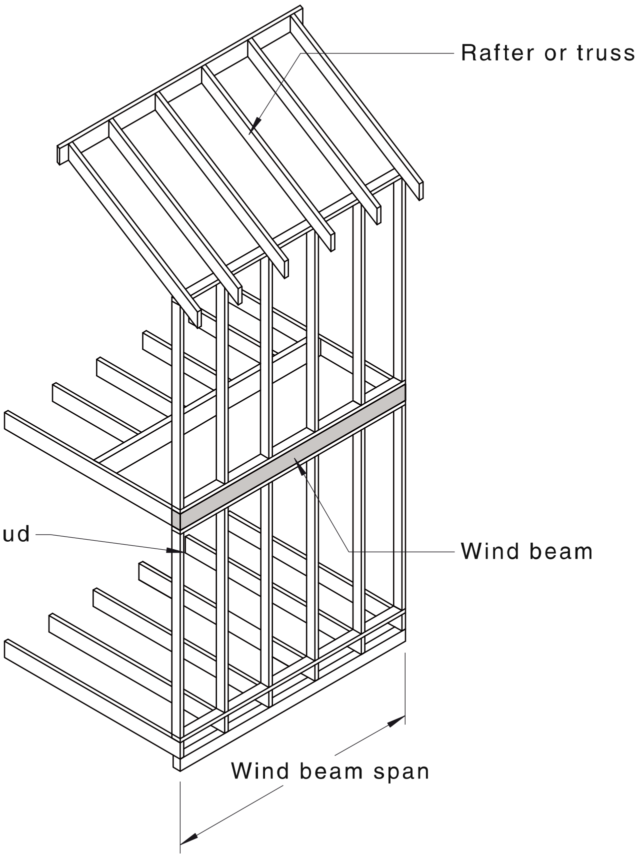


FIGURE 3.5.1 WIND BEAMS

3.5.2 Design for safety

3.5.2.1 *General consideration*

Design for safety requires consideration of the strength limit states in bending and shear.

3.5.2.2 *Design actions*

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

(a) *Permanent and imposed* Wind beams are not considered to support any permanent or imposed actions, including self-weight, as they are assumed fully supported by the lower wall frame and are only required to resist horizontal wind loads. (G = 0, Q = 0).

(b) *Wind* Wind actions for wind beams are considered applied as uniformly distributed lateral actions W_{uw} (in kN/m). Values of W_{uw} are determined from the following expression:

$$W_{uw} = q_s C_{ptw}(H_1 + H_2)0.5$$

where

q_s = free stream dynamic gust pressure for the serviceability limit state; values of q_s are given in Appendix A, Table A2, for each wind classification

C_{ptw} = net pressure coefficient for walls given in Table 3.5.2.2

H_1 = height of upper storey wall, in metres

H_2 = height of lower storey wall, in metres

TABLE 3.5.2.2

PRESSURE COEFFICIENTS FOR WALLS—STRENGTH

Wind classification	C_{ptw}
N1 to N4	0.9
C1 to C3	1.2

3.5.2.3 *Structural models and action categories for strength design*

Wind beams are designed as simply supported beams resisting a uniformly applied lateral wind load, $w = W_{uw}$. Determination of the corresponding member design capacity is as specified in Clause 3.5.2.4.

3.5.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending and in combined bending and shear. The following assumptions and modifications factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Use $k_1 = 1$.

(b) *Moisture content of timber*:

   (i) *Unseasoned timber*—use values of k_4 appropriate for thickness as specified in AS 1720.1.

   (ii) *Seasoned timber*—$k_4 = 1.0$.

(c) *Composite action* For nail-laminated multiple wind beam sections, composite action is ignored.

(d) *Member restraint* For the determination of bending capacity, wind beams are assumed torsionally restrained at supports and torsionally and laterally restrained between supports by the upper and lower wall frames; $L_{ay} = 600$ mm. In addition, the tension edge is assumed laterally restrained at intervals not greater than 600 mm.

   NOTE: Where nail-laminated members are used, the breadth of member used to derive the slenderness coefficient (S_1) is taken as the breadth of an individual lamination.

3.5.3 Design for serviceability

3.5.3.1 *General consideration*

The serviceability limit state for lateral deformation shall be considered.

NOTE: The application of a serviceability limit state for serviceability wind pressure on the wind beams is assumed to ensure adequate lateral rigidity for incidental horizontal live loads such as may be experienced in construction and maintenance.

3.5.3.2 *Design actions*

The distributed wind action (W_{uw}), used for the serviceability limit state, shall be as specified in Clause 3.5.2.2(b), with the value of C_{ptw} determined from Table 3.5.3.2.

TABLE 3.5.3.2

PRESSURE COEFFICIENTS FOR WALLS—SERVICEABILITY

Wind classification	Net pressure coefficient for walls (C_{ptw})
N1 to N4	0.9
C1 to C3	

3.5.3.3 *Structural model for serviceability design*

The structural model used to determine deflection shall be as for a simply supported beam supporting the uniformly distributed lateral wind load.

3.5.3.4 *Calculation of deflection*

Deflection of wind beams, under the serviceability wind action specified in Clause 3.5.3.2, shall be calculated assuming the structural model specified in Clause 3.5.3.3 taking into account the contribution of parallel members by adding an allowance for their rigidity, $EI = 10.6 \times 10^9$ Nmm^2, to the rigidity of the wind beam. No modification is required for duration of load for this case.

3.5.3.5 *Serviceability limit*

The deflection of wind beams under the serviceability wind action, given in Clause 3.5.3.2 and calculated in accordance with Clause 3.5.3.4, is limited to span/200, but not greater than 15 mm.

NOTE: This limit may not preclude damage to brittle surface finishes.

SECTION 4 DESIGN OF FLOOR MEMBERS

4.1 FLOOR JOISTS

4.1.1 Description

Floor joists are closely spaced parallel beams supporting overlying flooring or decking. Their primary purpose is to support floor loads. Floor joists may also be required to support ceilings and loadbearing walls that run either parallel or perpendicular to the direction of the joists (see Figure 4.1.1).

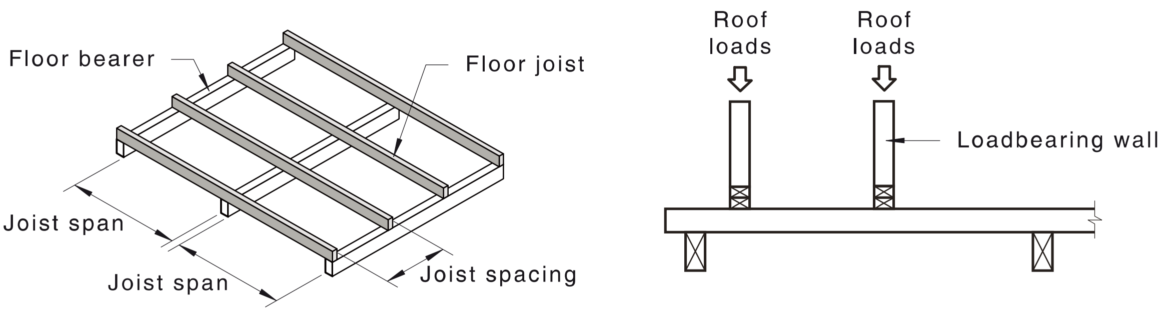


FIGURE 4.1.1 FLOOR JOISTS

4.1.2 Design for safety

4.1.2.1 *General consideration*

Floor joist design for safety requires consideration of the strength limit state for bending, shear and bearing.

4.1.2.2 *Design actions*

The values of the permanent actions and imposed actions used for design shall be as follows:

(a) *Permanent* Uniformly distributed permanent actions, G_1, and concentrated permanent actions, G_2, are determined from Table 4.1.2.2(A).

TABLE 4.1.2.2(A)

PERMANENT ACTIONS

Source of load	Distributed load, (G_1) kN/m	Concentrated load, (G_2) kN
Floor only:		
—floor mass up to 40 kg/m^2	0.4S + self weight	0
—floor mass up to 100 kg/m^2	1.0S + self weight	0
Loadbearing walls supporting roof loads. Wall perpendicular to joists but offset from supports:		
—tile roof	—	$(RLW \times 0.9 + 0.4)S$
—sheet roof	—	$(RLW \times 0.4 + 0.4)S$

NOTES:

1 S = spacing of joists in metres and RLW = roof load width, in metres.

2 For any particular case, combine the loads from each source to obtain the total.

(b) *Imposed* Distributed imposed actions, Q_1 to Q_4, and Q_7, and concentrated imposed actions, Q_5 and Q_6, are determined from Table 4.1.2.2(B).

For the determination of the concentrated imposed action, Q_5, a load distribution factor (g_{42}) is applied for the joist grid system as follows:

(i) *For bending*—the value of the load distribution factor, g_{42}, for concentrated loads applied anywhere within the middle half of the floor joist span and at least two joists in from the edge, is determined in accordance with the requirements of AS 1720.1 assuming the crossing member is flooring of the following nominal rigidity:

(A) For joist spacing ≤450 mm, $E_c I_c = 1.72 \times 10^6 \times L$ (Nmm^2) and $n_c = 1.0$.

(B) For joist spacing >450 mm but ≤600 mm, $E_c I_c = 2.66 \times 10^6 \times L$ (Nmm^2) and $n_c = 1.0$.

where

$E_c I_c$ = flexural rigidity of the flooring

L = span of floor joists, in mm

n_c = number of crossing members (flooring)

NOTE: The expression for rigidity, $E_B I_B$, given in AS 1720.1 is only valid for solid beams where the contribution of shear to overall rigidity is allowed for in the values of Modulus of Elasticity. For built-up sections such as I-sections or box sections, where the effect of shear may be significant, an effective overall rigidity should be calculated taking into account the effect of shear for the particular span and load case. Alternatively, adopt $g_{42} = 1$ for these cases.

(ii) *For shear and bearing*—$g_{42} = 1.0$.

TABLE 4.1.2.2(B)

IMPOSED ACTIONS

Type of load	Imposed action	Unit
Permanent: —*UDL*	$Q_1 = 0.5S$	kN/m
Transient:		
(a) *UDL*—general	$Q_2 = 1.5S$	kN/m
(b) Partial *UDL*	$Q_3 = 0.75S$	kN/m
(c) *UDL*—balcony or deck	$Q_4 = 2.0S$	kN/m
(d) Concentrated—general	$Q_5 = g_{42} \times 1.8$	kN
(e) Balcony or deck line load (concentrated)	$Q_6 = 1.5S$	kN
(f) Balcony or deck line load (*UDL*)	$Q_7 = 1.5$	kN/m

NOTES:

1 S = spacing of joists, in metres and g_{42} as defined in 4.1.2.2(b).

2 Where a balcony or deck is less than 1 m above ground level, the transient *UDL*—general may be replaced with $Q_4 = 1.5S$ kN/m.

4.1.2.3 *Structural models and action categories for strength*

The structural models used to calculate the member design action effects shall be as given in Table 4.1.2.4(A). Action combinations shown in Table 4.1.2.4(A) are divided into action categories used for the determination of member design capacity as specified in Clause 4.1.2.4.

4.1.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending, shear and bearing. The following assumptions and modification factors shall be used:

(a) *Load duration factor* the member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category are given in Table 4.1.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for load categories 2 and 3 given in Table 4.1.2.4(A), use values of k_4 appropriate to thickness as given in AS 1720.1. For Action category 1, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all action categories.

(c) *Strength sharing* For sawn timber floor joists, the strength sharing factor (k_9) is applied, assuming $n_{mem} = 5$ and $n_{com} = 1.0$. For other timber products $k_9 = 1.0$.

(d) *Member restraint* For the determination of bending capacity, the following assumptions regarding lateral restraint are used:

(i) *At supports*—floor joists are assumed rotationally and torsionally restrained at their supports.

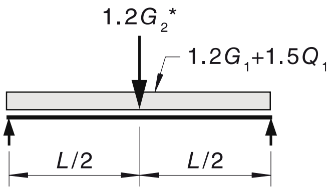
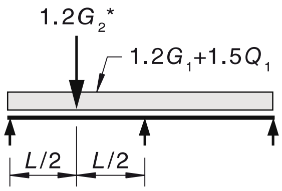
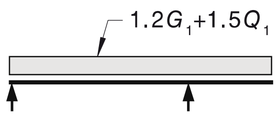
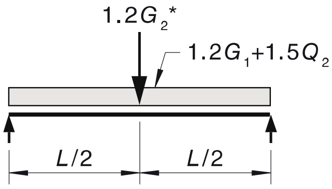
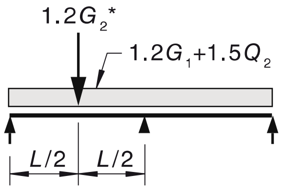
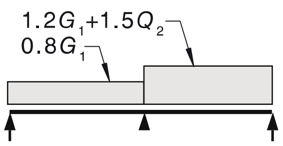
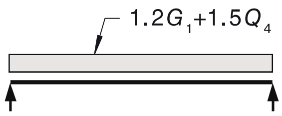
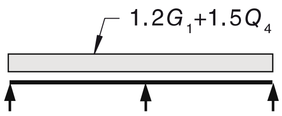
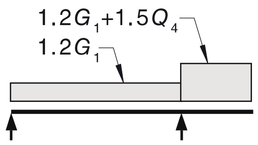
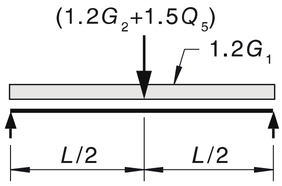
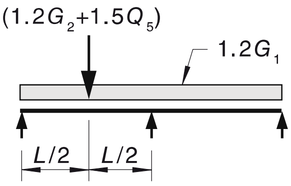
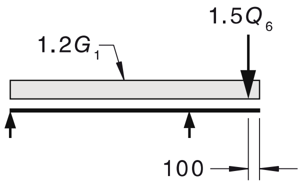
(ii) *Between supports*:

(A) The top edges of joists are assumed continuously laterally restrained.

(B) Continuous span joists are assumed restrained against buckling at the points of contraflexure. That is, for the negative moment case, $L_{ay} = L/4$.

TABLE 4.1.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

Load category	Single span	Continuous span	Overhang (cantilevered)
1			
2		 	
	For balcony or deck joists only:  (Edge joists—substitute Q_7 for Q_4)	For balcony or deck joists only:  (Edge joists—substitute Q_7 for Q_4)	For balcony or deck joists only:  (Edge joists—substitute Q_7 for Q_4)
3			

NOTES:

1 Concentrated loads, G_2 and Q_5, are considered applied at mid-span (as shown) for bending, or at 1.5d from supports for shear, or at supports for bearing. G_2 does not apply where joists do not support loadbearing walls perpendicular to the joists.

2 Action category 3—Imposed action Q_5 is considered to represent loads applied for short periods and at infrequent intervals such as might be experienced during erection and maintenance or periods of crowd loading. Where this is not the case consideration should be given to adopting a lower value of k_1 than that given for this action category in Table 4.1.2.4(B).

3 Edge joists are joists located along the edge of a balcony or deck.

TABLE 4.1.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.80
3	0.94

4.1.3 Design for serviceability

4.1.3.1 *General consideration*

Floor joist design for serviceability requires consideration of the serviceability limit states for flexural deformation and dynamic behaviour.

4.1.3.2 *Design actions*

Permanent actions and imposed actions used for the serviceability limit state shall be as follows:

(a) *Permanent* The concentrated and uniformly distributed permanent actions are determined as described in Clause 4.1.2.2(a).

(b) *Imposed* The relevant concentrated and uniformly distributed imposed actions are determined as described in Clause 4.1.2.2(b) and as follows for an additional concentrated action, Q_8 (in kN):

$$Q_8 = g_{41} \times 1.0 \qquad \text{. . . 4.1.3.2}$$

Where g_{41} is the load distribution factor given in AS 1720.1 for point loads applied at the mid-span of beams in a grid system. The factor g_{41} is calculated using the same assumptions as used to calculate g_{42} in Clause 4.1.2.2 including the provision for considering the rigidity of built-up versus solid sections. Where this provision is to be ignored, adopt g_{41} = 1.

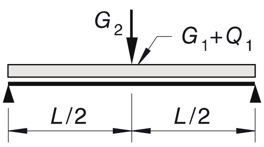
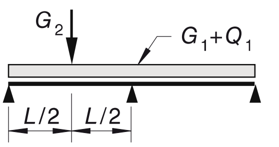
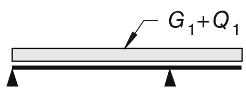
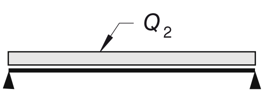
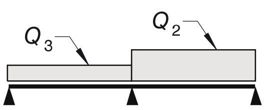
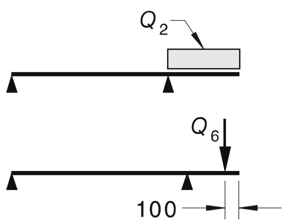
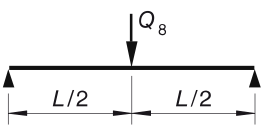
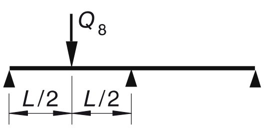
NOTE: The limit on deflection resulting from the application of the Q_8 action is intended to ensure satisfactory dynamic performance. The application of this criterion replaces the need to separately consider deflection due to the 1.8 kN concentrated imposed action for floors.

4.1.3.3 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 4.1.3.3. The action cases given in Table 4.1.3.3 are divided into action categories for the purpose of allowing for the effect of duration of load on stiffness, as specified in Clause 4.1.3.4.

TABLE 4.1.3.3

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

Load category	Single span	Continuous span	Overhang (cantilevered)
1			
2			
3			

4.1.3.4 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflections shall be applied using the load duration factor for flexural deformation (j_2) as given for each action category in Table 4.1.3.5(A).

4.1.3.5 *Serviceability limits*

For the purpose of assessing the serviceability limit states, the limits on deflection for each of the action categories detailed in Table 4.1.3.3 are given in Table 4.1.3.5(B).

TABLE 4.1.3.5(A)

LOAD DURATION FACTORS FOR DEFORMATION

Moisture content	Load duration factor (j_2)	
	Action category 1 (permanent loads)	Action categories 2 and 3 (transient loads)
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

TABLE 4.1.3.5(B)

LIMITS ON DEFLECTION

Action category	Limits on deflection	
	Single or continuous span	Overhang (cantilever)
1	Span/300 or 15 mm max.	Overhang/150* or 6 mm max.
2	Span/360 or 9 mm max.	Overhang/180* or 4.5 mm max.
3	2 mm	—

* Where the deflection at the end of the cantilever is upwards, the overhang/150 or overhang/180 limit shall be ignored.

NOTE: The floor dynamics deflection limit for Action category 3 has been developed for solid section floor joists up to 6 m spans. A tighter deflection limit may be required for larger spans or where products with deflection characteristics different from solid sections, such as I-sections, are used.

4.2 BEARERS

4.2.1 Description

Bearers are beams providing direct support for floor joists but in addition may support loads from loadbearing walls supporting roof loads and/or from upper storey floors.

Design includes consideration of single, continuous or cantilevered span applications. Concentrated permanent actions resulting from support to posts or intersecting loadbearing walls at locations other than at or near bearer supports are not considered (see Figure 4.2.1).

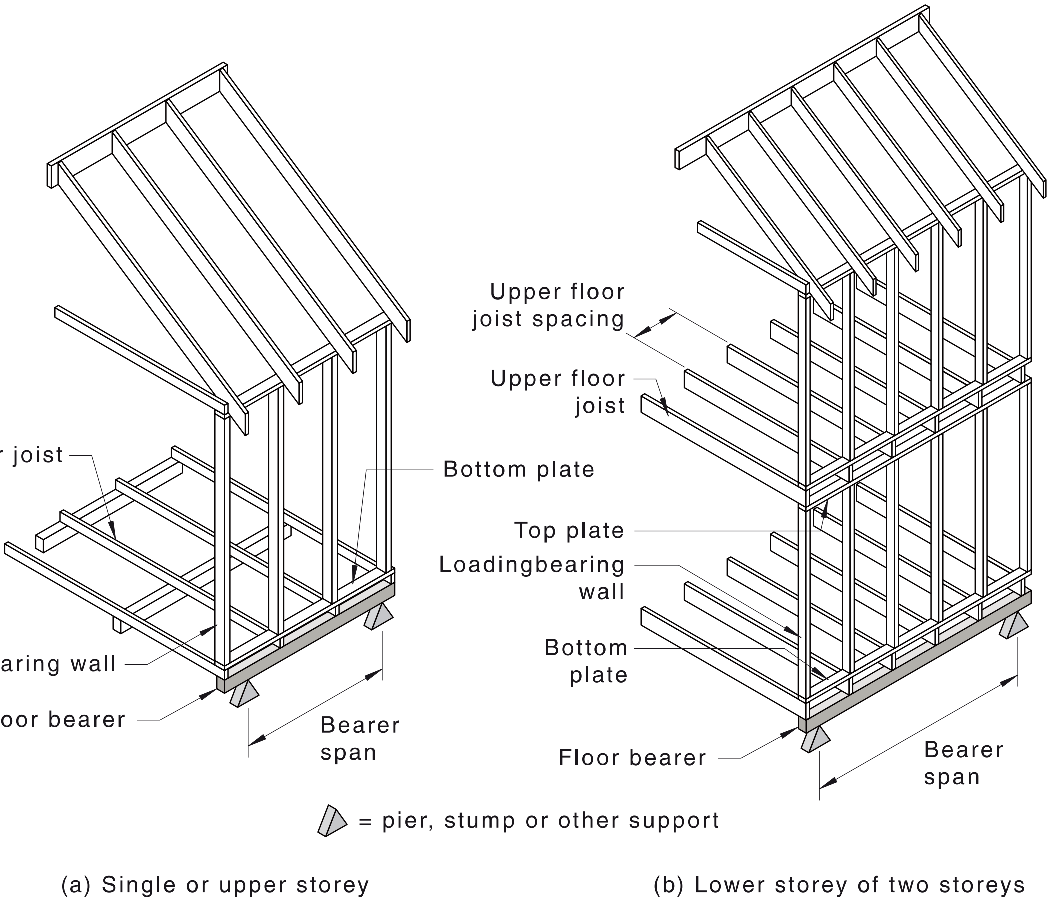


FIGURE 4.2.1 BEARER SUPPORTING LOADBEARING WALL

4.2.2 Design for safety

4.2.2.1 *General consideration*

Design for safety requires consideration of the strength limit states for bending, shear and bearing.

4.2.2.2 *Design actions*

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

(a) *Permanent* The uniformly distributed permanent action, *G*, for each bearer type is obtained by summing the loads from each applicable load source. Actions used for each load source are determined from Table 4.2.2.2(A).

TABLE 4.2.2.2(A)
PERMANENT ACTIONS

Source of load	Distributed permanent action, (G) kN/m
Bearer supporting floor joists only	0.4 (FLW) + 0.025 (FLW)2 + self weight
Add the following, as applicable:	
(a) Support to parallel loadbearing walls	
—single storey	0.4
—double storey	0.8
(b) Support to roofs	
—sheet roof	0.4 (RLW)
—tile roof	0.9 (RLW)
(c) Support to floor above	0.4 (FLW) + 0.025 (FLW)2

LEGEND:
FLW = floor load width, in metres, for the relevant floor
RLW = roof load width, in metres, for the roof supported by the bearer

(b) *Imposed* Distributed imposed actions Q_1, Q_2, Q_4 and Q_6 and concentrated imposed actions Q_3 and Q_5 are determined for each application, as appropriate, from Table 4.2.2.2(B).

TABLE 4.2.2.2(B)
IMPOSED ACTIONS

Type of load	Imposed action	Unit
Permanent:		
—due to floor directly supported	$Q_1 = 0.5(FLW)_1$	kN/m
—due to floor above, if applicable	$+0.5(FLW)_2$	kN/m
Transient:		
(a) *UDL*—general		
—due to floor directly supported	$Q_2 = 1.5(FLW)_1$	kN/m
—due to floor above, if applicable	$+0.75(FLW)_2$	kN/m
(b) Concentrated—general	$Q_3 = 1.8$	kN
(c) *UDL*—balcony or deck	$Q_4 = 2.0(FLW)_1$	kN/m
(d) Balcony or deck line load (concentrated)	$Q_5 = 1.5(FLW)_1$	kN
(e) Balcony or deck line load (*UDL*)	$Q_6 = 1.5$	kN/m

LEGEND:
$(FLW)_1$ = floor load width, in metres, for the directly supported floor
$(FLW)_2$ = floor load width, in metres, for a supported floor above

NOTES:

1 Roof imposed actions shall not be considered to be applied at the same time as full floor imposed actions.

2 Where a balcony or deck is less than 1 m above ground level, the transient *UDL*—balcony or deck may be replaced with $Q_4 = 1.5(FLW)_1$ kN/m.

4.2.2.3 *Structural models and action categories for strength*

The structural models used to calculate the member design action effects shall be as given in Table 4.2.2.4(A). Action combinations shown in Table 4.2.2.4(A) are divided into action categories that are used for the determination of member design capacity as specified in Clause 4.2.2.4.

4.2.2.4 *Member design capacity*

The requirements of AS 1720.1 shall be applied to determine member design capacities in bending, shear and bearing. The following assumptions and modification factors shall be used:

(a) *Load duration factor* The member design capacity includes the modification factor for load duration (k_1). Values of k_1 appropriate for each action category, as defined in Table 4.2.2.4(A), are given in Table 4.2.2.4(B).

(b) *Moisture content of timber*:

(i) *Unseasoned timber*—for load categories 2 and 3 given in Table 4.2.2.4(A), use values of k_4 appropriate to thickness as given in AS 1720.1. For Action category 1, $k_4 = 1.0$.

(ii) *Seasoned timber*—$k_4 = 1.0$ for all action categories.

(c) *Strength sharing* For nail-laminated members, the strength sharing factor (k_9) is applied for the combined member, assuming $n_{mem} = 1.0$ and n_{com} = number of combined sections.

(d) *Member restraint* For the determination of bending capacity the following assumptions relating to lateral restraint are used:

(i) *At supports*—bearers are assumed torsionally restrained at their supports.

(ii) *Between supports*:

(A) The top edges of bearers are assumed laterally restrained along the top edge by floor joists spaced at 600 mm centres.

(B) Continuous span bearers are assumed restrained against buckling at the point of contraflexure. That is, for the negative moment case, $L_{ay} = L/4$.

TABLE 4.2.2.4(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—STRENGTH

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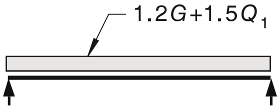
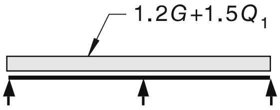
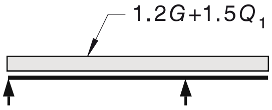
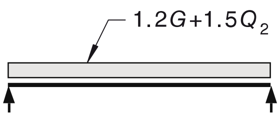
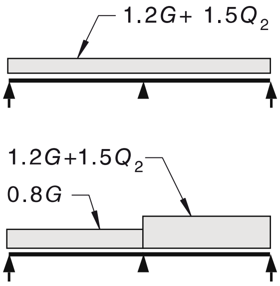
<tr><td>1</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">2</td><td></td><td></td><td></td></tr>
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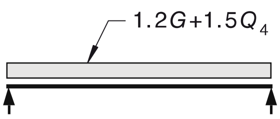
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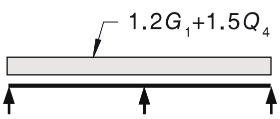
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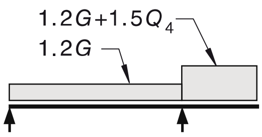
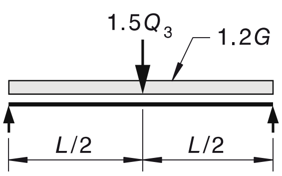
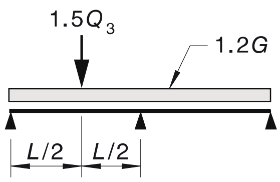
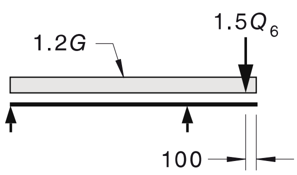

(Edge bearers—use Q_6 if $Q_6 > Q_4$)</td></tr>
<tr><td>3</td><td></td><td></td><td></td></tr>
</table>

NOTES:

1 Concentrated action Q_3 is considered applied at mid-span (as shown) for bending, or at 1.5d from supports for shear, or at supports for bearing.

2 Concentrated action Q_3 is considered to represent loads applied for short periods and at infrequent intervals such as might be experienced during erection and maintenance or periods of crowd loading. Where this is not the case adopt an appropriate lower value of k_1 than that given for this action category in Table 4.1.2.4(B).

3 Edge bearers are bearers located along the edge of a balcony or deck.

TABLE 4.2.2.4(B)

LOAD DURATION FACTORS FOR STRENGTH

Action category	Load duration factor (k_1)
1	0.57
2	0.80
3	0.94

4.2.3 Design for serviceability

4.2.3.1 *Design actions*

The permanent actions and imposed actions used to determine deflections for the serviceability limit state shall be as follows:

(a) *Permanent* The uniformly distributed permanent action, G, is determined as described in Clause 4.2.2.2.

(b) *Imposed* Uniformly distributed imposed actions Q_1, Q_2 and Q_3 and concentrated imposed actions Q_4 and Q_5 are determined from Table 4.2.3.1.

TABLE 4.2.3.1

IMPOSED ACTIONS—SERVICEABILITY

Type of load	Imposed action	Unit
Permanent:		
—due to floor directly supported	$Q_1 =$ 0.5$(FLW)_1$	kN/m
—due to floor above, if applicable	+0.5$(FLW)_2$	kN/m
Transient (see Note below):		
(a) *UDL*—general	$Q_2 =$ 1.5$(FLW)_1$	kN/m
(b) Partial *UDL*	$Q_3 =$ 0.75$(FLW)_1$	kN/m
(c) Concentrated load	$Q_4 =$ 1.8	kN
(d) Balcony or deck line load	$Q_5 =$ 1.5$(FLW)_1$	kN

LEGEND:

$(FLW)_1$ = floor load width, in metres, for the floor directly supported by the bearer

$(FLW)_2$ = floor load width, in metres, for floor above, if applicable

NOTE: Only the transient imposed action on the floor directly supported shall be considered.

4.2.3.2 *Structural models and action categories for serviceability design*

The structural models for which deflections are calculated shall be as given in Table 4.2.3.3(A). The action cases given in Table 4.2.3.3(A) are divided into action categories for the purpose of allowing for the effect of duration of load on stiffness as specified in Clause 4.2.3.3.

4.2.3.3 *Calculation of deflection*

The requirements of AS 1720.1 for the calculation of deflection shall be applied using the load duration factor for flexural deformation (j_2), as given in Table 4.2.3.3(B), for each action category in Table 4.2.3.3(A).

TABLE 4.2.3.3(A)

STRUCTURAL MODELS AND ACTION CATEGORIES—SERVICEABILITY

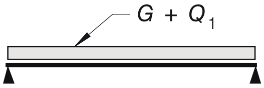
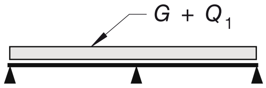
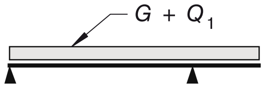
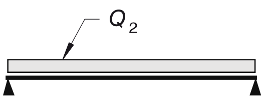
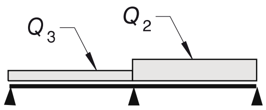
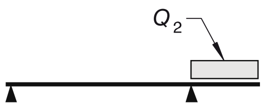
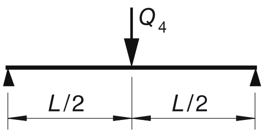
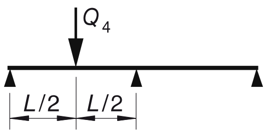
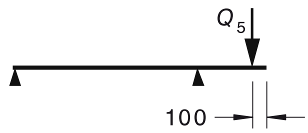
Action category	Single span	Continuous span	Cantilevered
1			
2			
			

TABLE 4.2.3.3(B)

LOAD DURATION FACTORS FOR DEFORMATION

Initial moisture content	Load duration factor (j_2)	
	Action category 1 (permanent actions)	Action category 2 (transient actions)
Seasoned	2.0	1.0
Unseasoned	3.0	1.0

4.2.3.4 *Serviceability limits*

For the purpose of assessing the serviceability limit states, the limits on deflection used for each of the action categories detailed in Table 4.2.3.3(A) are given in Table 4.2.3.4.

TABLE 4.2.3.4

LIMITS ON DEFLECTION

Action category	Limits on deflection	
	Single or continuous span	Overhang (cantilever)
1	Span/300 or 12 mm max.	Overhang/150* or 6 mm max.
2	Span/360 or 9 mm max.	Overhang/180* or 4.5 mm max.

* Where the deflection at the end of the cantilever is upwards, the overhang/150 or overhang/180 limits shall be ignored.

NOTE: Floor dynamics is not specifically considered for bearers but may require consideration in some design applications such as lightly loaded long span bearers. Tighter deflection limits may be required under these conditions. The above deflection limits are considered generally suitable for conventional solid section joist and bearer spans.

SECTION 5 DETERMINATION OF UPLIFT FORCES

5.1 SCOPE AND GENERAL

5.1.1 Scope of Section

This Section describes how net uplift pressures and net uplift forces for the determination of tie-down requirements are determined.

5.1.2 General

Net uplift forces are the difference between the ultimate uplift forces due to wind and the factored gravity loads due to permanent action and any permanent component of imposed action resisting uplift.

For the purposes of this Standard the uplift forces to be resisted for tie-down shall be determined as the product of the roof area supported and the net uplift pressures given for the level of the building where the tie-down is located.

For tie-down at bottom plate or subfloor level where overturning may contribute to uplift and, therefore, height and width of the structure are also relevant, the uplift pressures shall be equivalent values derived assuming the uplift load width is one half the building width. The resulting values are upper bound values applicable where the ratio of height (h) to width (w) does not exceed one (see Figure 5.2.2).

5.2 DETERMINATION OF NET UPLIFT PRESSURES

5.2.1 Roof uplift

Net uplift forces for tie-down connections between roof members or for the roof frame assembly connection to supporting walls or directly to floor frames or concrete slabs shall be as follows:

(a) *Roof battens* The net uplift pressures to be resisted by tie-down connectors at each rafter, p_u^* (kPa) are determined from—

$$p_u^* = q_u C_{pt} - 0.9G \qquad \ldots 5.2.1(1)$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients for roof battens given in Table 5.2.1(A)

G = roof permanent action taken as 0.1 kPa for sheet roofs or 0.6 kPa for tile roofs

TABLE 5.2.1(A)

NET PRESSURE COEFFICIENTS FOR ROOF BATTENS—UPLIFT

Wind classification	C_{pt}		
	General areas	Areas within 1.2 m of an edge	Areas within 1.2 m of an eaves corner
N1 to N4	1.0	1.8	−2.61
C1 to C3	1.44	2.25	−3.06

NOTE: The value of C_{pt} for areas within 1.2 m of an eaves corner applies only to roof slopes less than 10°. Where there is no internal pressure, such as in open verandah or eave construction, a value of C_{pt} = −2.7 shall be used for both wind classifications.

(b) *Roof frame to wall or directly to floor frame or slab* The net uplift pressure at each tie-down, p_u^* (kPa), is determined from—

$$p_u^* = q_u C_{pt} - 0.9G \qquad \text{. . . 5.2.1(2)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

C_{pt} = net pressure coefficients for roof uplift as given in Table 5.2.1(B)

G = roof permanent action taken as 0.4 kPa for sheet roofs, or 0.9 kPa for tile roofs

TABLE 5.2.1(B)

NET PRESSSURE COEFFICIENTS FOR ROOF UPLIFT

Wind classification	C_{pt}	
	Tile roof	Sheet roof
N1 and N2	−1.0	−1.0
N3 and N4	−1.0	−1.0
C1 to C3	−1.44	−1.44

5.2.2 Net uplift pressures at bottom plate or subfloor level

The net uplift pressure (p_u^*) at bottom plate or subfloor level is determined as the greater of the net uplift pressure due to direct uplift on the roof (p_{u1}^*) and the net uplift pressure resultant from the overturning effect of wind pressure on the wall and roof due to lateral wind (p_{u2}^*).

The net uplift pressures, p_{u1}^* and p_{u2}^* (kPa), shall be as follows:

(a) $$p_{u1}^* = q_u\left(K_a C_{pe} + C_{pi}\right) - 0.9\left(G + Q_p\right) \qquad \text{. . . 5.2.2(1)}$$

where

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

K_{a} = 0.8; roof area reduction factor given in AS/NZS 1170.2 for areas greater than 100 m^2, applied assuming that for uplift the house above bottom plate level acts as a ‘rigid box’

C_{pe} = 0.9; maximum value of external pressure coefficient for uplift

C_{pi} = value from Table 5.2.2(A) according to wind classification and location where the net uplift is being determined

G = permanent action resisting uplift from Table 5.2.2(B) according to the level where the net uplift is being determined

Q_{p} = permanent floor imposed action resisting uplift from Table 5.2.2(B) according to the level where the net uplift is being determined

TABLE 5.2.2(A)

INTERNAL PRESSURE COEFFICIENTS FOR DETERMINATION OF NET UPLIFT PRESSURE

Wind classification	Location	C_{pi}
N1 to N4	Bottom plate level	+0.2
	Subfloor level	0
C1 to C3	Bottom plate level	+0.7
	Subfloor level	0

NOTE: At bottom plate level, internal pressure on the roof contributes to uplift, whereas for subfloors (either single, upper, or lower storey) the internal pressure on floor and roof equalizes.

TABLE 5.2.2(B)

PERMANENT AND PERMANENT IMPOSED ACTION RESISTING UPLIFT

Location		Permanent action (G), kPa		Permanent imposed action (Q_p), kPa
Single or upper storey	Bottom plate level	Sheet roof	0.4 + 2/W	0
		Tile roof	0.9 + 2/W	
	Subfloor level	Sheet roof	0.8 + 2/W	0.5
		Tile roof	1.3 + 2/W	
Lower storey of two storeys	Bottom plate level	Sheet roof	0.8 + 4/W	0.5
		Tile roof	1.3 + 4/W	
	Subfloor level	Sheet roof	1.2 + 4/W	1.0
		Tile roof	1.7 + 4/W	

(b)

$$p^*_{u2} = \frac{K_c q_u}{W^2}\left[C_{ptw}h^2 - \left(C_{cel} - C_{pi}\right)\left(0.75h^2 - 2h_r h - h_r^2\right) - \left(C_{pe2} - C_{pi}\right)\left(0.25h^2 + 2h_r h + h_r^2\right)\right] - 0.9\left(G + Q_p\right) \qquad \ldots 5.2.2(2)$$

where

q_{u} = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

K_c = 0.8, the value for both $K_{c,e}$ and $K_{c,i}$ is in AS/NZS 1170.2 for four effective surfaces including both internal and external surfaces.

C_{ptw} = combined pressure coefficient for the windward and leeward walls from Table 5.2.2(C) according to roof pitch (α)

C_{pe1} = external pressure coefficient for the windward roof slope from Table 5.2.2(D) according to roof slope (α) and h/W ratio

C_{pe2} = external pressure coefficient for the leeward roof slope from Table 5.2.2(E) according to roof slope (α) and h/W ratio

C_{pi} = internal pressure coefficient from Table 5.2.2(A) according to wind classification and location where net uplift pressure is being determined

h = height from lowest floor to single or upper storey ceiling level for single or two storey, respectively

h_r = $(W/2)\tan\alpha$, where W is width across the outer walls and α is roof pitch (see Figure 5.2.2)

G = permanent action resisting uplift from Table 5.2.2(B) according to level where net uplift is being determined

Q_p = permanent floor imposed action resisting uplift from Table 5.2.2(B) according to level where net uplift is being determined

W = overall width across external walls (see Figure 5.2.2)

TABLE 5.2.2(C)

COMBINED PRESSURE COEFFICIENTS FOR WINDWARD AND LEEWARD WALLS ($\theta = 0°$)

Roof pitch (α)	$\alpha < 10°$	$10° \leq \alpha \leq 15°$	$\alpha = 20°$	$\alpha \geq 25°$
Pressure coefficient (C_{ptw})	1.1	1.1	1.1	1.2

TABLE 5.2.2(D)

EXTERNAL PRESSURE COEFFICIENTS FOR WINDWARD ROOF AREAS

h/W ratio	**Pressure coefficient (C_{pe1})**						
	Roof pitch (α)						
	<10°	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**
≤0.25	−0.9 or −0.4	−0.7 or −0.3	−0.5 or 0	−0.3 or +0.2	−0.2 or +0.3	−0.2 or +0.4	+0.5 or 0
0.5	−0.9 or −0.4	−0.9 or −0.4	−0.7 or −0.3	−0.4 or 0	−0.3 or +0.2	−0.2 or +0.3	−0.2 or +0.4
≥1.0	−1.3 or −0.3	−1.3 or −0.6	−0.1 or −0.5	−0.7 or −0.3	−0.5 or 0	−0.3 or +0.2	−0.2 or +0.3

NOTES:

1 Where two values are given, both values are considered.

2 Interpolation between the respective coefficients for the given roof pitches and h/W ratios may be used to obtain more accurate coefficients for intermediate values of roof pitch and/or h/W ratio.

TABLE 5.2.2(E)

EXTERNAL PRESSURE COEFFICIENTS FOR LEEWARD ROOF AREAS

h/*W* ratio	Pressure coefficient (C_{pe2})			
	Roof pitch (α)			
	<10°	10°	15°	≥20°
≤0.25	–0.3	–0.3	–0.5	–0.6
0.5	–0.5	–0.5	–0.5	–0.6
≥1.0	–0.7	–0.7	–0.6	–0.6

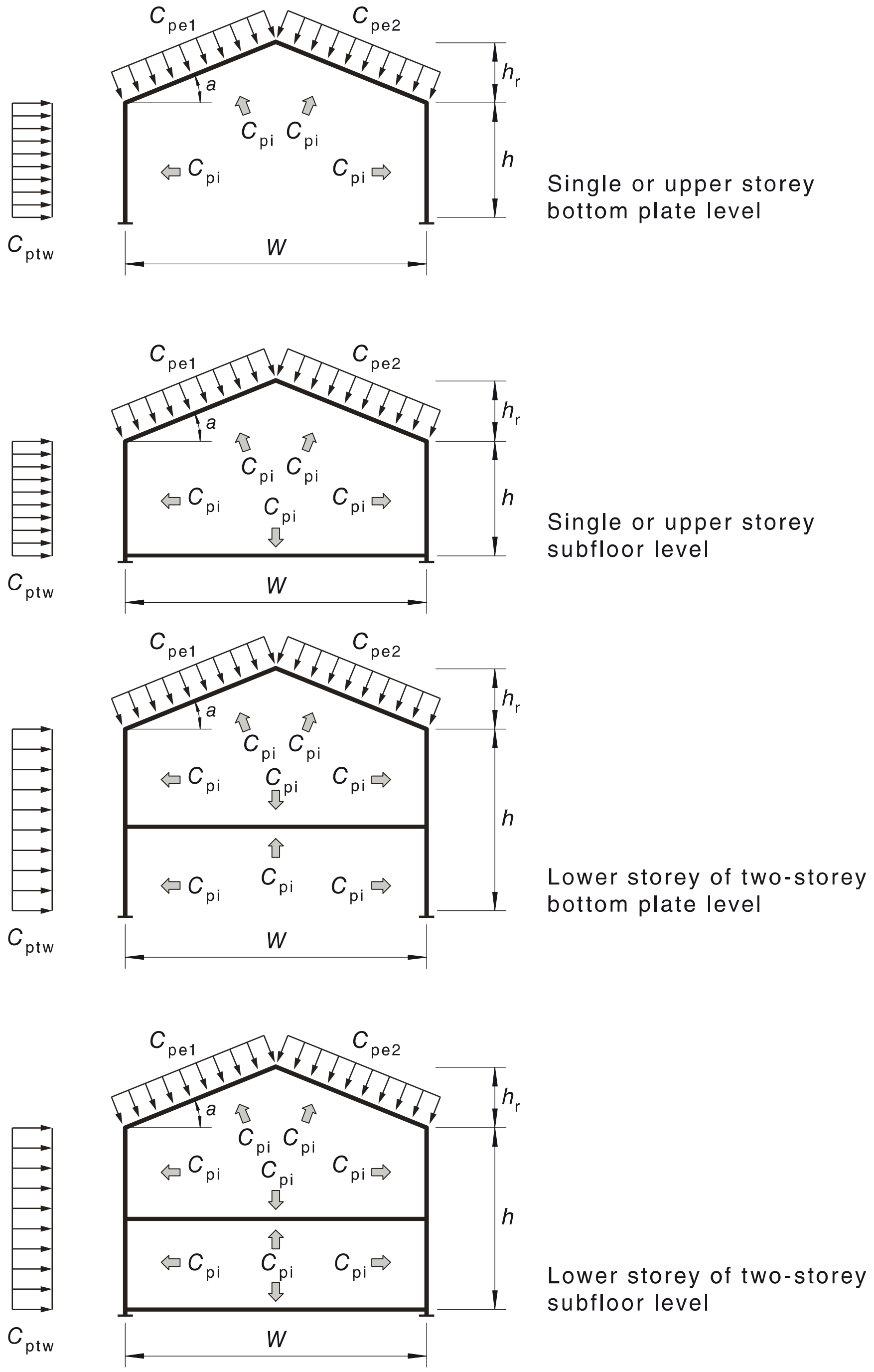


FIGURE 5.2.2 NOTATION

SECTION 6 PRESSURES FOR DETERMINATION OF RACKING FORCES

6.1 SCOPE AND GENERAL

6.1.1 Scope of Section

This Section describes how equivalent pressures for use with projected areas, for the calculation of racking loads to be resisted by bracing, are derived. The methods of determination of equivalent pressures for the calculation of racking forces in orthogonal directions for single or upper storey, for lower of two storeys and for subfloor level are given.

NOTE: The information in this Section is extracted, with minor editorial modification, from AS 4055 (Appendix A5). AS 4055 also provides a method of calculating racking forces based on Area of elevation and lateral wind pressure, with wind pressures for various wind classes tabulated in Section 5 of that Standard.

6.1.2 Notation

Notation symbols for this Section are as follows:

H_u = height, floor to ceiling for single or upper storey, in metres

H_L = height, floor to ceiling for lower storey of two storeys, in metres

H_F = depth of upper floor, in metres

W = width of building, in metres (see Figure 6.1.3)

L = length of building, in metres (see Figure 6.1.3)

α = roof pitch, in degrees (see AS/NZS 1170.2 and Figure 6.1.3)

θ = wind direction, in degrees (see AS/NZS 1170.2)

h = height to eaves, in metres (see AS/NZS 1170.2)

d = plan dimension of building or part of building parallel to the wind direction, in metres (see AS/NZS 1170.2)

b = plan dimension of building or part of building perpendicular to wind direction, in metres (see AS/NZS 1170.2)

K_c = pressure combination factor

$C_{pt,roof}$ = combined pressure coefficient for the windward and leeward roof areas

$C_{pt,wall}$ = combined pressure coefficient for the windward and leeward walls

q_u = free stream dynamic gust pressure, in kPa, for the ultimate limit state; values of q_u are given in Appendix A, Table A2, for each wind classification

6.1.3 Assumptions

The following assumptions have been made in the derivation of equivalent pressures for use with projected areas for the determination of racking forces:

(a) The geometry assumed is a simple outline of the building, which ignores eave overhangs, fascias and gutters. The projected area for the roof is taken as the area above ceiling level for the single or upper storey (see Figure 6.1.3).

(b) Buildings are assumed enclosed underneath the lower floor.

(c) The floor depth of upper floors (H_F) is assumed to be 0.3 m.

(d) $H_u = H_L$ = 2.4 m. Pressures calculated for 2.4 m floor to ceiling heights are assumed to apply for walls up to 3.0 m high.

(e) A pressure combination factor K_c = 0.8 is applied where the load effect is the result of the combination of pressures on two or more surfaces. [K_c is not applied in combination with the area reduction factor (K_a).]

(f) The assumed combined pressure coefficients for the windward and leeward walls ($C_{pt,wall}$) for wind directions $\theta = 0°$ and $\theta = 90°$ are given in Table 6.1.3(A) and Table 6.1.3(B), respectively.

(g) The assumed combined pressure coefficients for the windward and leeward roofs ($C_{pt,roof}$) for wind parallel to the slope (pitch) of roof are given in Table 6.1.3(C).

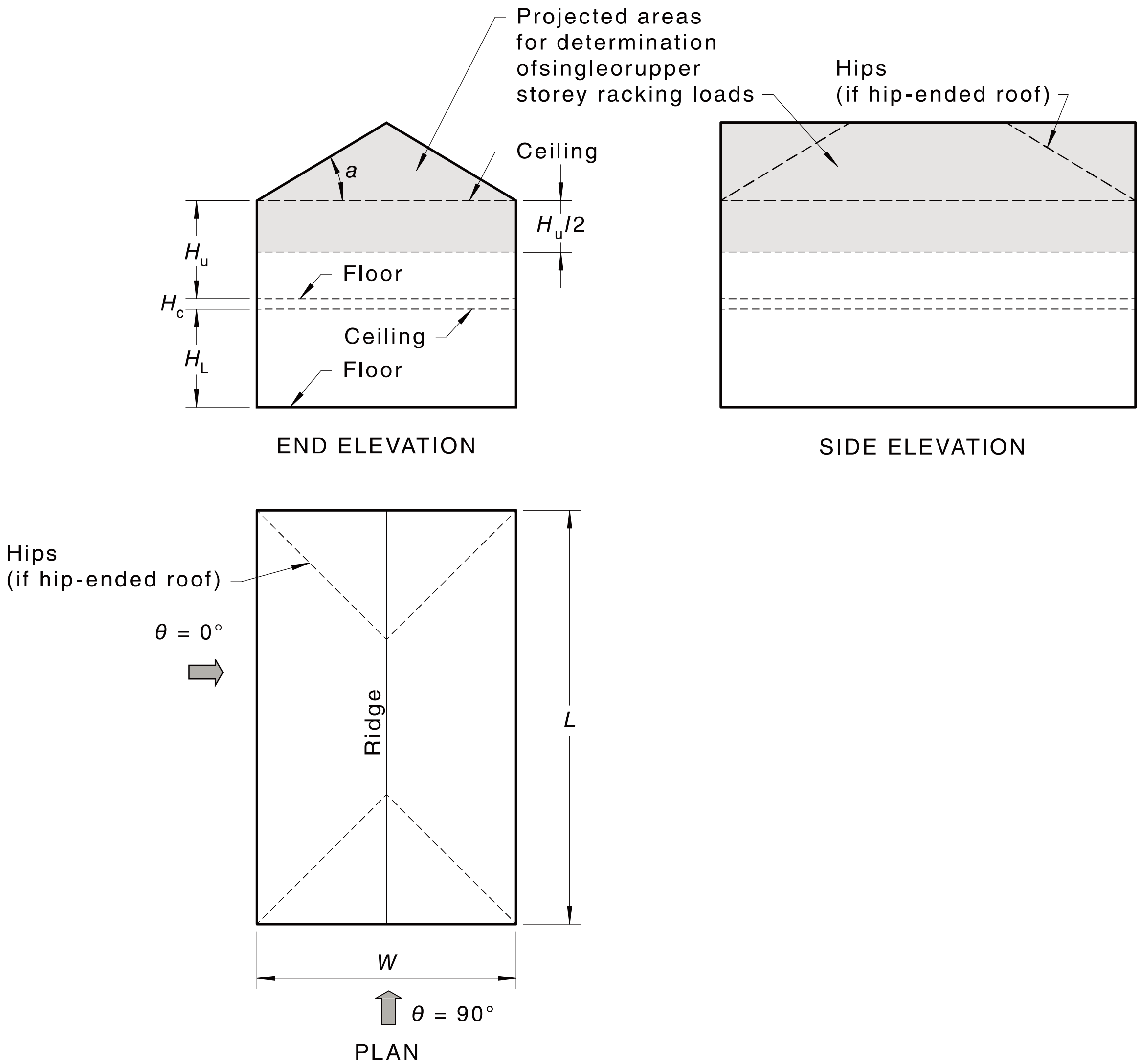


FIGURE 6.1.3 NOTATION

TABLE 6.1.3(A)

**COMBINED PRESSURE COEFFICIENTS FOR WALLS—
WIND DIRECTION PARALLEL TO ROOF SLOPE***

Roof pitch (α)	$\alpha < 10°$	$10° \leq \alpha \leq 15°$	$\alpha = 20°$	$\alpha \geq 25°$
$C_{pt,wall}$	1.1	1.1	1.1	1.2

* Applies to general roof slopes ($\theta = 0°$) and hip ends ($\theta = 90°$).

TABLE 6.1.3(B)

**COMBINED PRESSURE COEFFICIENTS FOR WALLS—
WIND DIRECTION PERPENDICULAR TO ROOF SLOPE***

d/b	≤1	2	≥4
$C_{pt,wall}$	1.2	1.0	0.9

* Applies to gable or skillion roof ends ($\theta = 90°$).

TABLE 6.1.3(C)

**COMBINED PRESSURE COEFFICIENTS FOR ROOFS—
WIND DIRECTION PARALLEL TO ROOF SLOPE***

Ratio h/d	$C_{pt,roof}$						
	Roof pitch (α)						
	<10°	10°	15°	20°	25°	30°	35°
≤0.25	0	0	+0.5	+0.8	+0.9	+0.9	+1.0
0.5	0	+0.1	+0.2	+0.6	+0.8	+0.8	+0.9
≥1.0	0	+0.1	+0.1	+0.3	+0.6	+0.8	+0.8

* Applies to general roof slopes ($\theta = 0°$) and hip ends ($\theta = 90°$).

6.2 EQUIVALENT PRESSURES ON PROJECTED AREAS

6.2.1 For flat wall surfaces, gable or skillion roof ends

The equivalent pressure (p) on the projected area shown in Figure 6.2.1 for calculation of the racking load for bracing in single or upper storey, or the lower of two-storey or subfloor walls shall be determined from—

$$p = q_u C_{pt,wall} K_c \quad \text{. . . 6.2.1}$$

where

$C_{pt,wall}$ = 1.2, as given in Table 6.1.3(B) for $d/b = 1$

K_c = 0.8; pressure combination factor applicable for the combined effect of pressure on two or more surfaces

NOTE: The assumption that $d = b$, i.e. $L = W$ corresponds to the maximum combined pressure coefficient for the walls.

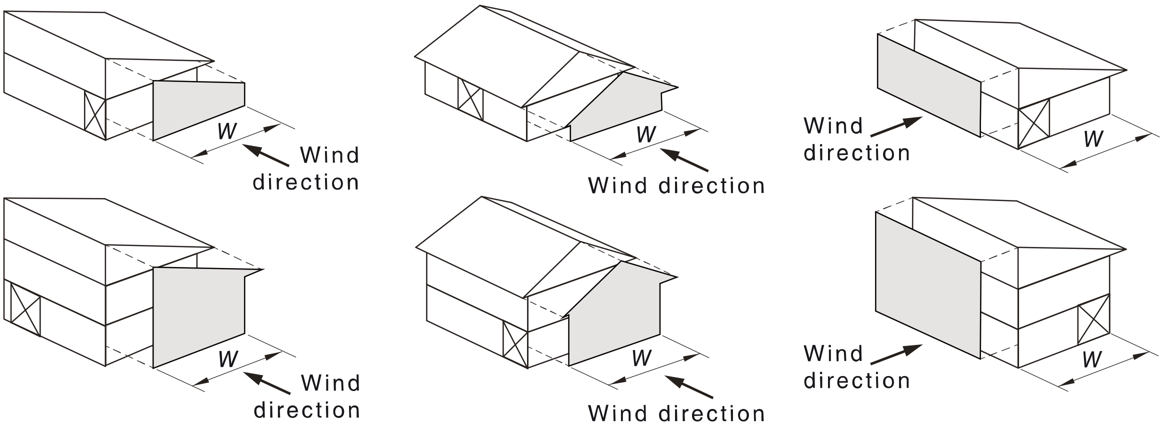


FIGURE 6.2.1 FLAT WALL SURFACES—GABLE AND SKILLION ROOF ENDS

6.2.2 For side elevations, single or upper storey, gable or hip-ended roofs

The equivalent pressure (p) for the projected areas shown in Figure 6.2.2 for calculation of the racking load for bracing in single or upper storey walls shall be determined from—

$$p = \frac{q_u K_c \left[C_{pt,wall}(H_u/2) + C_{pt,roof}(W/2)\tan\alpha\right]}{(H_u/2) + (W/2)\tan\alpha} \qquad \text{. . . 6.2.2}$$

where

$C_{pt,wall}$ = value from Table 6.1.3(A) for roof pitch, α

$C_{pt,roof}$ = value from Table 6.1.3(C), for roof pitch α, and assuming $(h/d) = (H_u/W)$

K_c = 0.8; pressure combination factor

NOTES:

1 The assumption that $h/d = H_u/W$ maximizes the assumed combined pressure coefficients for the roof.

2 The reduction in projected area for hip-ended roofs has been ignored in the determination of the equivalent pressures to be applied to the projected areas corresponding to either hip or gable ended roofs.

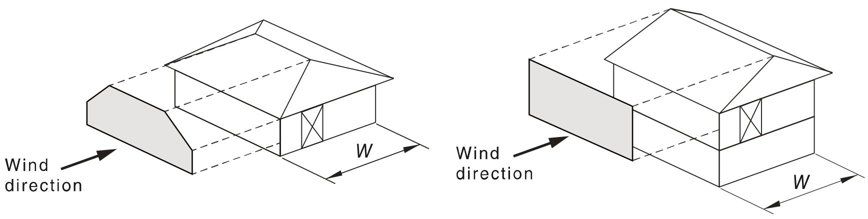


FIGURE 6.2.2 SIDE ELEVATIONS—SINGLE OR UPPER STOREY—GABLE OR HIP-ENDED ROOFS

6.2.3 Side elevation, lower storey of two storeys or subfloor, gable or hip-ended roof

The pressure (p) on the projected area shown in Figure 6.2.3 for calculation of the racking force for bracing in the lower storey of two-storey walls shall be determined from—

$$p = \frac{q_u K_c \left[C_{pt,wall}(H_u + H_F + H_L/2) + C_{pt,roof}(W/2)\tan\alpha\right]}{(H_u + H_F + H_L/2) + (W/2)\tan\alpha} \qquad \text{. . . 6.2.3}$$

where

$C_{pt,wall}$ = value determined from Table 6.3.1(A) for roof pitch (α)

$C_{pt,roof}$ = value determined from Table 6.3.1(C) for roof pitch (α) and assuming $(h/d) = (H_u + H_F + H_L)/W$

K_c = 0.8; pressure combination factor

NOTES:

1 The assumption that $h/d = (H_u + H_F + H_L)/W$ maximizes the assumed combined pressure coefficients for the roof.

2 The reduction in projected area for hip-ended roofs has been ignored in the determination of equivalent pressures to be applied for projected areas for either hip- or gable-ended roofs.

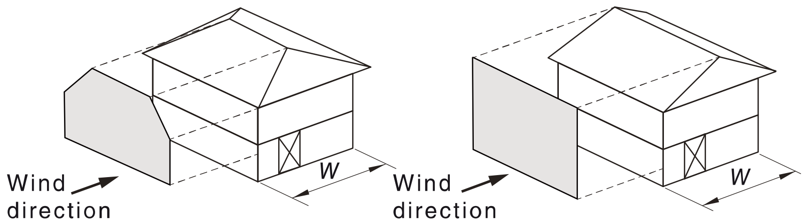


FIGURE 6.2.3 SIDE ELEVATION—LOWER STOREY OF TWO STOREYS OR SUBFLOOR—GABLE OR HIP-ENDED ROOF

6.2.4 End elevation, single or upper storey, hip-ended roof

The pressure (p) on the projected area shown in Figure 6.2.4 for calculation of racking loads for bracing in single or upper storey walls shall be determined from—

$$p = \frac{q_u K_c \left[C_{pt,wall}(H_u/2) + C_{pt,roof}(W/4)\tan\alpha\right]}{(H_u/2) + (W/4)\tan\alpha} \qquad \text{. . . 6.2.4}$$

where

$C_{pt,wall}$ = 1.2

$C_{pt,roof}$ = value obtained from Table 6.1.3(C) for roof pitch (α) with $h/d = H_u/L$ and assuming $L = W$

K_c = 0.8; pressure combination factor

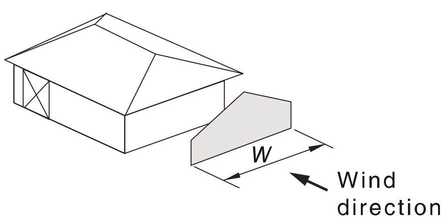


FIGURE 6.2.4 END ELEVATION—SINGLE OR UPPER STOREY—HIP-ENDED ROOF

6.2.5 End elevation, lower storey of two storeys, hip-ended roof

The equivalent pressure (p) on the projected area shown in Figure 6.2.5 for calculating racking loads for bracing in walls of the lower storey of two-storey walls shall be determined from—

$$p = \frac{q_u K_c \left[C_{pt,wall}(H_u/2) + C_{pt,roof}(W/4)\tan\alpha\right]}{(H_u/2) + (W/4)\tan\alpha} \quad \ldots 6.2.5$$

where

$C_{pt,wall}$ = 1.2

$C_{pt,roof}$ = value obtained from Table 6.1.3(C) for roof pitch (α) and assuming $h/d = H_u + H_F + H_L)/L$ and $L = 1.5W$

K_c = 0.8; pressure combination factor

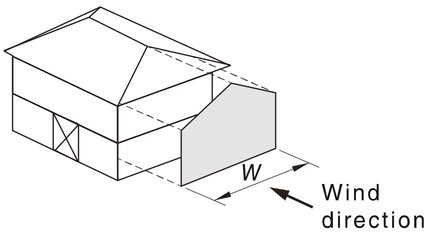


FIGURE 6.2.5 END ELEVATION—LOWER STOREY OF TWO STOREYS—HIP-ENDED ROOF

APPENDIX A

WIND CLASSIFICATIONS AND DESIGN WIND PRESSURES

(Normative)

A1 WIND CLASSIFICATIONS

The member design criteria, racking pressures and uplift forces given in this Standard have been determined using wind classifications—

(a) N1 to N4 applicable for non-cyclonic regions A and B; and

(b) C1, C2, and C3 for cyclonic regions.

The wind classifications correspond to bands of design gust wind speed for the ultimate and serviceability limit states. Wind classifications corresponding to the maximum design gust wind speeds for the ultimate and serviceability limit states are given in Table A1.

TABLE A1

WIND CLASSIFICATIONS

Maximum design gust wind speed (m/s)		Wind classification	
Ultimate limit state	**Serviceability limit state**	**Non-cyclonic**	**Cyclonic**
34	26	N1	—
40	26	N2	—
50	32	N3	C1
61	39	N4	C2
74	47	—	C3

NOTES:

1 The above wind classifications have been adopted from AS 4055.

2 The above classifications may be adopted for design wind speeds up to 5% greater than the maximum values given for each classification.

A2 DESIGN WIND PRESSURES

Design wind pressures shall be determined using the free stream dynamic gust pressures for the ultimate and serviceability limit states as given in Table A2. The gust pressures in Table A2 have been calculated from AS 4055 using the maximum design gust wind speeds given in Table A1.

TABLE A2

FREE STREAM DYNAMIC GUST PRESSURES

Wind classification		Free stream dynamic gust pressure (kPa)	
Regions A and B	**Regions C and D**	**Ultimate limit state (q_u)**	**Serviceability limit state (q_s)**
N1	—	0.69	0.41
N2	—	0.96	0.41
N3	C1	1.50	0.61
N4	C2	2.23	0.91
—	C3	3.29	1.33

APPENDIX B

DESIGN OF OVERHANGS FOR BIRDSMOUTH NOTCHED RAFTERS

(Normative)

B1 GENERAL

Rafters are often birdsmouth-notched at their lower support point so as to provide bearing to a wall and to permit an overhang.

The following design method, which differs from that given for notches in AS 1720.1, applies for birdsmouth notches to a maximum depth of one third of the rafter depth.

The design method allows for the load sharing effect obtained when the overhanging ends of parallel rafters are attached to a fascia and the connection and fascia are capable of transferring load to adjacent rafters.

B2 EFFECT OF BIRDSMOUTH NOTCH ON RIGIDITY

In determining the deflection of the overhanging portion of a birdsmouth notched rafter, the rigidity of the rafter (for both the overhang span and the backspan) shall be taken as $g_{47}E_rI_r$, where g_{47} is a birdsmouth geometry factor that accounts for reduced rigidity due to the birdsmouth notch and (E_rI_r) is the rigidity of the unnotched rafter.

The birdsmouth geometry factor is bounded by the range $0.25 \leq g_{47} \leq 1.0$ and in this range is given by—

$$g_{47} = 1 - (5.7d_{notch}/L_o) \quad \ldots \text{B2}$$

where

d_{notch} = depth of the birdsmouth notch, in mm (see Figure B2)

L_o = horizontal span of the overhang, in mm

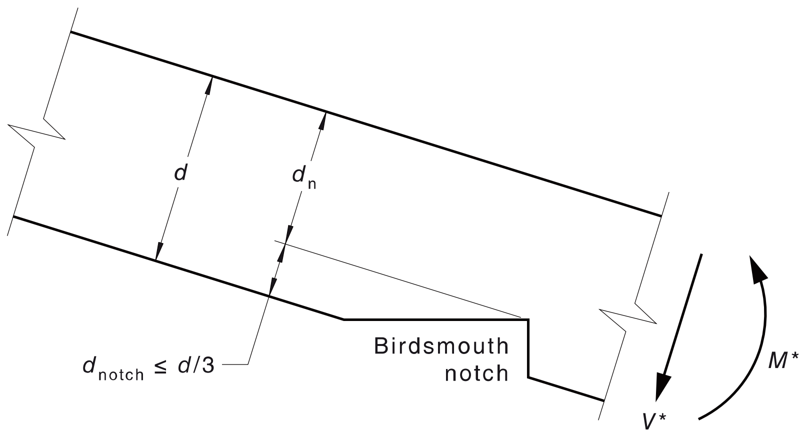


FIGURE B2 NOTATION AND SIGN CONVENTION

B3 LOAD SHARING FOR PARALLEL RAFTER OVERHANGS

In the determination of the strength and serviceability limit states, concentrated and partial area actions (P^* and w^*) applied to the overhanging portion of parallel rafters shall be assumed laterally distributed to adjacent rafters such that the effective concentrated action (P^*_{eff}) or effective distributed action (w^*_{eff}) used for the design of an individual rafter is obtained as follows:

$$P^*_{\text{eff}} = g_{45} P^* \qquad \text{. . . B3(1)}$$

and

$$w^*_{\text{eff}} = g_{45} w^* \qquad \text{. . . B3(2)}$$

where

P^* = design concentrated action

w^* = design partial area action

g_{45} = the load distribution factor, which is bounded by the range $0.3 \leq g_{45} \leq 1.0$ and in this range is given by the following equation:

$$g_{45} = 0.2 \log_{10}\left(\frac{h_r}{h_f}\right) + 0.69 \qquad \text{. . . B3(3)}$$

where

$$h_r = g_{47} \frac{E_r I_r}{L_o^3} \qquad \text{. . . B3(4)}$$

$$h_f = \frac{E_f I_f}{S^3} \qquad \text{. . . B3(5)}$$

$g_{47} E_r I_r$ = flexural rigidity of the rafter overhang, calculated in accordance with Paragraph B2

$E_f I_f$ = flexural rigidity of the fascia

L_o, S = horizontal span of overhang and spacing of rafters, respectively

B4 RAFTER STRENGTH AT BIRDSMOUTH NOTCH

B4.1 Bending strength

The design capacity in bending (ϕM) at the birdsmouth notch, for the strength limit state, shall satisfy the following equation:

$$(\phi M) \geq M^* \qquad \text{. . . B4.1(1)}$$

where

$$(\phi M) = \phi k_1 k_4 k_6 k_9 [f'_b] Z_n \qquad \text{. . . B4.1(2)}$$

and

M^* = design action effect in bending for negative moment as defined in Figure B2

ϕ = capacity factor given in AS 1720.1

k_1, k_4, k_6 = modification factors given in AS 1720.1

k_9 = strength sharing modification factor for parallel overhanging rafters rigidly connected to a fascia

= $1.24 - 0.24(S/L_o)$, but is not less than 1.0

f'_b = characteristic strength in bending

Z_n = net section modulus at notch

= $\left(bd_n^2/6\right)$, where b equals the breadth and d_n equals the depth of rafter above the birdsmouth notch (see Figure B2) ($d_n \geq 2d/3$)

B4.2 Shear strength at birdsmouth notch

The design capacity in shear at the birdsmouth notch for the strength limit state shall satisfy the following equation:

$$\phi V \geq V^* \qquad \text{. . . B4.2(1)}$$

where

$$\phi V = \phi k_1 k_4 k_6 \left[f'_s\right] A_s \qquad \text{. . . B4.2(2)}$$

and

V^* = design action effect in positive shear (see Figure B2)

ϕ = capacity factor, given in AS 1720.1

k_1, k_4, k_6 = modification factors given in AS 1720.1

f'_s = characteristic strength in shear

A_s = $\dfrac{2}{3} bd_n$

B4.3 Combined bending and shear (fracture strength) at the birdsmouth notch

For a rafter of depth (d), birdsmouth-notched to a maximum depth of one third of its depth, as shown in Figure B2, the maximum bending moment action effect (M^*) and nominal maximum shear force action effect (V^*), calculated for the net section, shall comply with the following interaction equation:

$$\frac{6M^*}{bd_n^2} + \frac{6V^*}{bd_n} \leq \phi g_{50} k_1 k_4 k_6 f'_{sj} \qquad \text{. . . B4.3}$$

where

b = breadth of the rafter

d_n = net depth of rafter above the notch

ϕ = capacity factor, given in AS 1720.1

k_1, k_4, k_6 = modification factors given in AS 1720.1

f'_{sj} = characteristic shear strength at joint details

g_{50} = coefficient for birdsmouth notch

= $18/\left(d^{0.333}\right)$

If, according to the sign convention shown in Figure B2, M^* is negative, it may be taken as zero in the application of Equation B4.3. Similarly, if V^* is positive, it may also be taken as zero in the application of Equation B4.3.

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