AS 1720.1—2010
(Incorporating Amendment Nos 1, 2 and 3)

AS 1720.1—2010

Australian STANDARD

# Timber structures

# Part 1: Design methods

STANDARDS Australia

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- A3P
- Association of Consulting Engineers Australia
- Australian Building Codes Board
- Australian Timber Importers' Federation
- Australian Wood Panels Association
- BRANZ
- CSIRO Manufacturing and Materials Technology
- Curtin University of Technology
- Department of Primary Industries and Fisheries Queensland
- Engineers Australia
- Frame and Truss Manufacturers Association Australia
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- Master Builders Australia
- Monash University
- New Zealand Timber Industry Federation
- Scion
- Standards New Zealand
- Timber Queensland
- University of Auckland
- University of Technology, Sydney
- Wood Processors Association

Additional Interests:

- Mr Bruce Hutchings

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AS 1720.1—2010
(Incorporating Amendment Nos 1, 2 and 3)



Australian Standard®

# Timber structures

# Part 1: Design methods

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# PREFACE

This Standard was prepared by the Joint Standards Australia/Standards New Zealand Committee TM-001, Timber Structures, to supersede AS 1720.1—1997.

*This Standard incorporates Amendment No. 1 (December 2010), Amendment No. 2 (August 2011) and Amendment No. 3 (August 2015). The changes required by the Amendments are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.*

The decision to prepare this Standard as an Australian Standard was by consensus agreement of the Joint Committee.

The objective of this Standard is to provide a code of practice for the design and acceptance of timber structures and elements. It includes design methods and design data appropriate for commonly encountered structural elements and materials and requirements to be met for specification of the design, installation and maintenance of timber structures.

Capacity factors for the timber materials represented in this Standard have been reviewed and, in some cases, modified to better reflect the safety levels appropriate for the wide range of applications for which timber structural elements may be used.

For housing, the increasing sizes of houses and increasingly larger areas that are in some cases supported by a single structural element has resulted in a need to limit application of category 1 capacity factors according to the area likely to be affected by failure of the individual element. For structures other than houses the definition of 'primary structural element' has been changed to recognise that even a partial structural collapse of some structures can have severe consequences.

Conceptually, the limit state design principles of this Standard do not differ from the 1997 version. Only essential changes and editorial improvements have been made, which reflect experience with the application of the Standard over the past decade; these changes relate to layout improvements and clarification of meaning.

Differences from the 1997 edition include the following:

(a) The notation and terminology for actions have been aligned with AS/NZS 1170 series.

(b) For easier referencing, the design properties for commonly available structural sawn timber (F-grades, MGP-grades and A17-grade) are now consolidated and presented together in an appendix.

(c) The presentation of requirements for selection of capacity factors for member and joint design has been simplified and clarified.

(d) For consistency with the AS/NZS 4063 series, characteristic properties are now uniformly defined as including the effect of size.

(e) Issues associated with evaluation methods, verification procedures, monitoring and quality control in production and manufacture are not relevant to design and are not therefore directly referred to in this revised Standard.

The terms 'normative' and 'informative' have been used in this Standard to define the application of the appendix to which they apply. A 'normative' appendix is an integral part of a Standard, whereas an 'informative' appendix is only for information and guidance.

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STANDARDS AUSTRALIA

**Australian Standard**

**Timber structures**

Part 1: Design methods

# SECTION 1 SCOPE AND GENERAL

## 1.1 SCOPE AND APPLICATION

### 1.1.1 Scope

This Standard sets out limit state design methods for the structural use of timber, which are based on the principles of structural mechanics and on data established by research. It provides design data for sawn timber, laminated timber, timber in pole form, plywood, laminated veneer lumber and various types of fastenings. In addition, it provides methods of test for components or assemblies of unconventional design which may not be readily amenable to detailed analysis.

For ease of use, the simpler design situations are set in the main body of the text. Related appendices, which form an integral part of the Standard, give acceptable procedures for detailed design situations.

### 1.1.2 Application

This Standard is intended for use in the design or appraisal of structural elements or systems comprised of timber or wood products and of structures comprised substantially of timber.

## 1.2 NORMATIVE REFERENCES

The normative documents referenced in this Standard are listed in Appendix A.

NOTE: Documents referenced for informative purposes are listed in the Bibliography.

## 1.3 TIMBER

### 1.3.1 General

All timber used in accordance with this Standard shall comply with the requirements of the appropriate Australian Standards, as follows:

(a) *Visually graded sawn timber* Visually graded sawn timber shall conform to the requirements of AS 2082 or AS 2858.

(b) *Mechanically graded timber* Mechanically graded timber shall conform to the requirements of AS/NZS 1748.

(c) *Proof-graded timber* Proof-graded timber shall conform to the requirements of AS 3519.

(d) *Structural plywood* Structural plywood shall conform to the requirements of AS/NZS 2269.0.

(e) *Structural laminated veneer lumber* Structural laminated veneer lumber shall conform to the requirements of AS/NZS 4357.0.

(f) *Glued laminated timber* Glued laminated timber shall conform to the requirements of AS/NZS 1328.1

(g) *Round timber* Round timber shall conform to the requirements of AS 3818.3 or AS 3818.11, as appropriate.

(h) *Other timber products or grades* Timber products or grades not listed in this Standard may have properties established by testing and evaluation methods consistent with those described in the AS/NZS 4063 series. In addition, for other timber-based products, modification factors for duration of load effect for strength and stiffness shall be determined based on authoritative research. Consideration shall also be given to the determination of the structural durability of adhesives used for manufacture.

### 1.3.2 Identification

Structural timber used in conjunction with this Standard shall have its stress grade identified in accordance with the relevant Australian timber product Standard given in Clause 1.3.1.

For various purposes, it may also be necessary to specify a particular species. When a particular species is specified, the specification shall require that all pieces of timber be suitably identified as to species.

NOTES:

1 The design properties recommended in this Standard have been chosen on the assumption that structures of unseasoned timber that are allowed to dry will not receive their full design load until a period of air-drying of at least 2 weeks has taken place. Freshly sawn timber that is unseasoned, or has recently been treated with waterborne chemicals, tends to have a reduced resistance and stiffness to sustained loads during the initial drying period. Under normal circumstances, unseasoned timber will have had this drying period before being subjected to its full design load.

2 Usually, only a limited number of the timber species and stress grades listed in this Standard will be readily available at any particular place and time.

### 1.3.3 Change of grade

The strength properties of graded timber or timber products may alter as a consequence of subsequent processes such as longitudinal resawing, chemical treatments and redrying processes.

NOTE: It may be necessary to reassess strength properties to ensure that graded timber or timber products still satisfy design requirements after having been subjected to such processes.

### 1.3.4 Special provisions

Design loads for timber joints and design rules for notched beams given herein are based on the assumption that there are no loose knots, severe sloping grain, gum veins, gum or rot pockets, lyctid-susceptible sapwood, holes or splits in the vicinity of any fasteners or notch roots.

### 1.3.5 Treated timber

Timber, treated by impregnation with waterborne chemicals such as preservatives, is classified as unseasoned timber unless seasoning is specified.

NOTE: Where the material is dried after treatment, re-grading may be required.

## 1.4 GENERAL DESIGN CONSIDERATIONS

### 1.4.1 Loads

#### 1.4.1.1 *General*

A structure and its structural elements shall be designed to resist the design action effects produced by—

(a) the combinations of actions for the strength, stability and serviceability limit states as specified in AS/NZS 1170.0; or

(b) such other design actions deemed to be acceptable for the limit state as appropriate to the intended end-use service conditions of a structure or its structural elements.

**1.4.1.2** *Duration of load*

Duration of load effects need to be considered in the determination of the critical combination of actions.

NOTE: For definition of duration of loading, see Clause 1.7.2.12, and for further information, see Clause 2.4.1.

### 1.4.2 Design methods

**1.4.2.1** *General*

A structure and its structural elements shall be designed to resist the design action effects resulting from the appropriate combinations of actions in order to satisfy the requirements for strength, stability and serviceability limit states.

**1.4.2.2** *Strength limit state*

This limit state condition shall be deemed to be satisfied when the structural elements of a structure are proportioned so that their design capacities ($R_d$) equal or exceed the design action effects ($S^*$). For a given failure mode the strength limit state takes the general form—

$$R_d \geq S^*$$

where

$R_d$ = design capacity of a structural element determined in accordance with the relevant Sections of this Standard.

NOTE: The general requirements are given in Section 2.

$S^*$ = design action effect, such as bending moment, axial force or shear force resulting from the combinations of actions for the strength limit states

NOTE: The design capacity ($R_d$) of a structural element is also referred to as the factored resistance of the structural element for a given failure mode.

**1.4.2.3** *Stability limit state*

The structure as a whole (or any part of it) shall be designed to prevent instability due to overturning, uplift or sliding as follows:

(a) The design action effect and the design resistance effect shall be determined in accordance with AS/NZS 1170.0.

(b) The whole or part of the structure shall be proportioned so that the design resistance effect is not less than the design action effect.

**1.4.2.4** *Serviceability limit state*

The structure and its structural elements shall be designed to give satisfactory performance under the specified service conditions by controlling or limiting deflections, vibration and displacement of fasteners as follows:

(a) Unless otherwise agreed, the combinations of actions for serviceability limit states shall be in accordance with AS/NZS 1170.0.

(b) Deflections due to combinations of actions appropriate for the serviceability limit state shall be determined by elastic analysis methods.

NOTES:

1 The deflection limits for the serviceability limit state should be appropriate to the structure and its intended use, the nature of loading, the relationship between adjacent members and the effect on other elements supported.

2 Guidance on selection of deflection limits is given in Appendix B.

3 The determination of acceptable deflection limits is part of the design process performed by the design engineer.

4 The characteristic modulus of elasticity values given in this Standard are mean values only and, accordingly, deflection of individual members subjected to the design loads may vary both above and below the calculated estimate. Where the actual deflection is critical, it is suggested that lower fifth-percentile estimates of modulus of elasticity should be obtained and used. A procedure for estimating lower fifth-percentile values of modulus of elasticity is given in Appendix B.

(c) Where required, structures shall be designed to ensure that the vibration induced by machinery, or vehicular or pedestrian traffic does not adversely affect the serviceability of the structure.

NOTES:

1 Where there is likelihood of a structure being subjected to vibration effects, measures should be taken to minimize any dynamic behaviour so as to prevent possible discomfort or alarm, damage to the structure, or interference with its proper function.

2 The AS 2670 series gives guidance on the evaluation of human exposure to whole-body vibrations of the type likely to be transmitted by structures.

(d) Where required, the displacement associated with various types of fasteners shall be assessed in accordance with the methods given in Appendix C.

**1.4.2.5** *Experimentally based design*

Where a structure or a structural element is demonstrated by the full-scale tests specified in Appendix D to satisfy requirements for strength, deformation, stability and serviceability, the corresponding design requirements of this Standard shall also be deemed to have been satisfied.

NOTE: Other design considerations will still be required to be met.

### 1.4.3 Timber dimensions for engineering calculations

All engineering calculations shall be based on the minimum cross-section after taking account of tolerances. Such calculations shall not be based on the nominal cross-section.

NOTE: Nominal cross-section is sometimes referred to as 'called' cross-section.

### 1.4.4 Other design considerations

**1.4.4.1** *Buckling restraints*

Where there is some doubt as to the effectiveness of buckling restraints, appropriate computations, as given in Appendix E, shall be made to check the stiffness and strength of the restraints.

**1.4.4.2** *Erection and other extraneous forces*

Adequate provision shall be made to resist the lateral and other forces that can occur during the transport of structural elements, and during the erection of a structure.

**1.4.4.3** *Secondary stresses*

Careful consideration shall be given to the influence of secondary stresses. Where secondary stresses cannot be reduced to negligible proportions, suitable provisions in the design or some reduction in permissible primary stresses shall be made.

**1.4.4.4** *Shrinkage*

When using unseasoned timber, consideration shall be given to the effects of shrinkage. Detailing of the joints shall not restrain shrinkage where splitting could render the joint ineffective. Consideration shall also be given to detailing to avoid damage or unsightly appearances.

NOTE: For most timbers the magnitude of shrinkage is in the range of 0.1% to 0.3% in the direction of the wood grain and 2% to 10% transverse to this direction. Information on shrinkage for specific species can be obtained from the following:

(a) KINGSTON, R.S.T. and RISDON C.J.E. *Shrinkage and Density of Australian and Other South-west Pacific Woods*. Division of Forest Products Technological Paper No. 13, CSIRO, 1961.

(b) BUDGEN, B. *Shrinkage and Density of some Australian and South-east Asian Timbers*. Division of Building Research Technological Paper (Second Series) No. 38, CSIRO, 1981.

**1.4.4.5** *Durability*

A1 The structure and its structural elements (including timber, metal, adhesives and other structural material) shall be designed to satisfy the strength, stability and serviceability requirements for the design life of the structure. Any assumed maintenance program shall be specified. Due consideration shall be given to environmental conditions, such as the thermal, physical, chemical, mechanical and biological agents that may act on the structure to reduce its performance characteristics.

NOTES:

1 Generally, timber under cover and in well-ventilated conditions and not in contact with the ground water or free water is not subject to fungal attack. However, such timber may be subject to termite attack and to attack by other insects in parts of Australia. If conditions favourable for biological attack exist, then steps should be taken to eliminate the hazards. This is particularly important in structures where there is no load-sharing capacity, or there is a high consequence of failure.

2 Information on the durability design of timber structures and components thereof can be obtained from the following:

(a) All hazards:

*Design for Durability*, Datafile P4, National Association of Forest Industries Timber Manual.

STRINGER G.R. *The Development of Reliability Based Durability Design Methods for Timber Structures*, Gottstein Report 1994.

(b) Fire—AS 1720.4.

(c) Biological hazards:

Subterranean termites—The AS 3660 series.

Marine organisms—CSIRO, Marine Borers and Timber Piling Options.

Insects and fungi—AS 1604.1.

(d) Timber for houses:

Timber framing—The AS 1684 series.

(e) Metal connectors, corrosion—AS 4100.

(f) Timber adhesives—AS 2754.1(Int).

(g) Timber service life—*Timber Service Life Design Guide*, Forest and Wood Products Australia, 2007.

## 1.5 DESIGN AND SUPERVISION

### 1.5.1 Design

The design of timber structures to which this Standard applies, including the specification of materials and any protective treatment, shall be carried out in accordance with the requirements of this Standard.

### 1.5.2 Design information

The following information, appropriate to the design, shall be shown on the drawings:

(a) Reference number and date of the Standards used for the design.

(b) Dead, live, wind, snow and earthquake loads used in design.

(c) Strength group, joint group, stress grade and grading Standard used for design.

(d) Species or group of species of timber or plywood to be used.

(e) In-service moisture and temperature regime for which the structure has been designed.

(f) Protective treatment, if applicable.

(g) Fire-resistance rating, if applicable.

(h) Type of glue, if applicable.

(i) Standard of inspection to be employed during construction.

(j) Any other information that is critical to design.

### 1.5.3 Design details

The drawings and specification shall include the following, where appropriate:

(a) Size, characteristic value and moisture content of each member.

(b) Dimensioned details of all joints, connections and splices and their locations.

(c) Sizes and types of fasteners.

(d) Camber in any member.

(e) Any constraint on construction assumed in design.

(f) Surface finish of any member.

(g) Any special maintenance requirements.

(h) Any other requirements.

### 1.5.4 Supervision

The fabrication and erection of the timber structures or the parts of structures to which this Standard applies shall be supervised to ensure that all of the requirements of the design are satisfied in the completed structure.

## 1.6 WORKMANSHIP AND MAINTENANCE

### 1.6.1 General

The requirements of the designer, as specified in the design documents, shall be satisfied in the completed structure during its service life.

### 1.6.2 Moisture content

When structures or elements are to be fabricated with seasoned timber in situations where dimensional stability is critical, the designer of the structure shall ascertain the average equilibrium moisture content for the environment in which the structures or elements are to be erected, and shall specify that each piece of timber to be used shall have an average moisture content at the time of fabrication that is within 3% of the equilibrium value.

NOTE: Information on equilibrium moisture content values in timbers located in Australia can be obtained from the following references:

(a) FINIGHAN, R. *Moisture Content Predictions for Eight Seasoned Timbers under Sheltered Outdoor Conditions in Australia and New Guinea*. Division of Forest Products Technological Paper No. 44, CSIRO, 1966.

(b) BRAGG, C. *An Equilibrium Moisture Content Survey of Timber in Queensland*. Queensland Department of Forestry Technical Paper No. 40, QFD, 1986.

(c) BRENNAN G.K. and PITCHER J.A. *Equilibrium Moisture Content Variation of Timbers commonly used in Western Australia*. CALM Science Volume 2 No. 1 p.25-50, 1995.

### 1.6.3 Corrosion

The designer of the structure shall take due account of any possible corrosive effects on metal connectors or fasteners.

NOTE: Information on the protection of steel can be obtained from AS/NZS 2312.

### 1.6.4 Shop drawings

Where specified, shop drawings shall be prepared for structural members.

### 1.6.5 Fabrication and erection

The structure shall be fabricated and erected in accordance with the designer's instructions.

### 1.6.6 Maintenance

The structure shall be maintained during its service life in a condition that satisfies the designer's requirements as specified in the design documents.

## 1.7 DEFINITIONS

For the purpose of this Standard, the definitions given in AS/NZS 4491 and those below apply. Where the definitions in AS/NZS 4491 differ from those listed herein, those in this Standard shall apply.

### 1.7.1 Administrative definitions

**1.7.1.1** *Building authority or other regulatory authority*

Body having statutory powers to control the design and erection of buildings or structures, including scaffolding, in the area in which the building or structure concerned is to be erected.

**1.7.1.2** *Professional engineer*

A person who is—

(a) if legislation is applicable, a registered *professional engineer* in the relevant discipline, who has appropriate experience and competence in the relevant field; or

(b) if legislation is not applicable—

(i) a Corporate Member of Engineers Australia or the Institution of Professional Engineers New Zealand; or

(ii) eligible to become a Corporate Member of Engineers Australia or the Institution of Professional Engineers New Zealand and has appropriate experience and competence in the relevant field.

### 1.7.2 Technical definitions

**1.7.2.1** *A17-grade timber*

A stress grade of timber for which the specific suite of characteristic values given in Table H3.1, Appendix H, are applicable.

NOTE: A17-grades are assigned to seasoned hardwood timber in accordance with the grading Standard AS 2082.

**1.7.2.2** *Characteristic capacity*

The product of characteristic value and geometric section property appropriate to the action effect (see Clause 2.1.2).

**1.7.2.3** *Capacity factor*

A factor applied to the characteristic capacity to obtain the design capacity (see Clause 2.1.2).

**1.7.2.4** *Design characteristic value/characteristic value for design*

A specific design property suitable for use in the calculation of design capacity of a structural element.

NOTE: This definition may differ from the meaning given or applied in other contexts

**1.7.2.5** *Collapse-susceptible timber*

Timber, for which the shrinkage values before and after reconditioning differ by more than 2%.

NOTE: Information on shrinkage values can be obtained from the following references:

(a) KINGSTON, R.S.T. and RISDON C.J.E. *Shrinkage and Density of Australian and Other South-west Pacific Woods*. Division of Forest Products Technological Paper No. 13, CSIRO, 1961.

(b) BUDGEN, B. *Shrinkage and Density of some Australian and South-east Asian Timbers*. Division of Building Research Technological Paper (Second Series) No. 38, CSIRO, 1981.

**1.7.2.6** *Corewood*

Timber adjacent to or including the pith, that is, of density less than 80% that of the density of mature trees.

NOTE: For plantation grown softwoods, corewood may be avoided by excluding all timber within a radius of 50 mm from the centre of the pith that has a growth ring width greater than 6 mm (see also Clause 4.1.4).

**1.7.2.7** *Design action effect or design load effect*

The action or load effect computed from the design actions or design loads.

**1.7.2.8** *Design action or design load*

The combination of the nominal actions or loads and the load factors, as specified in AS/NZS 1170.0.

**1.7.2.9** *Design capacity*

The product of the capacity factor, modification factors and characteristic capacity of a structural element (see Clause 2.1.2).

**1.7.2.10** *Design life*

Period over which a structure or structural element is required to perform its function without repair.

**1.7.2.11** *Design resistance effect*

The resistance effect computed from the loads and design capacities contributing towards the stability limit state resistance.

**1.7.2.12** *Duration of loading*

Period during which a member, a structural element or a complete structure is subject to a specified load level.

NOTES:

1 For the purposes of interpretation in the use of load-duration factors in this Standard, see Clause 2.4.1.

2 The strength and stiffness properties of timber under load are time dependent.

**1.7.2.13** *F-grade timber*

A stress grade of timber for which the specific suite of characteristic values given in Table H2.1, Appendix H are applicable.

NOTE: F-grades may be assigned to graded timber in accordance with the appropriate grading Standard, i.e., AS 2082, AS 2858, AS/NZS 1748 or AS 3519.

**1.7.2.14** *Hardwood*

A wood from trees classified botanically as *Angiosperm*.

NOTES:

1 Although the term 'hardwood' is popularly interpreted to indicate the relative hardness of the wood, this interpretation is misleading as the wood of many hardwoods is relatively soft (e.g., balsa wood).

2 The structural difference between hardwoods and softwoods is that hardwoods have vessels or pores and softwoods do not; that is, the wood of softwoods does not include pores.

**1.7.2.15** *Joint group*

The classification assigned to a timber species or species group for the purpose of calculating joint capacity.

NOTES:

1 Unseasoned timber species or species groups are classified into six joint groups (i.e., J1 to J6). Seasoned timber species or species groups are classified into six joint groups (i.e., JD1 to JD6).

2 The joint group classifications for a range of timber species or species groups are given in Tables H2.3, H2.4, and H3.1, Appendix H for hardwoods and softwoods.

3 Joint group is defined for the purpose of this Standard only; the classifications are not necessarily applicable for proprietary fasteners (e.g., multi-toothed nailplates).

**1.7.2.16** *Limit state*

Any limiting condition beyond which the structure ceases to fulfil its intended function.

**1.7.2.17** *MGP grade timber*

A stress grade of timber for which the specific suite of characteristic values given in Table H3.1, Appendix H, are applicable.

NOTE: MGP grades are assigned to seasoned softwood timber graded in accordance with AS/NZS 1748.

**1.7.2.18** *Primary structural element*

Members and joints whose failure could result in collapse of a significant portion of a structure.

**1.7.2.19** *Proof testing*

Application of test loads to a structure or element to ascertain the structural characteristics of only that one unit under test.

**1.7.2.20** *Prototype testing*

Application of test loads to a structure or element to ascertain the structural characteristics of structures or elements which are nominally identical to the unit or units tested.

**1.7.2.21** *Seasoned timber*

Timber in which the average moisture content is nominally between 10 and 15%.

NOTE: Seasoned timber is sometimes referred to as 'dry', 'air-dried' or 'kiln-dried' timber.

**1.7.2.22** *Secondary structural elements*

Members and joints whose failure would result in a localized collapse.

**1.7.2.23** *Serviceability limit state*

A limit state of acceptable in-service condition.

**1.7.2.24** *Softwood*

Wood from trees classified botanically as *Gymnosperm*.

NOTES:

1 Although the term 'softwood' is popularly interpreted to indicate the relative softness of the wood, this interpretation is misleading as the wood of many softwoods can be relatively hard (e.g., cypress pine).

2 Commercial species of this group are nearly always conifers.

**1.7.2.25** *Stability limit state*

A limit state corresponding to the loss of static equilibrium of a structure considered as a rigid body.

**1.7.2.26** *Strength group*

The classification assigned to a timber species or species group for the purposes of determining characteristic values for bearing, shear at joint details and tension perpendicular to grain.

NOTES:

1 Unseasoned timber species or species groups are classified into seven strength groups (i.e., S1 to S7). Seasoned timber species or species groups are classified into eight strength groups ( i.e., SD1 to SD8).

2 The strength group classifications for a range of timber species or species groups are given in Tables H2.3, H2.4, and H3.1, Appendix H, for hardwoods and softwoods.

3 Procedures for assigning species to strength group classifications are given in AS 2878.

**1.7.2.27** *Strength limit state*

A limit state of collapse or loss of structural integrity.

**1.7.2.28** *Stress grade*

A classification assigned to structural timber or wood products that indicates a suite of characteristic values of strength and stiffness properties suitable for structural design.

NOTE: Typical examples of stress grades are—

(a) F-grades for sawn timber as given in Table H2.1 of Appendix H;

(b) MGP grades for sawn timber as given in Table H3.1 of Appendix H;

(c) A17 grades for sawn timber as given in Table H3.1 of Appendix H;

(d) F-grades for plywood as given in Table 5.1;

(e) GL grades for glued laminated timber as given in Table 7.1; and

(f) proprietary stress grades detailed in a manufacturer's product specification.

**1.7.2.29** *Structural element*

A member and/or joint that fulfils a structural function.

**1.7.2.30** *Structural timber*

Timber with defined structural properties that meet the requirements of this Standard.

**1.7.2.31** *Unseasoned timber*

Timber in which the average moisture content exceeds 25%.

NOTE: Unseasoned timber is sometimes referred to as 'green' timber.

## 1.8 NOTATION

Except where specifically defined in a particular Clause, the quantity symbols and factors shall be as listed in Appendix F.

## 1.9 UNITS

Unless otherwise stated, the units of measurement used in this Standard shall be in accordance with the International System of Units (SI).

NOTE: In general, newtons (N), millimetres (mm) and megapascals (MPa) are appropriate units to be used.

# SECTION 2 DESIGN PROPERTIES OF STRUCTURAL TIMBER ELEMENTS

## 2.1 GENERAL

### 2.1.1 General procedure

Design capacities for structural timber shall be obtained through modifying characteristic capacities by factors appropriate to the service conditions and material property type. This general procedure applies to all types of structural timber, including sawn timber, laminated timber, natural round timber, plywood and laminated veneer lumber except where otherwise specified.

NOTE: Appendix G gives references for additional information on species design properties and examples of load duration factors for typical combinations of actions.

### 2.1.2 Member design capacity

The design capacity ($R_d$) of a structural member is the product of the characteristic value of the material, the appropriate geometric property, factors to allow for variation in strength with the environment and configuration of the element in use, and a capacity factor, and is expressed as follows:

$$R_d = \phi \, k_{mod} \, f'_o \, X \qquad \ldots 2.1$$

where

$\phi$ = capacity factors (see Clause 2.3)

$k_{mod}$ = product of modification factors as defined in Clause 2.4 and elsewhere in this Standard ($k_1 \times k$......... $\times k_n$)

$f'_o X$ = characteristic capacity appropriate to the action effect

$f'_o$ = characteristic value of material design property (e.g., bending, tension, etc.), in megapascals

$X$ = geometric sectional property of member appropriate to the method of load application to produce a particular failure mode

NOTE: As an example the factor $k_{mod}$ for design moment capacity in bending of a solid timber member is typically given by $k_{mod} = k_1 \, k_4 \, k_6 \, k_9 \, k_{12}$. The appropriate geometric property is section modulus ($Z$) and the appropriate characteristic value is $f'_b$, giving the following design capacity:

$$M_d = \phi \, k_1 \, k_4 \, k_6 \, k_9 \, k_{12} \, f'_b \, Z$$

### 2.1.3 Design rigidity of members

The design rigidity is the product of the relevant section property and the average modulus of elasticity (or modulus of rigidity) applicable for the stress grade.

### 2.1.4 Design capacity of joints

The design capacity of joints ($R_d$) required to resist direct loads, as a generalization, is the product of the characteristic capacity of the fastener, factors to allow for the environment of use and configuration of the joint and a capacity factor, expressed as follows:

$$R_d = \phi \, k_{mod} \, n \, Q \qquad \ldots 2.2$$

where

$\phi$ = capacity factor (see Clause 2.3)

$k_{mod}$ = product of modification factors as defined in Clause 2.4 and Section 4 ($k_1 \times k$......... $\times k_n$)

$Q$ = characteristic capacity for the fastener type, size and load and fastener orientation, as appropriate, in newtons

$n$ = number of fasteners

NOTE: Expressions for the design capacity of joints ($R_d$) required to resist in-plane moments are given in Section 4.

## 2.2 DESIGN PROPERTIES

### 2.2.1 Characteristic values for calculating the design capacity of members

The characteristic values for strength and elastic moduli for the design of structural elements using standardized stress grades of structural sawn timber, structural plywood, round timber, glue-laminated timber and structural laminated veneer lumber are obtained as follows:

(a) For F-grades, MGP grades and A17 grades of solid sawn timber refer to Appendix H.

(b) For F-grades for structural plywood, refer to Section 5.

(c) For round timbers, refer to Section 6 and Appendix H.

(d) For GL-grades for glue-laminated timber, refer to Section 7.

(e) For structural laminated veneer lumber, refer to Section 8.

### 2.2.2 Characteristic values for calculating the design capacity of joints

The characteristic capacities used to calculate the design capacity of joints using nails, screws, bolts, coach screws, split-ring and shear-plate connectors shall be as given in Section 4 for species joint groups. Joint group classifications for species and moisture condition shall be as given in Appendix H.

## 2.3 CAPACITY FACTOR

Values of the capacity factor ($\phi$) for calculating the design capacity, $R_d$, of structural members and structural joints are listed in Table 2.1 and Table 2.2, respectively, for material and fastener types and structural application.

## TABLE 2.1

## VALUES OF CAPACITY FACTOR ($\phi$) FOR CALCULATING THE DESIGN CAPACITY ($R_d$) OF STRUCTURAL MEMBERS

| Structural timber material | Application of structural member | | |
|---|---|---|---|
| | **Category 1** | **Category 2** | **Category 3** |
| | Structural members for houses for which failure would be unlikely to affect an area* greater than 25 $m^2$;<br>OR<br>secondary members in structures other than houses | Primary structural members in structures other than houses;<br>OR<br>elements in houses for which failure would be likely to affect an area* greater than 25 $m^2$ | Primary structural members in structures intended to fufill an essential service or post disaster function |
| | Values of capacity factor ($\phi$) | | |
| Sawn timber—AS 2082, AS 2858, AS/NZS 1748, AS 3519: | | | |
| —Stress grades: MGP 15, A17, F17 and higher F-grades | 0.95 | 0.85 | 0.75 |
| —All other timber† and stress grades | 0.90 | 0.70 | 0.60 |
| Round timber— AS 3818.3 or AS 3818.11, as appropriate | 0.90 | 0.70 | 0.60 |
| Glue-laminated timber—AS/NZS 1328.1 | 0.95 | 0.85 | 0.75 |
| Structural plywood—AS/NZS 2269.0 | 0.95 | 0.85 | 0.75 |
| Structural laminated veneer lumber—AS/NZS 4357.0 | 0.95 | 0.90 | 0.80 |

NOTE: Refer to definitions: secondary structural element in Clause 1.7.2.22; primary structural element in Clause 1.7.2.18.

* In this context, area should be taken as the plan area.

† Indicates design capacities determined using the characteristic values from Table H2.2, Appendix H.

## TABLE 2.2

## VALUES OF CAPACITY FACTOR ($\phi$) FOR CALCULATING THE DESIGN CAPACITY ($N_d$) OF STRUCTURAL JOINTS

| Type of fastener and applicable Australian product standards | Application of structural member | | |
|---|---|---|---|
| | **Category 1** | **Category 2** | **Category 3** |
| | Structural joints for houses for which failure would be unlikely to affect an area[(1)] greater than 25 m$^2$;<br>OR<br>Joints for secondary elements[(2)] in structures other than houses | Primary structural joints[(2)] in structures other than houses;<br>OR<br>joints for house construction for which failure of the joint would be likely to affect an area[(1)] greater than 25 m$^2$ | Primary structural joints[(1)] in structures intended to fulfill an essential services or post disaster function |
| | **Capacity factor ($\phi$)** | | |
| Nails conforming to AS 2334 | 0.85 | 0.80 | 0.75 |
| Screws conforming to the AS 3566 series | 0.85 | 0.80 | 0.75 |
| Multi-toothed nail plates conforming to manufacturer's specifications | 0.85 | 0.80 | 0.75 |
| Bolts conforming to AS 1111.1<br>—M16 and smaller<br>—larger than M16 | <br>0.85<br>0.75 | <br>0.80<br>0.65 | <br>0.75<br>0.60 |
| Coach screws conforming to AS/NZS 1393<br>—M16 and smaller<br>—larger than M16 | <br>0.85<br>0.75 | <br>0.80<br>0.65 | <br>0.75<br>0.60 |
| Split-ring fasteners conforming to AS 1442 | 0.75 | 0.65 | 0.60 |
| Shear-plate fasteners conforming to AS 1442 | 0.75 | 0.65 | 0.60 |

NOTES:

1 In this context, area should be taken as the plan area. In most instances, a category 2 joint will be required for support of a category 2 member.

2 Refer to definitions: secondary structural elements in Clause 1.7.2.22; primary structural elements (joints) in Clause 1.7.2.18.

## 2.4 MODIFICATION FACTORS

### 2.4.1 Duration of load

**2.4.1.1** *Effect on strength*

The effect of duration of load on strength is given by the modification factor $k_1$ in Table 2.3.

In checking the design capacity of a structural element, all combinations of actions shall be considered.

For any given combination of loads of differing duration, the factor $k_1$ to be used is that appropriate to the action that is of the shortest duration. In Table 2.3, the effective duration of a peak action refers to the cumulative duration for which the peak action occurs.

For the purposes of interpretation in the selection of load-duration factors for calculation of design capacity, the following shall apply:

(a) Permanent actions and the long-term components of imposed actions shall be considered of '50 or more years duration'.

(b) Imposed actions that act on floors (such as those due to vehicles or people), and are applied at frequent but irregular intervals such that the structure is unloaded or loaded well below the nominal imposed action for most of each day, shall be considered to be of 'five months duration'.

(c) Imposed actions, such as those arising during erection and maintenance, and imposed actions that are infrequent (e.g., crowd loads), applied for periods of a few days and at infrequent intervals, shall be considered to be of 'five days duration'.

(d) Impact actions, such as those caused by falling weights or snatch lifts, shall be considered to be of 'five seconds duration'.

(e) Snow action shall be considered to be of the following durations:

    (i) For alpine areas ........................................................................ 5 months.

    (ii) For sub-alpine areas ....................................................................5 days.

(f) Earthquake actions shall be considered to be of 'five seconds duration'.

(g) Strength limit state gust wind actions referred to in AS/NZS 1170.2 shall be considered to be of 'five seconds duration'.

(h) For the fire limit state, the duration of actions shall be considered to be of 'five days duration'.

NOTE: The critical strength limit state load combination may not be the one with the highest value because the resistance of timber to load is a function of duration of load. In some cases, load combinations involving longer term loads of lesser load effect may be more critical than combinations involving shorter term loads.

### TABLE 2.3
### DURATION OF LOAD FACTOR FOR STRENGTH

| Effective duration of peak action | Modification factor ($k_1$)* | |
|---|---|---|
| | For the strength of timber | For the strength of joints using laterally loaded fasteners† |
| 5 seconds | 1.00 | 1.14 |
| 5 minutes | 1.00 | 1.00 |
| 5 hours | 0.97 | 0.86 |
| 5 days | 0.94 | 0.77 |
| 5 months | 0.80 | 0.69 |
| 50+ years | 0.57 | 0.57 |

* Typical values of $k_1$ for various load combinations are given in Table G1, Appendix G.

† For the strength of joints with fasteners loaded in withdrawal and for the strength of steel in joints, $k_1$ = 1.00.

#### 2.4.1.2 *Effect on deformation*

For members in bending and compression and shear deformation or tensile deformation, the calculated short-term deformation shall be multiplied by the appropriate modification factor for creep ($j_2$ or $j_3$), as given in Table 2.4 and illustrated graphically in Figures 2.1 and 2.2.

Values intermediate between those given in Table 2.4 shall be obtained by interpolation involving the logarithm of time, and a linear function of initial moisture content as shown in Figures 2.1 and 2.2.

When several types of action contribute to deformation, the maximum deformation shall be taken to be equal to the sum of the deformations computed for each type of load acting alone.

Where there is a recovery period of more than 10 times the period of the applied action between applications, the creep component of deformation for that action type shall be assumed to be totally recovered.

The modification factors $j_2$ and $j_3$, given in Table 2.4, are not applicable to collapse-susceptible hardwoods (see Clause 1.7.2) that are unseasoned at the time of loading (see Note 3 below).

NOTES:

1 In general, peak values of serviceability-imposed actions are not of a permanent nature; accordingly, if a designer wishes to compute the long-term deformations of a structure to satisfy a specified serviceability limit state, the designer first estimates the portion of the serviceability load that is permanently or semi-permanently applied and then uses an appropriate creep factor to compute the deformation for that portion.

2 The duration of actions specified for calculation of design capacity are not intended for estimation of deformation.

3 For collapse susceptible hardwoods in the unseasoned condition, the creep factors $j_2$ and $j_3$ may be considerably greater than values given in Table 2.4.

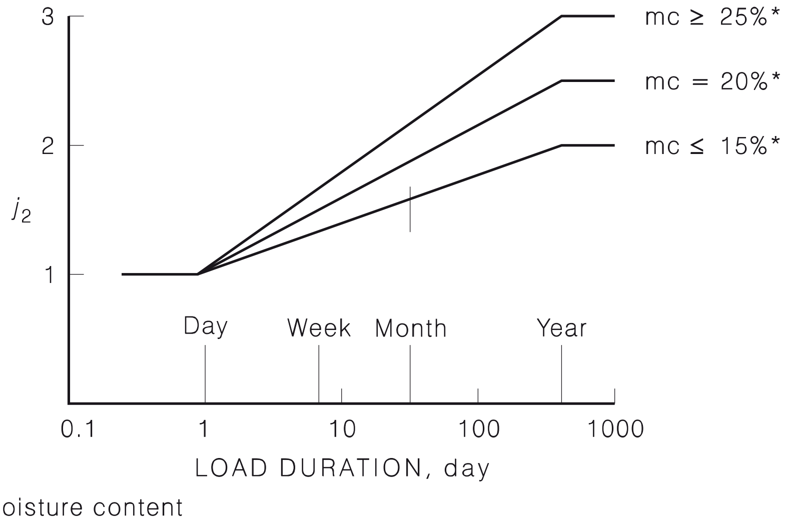



FIGURE 2.1 DURATION FACTOR ($j_2$) FOR BENDING AND COMPRESSION DEFORMATIONS

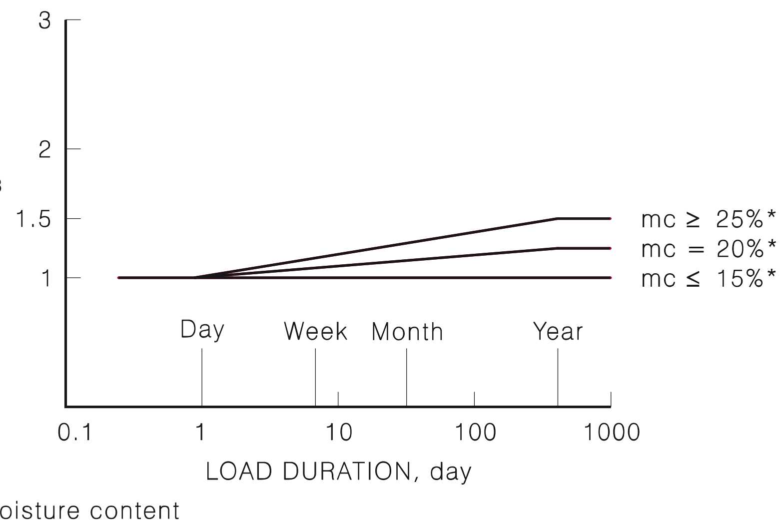



FIGURE 2.2 DURATION FACTOR ($j_3$) FOR TENSION DEFORMATIONS

## TABLE 2.4

## DURATION OF LOAD FACTOR FOR CREEP DEFORMATION

| Initial moisture content* | For bending, compression and shear members ($j_2$) | | For tension members ($j_3$) | |
|---|---|---|---|---|
| | Duration of action | | Duration of action | |
| % | ≤1 day | ≥1 year | ≤1 day | ≥1 year |
| ≤15 | 1 | 2 | 1 | 1 |
| ≥25 | 1 | 3 | 1 | 1.5 |

* Moisture content at the time of load application.

### 2.4.2 Moisture condition

#### 2.4.2.1 *General*

Depending on the initial moisture content of the timber and the moisture content at time of loading and throughout its life, the characteristic capacity of timber shall be modified by a factor denoted by $k_4$.

NOTE: For appropriate values of $k_4$ for glulam, refer to Clause 7.4.2, and for laminated veneer lumber (LVL) to Clause 8.4.3.

#### 2.4.2.2 *Unseasoned timber*

For unseasoned timber, generally $k_4$ equals 1.0. Where unseasoned timber is used under normal conditions of temperature and humidity and will not be subject to its full design load until it has partially seasoned (i.e., to below 25% moisture content), it is permissible to increase the characteristic capacity for unseasoned timber by multiplying by the factor $k_4$ given in Table 2.5.

**TABLE 2.5**

**PARTIAL SEASONING FACTOR ($k_4$)**

| Least dimension of member | 38 mm or less | 50 mm | 75 mm | 100 mm or more |
|---|---|---|---|---|
| Value of $k_4$ | 1.15 | 1.10 | 1.05 | 1.00 |

#### 2.4.2.3 *Seasoned timber*

For seasoned timber, generally $k_4$ equals 1.0. Where seasoned timber is subjected to conditions in which its average moisture content for a 12 month period is expected to exceed 15%, the characteristic capacity shall be decreased.

The value of $k_4$ shall be determined as the greater of—

(a) $k_4 = 1 - 0.3\dfrac{EMC - 15}{10}$; and

(b) $k_4 = 0.7$;

where $EMC$ is the highest value of the annual average moisture content (percent) that the timber will attain in service.

### 2.4.3 Temperature

For covered timber structures under ambient conditions, no modification for strength need be made for the effect of temperature (i.e., $k_6$ equals 1.0) except that where seasoned timber is used in structures erected in coastal regions of Queensland north of latitude 25°S, and all other regions of Australia north of latitude 16°S, the strength shall be modified by a factor $k_6$ of 0.9.

NOTE: Information on the effects of high temperatures can be obtained from MEYER, R.W. and KELLOG, R.M. *Structural Use of Wood in Adverse Environments*, Van Nostrand, 1982.

### 2.4.4 Length and position of bearing

Where rectangular bearing areas are located 75 mm or more from the end of a piece of timber, it is permissible to increase the characteristic capacity in bearing perpendicular to the grain (refer to Clause 3.2.6) by the appropriate value of factor $k_7$ in Table 2.6. The length of bearing shall be measured parallel to the grain of the loaded member. For all other conditions $k_7$ equals 1.0.

For circular bearing areas, the effective bearing length shall be taken as being equal to the diameter of the bearing area.

**TABLE 2.6**

**LENGTH OF BEARING FACTOR**

| **Length of bearing of member** | 12 | 25 | 50 | 75 | 125 | 150 or more |
|---|---|---|---|---|---|---|
| **Value of $k_7$** | 1.75 | 1.40 | 1.20 | 1.15 | 1.10 | 1.00 |

### 2.4.5 Strength sharing between parallel members

#### 2.4.5.1 *General*

When a sawn timber structural member is one of a number of parallel acting elements that interact to assist each other to form a strength-sharing structural system, it is permissible to increase the characteristic capacities in bending for sawn structural timber by the strength-sharing factor ($k_9$) determined in accordance with Clause 2.4.5.3. Values of the strength-sharing factor ($k_9$) specified in this Clause shall not apply to plywood, glued laminated timber and LVL members (see Sections 5, 7 and 8).

NOTE: In addition to strength-sharing structural systems, grid systems provide a method for laterally distributing concentrated loads (see Appendix E).

#### 2.4.5.2 *Strength-sharing structural systems*

Parallel acting systems are classified as follows:

(a) Combined parallel systems.

(b) Discrete parallel systems.

A combined parallel system is comprised of two or more elements that are effectively fastened together.

In determining the factor $k_9$ for a combined parallel system, the effective number of elements shall be taken as the number of elements ($n_{com}$) that are effectively fastened together to form a single group.

NOTES:

1 An example of a combined parallel system is illustrated in Figure 2.3(a).

2 Effectively fastened together means that all of the elements are constrained to the same deformation under load.

3 One method of effectively fastening elements together is by rigidly connecting all of the elements together at a large number of points along their length.

A discrete parallel system has three or more members that are discretely spaced parallel to each other. These discretely spaced members support either an overlying set of members, usually laid at right angles to the supporting members, or a structural sheathing material.

In determining the factor $k_9$ for a discrete parallel system, the effective number of elements shall be taken as the product of—

$$n_{com} \times n_{mem}$$

where

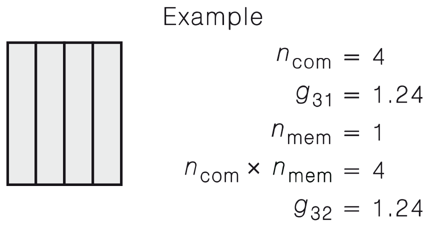
$n_{com}$ = number of elements that are effectively fastened together to form a single group

$n_{mem}$ = number of members that are discretely spaced parallel to each other

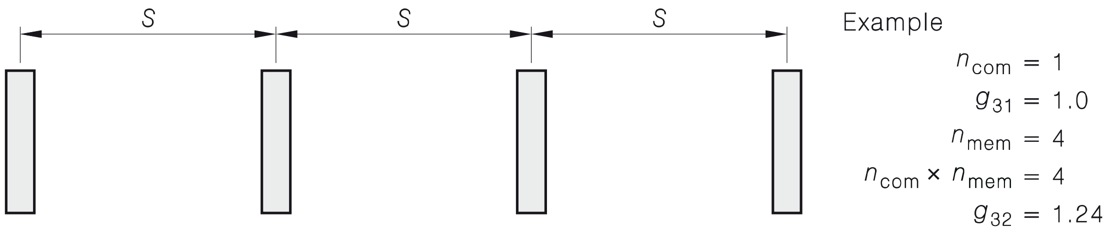
NOTES:

1 Examples of discrete parallel systems are illustrated in Figures 2.3(b) and 2.3(c).

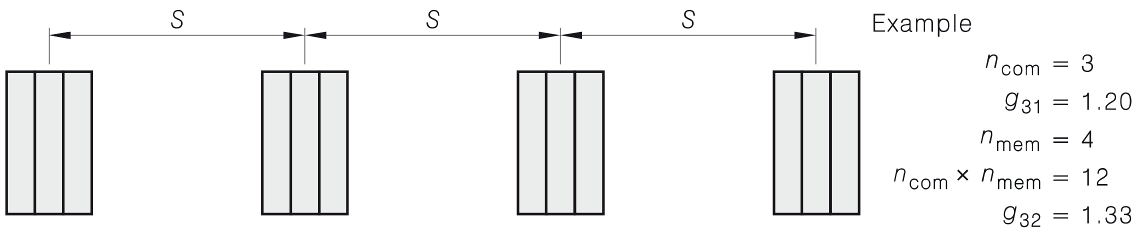
2 For a parallel system, in the event of the failure of a single supporting member, then the overlying members or sheathing material should be capable of transferring loads to adjacent supporting members.




(a) Combined parallel system




(b) Discrete parallel system




(c) Combined and discrete parallel system

FIGURE 2.3 PARALLEL SYSTEMS

**2.4.5.3** *Modification factor for strength sharing*

The strength-sharing factor $k_9$ is given by the following equation:

$$k_9 = g_{31} + (g_{32} - g_{31})\left[1 - \frac{2s}{L}\right], \text{ but not less than 1.0} \qquad \text{. . . 2.4.5.3}$$

where

$g_{31}$ = geometric factor appropriate to the number of members ($n_{com}$) in a combined parallel system given in Table 2.7

$g_{32}$ = geometric factor appropriate to the number of members ($n_{com} \times n_{mem}$) in a discrete system given in Table 2.7

$s$ = centre-to-centre spacing of the discrete parallel members

$L$ = effective span of the parallel members

The strength-sharing factor $k_9$ is illustrated graphically in Figure 2.4.

**TABLE 2.7**

**GEOMETRIC FACTORS FOR PARALLEL SYSTEMS**

| Number of members in combined parallel ($n_{com}$) | $g_{31}$ | Total number of members in parallel system ($n_{com} \times n_{mem}$) | $g_{32}$ |
|---|---|---|---|
| 1 | 1.00 | 1 | 1.00 |
| 2 | 1.14 | 2 | 1.14 |
| 3 | 1.20 | 3 | 1.20 |
| 4 | 1.24 | 4 | 1.24 |
| 5 | 1.26 | 5 | 1.26 |
| 6 | 1.28 | 6 | 1.28 |
| 7 | 1.30 | 7 | 1.30 |
| 8 | 1.31 | 8 | 1.31 |
| 9 | 1.32 | 9 | 1.32 |
| 10 or more | 1.33 | 10 or more | 1.33 |

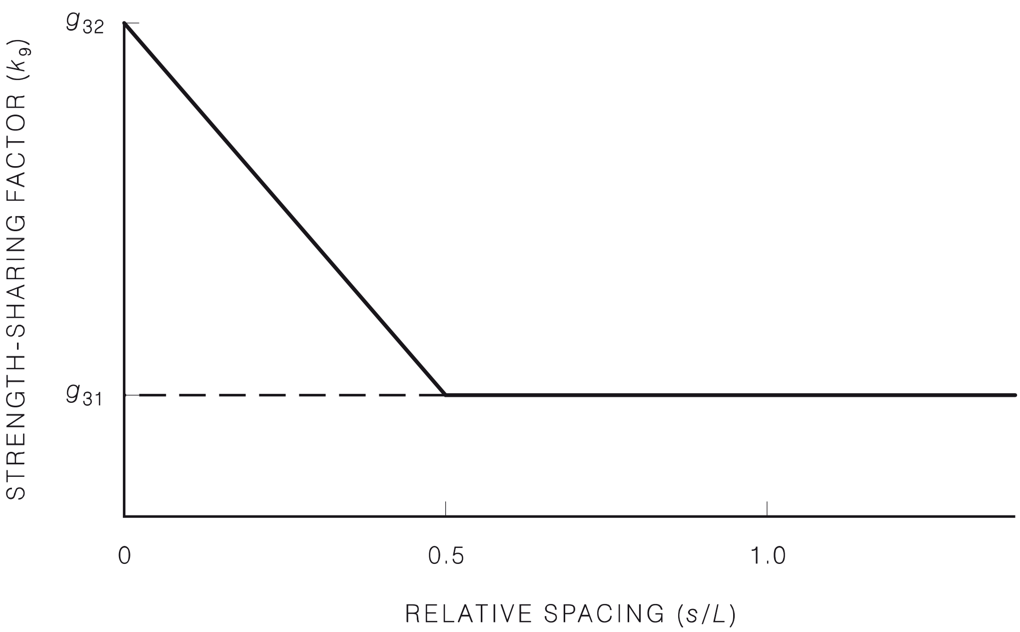



FIGURE 2.4 STRENGTH-SHARING FACTOR ($k_9$)

### 2.4.6 Stability factor

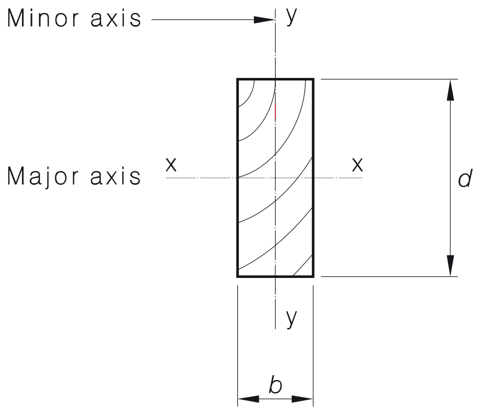
In the design of slender bending or axial compression members, a factor $k_{12}$ shall be used to allow for the effects of slenderness on strength. The stability factor is defined in Clause 3.2.4 for bending members and Clause 3.3.3 for compression members.

# SECTION 3 DESIGN CAPACITY OF BASIC STRUCTURAL MEMBERS

## 3.1 GENERAL

This Section shall be applied in conjunction with Section 2. This Section applies to the design of basic structural members such as beams, columns and tension members. In particular, many of the design parameters given herein refer to members of rectangular cross-section, for which the notation used is shown in Figure 3.1. The corresponding parameters for members of less usual shape shall be as given in Appendix E. Special design requirements related to the use of plywood, pole timbers, glued-laminated construction and laminated veneer lumber are given in Sections 5, 6, 7 and 8 respectively. Methods using a higher order approach to the design of members and for particular cases are given in Appendix E.

NOTE: In beam design, deflection considerations may govern member sizes (see Clauses 1.4.2.4 and 2.4.1.2).




NOTE: For design dimensions, see Clause 1.4.3.

FIGURE 3.1 NOTATION FOR A RECTANGULAR CROSS-SECTION

## 3.2 BEAM DESIGN

### 3.2.1 Bending strength

#### 3.2.1.1 *Design capacity*

The design capacity in bending ($M_d$) of unnotched beams, for the strength limit state, shall satisfy the following:

$$M_d \geq M^* \quad \ldots 3.2(1)$$

where

$$M_d = \phi k_1 k_4 k_6 k_9 k_{12} f'_b Z \quad \ldots 3.2(2)$$

and

$M^*$ = design action effect in bending (see Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ to $k_9$ = the modification factors given in Section 2

$k_{12}$ = stability factor (see Clause 3.2.4)

$f'_b$ = characteristic value in bending for the section size

$Z$ = section modulus of beam about the axis of bending (for rectangular beams $Z_x$ equals $bd^2/6$ and $Z_y$ equals $db^2/6$, where $b$ equals the breadth and $d$ equals the depth of the beam).

**3.2.1.2** *Bending about both axes*

For beams that can bend about both the major and minor axes simultaneously, for strength limit states, the design action effects in bending shall satisfy the following equation:

$$\frac{M_x^*}{M_{d,x}} + \frac{M_y^*}{M_{d,y}} \leq 1 \qquad \text{. . . 3.2(3)}$$

where

$M_x^*$ = design action effect in bending about the major principal x-axis (see Clause 1.4.2.2)

$M_{d,x}$ = design capacity in bending about the major principal x-axis determined according to Clause 3.2.1

$M_y^*$ = design action effect in bending about the minor principal y-axis (see Clause 1.4.2.2)

$M_{d,y}$ = design capacity in bending about the minor principal y-axis determined according to Clause 3.2.1

For less conservative criteria than given by Equation 3.2(3), see Equations E5(1) and E5(2), Appendix E.

### 3.2.2 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 this Standard.

### 3.2.3 Slenderness coefficient for lateral buckling under bending

**3.2.3.1** *General*

For the general case, and for several useful specific cases, equations for evaluating the slenderness coefficient are given in Appendix E. For special cases of solid beams of rectangular cross-section, the simple approximations given in Clause 3.2.3.2 are acceptable. The notation for beam restraints is shown in Figures 3.2 to 3.6.

**3.2.3.2** *Beams of rectangular cross-section*

Approximations of slenderness coefficients for typical load and restraint conditions are given as follows:

(a) *Beams that bend about their major axis having discrete lateral restraint systems* For a beam that is loaded along its compression edge and has discrete lateral restraints at points $L_{ay}$ apart, along the compression edge of the beam as indicated in Figure 3.2, the slenderness coefficient, denoted by $S_1$, is—

$$S_1 = 1.25\frac{d}{b}\left(\frac{L_{ay}}{d}\right)^{0.5} \qquad \text{. . . 3.2(4)}$$

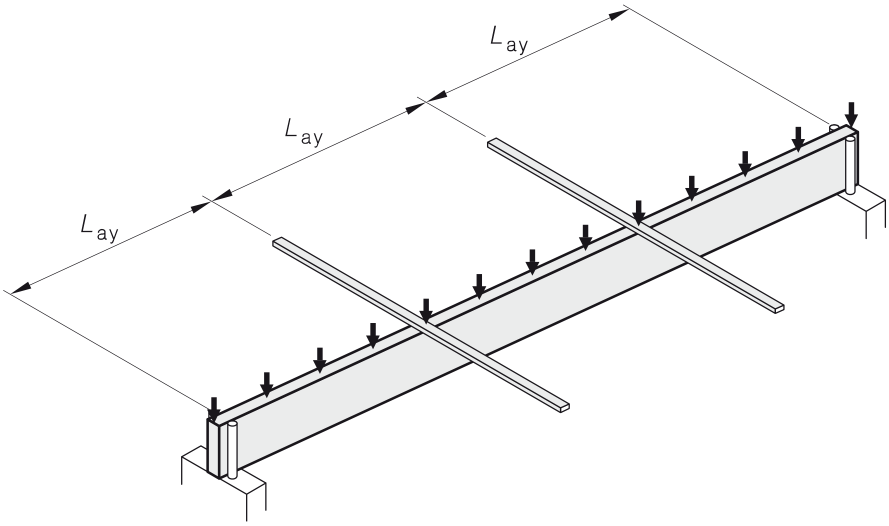



FIGURE 3.2 DISCRETE RESTRAINTS TO THE COMPRESSION EDGE

For a beam that is loaded along its tension edge and has discrete lateral restraints at points $L_{ay}$ apart, along the tension edge of the beam as indicated in Figure 3.3, then the slenderness coefficient, denoted by $S_1$, is—

$$S_1 = \left(\frac{d}{b}\right)^{1.35} \left(\frac{L_{ay}}{d}\right)^{0.25} \qquad \text{. . . 3.2(5)}$$

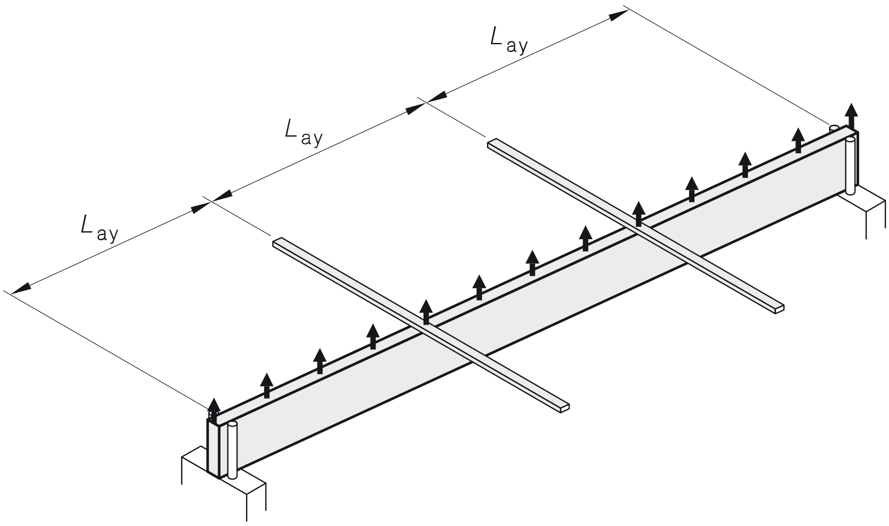



FIGURE 3.3 DISCRETE RESTRAINTS TO THE TENSION EDGE

(b) *Beams that bend about their major axis having continuous lateral restraint systems* A continuous lateral restraint system (see Figures 3.4 and 3.5) shall be assumed to exist when—

$$\frac{L_{ay}}{d} \leq 64\left(\frac{b}{\rho_b\, d}\right)^2 \qquad \text{. . . 3.2(6)}$$

For a beam that is loaded along its compression edge and has a continuous lateral restraint system along the compression edge (see Figure 3.4), the slenderness coefficient, denoted by $S_1$, shall be taken to be equal to zero.

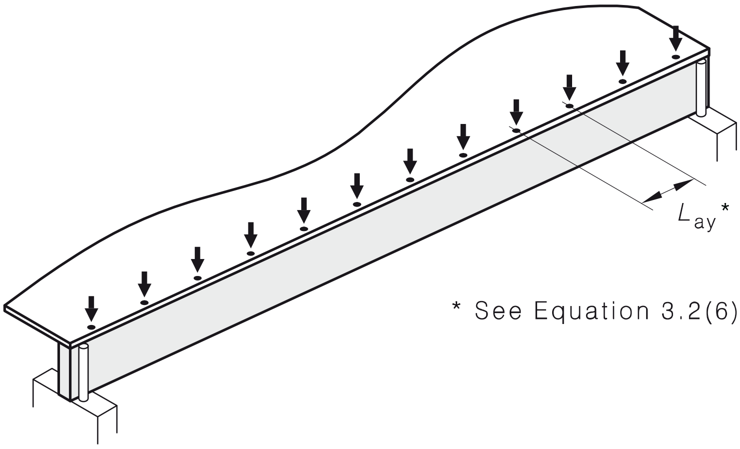



FIGURE 3.4 CONTINUOUS RESTRAINT ALONG THE COMPRESSION EDGE

For a beam that is loaded along its tension edge and has a continuous lateral restraint system along the tension edge (see Figure 3.5), the slenderness coefficient, denoted by $S_1$, is—

$$S_1 = 2.25\frac{d}{b} \qquad \ldots 3.2(7)$$

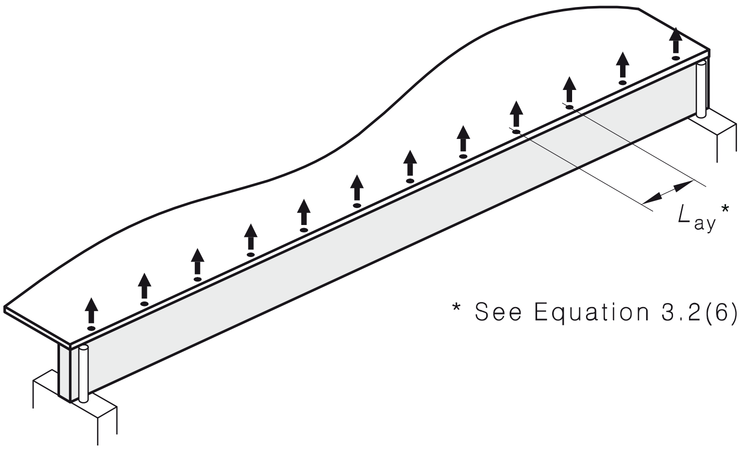



FIGURE 3.5 CONTINUOUS RESTRAINT ALONG THE TENSION EDGE

For a beam that is loaded along its tension edge and, in addition to having a continuous lateral restraint system along its tension edge, has equally spaced torsional restraints at points $L_{a\phi}$ apart, indicated in Figure 3.6, to prevent rotation about the beams Z axis, the slenderness coefficient, denoted by $S_1$, is—

$$S_1 = \frac{1.5\, d/b}{\left[\left(\frac{\pi d}{L_{a\phi}}\right)^2 + 0.4\right]^{0.5}} \qquad \ldots 3.2(8)$$

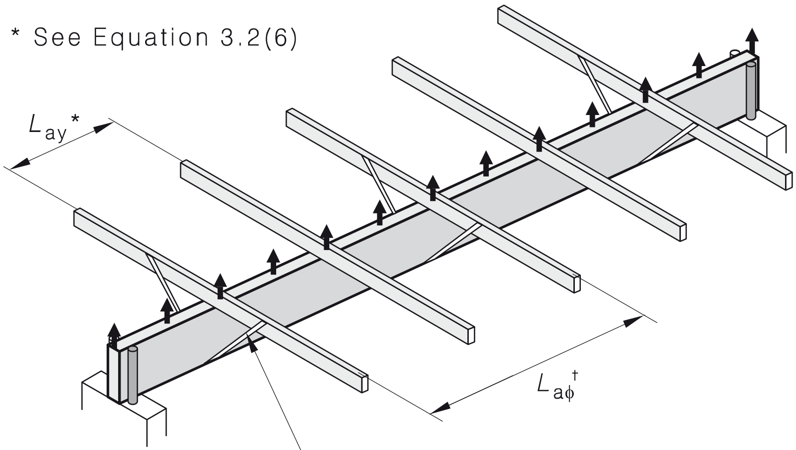



FIGURE 3.6 CONTINUOUS RESTRAINT ALONG THE TENSION EDGE COMBINED WITH DISCRETE TORSIONAL RESTRAINTS

(c) *Beams that bend only about their minor axis* For all cases, the slenderness coefficient, denoted by $S_2$, is—

$$S_2 = 0.0 \quad \ldots 3.2(9)$$

(d) *Beams that bend about both axes* The design of such beams, described in Clause 3.2.1.2, is based on an interaction of the two special cases for bending about single axis only, and hence no special definition of slenderness is required for this case.

### 3.2.4 Stability factor

The stability factor ($k_{12}$) for modification of the characteristic value in bending shall be given by the following:

(a) For $\rho_b S_1 \leq 10$—

$$k_{12} = 1.0 \quad \ldots 3.2(10)$$

(b) For $10 \leq \rho_b S_1 \leq 20$—

$$k_{12} = 1.5 - 0.05\, \rho_b S_1 \quad \ldots 3.2(11)$$

(c) For $\rho_b S_1 \geq 20$—

$$k_{12} = \frac{200}{(\rho_b S_1)^2} \quad \ldots 3.2(12)$$

For bending about the minor axis, where $S_2$ equals 0 then $k_{12}$ equals 1.0.

Conservative values of the material constant ($\rho_b$) for F-grade, MGP-grade, and A17-grade materials are given in Table 3.1. More accurate values of $\rho_b$ are given by Equations E2(1) and E2(2), Appendix E, and tabulated for F-grades, MGP-grade, and A17-grade in Paragraph E2, Appendix E. The shape of the stability factor curve is illustrated in Figure 3.7.

For material constants for GL-grades and structural laminated veneer lumber, refer to Sections 7 and 8, respectively.

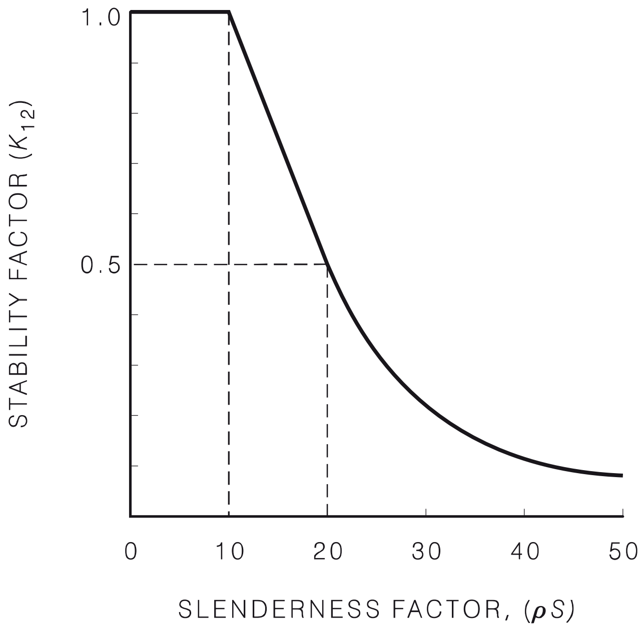



FIGURE 3.7 EFFECT OF SLENDERNESS COEFFICIENT ON THE STABILITY FACTOR FOR BEAMS AND COLUMNS

## TABLE 3.1

## MATERIAL CONSTANT ($\rho_b$) FOR SAWN TIMBER BEAMS

| Stress grade | Material constant ($\rho_b$) | |
|---|---|---|
| | Seasoned timber | Unseasoned timber |
| F34 | 1.12 | 1.21 |
| F27 | 1.08 | 1.17 |
| F22 | 1.05 | 1.15 |
| F17 | 0.98 | 1.08 |
| F14 | 0.98 | 1.08 |
| F11 | 0.98 | 1.07 |
| F8 | 0.89 | 0.99 |
| F7 | 0.86 | 0.96 |
| F5 | 0.82 | 0.91 |
| F4 | 0.80 | 0.90 |
| MGP 15 | 0.91 | — |
| MGP 12 | 0.85 | |
| MGP 10 | 0.75 | |
| A17 | 0.95 | — |

NOTES:

1 The values of $\rho_b$ for F-grade seasoned and unseasoned timber correspond to $r = 0.25$ in Tables E1 and E2, Appendix E.

2 The values of $\rho_b$ for MGP grades and A17 grade seasoned timber correspond to $r = 0.25$ in Table E1, Appendix E.

### 3.2.5 Flexural shear strength

The design capacity in shear ($V_d$) of un-notched beams, for the strength limit state, shall satisfy the following:

$$V_d \geq V^* \quad \ldots 3.2(13)$$

where

$$V_d = \phi k_1 k_4 k_6 f'_s A_s \quad \ldots 3.2(14)$$

and

$V^*$ = design action effect in shear (see Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1, k_4, k_6$ = modification factors given in Section 2

$f'_s$ = characteristic value in shear

$A_s$ = shear plane area (for a rectangular beam loaded about its major axis in bending, $A_s = ⅔(bd)$, where $b$ equals the breadth and $d$ equals the depth of the beam).

In calculating the design action effect in shear, it is appropriate to disregard the design actions located within a distance of 1.5 times the depth of the beam from the inside face of the support. This does not apply for the design of notched beams (see Appendix E).

### 3.2.6 Bearing capacity

#### 3.2.6.1 *Design capacity in bearing perpendicular to the grain*

The design capacity in bearing perpendicular to the grain ($N_{d,p}$) of a structural element (see Figure 3.8), for strength limit state, shall satisfy the following:

$$N_{d,p} \geq N_p^* \quad \ldots 3.2(15)$$

where

$$N_{d,p} = \phi k_1 k_4 k_6 k_7 f'_p A_p \quad \ldots 3.2(16)$$

and

$\phi$ = capacity factor (see Clause 2.3)

$N_p^*$ = design load effect in bearing (see Figure 3.8 and Clause 1.4.2.2)

$k_1$ to $k_7$ = modification factors given in Section 2

$f'_p$ = characteristic value in bearing perpendicular to grain

$A_p$ = bearing area for loading perpendicular to grain.

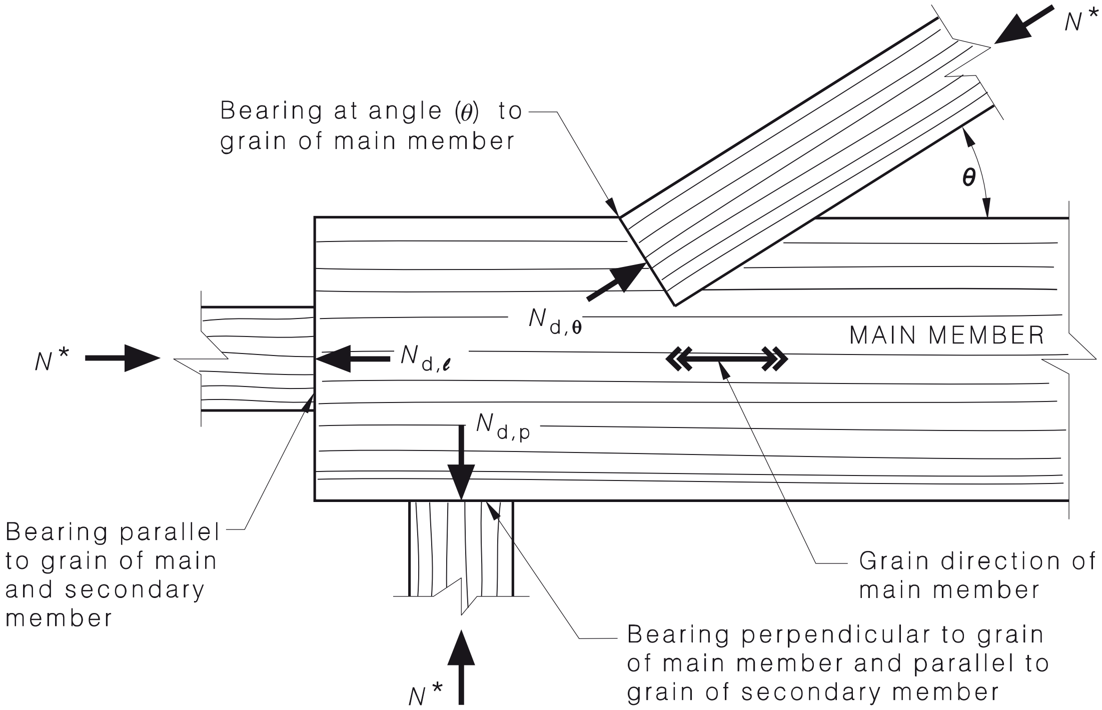



FIGURE 3.8 NOTATION FOR BEARING

**3.2.6.2** *Design capacity in bearing parallel to grain*

The design capacity in bearing parallel to the grain ( $N_{d,\ell}$ ) of a structural element (see Figure 3.8), for strength limit state, shall satisfy the following:

$$N_{d,\ell} \geq N_{\ell}^{*} \qquad \text{. . . 3.2(17)}$$

where

$$N_{d,\ell} = \phi\, k_1\, k_4\, k_6\, f'_{\ell}\, A_{\ell} \qquad \text{. . . 3.2(18)}$$

and

$N_{\ell}^{*}$ = design load effect in bearing (see Figure 3.8 and Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1, k_4, k_6$ = modification factors given in Section 2

$f'_{\ell}$ = characteristic value in bearing parallel to grain

$A_{\ell}$ = bearing area for loading parallel to grain

**3.2.6.3** *Design bearing capacity at an angle to grain*

The design capacity ($N_{d,\theta}$) in bearing at an angle ($\theta$) to the grain of wood is given by the following equation:

$$N_{d,\theta} = \frac{N_{d,\ell}\, N_{d,p}}{N_{d,\ell} \sin^2\theta + N_{d,p} \cos^2\theta} \qquad \text{. . . 3.2(19)}$$

where

$N_{d,\theta}$ = design bearing capacity at an angle to the grain of wood (see Figure 3.8)

$N_{d,\ell}$ = design capacity in bearing parallel to grain (see Clause 3.2.6.2)

$N_{d,p}$ = design capacity in bearing perpendicular to grain (see Clause 3.2.6.1)

### 3.2.7 Strength of notched beams

Requirements for the design strength in notched beams are given in Appendix E.

### 3.2.8 Concentrated loads and partial area loads on grid systems

A method to assist in the design of floor grid systems to resist concentrated and partial area loads is given in Appendix E.

## 3.3 COLUMN DESIGN

### 3.3.1 Compressive strength

**3.3.1.1** *Design compressive capacity parallel to grain*

The design capacity in compression parallel to the grain ($N_{d,c}$) of un-notched columns, for strength limit state, shall satisfy the following:

$$N_{d,c} \geq N_c^* \qquad \text{. . . 3.3(1)}$$

where

$$N_{d,c} = \phi\, k_1\, k_4\, k_6\, k_{12}\, f'_c\, A_c \qquad \text{. . . 3.3(2)}$$

and

$N_c^*$ = design action effect in compression (see Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$, $k_4$, $k_6$ = modification factors given in Section 2

$k_{12}$ = stability factor (see Clause 3.3.3)

$f'_c$ = characteristic value in compression parallel to grain

$A_c$ = cross-sectional area of column

**3.3.1.2** *Buckling about both axes*

For columns that can buckle about both axes, the strength limit states shall satisfy the following:

$$N_{d,cx} \geq N_c^* \qquad \text{. . . 3.3(3)}$$

and

$$N_{d,cy} \geq N_c^* \qquad \text{. . . 3.3(4)}$$

where

$N_{d,cx}$ and $N_{d,cy}$ = design capacity in compression for buckling about the major x-axis and minor y-axis respectively, determined in accordance with Clauses 3.3.1.1 and 3.3.1.2, respectively

**TABLE 3.2**

**EFFECTIVE LENGTH FACTOR ($g_{13}$)
FOR COLUMNS WITHOUT INTERMEDIATE LATERAL RESTRAINT**

| Condition of end restraint | Effective length factor ($g_{13}$) |
|---|---|
| Flat ends | 0.7 |
| Restrained at both ends in position and direction | 0.7 |
| Each end held by two bolts (substantially restrained) | 0.75 |
| One end fixed in position and direction, the other restrained in position only | 0.85 |
| Studs in light framing | 0.9 |
| Restrained at both ends in position only | 1.0 |
| Restrained at one end in position and direction and at the other end partially restrained in direction but not in position | 1.5 |
| Restrained at one end in position and direction but not restrained in either position or direction at other end | 2.0 |

NOTE: 'Flat ends' refers to perfectly flat ends bearing on flat unyielding bases.

### 3.3.2 Slenderness coefficient for lateral buckling under compression

#### 3.3.2.1 *General*

For the general case, and for several useful specific cases, equations for evaluating the slenderness coefficient are given in Paragraph E4, Appendix E. For the case of solid columns of rectangular cross-section as shown in Figure 3.1, the simple approximations given in Clause 3.3.2.2 are acceptable.

#### 3.3.2.2 *Columns of rectangular cross-section*

For columns of rectangular cross-section, the slenderness coefficients are as follows:

(a) *Slenderness coefficient for buckling about the major axis* For the case of discrete restraint systems, the slenderness coefficient, denoted by $S_3$, shall be taken to be the lesser of the following:

$$S_3 = \frac{L_{ax}}{d} \qquad \text{. . . 3.3(5)}$$

and

$$S_3 = \frac{g_{13}\,L}{d} \qquad \text{. . . 3.3(6)}$$

where

$L_{ax}$ = distance between points of effectively rigid restraint between which bending about the major (x) axis would be produced by buckling under load (see Figure 3.9)

$g_{13}$ = coefficient given in Table 3.2

For restraint systems that restrain movement in the direction of the y-axis, and are continuous along the length of the column, the slenderness coefficient shall be taken to be—

$$S_3 = 0.0 \qquad \text{. . . 3.3(7)}$$

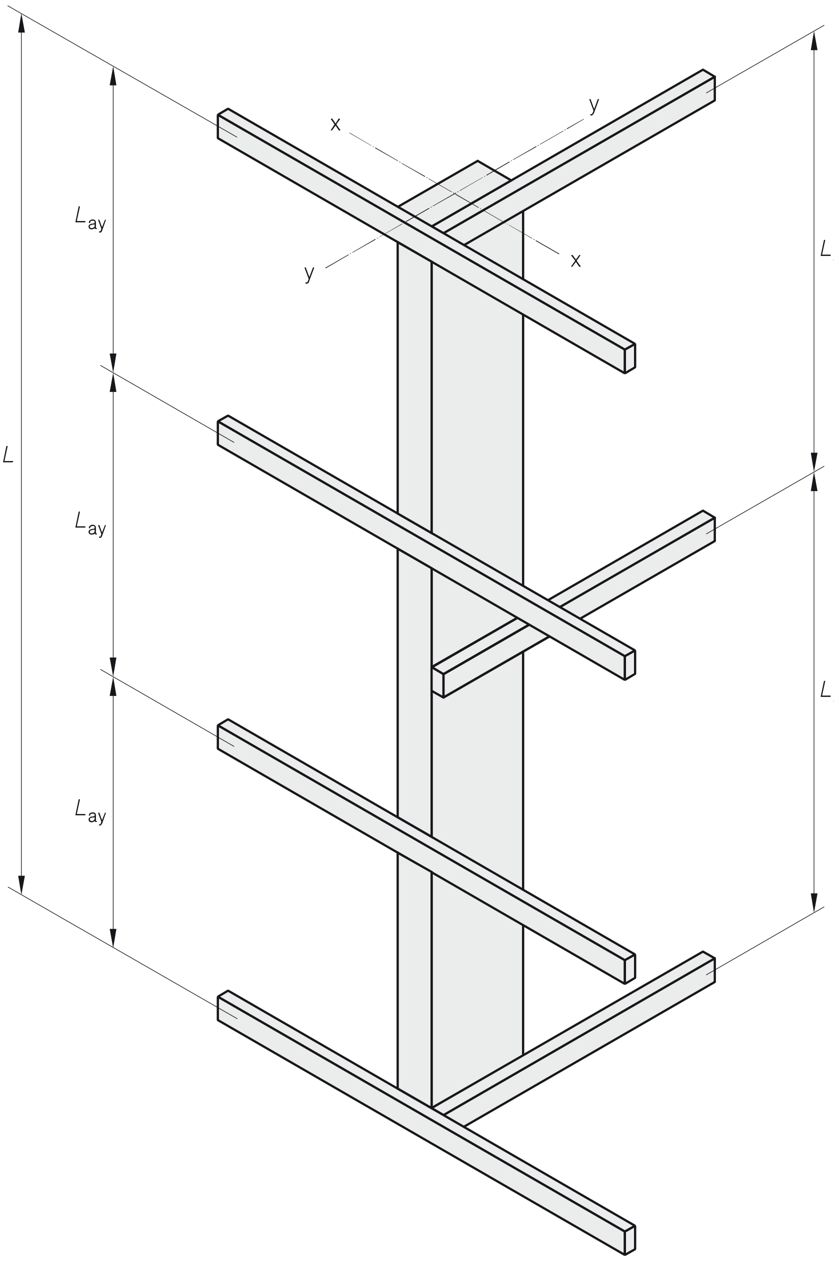



FIGURE 3.9 NOTATION FOR COLUMN RESTRAINTS

(b) *Slenderness coefficient for buckling about the minor axis* For discrete restraint systems, the slenderness coefficient, denoted by $S_4$, shall be taken to be the lesser of the following:

$$S_4 = \frac{L_{ay}}{b} \qquad \text{. . . 3.3(8)}$$

and

$$S_4 = \frac{g_{13}\, L}{b} \qquad \text{. . . 3.3(9)}$$

where

$L_{ay}$ = distance between points of effectively rigid restraint between which bending about the minor ($y$) axis would be produced by buckling under load (see Figure 3.9)

$g_{13}$ = coefficient given in Table 3.2

For restraint systems that act continuously along one edge only and which restrain movement in the direction of the x-axis, the slenderness coefficient is approximated by the following equation:

$$S_4 = \frac{3.5\, d}{b} \qquad \text{. . . 3.3(10)}$$

(c) *Columns that can bend about both axes* The design of such columns, described in Clause 3.3.1.2, is based on an interaction of the two special cases for bending about single axes only, and hence no special definition of slenderness is required for this case.

### 3.3.3 Stability factor

The stability factor ($k_{12}$) for modification of the characteristic value in compression shall be given by the following:

(a) For $\rho_c S \leq 10$—

$$k_{12} = 1.0 \qquad \text{. . . 3.3(11a)}$$

(b) For $10 \leq \rho_c S \leq 20$—

$$k_{12} = 1.5 - 0.05\, \rho_c S \qquad \text{. . . 3.3(11b)}$$

(c) For $\rho_c S \geq 20$—

$$k_{12} = \frac{200}{(\rho_c S)^2} \qquad \text{. . . 3.3(11c)}$$

where

$S$ = $S_3$ for buckling about the major axis

= $S_4$ for buckling about the minor axis

and where a conservative value of the material constant $\rho_c$ is given in Table 3.3; more accurate values of $\rho_c$ are given by Equations E2(3) and E2(4) and tabulated in Tables E3 and E4, Appendix E. The shape of the stability factor curve is illustrated in Figure 3.7.

For material constants for GL-grades and structural laminated veneer lumber, refer to Sections 7 and 8, respectively.

**TABLE 3.3**

**MATERIAL CONSTANT ($\rho_c$) FOR SAWN TIMBER COLUMNS**

| Stress grade | Material constant ($\rho_c$) | |
|---|---|---|
| | Seasoned timber | Unseasoned timber |
| F34 | 1.17 | 1.34 |
| F27 | 1.14 | 1.31 |
| F22 | 1.12 | 1.28 |
| F17 | 1.08 | 1.25 |
| F14 | 1.05 | 1.21 |
| F11 | 1.02 | 1.18 |
| F8 | 1.00 | 1.16 |
| F7 | 0.92 | 1.08 |
| F5 | 0.91 | 1.07 |
| F4 | 0.87 | 1.02 |
| MGP 15 | 0.99 | — |
| MGP 12 | 0.98 | |
| MGP 10 | 0.96 | |
| A17 | 1.10 | — |

NOTES:

1 The values of $\rho_c$ for F-grade seasoned and unseasoned timber correspond to $r = 0.25$ in Tables E3 and E4, Appendix E, respectively.

2 The values of $\rho_c$ for MGP grades and A17 grade seasoned timber correspond to $r = 0.25$ in Table E3, Appendix E.

### 3.3.4 Strength of notched columns

The appropriate design procedure for notched columns shall be as given in Appendix E.

### 3.3.5 Spaced columns

The slenderness coefficients required for computing the design axial strength of spaced columns shall be as given in Appendix E.

## 3.4 TENSION MEMBER DESIGN

### 3.4.1 Design tensile capacity parallel to grain

The design capacity in tension parallel to grain ($N_{d,t}$) of un-notched tension members, for strength limit state, shall satisfy the following:

$$N_{d,t} \geq N_t^* \qquad \ldots 3.4(1)$$

where

$$N_{d,t} = \phi \, k_1 \, k_4 \, k_6 \, f_t' \, A_t \qquad \ldots 3.4(2)$$

$N_t^*$ = design action effect in tension parallel to grain (see Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ to $k_6$ = modification factors given in Section 2

$f_t'$ = characteristic value in tension parallel to grain

$A_t$ = net cross-sectional area of tension member

The appropriate design procedure for the design of notched tension members is given in Appendix E.

### 3.4.2 Design capacity in tension perpendicular to grain

The design capacity in tension perpendicular to grain ($N_{d,tp}$) of un-notched members, for strength limit, shall satisfy the following:

$$N_{d,tp} \geq N_{p}^{*} \qquad \ldots 3.4(3)$$

where

$$N_{d,tp} = \phi\, k_1\, k_{11}\, f'_{tp}\, A_{tp} \qquad \ldots 3.4(4)$$

and

$N_{tp}^{*}$ = design action effect produced by strength limit states design loads (tension perpendicular to grain) (see Clause 1.4.2.2)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = modification factors for duration of load, given in Section 2

$k_{11}$ = modification factors for the effect of volume = $(V_o/V)^{0.2}$

where $V_o = 10^7$ (a reference volume) and $V$ is the volume, in $mm^3$, stressed in tension perpendicular to the grain at a level of stress greater than 80% of the maximum stress

$f'_{tp}$ = characteristic value in tension perpendicular to grain

$A_{tp}$ = member width (thickness) by effective length stressed in tension (see Figure 3.10), which is the effective area resisting tension perpendicular to the grain

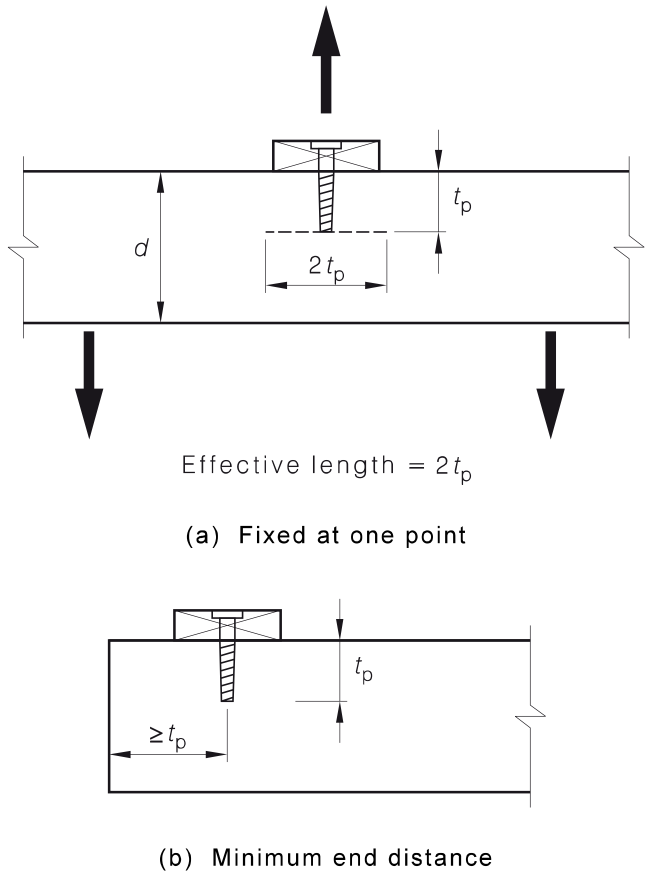



FIGURE 3.10 EFFECTIVE LENGTH STRESSED IN TENSION PERPENDICULAR TO GRAIN

## 3.5 COMBINED BENDING AND AXIAL ACTIONS

### 3.5.1 Combined bending and compression

Rectangular members with cross-section as shown in Figure 3.1 subject to combined axial compression and bending about the x-axis only, shall be proportioned so that—

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{N_c^*}{N_{d,cy}} \leq 1 \qquad \ldots 3.5(1)$$

and

$$\frac{M_x^*}{M_{d,x}} + \frac{N_c^*}{N_{d,cx}} \leq 1 \qquad \ldots 3.5(2)$$

where

$M_x^*$ = design action effect in bending about the major principal x-axis

$M_{d,x}$ = design capacity in bending about the major principal x-axis (see Clause 3.2.1.1)

$N_c^*$ = design action effect in compression, (see Clause 1.4.2.2)

$N_{d,cx}$ = design capacity in compression for buckling about the x-axis.

$N_{d,cy}$ = design capacity in compression for buckling about the y-axis

NOTE: Equations 3.5(1) and 3.5(2) contain an allowance for the effect of bending moment amplification due to the axial load. For non-rectangular members, Equations 3.5(1) and 3.5(2) may be used in the absence of other information.

For the unusual case of a beam-column subjected to bending simultaneously about both the x- and y-axes, a conservative criterion of strength is given in Appendix E.

### 3.5.2 Combined bending and tension

For a rectangular member subject to combined bending and tension, action effects shall be proportioned so that—

$$\frac{k_{12}\, M^*}{M_d} + \frac{N_t^*}{N_{d,t}} \leq 1 \qquad \ldots 3.5(3)$$

and

$$\frac{M_x^*}{M_{d,x}} - \frac{Z}{A}\frac{N_t^*}{M_{d,x}} \leq 1 \qquad \ldots 3.5(4)$$

where

$k_{12}$ = stability factor used in bending strength calculation (see Clause 3.2.4)

$M^*$ = design action effect in bending about the appropriate axis (see Clause 1.4.2.2)

$M_d$ = design capacity in bending about the appropriate axis (see Clause 3.2.1.1)

$N_t^*$ = design action effect in tension (see Clause 1.4.2.2)

$N_{d,t}$ = design capacity of a member in tension (see Clause 3.4.1)

$M_x^*$ = design action effect in bending about the major principal axis (see Clause 3.2.1.1)

$M_{d,x}$ = design capacity in bending about the major principal axis (see Clause 3.2.1.1)

$Z$ = section modulus about the appropriate axis

$A$ = cross-sectional area.

# SECTION 4 DESIGN CAPACITY OF JOINTS IN TIMBER STRUCTURES

## 4.1 GENERAL

### 4.1.1 Scope of Section

This Section applies to joints in solid timber fabricated with mechanical fasteners and characterized by a long history of use in timber structures. These include joints fabricated with the following mechanical fasteners:

(a) Nails.

(b) Wood screws.

(c) Bolts.

(d) Coach screws.

(e) Split-ring fasteners.

(f) Shear-plate fasteners.

Joints with plywood and the calculation of characteristic capacities for bolts are covered in Appendix C.

NOTES:

1 Appendix C gives further information on joints.

2 Testing of specialized and patented mechanical fasteners and variants of conventional fasteners is specified in AS 1649.

3 This Standard does not specifically cover glued timber-to-timber or timber-to-plywood connections as occur in fabricated components such as stressed skin panels or plywood webbed beams. In such cases, joint design may be based on the timber components in the connection, provided the joint is fabricated using a rigid, durable adhesive. Phenolic type adhesives meet these requirements. The design of fabricated components comprising glued connections is therefore based on the fact that with correct bonding practice and quality control, a joint is developed in which the adhesive bond strength and durability will be superior to the timber components comprising the joint.

4 Information on methods for assessing the deformation of joints is given in Paragraph C3, Appendix C.

### 4.1.2 Joint groups

For the purpose of joint design, timber species have been classified into six joint groups: J1, J2, J3, J4, J5 and J6 for unseasoned timber, and JD1, JD2, JD3, JD4, JD5 and JD6 for seasoned timber. The joint group classifications for timber species and moisture condition shall be as listed in Tables H2.3 and H2.4, Appendix H.

The joint group appropriate for the calculation of the design capacity of the joint shall be determined such that—

(a) where a joint comprises more than one species of timber, the species with the lowest joint group classification shall be used to calculate the capacity of the joint; except

(b) where the capacity of each part of the joint can be determined independently, it is permissible to calculate the design capacity of the joint as the lesser of the capacities of the individual parts based upon their individual joint group classification.

### 4.1.3 Joint types

For the purpose of joint design, joints are classified into two types as follows:

(a) *Type 1 joint* Fasteners subject to shear loads where the fastener is into the side or end grain of connected members (see Figure 4.1 and Figures 4.2(a) and 4.2(b)).

(b) *Type 2 joint* Fasteners subject to axial loads where the fastener is installed into the side or end grain of connected members (see Figure 4.1 and Figures 4.4(a), 4.4(b), and 4.4(c)).

NOTE: Where timber elements experience tension perpendicular to the grain, reference should be made to Clause 3.4.2.

Design capacities for Type 1 and Type 2 joints of various configurations are given in Clauses 4.2 to 4.7.

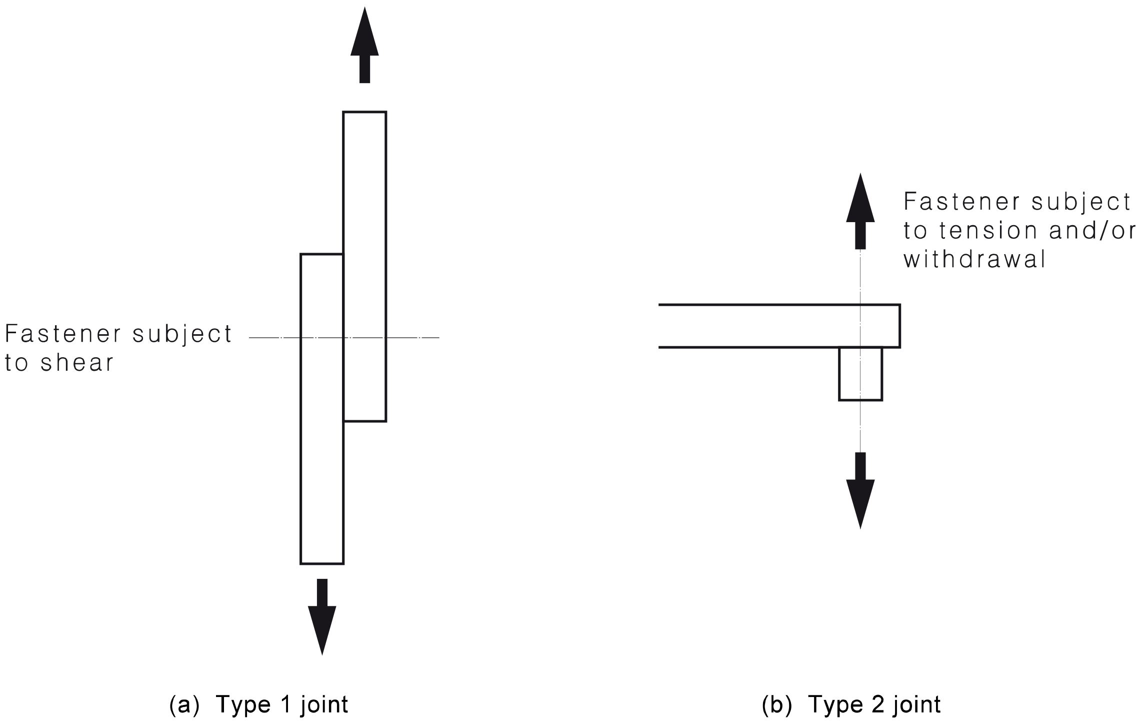



FIGURE 4.1 JOINT TYPES

### 4.1.4 Strength-reducing characteristics at joints

The characteristic capacities for fasteners in timber given in this Standard do not include any allowance for the presence of strength-reducing characteristics in the vicinity of the fastener. Fasteners shall be placed at locations where there are no significant strength-reducing characteristics or appropriate reductions shall be made to the capacity of the joint. Where joints are made in timber containing corewood, the characteristic capacities for fasteners shall be taken to correspond to the next lowest joint group to that assigned to the species of timber free of corewood.

### 4.1.5 Tendency to split

Special precautions shall be specified in the use of timber that has a tendency to split to an extent that is detrimental to fastener strength (see Clauses 4.2.4 and 4.2.6). In the absence of other guidance, the criterion for tendency to split shall be based on the parameter α defined by the following equation:

$$\alpha = \varepsilon^2/\gamma \quad \text{. . . 4.1(1)}$$

where

$\varepsilon$ = tangential shrinkage, in percentage

$\gamma$ = tangential cleavage strength of unseasoned timber, in newtons per millimetre, as measured by BS 373 or ASTM D143

Species for which $\alpha > 0.8$ often have a high tendency to split, particularly in exposed locations; species for which $\alpha < 0.55$ usually have little tendency to split.

NOTES:

1 Most eucalypt and corymbia species and most hardwood species of dry sclerophyll forests that have a basic density of less than 700 kg/m$^3$ have a splitting parameter $\alpha$>0.8; most softwoods and most rainforest hardwoods have a splitting parameter $\alpha$<0.8.

2 Information on shrinkage and cleavage strength for specific species can be obtained from the following:

(a) KINGSTON, R.S.T. and RISDON, C.J.E. *Shrinkage and Density of Australian and other South-west Pacific Woods*. Division of Forest Products Technological Paper No. 13, CSIRO, 1961.

(b) BUDGEN, B. *Shrinkage and Density of some Australian and South-east Asian Timbers*. Division of Building Research Technological Paper (Second Series) No. 38, CSIRO, 1981.

(c) BOLZA, E. and KLOOT, N.H. *The Mechanical Properties of 174 Australian Timbers*. Division of Forest Products Technological Paper No. 25, CSIRO, 1963.

### 4.1.6 Eccentric joints

When it is impracticable to ensure that all the members meeting at a joint are arranged symmetrically, with their centre-lines intersecting on a common axis, which is also the axis of resistance of the fastener or group of fasteners, the combined effects of primary stresses and secondary stresses due to the resulting bending and shear stress shall be checked.

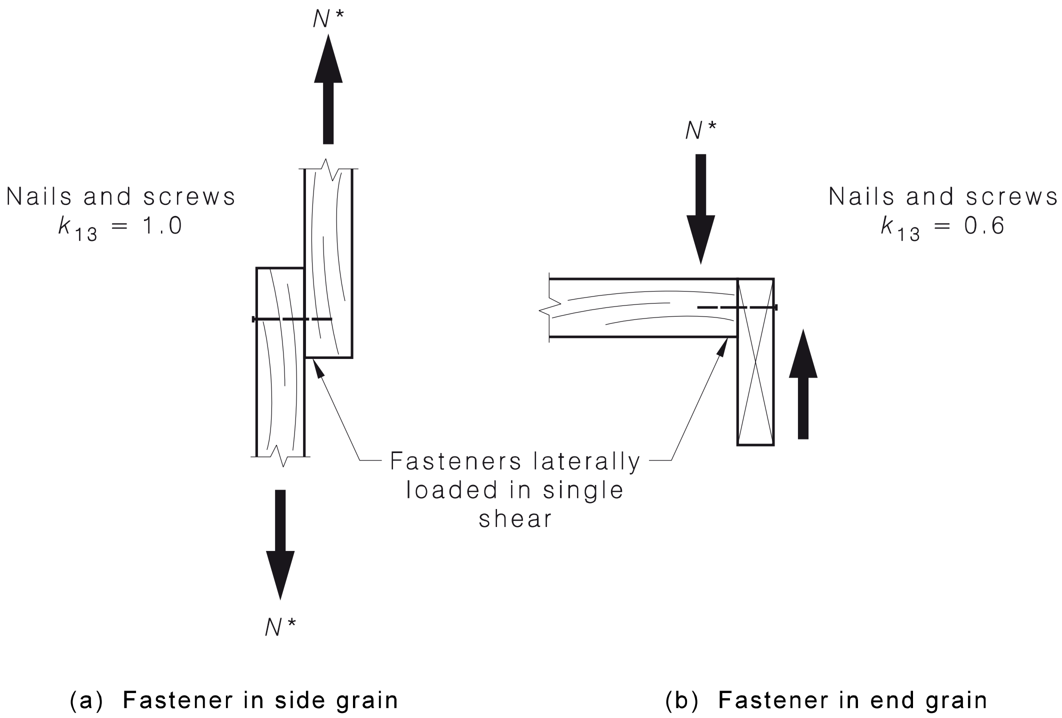



FIGURE 4.2 TYPE 1 JOINTS FOR NAILS AND SCREWS LOADED IN SHEAR

## 4.2 DESIGN OF NAILED JOINTS

### 4.2.1 General

The characteristic capacities for nails, listed in Tables 4.1 and 4.2, apply to plain shank low carbon steel nails, as specified in AS 2334.

### 4.2.2 Characteristic capacities for nails

**4.2.2.1** General

The characteristic capacities for nails are applicable for load directions at any angle to the grain (see Figure 4.3) and are given in Clauses 4.2.2.2 to 4.2.2.3.

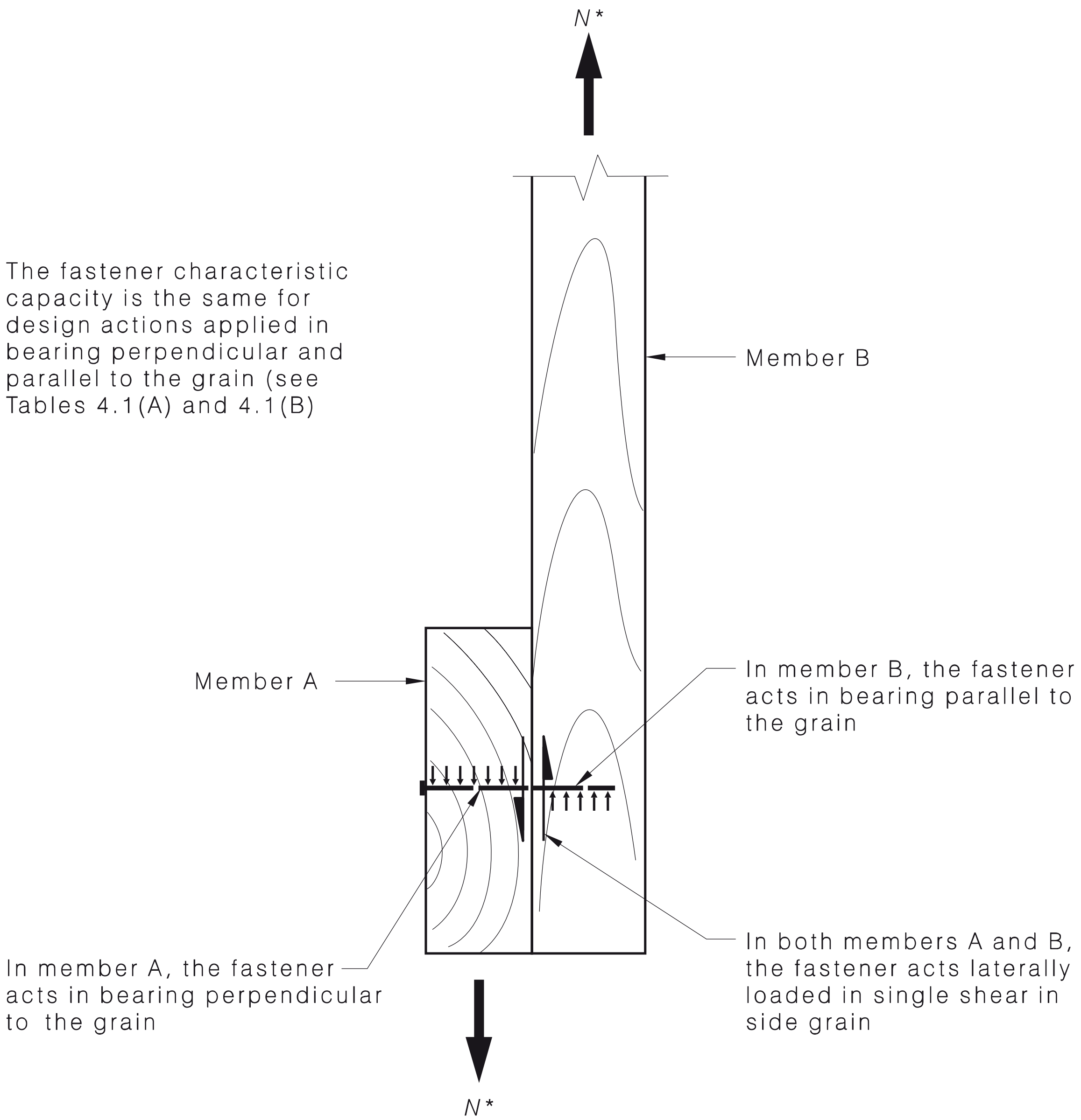



DIAGRAM ILLUSTRATES A TYPE 1 JOINT

FIGURE 4.3 FASTENER (NAIL OR SCREW) IN BEARING PERPENDICULAR AND PARALLEL TO THE GRAIN

**4.2.2.2** *Type 1 joints*

Characteristic capacities for Type 1 joints are as follows:

(a) *Lateral loads in side grain (see Figure* 4.2*(a))* The characteristic capacities for nails, whether hand or machine driven, laterally loaded in single shear in the side grain of unseasoned timber are given in Table 4.1(A) and in seasoned timber are given in Table 4.1(B).

(b) *Lateral loads in end grain (see Figure* 4.2*(b))* The characteristic capacities for nails, whether hand or machine driven, laterally loaded in single shear in the end grain of unseasoned and seasoned timber shall be taken as 60% of the values given in Tables 4.1(A) and 4.1(B), respectively.

NOTE: The modification factor $k_{13}$ accounts for end grain effects in Equations 4.2(2) and 4.2(4) (see Clause 4.2.3.2 and 4.2.3.3).

**TABLE 4.1(A)**

**CHARACTERISTIC CAPACITY ($Q_k$)—SINGLE PLAIN SHANK STEEL NAIL LATERALLY LOADED IN SINGLE SHEAR IN SIDE GRAIN—UNSEASONED TIMBER**

| Species group | Characteristic capacity per nail, N | | | | | | |
|---|---|---|---|---|---|---|---|
| | Nail diameter, mm | | | | | | |
| | 2.5 | 2.8 | 3.15 | 3.75 | 4.5 | 5.0 | 5.6 |
| J1 | 975 | 1180 | 1445 | 1960 | 2700 | 3245 | 3955 |
| J2 | 765 | 930 | 1135 | 1550 | 2125 | 2565 | 3125 |
| J3 | 545 | 665 | 810 | 1105 | 1520 | 1830 | 2225 |
| J4 | 385 | 470 | 575 | 780 | 1075 | 1300 | 1595 |
| J5 | 295 | 355 | 445 | 590 | 810 | 975 | 1180 |
| J6 | 220 | 265 | 325 | 445 | 620 | 740 | 885 |

**TABLE 4.1(B)**

**CHARACTERISTIC CAPACITY ($Q_k$)—SINGLE PLAIN SHANK STEEL NAIL LATERALLY LOADED IN SINGLE SHEAR IN SIDE GRAIN— SEASONED TIMBER**

| Species group | Characteristic capacity per nail, N | | | | | | |
|---|---|---|---|---|---|---|---|
| | Nail diameter, mm | | | | | | |
| | 2.5 | 2.8 | 3.15 | 3.75 | 4.5 | 5.0 | 5.6 |
| JD1 | 1285 | 1565 | 1920 | 2610 | 3570 | 4310 | 5250 |
| JD2 | 975 | 1180 | 1445 | 1960 | 2700 | 3245 | 3955 |
| JD3 | 765 | 930 | 1135 | 1550 | 2125 | 2565 | 3125 |
| JD4 | 545 | 665 | 810 | 1110 | 1520 | 1830 | 2225 |
| JD5 | 445 | 545 | 680 | 915 | 1255 | 1505 | 1830 |
| JD6 | 340 | 415 | 500 | 695 | 945 | 1135 | 1385 |

NOTE: See Clause 4.2.5 for nail length and timber thicknesses.

**4.2.2.3** *Type 2 joints*

Characteristic capacities for Type 2 joints are as follows:

(a) *Withdrawal loads from side grain (see Figure* 4.4*(a))* The characteristic capacities for nails, driven by hand only, in withdrawal from the side grain of unseasoned timber are given in Table 4.2(A) and in seasoned timber are given in Table 4.2(B).

NOTE: Withdrawal loads for gun-driven plain shank nails may be considerably less than withdrawal loads for the same nails driven by hand.

(b) *Withdrawal loads from end grain (see Figure* 4.4*(b))* The characteristic capacities for nails, driven by hand only, axially loaded in withdrawal from the end grain of unseasoned and seasoned timber, shall not exceed 25% of the values given in Tables 4.2(A) and 4.2(B) respectively. A minimum of two nails shall be used in end grain of Type 2 joints.

For skew driven nails (see Figure 4.4(c)), the characteristic capacities for nails in withdrawal from the end grain of unseasoned and seasoned timber shall not exceed 60% of the values given in Tables 4.2(A) and 4.2(B) respectively. A minimum of two (oppositely driven) nails shall be used in end grain of Type 2 joints.

NOTE: The modification factor ($k_{13}$) accounts for end grain effects in Equation 4.2(6) (see Clause 4.2.3.4).

**TABLE 4.2(A)**

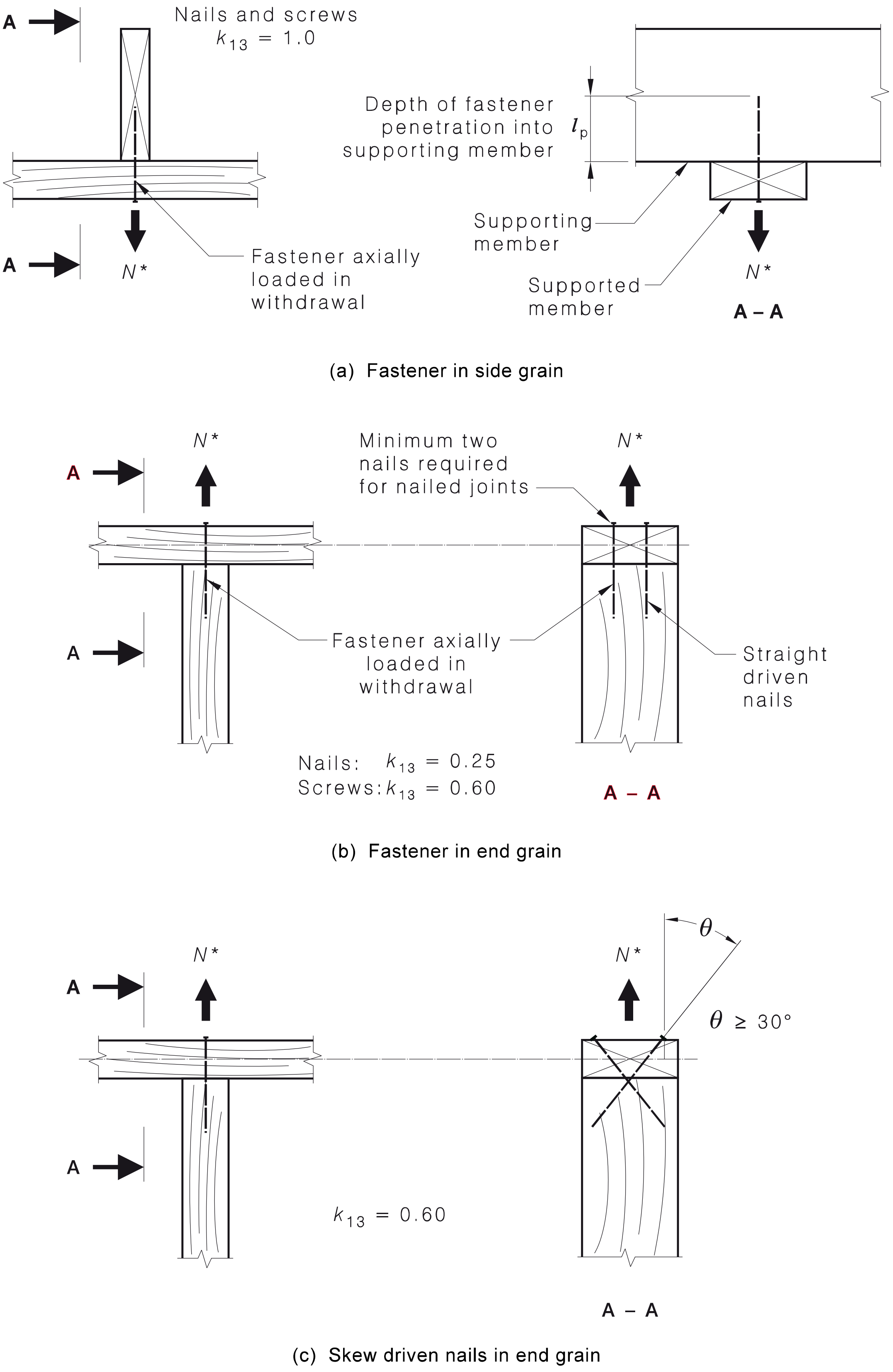
**CHARACTERISTIC CAPACITY ($Q_k$)—SINGLE PLAIN SHANK STEEL NAIL IN WITHDRAWAL AXIALLY LOADED IN SIDE GRAIN—UNSEASONED TIMBER**

| Species group | Characteristic capacity, N per mm penetration of nail | | | | | | |
|---|---|---|---|---|---|---|---|
| | Nail diameter, mm | | | | | | |
| | 2.5 | 2.8 | 3.15 | 3.75 | 4.5 | 5.0 | 5.6 |
| J1 | 19 | 20 | 24 | 27 | 32 | 35 | 41 |
| J2 | 15 | 17 | 19 | 22 | 27 | 30 | 34 |
| J3 | 13 | 14 | 16 | 19 | 24 | 25 | 29 |
| J4 | 11 | 13 | 14 | 17 | 20 | 22 | 25 |
| J5 | 9.1 | 10 | 11 | 14 | 17 | 19 | 20 |
| J6 | 6.8 | 7.6 | 8.6 | 10 | 12 | 14 | 15 |

**TABLE 4.2(B)**

**CHARACTERISTIC CAPACITY ($Q_k$)—SINGLE PLAIN SHANK STEEL NAIL IN WITHDRAWAL AXIALLY LOADED IN SIDE GRAIN—SEASONED TIMBER**

| Species group | Characteristic capacity, N per mm penetration of nail | | | | | | |
|---|---|---|---|---|---|---|---|
| | Nail diameter, mm | | | | | | |
| | 2.5 | 2.8 | 3.15 | 3.75 | 4.5 | 5.0 | 5.6 |
| JD1 | 29 | 32 | 35 | 42 | 51 | 57 | 64 |
| JD2 | 20 | 22 | 25 | 29 | 35 | 39 | 44 |
| JD3 | 13 | 14 | 16 | 19 | 24 | 25 | 29 |
| JD4 | 8.3 | 9.3 | 10 | 13 | 15 | 17 | 19 |
| JD5 | 5.2 | 5.9 | 6.6 | 7.9 | 9.5 | 11 | 12 |
| JD6 | 3.7 | 4.2 | 4.7 | 5.6 | 6.8 | 7.6 | 8.5 |




NOTE: Flat-head nails should be used where head pull-through may otherwise precede withdrawal and limit axial load carrying capacity; this is more likely with softwoods and lower density hardwoods.

FIGURE 4.4 TYPE 2 JOINTS FOR NAILS AND SCREWS (AND COACH SCREWS) LOADED IN WITHDRAWAL

### 4.2.3 Design capacity for nailed joints

#### 4.2.3.1 *General*

The design capacity for Type 1 and Type 2 nailed joints shall be calculated in accordance with Clauses 4.2.3.2 and 4.2.3.3.

#### 4.2.3.2 *Type 1 joint to resist direct loads*

The design capacity ($N_{d,j}$) for a Type 1 joint containing *n* nails designed to resist direct loads, as illustrated in Figure 4.5(a), for strength limit states shall satisfy the following:

$$N_{d,j} \geq N^{*} \quad \ldots 4.2(1)$$

where

$$N_{d,j} = \phi\, k_1\, k_{13}\, k_{14}\, k_{16}\, k_{17}\, n\, Q_k \quad \ldots 4.2(2)$$

and

$N^{*}$ = design action effect on joint

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = the factor for duration of load for joints (see Clause 2.4.1.1)

$k_{13}$ = 1.0 for nails in side grain (see Figure 4.2(a))

= 0.6 for nails in end grain (see Figure 4.2(b))

$k_{14}$ = 1.0 for nails in single shear (see Figure 4.6(a))

= 2.0 for nails in double shear(see Figure 4.6(b))

$k_{16}$ = 1.2 for nails driven through close fitting holes into metal side plates

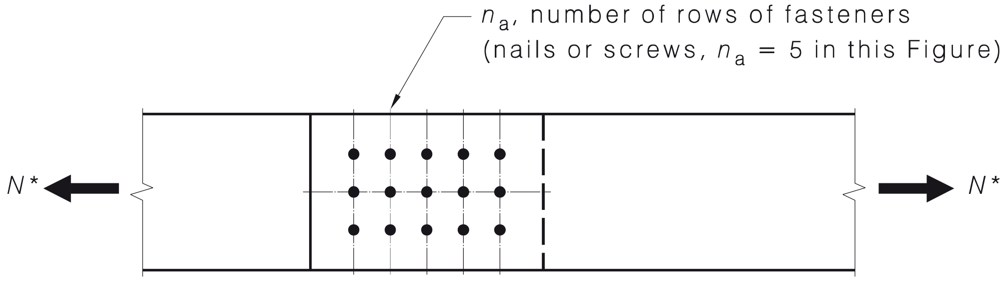
= 1.1 for nails driven through plywood gussets

= 1.0 otherwise

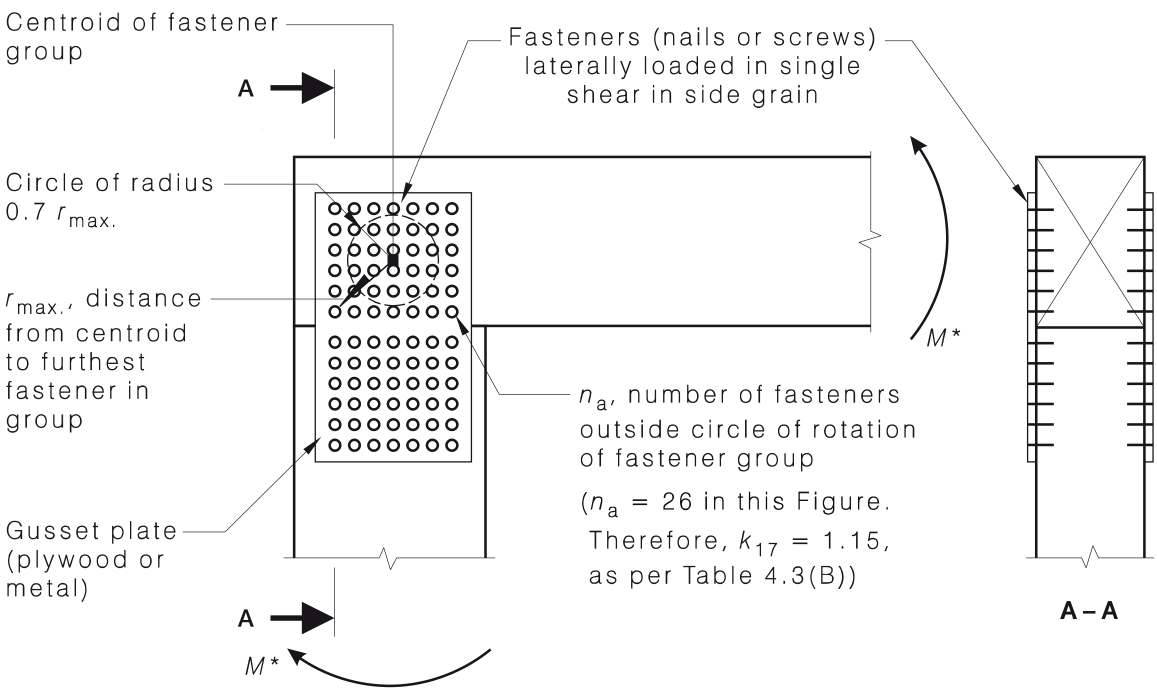
$k_{17}$ = factor for multiple nailed joints given in Table 4.3(A) for Type 1 joints designed to resist direct loads in either tension or compression

$n$ = total number of nails in connection resisting design action effect in shear

$Q_k$ = characteristic capacity given in Table 4.1(A) and Table 4.1(B). See also Clauses 4.2.2 and 4.2.5

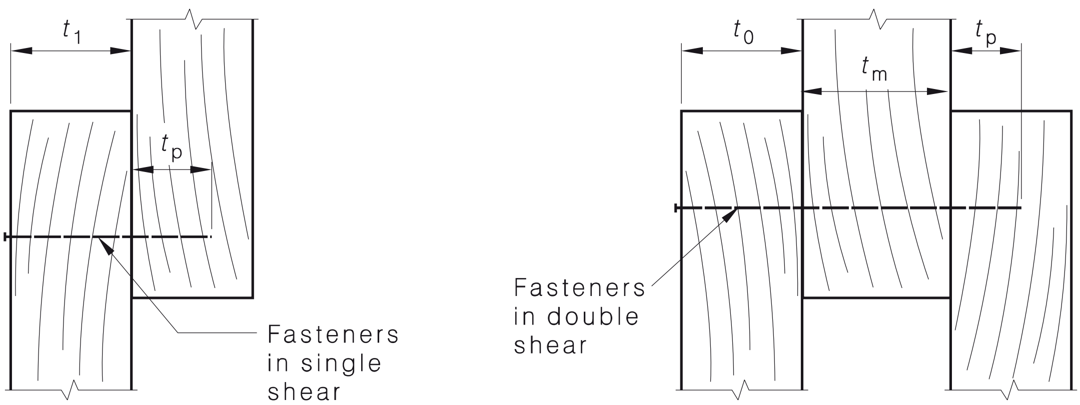



(a) Type 1 joint-resisting direct loads




(b) Type 1 joint-resisting in-plane moments

FIGURE 4.5 DETERMINATION OF MODIFICATION FACTOR $k_{17}$ FOR USE IN THE DESIGN OF TYPE 1 JOINTS CONTAINING A MULTIPLE NUMBER OF FASTENERS (NAILS OR SCREWS)




(a) Two-member Type 1 joint (b) Three-member Type 1 joint

FIGURE 4.6 TIMBER THICKNESSES AND FASTENER LENGTHS FOR NAILS, WOOD SCREWS AND COACH SCREWS

### TABLE 4.3(A)

### VALUES OF FACTOR $k_{17}$ FOR USE IN THE DESIGN OF MULTIPLE NAIL AND SCREW JOINTS—TO RESIST DIRECT LOADS

| Condition of timber | Values of $k_{17}$ | | | |
|---|---|---|---|---|
| | Number of rows of fasteners* | | | |
| | $n_a \leq 4$ | $n_a = 5$ | $n_a = 10$ | $n_a \geq 20$ |
| Unseasoned | 1.00 | 0.90 | 0.80 | 0.75 |
| Seasoned | 1.00 | 0.94 | 0.90 | 0.85 |

* $n_a$ refers to the number of rows of fasteners.

### TABLE 4.3(B)

### VALUES OF FACTOR $k_{17}$ FOR USE IN THE DESIGN OF MULTIPLE NAIL AND SCREW JOINTS—TO RESIST IN-PLANE MOMENTS

| $n$* | $k_{17}$ |
|---|---|
| 2 | 1.00 |
| 5 | 1.05 |
| 10 | 1.10 |
| 20 | 1.15 |
| 100 or greater | 1.20 |

* $n$ refers to the number of nails per interface for which $r_i / r_{max.} \geq 0.7$.

#### 4.2.3.3 *Type 1 joint to resist in-plane moment*

The design capacity ($M_{d,j}$) per shear plane interface for a Type 1 joint containing $n$ nails designed to resist in-plane moment, as illustrated in Figure 4.5(b), for strength limit states shall satisfy the following:

$$M_{d,j} \geq M^* \qquad \ldots 4.2(3)$$

where

$$M_{d,j} = \phi k_1 k_{13} k_{14} k_{16} k_{17} r_{max.} Q_k \sum_{i=1}^{n} \left( \frac{r_i}{r_{max.}} \right)^{3/2} \qquad \ldots 4.2(4)$$

$M^*$ = design action effect on joint (in-plane moment)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for joints (see Clause 2.4.1.1)

$k_{13}$ = 1.0 for nails in side grain (see Figure 4.2(a))

= 0.6 for nails in end grain (see Figure 4.2(b))

$k_{14}$ = 1.0 for nails in single shear (see Figure 4.6(a))

= 2.0 for nails in double shear (see Figure 4.6(b))

$k_{16}$ = 1.2 for nails driven through close fitting holes into metal side plates

= 1.1 for nails driven through plywood gussets

= 1.0 otherwise

$k_{17}$ = factor for multiple nailed joints given in Table 4.3(B) for Type 1 joints to resist in-plane moments

$r_{max.}$ = maximum value of $r_i$

$Q_k$ = characteristic capacity given in Table 4.1(A) and Table 4.1(B). See also Clauses 4.2.2 and 4.2.5

$n$ = total number of nails in single shear per interface resisting the design in-plane moment

$r_i$ = distance from the $i$-th nail to the centroid of the nail group

#### 4.2.3.4 *Type 2 joint*

The design capacity ($N_{d,j}$) for a Type 2 joint containing $n$ nails designed to resist axial loads tending to cause withdrawal, as illustrated in Figure 4.4, for strength limit states shall satisfy the following:

$$N_{d,j} \geq N^* \qquad \text{. . . 4.2(5)}$$

where

$$N_{d,j} = \phi\, k_{13}\, l_p\, n\, Q_k \qquad \text{. . . 4.2(6)}$$

$N^*$ = design action effect in the axial direction of the fastener

$\phi$ = capacity factor (see Clause 2.3)

$k_{13}$ = 1.0 for withdrawal from side grain (see Figure 4.4a))

= 0.25 for withdrawal from end grain for straight driven nails (see Figure 4.4b))

= 0.6 for withdrawal from end grain for skew driven nails (see Figure 4.4(c))

$l_p$ = depth of nail penetration, in millimetres, into supporting member (see Figure 4.4(a))

$n$ = total number of nails in the joint

$Q_k$ = characteristic capacity given in Tables 4.2(A) and Table 4.2(B). See also Clauses 4.2.2 and 4.2.5

NOTE: The duration of load factor $k_1$ does not apply to nails subject to the action of withdrawal loads.

### 4.2.4 Spacing, edge and end distances for nails

Table 4.4 provides recommended minimum spacings, edge and end distances for nails in terms of nail diameter ($D$). For spacings at an angle to the grain, interpolation by means of Hankinson's formula is appropriate. Where it can be demonstrated that splitting will not occur, it is appropriate to use spacings and end distances less than those in Table 4.4.

NOTE: For timber that has a tendency to split (see Clause 4.1.5), some mitigation measures (such as preboring or increased spacing) are recommended (see Clause 4.2.6). The fabrication of prototype joints is a useful method of checking the efficacy of mitigation measures.

**TABLE 4.4**

**MINIMUM SPACING, EDGE AND END DISTANCES FOR NAILS**

| Spacing type | Minimum distance | |
|---|---|---|
| | Holes not prebored | Holes prebored to 80% of nail diameter |
| End distance | 20*D* | 10*D* |
| Edge distance | 5*D* | 5*D* |
| Between nails | | |
| —along grain | 20*D* | 10*D* |
| —across grain | 10*D* | 3*D* |

### 4.2.5 Nail length and timber thickness

The characteristic capacities given in Tables 4.1(A) and 4.1(B) shall be applicable, where timber thicknesses and nail length as shown in Figure 4.6 are as follows:

(a) *Two-member joints (nails in single shear)* Thickness of first member, $t_l > 10D$; depth of penetration of nail into second member, $t_p > 10D$.

For lesser values of $t_l$ and $t_p$, the basic load shall be reduced in proportion to the decrease in $t_l$ or $t_p$ (whichever gives the greater decrease), and the nails shall be considered as non-loadbearing if $t_l$ or $t_p$ is less than $5D$.

NOTE: These limits do not apply to plywood (see Appendix C).

(b) *Three-member joints (nails in double shear)* Thickness of central member, $t_m > 10D$; thickness to outer member, $t_o > 7.5D$; depth of penetration of nail into outer member, $t_p > 7.5D$.

For lesser values of $t_m$, $t_o$ and $t_p$, the basic load shall be reduced in proportion to the decrease in $t_m$, $t_o$ and $t_p$ (whichever gives the greatest decrease), and the nails shall be regarded as being in single shear if $t_p$ is less than $5D$.

NOTE: These limits do not apply to plywood (see Appendix C).

### 4.2.6 Avoidance of splitting

The characteristic capacities for nails have been derived on the assumption that splitting of the timber does not occur to any significant extent. In unseasoned timber that shows a marked tendency to split (see Clauses 4.1.5 and 4.2.4), the use of prebored holes of diameter 80% of the nail diameter is recommended.

## 4.3 DESIGN OF SCREWED JOINTS

### 4.3.1 General

The characteristic capacities for screws are listed in Tables 4.5 and 4.6. These capacities may also apply for type 17 self-drilling screws, manufactured in accordance with the AS 3566 series.

### 4.3.2 Characteristic capacities for screws

#### 4.3.2.1 *General*

The characteristic capacities for screws (machine and hand driven) are applicable for load direction at any angle to the grain (see Figure 4.2) and shall be as given in Clauses 4.4.2.2 to 4.4.2.5.

#### 4.3.2.2 *Type 1 joints*

Characteristic capacities for Type 1 joints shall be as follows:

(a) *Lateral loads in side grain, joint (see Figure 4.2(a))* The characteristic capacities for screws, laterally loaded in single shear in the side grain of unseasoned timber shall be as given in Table 4.5(A) and in seasoned timber shall be as given in Table 4.5(B).

(b) *Lateral loads in end grain (see Figure 4.2(b))* The characteristic capacities for screws, laterally loaded in single shear in the end grain of unseasoned and seasoned timber shall be taken as 60% of the values given in Tables 4.5(A) and 4.5(B) respectively.

NOTE: The modification factor $k_{13}$ accounts for end grain effects in Equations 4.3(2) and 4.3(4). See Clauses 4.3.3.2 and 4.3.3.3.

#### 4.3.2.3 *Type 2 joints*

Characteristic capacities for Type 2 joints shall be as follows:

(a) *Withdrawal loads from side grain (see Figure* 4.4*(a))* The characteristic capacities for screws, axially loaded in withdrawal from the side grain of unseasoned timber shall be as given in Table 4.6(A) and in seasoned timber shall be as given in Table 4.6(B).

(b) *Withdrawal loads from end grain (see Figure* 4.4*(b))* The characteristic capacities for screws, axially loaded in withdrawal from the end grain of unseasoned and seasoned timber shall not exceed 60% of the values given in Tables 4.6(A) and 4.6(B) respectively.

NOTE: The modification factor $k_{13}$ accounts for end grain effects in Equation 4.3(6). See Clause 4.3.3.4.

**4.3.2.4** *Maximum tensile load capacity*

The maximum tensile load capacity for a screw subject to direct axial loading shall not exceed the value appropriate to the diameter and metal from which the screw is manufactured as given in Table 4.7.

**4.3.2.5** *Interpolation*

The characteristic capacity for other screw diameters shall be as appropriately determined by linear interpolation in Tables 4.5(A), 4.5(B), 4.6(A), 4.6(B) and 4.7.

NOTE: For Type 17 screws, screw size number may be used to select capacity.

**TABLE 4.5(A)**

**CHARACTERISTIC CAPACITY FOR A SINGLE SCREW IN SIDE GRAIN LATERALLY LOADED IN SINGLE SHEAR—UNSEASONED TIMBER**

| Joint group | Characteristic capacity per screw, N | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screw size number | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 18 |
| | Shank diameter, mm | | | | | | |
| | 2.74 | 3.45 | 4.17 | 4.88 | 5.59 | 6.3 | 7.72 |
| J1 | 1 280 | 1 950 | 2 700 | 3 570 | 4 520 | 5 560 | 7 950 |
| J2 | 1 010 | 1 520 | 2 120 | 2 800 | 3 570 | 4 380 | 6 270 |
| J3 | 710 | 1 080 | 1 520 | 2 020 | 2 530 | 3 130 | 4 480 |
| J4 | 510 | 780 | 1 080 | 1 420 | 1 790 | 2 220 | 3 170 |
| J5 | 370 | 570 | 780 | 1 010 | 1 310 | 1 620 | 2 290 |
| J6 | 240 | 370 | 510 | 670 | 840 | 1 040 | 1 480 |

**TABLE 4.5(B)**

**CHARACTERISTIC CAPACITY FOR A SINGLE SCREW IN SIDE GRAIN LATERALLY LOADED IN SINGLE SHEAR—SEASONED TIMBER**

| Joint group | Characteristic capacity per screw, N | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screw size number | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 18 |
| | Shank diameter, mm | | | | | | |
| | 2.74 | 3.45 | 4.17 | 4.88 | 5.59 | 6.3 | 7.72 |
| JD1 | 1 720 | 2 560 | 3 570 | 4 720 | 6 000 | 7 380 | 10 550 |
| JD2 | 1 280 | 1 950 | 2 700 | 3 570 | 4 520 | 5 560 | 7 950 |
| JD3 | 1 010 | 1 520 | 2 120 | 2 800 | 3 570 | 4 380 | 6 270 |
| JD4 | 710 | 1 080 | 1 520 | 2 020 | 2 530 | 3 130 | 4 480 |
| JD5 | 510 | 780 | 1 080 | 1 420 | 1 790 | 2 220 | 3 170 |
| JD6 | 370 | 570 | 780 | 1 010 | 1 310 | 1 620 | 2 290 |

## TABLE 4.6(A)

## CHARACTERISTIC CAPACITY FOR A SINGLE SCREW IN SIDE GRAIN LOADED IN WITHDRAWAL—UNSEASONED TIMBER

| Joint group | Characteristic capacity per screw, N per mm penetration of thread | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screw size number | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 18 |
| | Shank diameter, mm | | | | | | |
| | 2.74 | 3.45 | 4.17 | 4.88 | 5.59 | 6.3 | 7.72 |
| J1 | 56 | 71 | 85 | 100 | 116 | 129 | 158 |
| J2 | 42 | 54 | 66 | 77 | 87 | 100 | 122 |
| J3 | 33 | 41 | 50 | 58 | 66 | 75 | 91 |
| J4 | 23 | 31 | 37 | 42 | 48 | 54 | 68 |
| J5 | 19 | 25 | 29 | 35 | 41 | 44 | 54 |
| J6 | 15 | 19 | 23 | 27 | 31 | 35 | 42 |

## TABLE 4.6(B)

## CHARACTERISTIC CAPACITY FOR A SINGLE SCREW IN SIDE GRAIN LOADED IN WITHDRAWAL—SEASONED TIMBER

| Joint group | Characteristic capacity per screw, N per mm penetration of thread | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screw size number | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 18 |
| | Shank diameter, mm | | | | | | |
| | 2.74 | 3.45 | 4.17 | 4.88 | 5.59 | 6.3 | 7.72 |
| JD1 | 81 | 102 | 125 | 147 | 168 | 189 | 232 |
| JD2 | 62 | 79 | 97 | 112 | 127 | 145 | 178 |
| JD3 | 48 | 62 | 73 | 87 | 100 | 112 | 137 |
| JD4 | 37 | 46 | 56 | 66 | 75 | 85 | 104 |
| JD5 | 29 | 37 | 44 | 52 | 60 | 68 | 83 |
| JD6 | 23 | 29 | 35 | 41 | 46 | 52 | 64 |

## TABLE 4.7

## MAXIMUM TENSILE CAPACITY FOR SCREWS

| Metal | Maximum tensile capacity, N | | | | | | |
|---|---|---|---|---|---|---|---|
| | Screw size number | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 18 |
| Steel and 18/8 stainless steel | 1 410 | 2 140 | 3 180 | 4 380 | 5 710 | 7 300 | 10 810 |
| Brass and silicon bronze | 1 080 | 1 640 | 2 450 | 3 380 | 4 400 | 5 620 | 8 320 |
| Aluminium alloy | 830 | 1 250 | 1 870 | 2 590 | 3 360 | 4 300 | 6 370 |

### 4.3.3 Design capacity of screwed joints

**4.3.3.1** *General*

The design capacity of Type 1 and Type 2 screwed joints shall be calculated in accordance with Clauses 4.3.3.2 to 4.3.3.4.

**4.3.3.2** *Type 1 joint to resist direct loads*

The design capacity ($N_{d,j}$) for a Type 1 joint containing $n$ screws designed to resist direct loads as illustrated in Figure 4.5(a), for strength limit states shall satisfy the following:

$$N_{d,j} \geq N^* \qquad \text{. . . 4.3(1)}$$

where

$$N_{d,j} = \phi k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \qquad \text{. . . 4.3(2)}$$

and

$N^*$ = design action effect on joint produced by strength limit states design loads (direct loads)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for joints (see Clause 2.4.1.1)

$k_{13}$ = 1.0 for screws in side grain (see Figure 4.2(a))
= 0.6 for screws in end grain (see Figure 4.2(b))

$k_{14}$ = 1.0 for screws in single shear (see Figure 4.6(a))
= 2.0 for screws in double shear (see Figure 4.6(b))

$k_{16}$ = 1.2 where the load is applied through metal side plates of adequate strength to transfer the load and the screws are a close fit to the holes in these plates
= 1.1 for screws through plywood gusset plates
= 1.0 otherwise

$k_{17}$ = factor for multiple screwed joints given in Table 4.3(A) for Type 1 joints to resist direct loads in either compression or tension

$n$ = total number of screws in the joint

$Q_k$ = characteristic capacity given in Tables 4.5(A) and 4.5(B). (See also Clause 4.3.2 and 4.3.5)

**4.3.3.3** *Type 1 joint to resist in-plane bending moments*

The design capacity ($M_{d,j}$) for a Type 1 joint containing $n$ screws designed to resist in-plane moment, as illustrated in Figure 4.4(b), for strength limit state shall satisfy the following:

$$M_{d,j} \geq M^* \qquad \text{. . . 4.3(3)}$$

where

$$M_{d,j} = \phi k_1 k_{13} k_{14} k_{16} k_{17} r_{max.} Q_k \sum_{i=1}^{n} \left( \frac{r_i}{r_{max.}} \right)^{\frac{3}{2}} \qquad \text{. . . 4.3(4)}$$

and

$M^*$ = design action effect on joint (in-plane bending moment)

$\phi$ = capacity factor (see Clause 2.4.1.1)

$k_1$ = factor for duration of load for joints (see Clause 2.4.1.1)

$k_{13}$ = 1.0 for screws in side grain (see Figure 4.2(a))
= 0.6 for screws in end grain (see Figure 4.2(b))

$k_{14}$ = 1.0 for screws in single shear (see Figure 4.6(a))
= 2.0 for screws in double shear (see Figure 4.6(b))

$k_{16}$ = 1.2 where the load is applied through metal side plates of adequate strength to transfer the load and the screws are a close fit to the holes in these plates
= 1.1 for screws through plywood gusset plates
= 1.0 otherwise

$k_{17}$ = factor for multiple screwed joints given in Table 4.3(B) for Type 1 joints to resist in-plane moments

$r_{max.}$ = maximum value of $r_i$

$Q_k$ = characteristic capacity given in Table 4.5(A) and Table 4.5(B). See also Clause 4.3.2 and 4.3.5

$n$ = total number of screws in single shear per interface resisting the design in-plane moment

$r_i$ = distance from the $i$-th screw to the centroid of the screw group

**4.3.3.4** *Type 2 joint*

The design capacity ($N_{d,j}$) for Type 2 joint containing $n$ screws designed to resist axial loads tending to cause withdrawal, as illustrated in Figure 4.4, for strength limit states shall satisfy the following:

$$N_{d,j} \geq N^* \quad \text{. . . 4.3(5)}$$

where $N_{d,j}$ is the lesser of—

$$N_{d,j} = \phi\, k_{13}\, l_p\, n\, Q_k; \text{ or} \quad \text{. . . 4.3(6)}$$

$$N_{d,j} = n\, N_{d,ts} \quad \text{. . . 4.3(7)}$$

and

$N^*$ = design action effect in the axial direction of the fastener

$\phi$ = capacity factor (see Clause 2.3)

$k_{13}$ = 1.0 for withdrawal from side grain (see Figure 4.4(a))
= 0.6 for withdrawal from end grain (see Figure 4.4(b))

$l_p$ = depth of screw penetration, in millimetres, into supporting member (see Figure 4.4(a))

$n$ = total number of screws in the joint

$Q_k$ = characteristic capacity given in Table 4.6(A) and Table 4.6(B). (See also Clause 4.3.2)

$N_{d,ts}$ = design tensile capacity of screw; refer screw manufacturer's specifications.

NOTES:

1 The load duration factor $k_1$ does not apply to screws subject to the action of withdrawal loads.

2 For some kinds of screw, head pull-through may be a design consideration.

### 4.3.4 Spacing, edge and end distances for screws

Table 4.8 provides minimum spacings, edge and end distances for screws given in terms of the shank diameter ($D$).

For spacings at an angle to the grain, interpolation according to Hankinson's formula is appropriate (see Clause 4.4.2.4(c)).

**TABLE 4.8**

**MINIMUM SPACING, EDGE AND END DISTANCES FOR SCREWS**

| Spacing | Minimum distance |
|---|---|
| End distance | 10*D* |
| Edge distance | 5*D* |
| Between screws | |
| —along grain | 10*D* |
| —across grain | 3*D* |

*D* = shank diameter of screws.

### 4.3.5 Screw length and timber thickness

For the characteristic capacities given in Tables 4.5(A) and 4.5(B) to be applicable, timber thicknesses and screw length as shown in Figure 4.6(a) shall be such that—

(a) thickness of first member ........................................................................ $t_1 > 10D$; and

(b) depth of penetration into second member ........................................................ $t_p > 7D$.

For lesser values of $t_1$ and $t_p$, the characteristic capacity shall be reduced in proportion to the decrease in $t_1$ or $t_p$ and the screw shall be considered as non-load-bearing if $t_1$ or $t_p$ is less than 4*D*.

### 4.3.6 Preboring

The values given in Tables 4.5(A), 4.5(B), 4.6(A), and 4.6(B) apply when the correct size lead holes have been bored. The diameter of the hole for the shank shall be equal to the diameter of the shank, and the lead hole for the threaded portion of the screw shall not be greater than the root diameter of the screw.

NOTE: These limits do not apply to plywood (see Appendix C).

## 4.4 DESIGN OF BOLTED JOINTS

### 4.4.1 General

The characteristic capacities given in Clauses 4.4.2.2 and 4.4.2.3 are applicable to steel bolts as specified in AS 1111.1, when fitted into prebored holes of diameter approximately 10% greater than the bolt diameter and when fitted with washers as specified in Clause 4.4.5.

NOTE: For dome head coach bolts, washers should be fitted. Where dome head coach bolts are used shank diameter should be equivalent to thread diameter.

Characteristic capacities for values of bolt diameters and effective timber thicknesses that are not included in Tables 4.9 and 4.10 shall be calculated in accordance with Paragraph C3, Appendix C.

### 4.4.2 Characteristic capacities for bolts

#### 4.4.2.1 *General*

The characteristic capacities for bolts that are laterally loaded parallel and perpendicular to the grain and at other angles to the grain shall be as specified in Clauses 4.4.2.2 to 4.4.2.4.

#### 4.4.2.2 *Characteristic capacity parallel to the grain*

The characteristic capacity for a laterally loaded single bolt bearing parallel to the grain and acting in single shear ($Q_{kl}$) shall be as given for a selection of bolt diameters and effective timber thicknesses in Table 4.9(B) and 4.9(C).

#### 4.4.2.3 *Characteristic capacity perpendicular to grain*

The characteristic capacity for a laterally loaded single bolt bearing perpendicular to the grain and acting in single shear ($Q_{kp}$) shall be as given for a selection of bolt diameters and effective timber thicknesses in Tables 4.10(B) and 4.10(C).

**4.4.2.4** *Characteristic capacity for a bolted joint system*

The characteristic capacity for a laterally loaded single bolt in a bolted joint system ($Q_{sk}$) shall be derived as follows:

(a) For systems loaded parallel to the grain—

$$Q_{sk} = Q_{skl}$$

where $Q_{skl}$ is the system capacity given in Table 4.9(A).

(b) For systems loaded perpendicular to the grain—

$$Q_{sk} = Q_{skp}$$

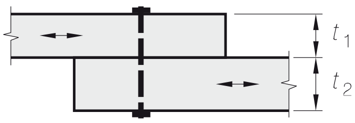
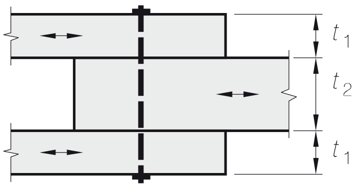
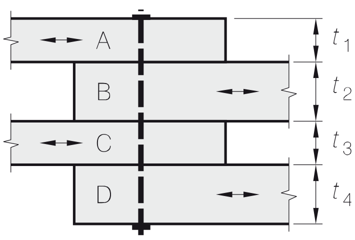
where $Q_{skp}$ is the system capacity given in Table 4.10(A).

(c) For systems loaded at an angle to the grain, ($\theta$), the system capacity shall be calculated by use of Hankinson's formula as follows:

$$Q_{sk} = \frac{Q_{skl}\, Q_{skp}}{Q_{skl} \sin^2 \theta + Q_{skp} \cos^2 \theta} \qquad \text{. . . 4.4(1)}$$

**TABLE 4.9(A)**

**CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PARALLEL TO GRAIN—SYSTEM CAPACITY**

| Joint configuration | Effective timber thickness ($b_{eff}$) | System capacity ($Q_{skl}$) |
|---|---|---|
| (1) Two member   | $b_{eff}$ equals smaller of $t_1$ and $t_2$ | $Q_{kl}$ |
| (2) Three member   | $b_{eff}$ equals smaller of $t_2$ and $2t_1$ | $2Q_{kl}$ |
| (3) Multiple member   | (i) Between A and B, $b_{eff}$ equals the smaller of $t_1$ and $t_2$<br>(ii) Between B and C, $b_{eff}$ equals the smaller of $t_2$ and $t_3$<br>(iii) etc. | (i) $Q_{kl}$<br>(ii) $Q_{kl}$<br>(iii) etc.<br>$Q_{skl}$ = sum of basic loads (i), (ii), etc. |

LEGEND:
↔ = Indicates load direction.

NOTE: Values of the characteristic capacity ($Q_{kl}$) are given in Tables 4.9(B) and 4.9(C) for unseasoned and seasoned timber respectively.

## TABLE 4.9(B)

## CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PARALLEL TO GRAIN—UNSEASONED TIMBER

| Joint group | Effect timber thickness ($b_{eff}$) mm | Characteristic capacity ($Q_{kl}$), N | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Bolt diameter ($D$) | | | | | | | | |
| | | M6 | M8 | M10 | M12 | M16 | M20 | M24 | M30 | M36 |
| J1 | 25 | 3 300 | 5 600 | 6 900 | 8 300 | 11 100 | 13 900 | 16 700 | 20 800 | 25 000 |
| | 38 | 3 300 | 5 900 | 9 200 | 12 700 | 16 900 | 21 100 | 25 300 | 31 600 | 38 000 |
| | 50 | 3 300 | 5 900 | 9 200 | 13 200 | 22 200 | 27 800 | 33 300 | 41 600 | 50 000 |
| | 75 | 3 300 | 5 900 | 9 200 | 13 200 | 23 400 | 36 600 | 50 000 | 62 400 | 74 900 |
| | 100 | 3 300 | 5 900 | 9 200 | 13 200 | 23 400 | 36 600 | 52 700 | 82 400 | 99 900 |
| | 150 | 3 300 | 5 900 | 9 200 | 13 200 | 23 400 | 36 600 | 52 700 | 82 400 | 118 700 |
| | 200 | 3 300 | 5 900 | 9 200 | 13 200 | 23 400 | 36 600 | 52 700 | 82 400 | 118 700 |
| J2 | 25 | 2 800 | 4 400 | 5 500 | 6 600 | 8 800 | 11 000 | 13 200 | 16 500 | 19 800 |
| | 38 | 2 800 | 4 900 | 7 700 | 10 000 | 13 400 | 16 700 | 20 100 | 25 100 | 30 100 |
| | 50 | 2 800 | 4 900 | 7 700 | 11 100 | 17 600 | 22 000 | 26 400 | 33 000 | 39 600 |
| | 75 | 2 800 | 4 900 | 7 700 | 11 100 | 19 700 | 30 800 | 39 600 | 49 500 | 59 400 |
| | 100 | 2 800 | 4 900 | 7 700 | 11 100 | 19 700 | 30 800 | 44 400 | 66 000 | 79 200 |
| | 150 | 2 800 | 4 900 | 7 700 | 11 100 | 19 700 | 30 800 | 44 400 | 69 300 | 99 800 |
| | 200 | 2 800 | 4 900 | 7 700 | 11 100 | 19 700 | 30 800 | 44 400 | 69 300 | 99 800 |
| J3 | 25 | 2 600 | 3 600 | 4 400 | 5 300 | 7 100 | 8 900 | 10 700 | 13 300 | 16 000 |
| | 38 | 2 600 | 4 500 | 6 700 | 8 100 | 10 800 | 13 500 | 16 200 | 20 200 | 24 300 |
| | 50 | 2 600 | 4 500 | 7 100 | 10 200 | 14 200 | 17 800 | 21 300 | 26 600 | 32 000 |
| | 75 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 26 600 | 32 000 | 39 900 | 47 900 |
| | 100 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 40 900 | 53 300 | 63 900 |
| | 150 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 40 900 | 63 900 | 92 000 |
| | 200 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 40 900 | 63 900 | 92 000 |
| J4 | 25 | 2 000 | 2 800 | 3 500 | 4 200 | 5 600 | 7 000 | 8 400 | 10 500 | 12 600 |
| | 38 | 2 000 | 3 600 | 5 300 | 6 400 | 8 500 | 10 600 | 12 800 | 16 000 | 19 200 |
| | 50 | 2 000 | 3 600 | 5 600 | 8 100 | 11 200 | 14 000 | 16 800 | 21 000 | 25 200 |
| | 75 | 2 000 | 3 600 | 5 600 | 8 100 | 14 300 | 21 000 | 25 200 | 31 500 | 37 800 |
| | 100 | 2 000 | 3 600 | 5 600 | 8 100 | 14 300 | 22 400 | 32 300 | 42 000 | 50 400 |
| | 150 | 2 000 | 3 600 | 5 600 | 8 100 | 14 300 | 22 400 | 32 300 | 50 400 | 72 600 |
| | 200 | 2 000 | 3 600 | 5 600 | 8 100 | 14 300 | 22 400 | 32 300 | 50 400 | 72 600 |
| J5 | 25 | 1 700 | 2 200 | 2 800 | 3 300 | 4 400 | 5 500 | 6 600 | 8 300 | 9 900 |
| | 38 | 1 700 | 3 100 | 4 200 | 5 000 | 6 700 | 8 400 | 10 000 | 12 500 | 15 000 |
| | 50 | 1 700 | 3 100 | 4 800 | 6 600 | 8 800 | 11 000 | 13 200 | 16 500 | 19 800 |
| | 75 | 1 700 | 3 100 | 4 800 | 7 000 | 12 400 | 16 500 | 19 800 | 24 800 | 29 700 |
| | 100 | 1 700 | 3 100 | 4 800 | 7 000 | 12 400 | 19 400 | 26 400 | 33 000 | 39 600 |
| | 150 | 1 700 | 3 100 | 4 800 | 7 000 | 12 400 | 19 400 | 27 900 | 43 600 | 59 400 |
| | 200 | 1 700 | 3 100 | 4 800 | 7 000 | 12 400 | 19 400 | 27 900 | 43 600 | 62 700 |
| J6 | 25 | 1 400 | 1 800 | 2 300 | 2 700 | 3 600 | 4 500 | 5 400 | 6 800 | 8 100 |
| | 38 | 1 600 | 2 700 | 3 400 | 4 100 | 5 500 | 6 800 | 8 200 | 10 300 | 12 300 |
| | 50 | 1 600 | 2 800 | 4 300 | 5 400 | 7 200 | 9 000 | 10 800 | 13 500 | 16 200 |
| | 75 | 1 600 | 2 800 | 4 300 | 6 200 | 10 800 | 13 500 | 16 200 | 20 300 | 24 300 |
| | 100 | 1 600 | 2 800 | 4 300 | 6 200 | 11 100 | 17 300 | 21 600 | 27 000 | 32 400 |
| | 150 | 1 600 | 2 800 | 4 300 | 6 200 | 11 100 | 17 300 | 24 900 | 38 900 | 48 600 |
| | 200 | 1 600 | 2 800 | 4 300 | 6 200 | 11 100 | 17 300 | 24 900 | 38 900 | 56 000 |

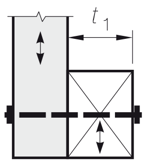
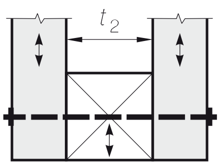
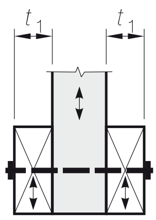
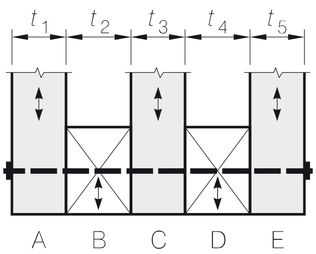
## TABLE 4.9(C)

## CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PARALLEL TO GRAIN—SEASONED TIMBER

| Joint group | Effect timber thickness ($b_{eff}$) mm | Characteristic capacity ($Q_{kl}$), N | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Bolt diameter ($D$) | | | | | | | | |
| | | M6 | M8 | M10 | M12 | M16 | M20 | M24 | M30 | M36 |
| JD1 | 25 | 4 100 | 6 900 | 8 600 | 10 400 | 13 800 | 17 300 | 20 700 | 25 900 | 31 100 |
| | 35 | 4 100 | 7 300 | 11 400 | 14 500 | 19 300 | 24 200 | 29 000 | 36 200 | 43 500 |
| | 40 | 4 100 | 7 300 | 11 400 | 16 400 | 22 100 | 27 600 | 33 100 | 41 400 | 49 700 |
| | 45 | 4 100 | 7 300 | 11 400 | 16 400 | 24 800 | 31 100 | 37 300 | 46 600 | 55 900 |
| | 70 | 4 100 | 7 300 | 11 400 | 16 400 | 29 100 | 45 500 | 58 000 | 72 500 | 86 900 |
| | 90 | 4 100 | 7 300 | 11 400 | 16 400 | 29 100 | 45 500 | 65 600 | 93 200 | 111 800 |
| | 105 | 4 100 | 7 300 | 11 400 | 16 400 | 29 100 | 45 500 | 65 600 | 102 500 | 130 400 |
| | 120 | 4 100 | 7 300 | 11 400 | 16 400 | 29 100 | 45 500 | 65 600 | 102 500 | 147 500 |
| JD2 | 25 | 3 500 | 5 600 | 6 900 | 8 300 | 11 100 | 13 900 | 16 700 | 20 800 | 25 000 |
| | 35 | 3 500 | 6 200 | 9 700 | 11 700 | 15 500 | 19 400 | 23 300 | 29 100 | 35 000 |
| | 40 | 3 500 | 6 200 | 9 700 | 13 300 | 17 800 | 22 200 | 26 600 | 33 300 | 40 000 |
| | 45 | 3 500 | 6 200 | 9 700 | 14 000 | 20 000 | 25 000 | 30 000 | 37 500 | 45 000 |
| | 70 | 3 500 | 6 200 | 9 700 | 14 000 | 24 900 | 38 900 | 46 600 | 58 300 | 69 900 |
| | 90 | 3 500 | 6 200 | 9 700 | 14 000 | 24 900 | 38 900 | 55 900 | 74 900 | 89 900 |
| | 105 | 3 500 | 6 200 | 9 700 | 14 000 | 24 900 | 38 900 | 55 900 | 87 400 | 104 900 |
| | 120 | 3 500 | 6 200 | 9 700 | 14 000 | 24 900 | 38 900 | 55 900 | 87 400 | 119 900 |
| JD3 | 25 | 3 200 | 4 400 | 5 500 | 6 600 | 8 800 | 11 000 | 13 200 | 16 500 | 19 800 |
| | 35 | 3 200 | 5 600 | 7 700 | 9 200 | 12 300 | 15 400 | 18 500 | 23 100 | 27 700 |
| | 40 | 3 200 | 5 600 | 8 800 | 10 600 | 14 100 | 17 600 | 21 100 | 26 400 | 31 700 |
| | 45 | 3 200 | 5 600 | 8 800 | 11 900 | 15 800 | 19 800 | 23 800 | 29 700 | 35 600 |
| | 70 | 3 200 | 5 600 | 8 800 | 12 700 | 22 500 | 30 800 | 37 000 | 46 200 | 55 400 |
| | 90 | 3 200 | 5 600 | 8 800 | 12 700 | 22 500 | 35 200 | 47 500 | 59 400 | 71 300 |
| | 105 | 3 200 | 5 600 | 8 800 | 12 700 | 22 500 | 35 200 | 50 700 | 69 300 | 83 200 |
| | 120 | 3 200 | 5 600 | 8 800 | 12 700 | 22 500 | 35 200 | 50 700 | 79 200 | 95 000 |
| JD4 | 25 | 2 600 | 3 600 | 4 400 | 5 300 | 7 100 | 8 900 | 10 700 | 13 300 | 16 000 |
| | 35 | 2 600 | 4 500 | 6 200 | 7 500 | 9 900 | 12 400 | 14 900 | 18 600 | 22 400 |
| | 40 | 2 600 | 4 500 | 7 100 | 8 500 | 11 400 | 14 200 | 17 000 | 21 300 | 25 600 |
| | 45 | 2 600 | 4 500 | 7 100 | 9 600 | 12 800 | 16 000 | 19 200 | 24 000 | 28 800 |
| | 70 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 24 900 | 29 800 | 37 300 | 44 700 |
| | 90 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 38 300 | 47 900 | 57 500 |
| | 105 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 40 900 | 55 900 | 67 100 |
| | 120 | 2 600 | 4 500 | 7 100 | 10 200 | 18 200 | 28 400 | 40 900 | 63 900 | 76 700 |
| JD5 | 25 | 2 100 | 2 800 | 3 500 | 4 200 | 5 600 | 7 000 | 8 400 | 10 500 | 12 600 |
| | 35 | 2 200 | 3 900 | 4 900 | 5 900 | 7 800 | 9 800 | 11 800 | 14 700 | 17 600 |
| | 40 | 2 200 | 3 900 | 5 600 | 6 700 | 9 000 | 11 200 | 13 400 | 16 800 | 20 200 |
| | 45 | 2 200 | 3 900 | 6 200 | 7 600 | 10 100 | 12 600 | 15 100 | 18 900 | 22 700 |
| | 70 | 2 200 | 3 900 | 6 200 | 8 900 | 15 700 | 19 600 | 23 500 | 29 400 | 35 300 |
| | 90 | 2 200 | 3 900 | 6 200 | 8 900 | 15 800 | 24 600 | 30 200 | 37 800 | 45 400 |
| | 105 | 2 200 | 3 900 | 6 200 | 8 900 | 15 800 | 24 600 | 35 300 | 44 100 | 52 900 |
| | 120 | 2 200 | 3 900 | 6 200 | 8 900 | 15 800 | 24 600 | 35 500 | 50 400 | 60 500 |
| JD6 | 25 | 1 700 | 2 200 | 2 800 | 3 300 | 4 400 | 5 500 | 6 600 | 8 300 | 9 900 |
| | 35 | 1 900 | 3 100 | 3 900 | 4 600 | 6 200 | 7 700 | 9 200 | 11 600 | 13 900 |
| | 40 | 1 900 | 3 400 | 4 400 | 5 300 | 7 000 | 8 800 | 10 600 | 13 200 | 15 800 |
| | 45 | 1 900 | 3 400 | 5 000 | 5 900 | 7 900 | 9 900 | 11 900 | 14 900 | 17 800 |
| | 70 | 1 900 | 3 400 | 5 300 | 7 600 | 12 300 | 15 400 | 18 500 | 23 100 | 27 700 |
| | 90 | 1 900 | 3 400 | 5 300 | 7 600 | 13 500 | 19 800 | 23 800 | 29 700 | 35 600 |
| | 105 | 1 900 | 3 400 | 5 300 | 7 600 | 13 500 | 21 100 | 27 700 | 34 700 | 41 600 |
| | 120 | 1 900 | 3 400 | 5 300 | 7 600 | 13 500 | 21 100 | 30 400 | 39 600 | 47 500 |

## TABLE 4.10(A)

## CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PERPENDICULAR TO THE GRAIN—SYSTEM CAPACITY

| **Joint configuration** | **Effective timber thickness ($b_{eff}$)** | **System capacity ($Q_{skp}$)** |
|---|---|---|
| (1) Two member   | $b_{eff}$ equals $2t_1$ | $Q_{kp}$ |
| (2) Three member, Type A   | $b_{eff}$ equals $t_2$ | $2Q_{kp}$ |
| (3) Three member, Type B   | $b_{eff}$ equals $2t_1$ | $2Q_{kp}$ |
| (4) Multiple member   | (i) Between A and B $b_{eff}$ is equal to $t_2$<br>(ii) Between B and C $b_{eff}$ is equal to $t_2$<br>(iii) Between C and D $b_{eff}$ is equal to $t_4$ | (i) $Q_{kp}$<br>(ii) $Q_{kp}$<br>(iii) etc.<br>$Q_{skp}$ = sum of basic loads (i), (ii), (iii), etc. |

NOTE: At each interface, the strength of the bolted joint with respect to the member aligned parallel to the direction of the stress shall be checked according to Table 4.9.

## TABLE 4.10(B)

## CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PERPENDICULAR TO THE GRAIN—UNSEASONED TIMBER

| Joint group | Effect timber thickness ($b_{eff}$) mm | Characteristic capacity ($Q_{kp}$), N | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Bolt diameter ($D$) | | | | | | | | |
| | | M6 | M8 | M10 | M12 | M16 | M20 | M24 | M30 | M36 |
| J1 | 25 | 1 650 | 2 200 | 2 750 | 3 300 | 4 400 | 5 500 | 6 600 | 8 250 | 9 900 |
| | 38 | 2 510 | 3 340 | 4 180 | 5 020 | 6 690 | 8 360 | 10 030 | 12 540 | 15 050 |
| | 50 | 3 230 | 4 400 | 5 500 | 6 600 | 8 800 | 11 000 | 13 200 | 16 500 | 19 800 |
| | 75 | 3 230 | 4 980 | 6 960 | 9 150 | 13 200 | 16 500 | 19 800 | 24 750 | 29 700 |
| | 100 | 3 230 | 4 980 | 6 960 | 9 150 | 14 080 | 19 680 | 25 870 | 33 000 | 39 600 |
| | 150 | 3 230 | 4 980 | 6 960 | 9 150 | 14 080 | 19 680 | 25 870 | 36 150 | 47 520 |
| | 200 | 3 230 | 4 980 | 6 960 | 9 150 | 14 080 | 19 680 | 25 870 | 36 150 | 47 520 |
| J2 | 25 | 1 310 | 1 750 | 2 190 | 2 630 | 3 500 | 4 380 | 5 250 | 6 560 | 7 880 |
| | 38 | 2 000 | 2 660 | 3 330 | 3 990 | 5 320 | 6 650 | 7 980 | 9 980 | 11 970 |
| | 50 | 2 630 | 3 500 | 4 380 | 5 250 | 7 000 | 8 750 | 10 500 | 13 130 | 15 750 |
| | 75 | 3 090 | 4 750 | 6 560 | 7 880 | 10 500 | 13 130 | 15 750 | 19 690 | 23 630 |
| | 100 | 3 090 | 4 750 | 6 640 | 8 730 | 13 440 | 17 500 | 21 000 | 26 250 | 31 500 |
| | 150 | 3 090 | 4 750 | 6 640 | 8 730 | 13 440 | 18 780 | 24 690 | 34 510 | 45 360 |
| | 200 | 3 090 | 4 750 | 6 640 | 8 730 | 13 440 | 18 780 | 24 690 | 34 510 | 45 360 |
| J3 | 25 | 830 | 1 100 | 1 380 | 1 650 | 2 200 | 2 750 | 3 300 | 4 130 | 4 950 |
| | 38 | 1 250 | 1 670 | 2 090 | 2 510 | 3 340 | 4 180 | 5 020 | 6 270 | 7 520 |
| | 50 | 1 650 | 2 200 | 2 750 | 3 300 | 4 400 | 5 500 | 6 600 | 8 250 | 9 900 |
| | 75 | 2 420 | 3 300 | 4 130 | 4 950 | 6 600 | 8 250 | 9 900 | 12 380 | 14 850 |
| | 100 | 2 420 | 3 730 | 5 220 | 6 600 | 8 800 | 11 000 | 13 200 | 16 500 | 19 800 |
| | 150 | 2 420 | 3 730 | 5 220 | 6 860 | 10 560 | 14 760 | 19 400 | 24 750 | 29 700 |
| | 200 | 2 420 | 3 730 | 5 220 | 6 860 | 10 560 | 14 760 | 19 400 | 27 110 | 35 640 |
| J4 | 25 | 530 | 710 | 890 | 1 070 | 1 420 | 1 780 | 2 130 | 2 660 | 3 200 |
| | 38 | 810 | 1 080 | 1 350 | 1 620 | 2 160 | 2 700 | 3 240 | 4 050 | 4 860 |
| | 50 | 1 070 | 1 420 | 1 780 | 2 130 | 2 840 | 3 550 | 4 260 | 5 330 | 6 390 |
| | 75 | 1 600 | 2 130 | 2 660 | 3 200 | 4 260 | 5 330 | 6 390 | 7 990 | 9 590 |
| | 100 | 1 770 | 2 730 | 3 550 | 4 260 | 5 680 | 7 100 | 8 520 | 10 650 | 12 780 |
| | 150 | 1 770 | 2 730 | 3 820 | 5 020 | 7 720 | 10 650 | 12 780 | 15 980 | 19 170 |
| | 200 | 1 770 | 2 730 | 3 820 | 5 020 | 7 720 | 10 800 | 14 190 | 19 830 | 25 560 |
| J5 | 25 | 350 | 470 | 590 | 710 | 940 | 1 180 | 1 410 | 1 760 | 2 120 |
| | 38 | 540 | 710 | 890 | 1 070 | 1 430 | 1 790 | 2 140 | 2 680 | 3 210 |
| | 50 | 710 | 940 | 1 180 | 1 410 | 1 880 | 2 350 | 2 820 | 3 530 | 4 230 |
| | 75 | 1 060 | 1 410 | 1 760 | 2 120 | 2 820 | 3 530 | 4 230 | 5 290 | 6 350 |
| | 100 | 1 310 | 1 880 | 2 350 | 2 820 | 3 760 | 4 700 | 5 640 | 7 050 | 8 460 |
| | 150 | 1 310 | 2 020 | 2 820 | 3 710 | 5 640 | 7 050 | 8 460 | 10 580 | 12 690 |
| | 200 | 1 310 | 2 020 | 2 820 | 3 710 | 5 720 | 7 990 | 10 500 | 14 100 | 16 920 |
| J6 | 25 | 180 | 240 | 300 | 360 | 480 | 600 | 720 | 900 | 1 080 |
| | 38 | 270 | 360 | 460 | 550 | 730 | 910 | 1 090 | 1 370 | 1 640 |
| | 50 | 360 | 480 | 600 | 720 | 960 | 1 200 | 1 440 | 1 800 | 2 160 |
| | 75 | 540 | 720 | 900 | 1 080 | 1 440 | 1 800 | 2 160 | 2 700 | 3 240 |
| | 100 | 720 | 960 | 1 200 | 1 440 | 1 920 | 2 400 | 2 880 | 3 600 | 4 320 |
| | 150 | 780 | 1 190 | 1 670 | 2 160 | 2 880 | 3 600 | 4 320 | 5 400 | 6 480 |
| | 200 | 780 | 1 190 | 1 670 | 2 190 | 3 380 | 4 720 | 5 760 | 7 200 | 8 640 |

## TABLE 4.10(C)

## CHARACTERISTIC CAPACITY FOR SINGLE BOLTS PERPENDICULAR TO THE GRAIN—SEASONED TIMBER

| Joint group | Effect timber thickness ($b_{eff}$) mm | Characteristic capacity ($Q_{kp}$), N | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Bolt diameter ($D$) | | | | | | | | |
| | | M6 | M8 | M10 | M12 | M16 | M20 | M24 | M30 | M36 |
| JD1 | 25 | 2 210 | 2 950 | 3 690 | 4 430 | 5 900 | 7 380 | 8 850 | 11 060 | 13 280 |
| | 35 | 3 100 | 4 130 | 5 160 | 6 200 | 8 260 | 10 330 | 12 390 | 15 490 | 18 590 |
| | 40 | 3 540 | 4 720 | 5 900 | 7 080 | 9 440 | 11 800 | 14 160 | 17 700 | 21 240 |
| | 45 | 3 980 | 5 310 | 6 640 | 7 970 | 10 620 | 13 280 | 15 930 | 19 910 | 23 900 |
| | 70 | 4 340 | 6 680 | 9 330 | 12 260 | 16 520 | 20 650 | 24 780 | 30 980 | 37 170 |
| | 90 | 4 340 | 6 680 | 9 330 | 12 260 | 18 880 | 26 390 | 31 860 | 39 830 | 47 790 |
| | 105 | 4 340 | 6 680 | 9 330 | 12 260 | 18 880 | 26 390 | 34 680 | 46 460 | 55 760 |
| | 120 | 4 340 | 6 680 | 9 330 | 12 260 | 18 880 | 26 390 | 34 680 | 48 470 | 63 720 |
| JD2 | 25 | 1 690 | 2 250 | 2 810 | 3 380 | 4 500 | 5 630 | 6 750 | 8 440 | 10 130 |
| | 35 | 2 360 | 3 150 | 3 940 | 4 730 | 6 300 | 7 880 | 9 450 | 11 810 | 14 180 |
| | 40 | 2 700 | 3 600 | 4 500 | 5 400 | 7 200 | 9 000 | 10 800 | 13 500 | 16 200 |
| | 45 | 3 040 | 4 050 | 5 060 | 6 080 | 8 100 | 10 130 | 12 150 | 15 190 | 18 230 |
| | 70 | 3 970 | 6 110 | 7 880 | 9 450 | 12 600 | 15 750 | 18 900 | 23 630 | 28 350 |
| | 90 | 3 970 | 6 110 | 8 540 | 11 220 | 16 200 | 20 250 | 24 300 | 30 380 | 36 450 |
| | 105 | 3 970 | 6 110 | 8 540 | 11 220 | 17 280 | 23 630 | 28 350 | 35 440 | 42 530 |
| | 120 | 3 970 | 6 110 | 8 540 | 11 220 | 17 280 | 24 150 | 31 750 | 40 500 | 48 600 |
| JD3 | 25 | 1 280 | 1 700 | 2 130 | 2 550 | 3 400 | 4 250 | 5 100 | 6 380 | 7 650 |
| | 35 | 1 790 | 2 380 | 2 980 | 3 570 | 4 760 | 5 950 | 7 140 | 8 930 | 10 710 |
| | 40 | 2 040 | 2 720 | 3 400 | 4 080 | 5 440 | 6 800 | 8 160 | 10 200 | 12 240 |
| | 45 | 2 300 | 3 060 | 3 830 | 4 590 | 6 120 | 7 650 | 9 180 | 11 480 | 13 770 |
| | 70 | 3 570 | 4 760 | 5 950 | 7 140 | 9 520 | 11 900 | 14 280 | 17 850 | 21 420 |
| | 90 | 3 750 | 5 770 | 7 650 | 9 180 | 12 240 | 15 300 | 18 360 | 22 950 | 27 540 |
| | 105 | 3 750 | 5 770 | 8 060 | 10 600 | 14 280 | 17 850 | 21 420 | 26 780 | 32 130 |
| | 120 | 3 750 | 5 770 | 8 060 | 10 600 | 16 320 | 20 400 | 24 480 | 30 600 | 36 720 |
| JD4 | 25 | 940 | 1 250 | 1 560 | 1 880 | 2 500 | 3 130 | 3 750 | 4 690 | 5 630 |
| | 35 | 1 310 | 1 750 | 2 190 | 2 630 | 3 500 | 4 380 | 5 250 | 6 560 | 7 880 |
| | 40 | 1 500 | 2 000 | 2 500 | 3 000 | 4 000 | 5 000 | 6 000 | 7 500 | 9 000 |
| | 45 | 1 690 | 2 250 | 2 810 | 3 380 | 4 500 | 5 630 | 6 750 | 8 440 | 10 130 |
| | 70 | 2 630 | 3 500 | 4 380 | 5 250 | 7 000 | 8 750 | 10 500 | 13 130 | 15 750 |
| | 90 | 3 120 | 4 500 | 5 630 | 6 750 | 9 000 | 11 250 | 13 500 | 16 880 | 20 250 |
| | 105 | 3 120 | 4 810 | 6 560 | 7 880 | 10 500 | 13 130 | 15 750 | 19 690 | 23 630 |
| | 120 | 3 120 | 4 810 | 6 720 | 8 830 | 12 000 | 15 000 | 18 000 | 22 500 | 27 000 |
| JD5 | 25 | 680 | 900 | 1 130 | 1 350 | 1 800 | 2 250 | 2 700 | 3 380 | 4 050 |
| | 35 | 950 | 1 260 | 1 580 | 1 890 | 2 520 | 3 150 | 3 780 | 4 730 | 5 670 |
| | 40 | 1 080 | 1 440 | 1 800 | 2 160 | 2 880 | 3 600 | 4 320 | 5 400 | 6 480 |
| | 45 | 1 220 | 1 620 | 2 030 | 2 430 | 3 240 | 4 050 | 4 860 | 6 080 | 7 290 |
| | 70 | 1 890 | 2 520 | 3 150 | 3 780 | 5 040 | 6 300 | 7 560 | 9 450 | 11 340 |
| | 90 | 2 430 | 3 240 | 4 050 | 4 860 | 6 480 | 8 100 | 9 720 | 12 150 | 14 580 |
| | 105 | 2 510 | 3 780 | 4 730 | 5 670 | 7 560 | 9 450 | 11 340 | 14 180 | 17 010 |
| | 120 | 2 510 | 3 870 | 5 400 | 6 480 | 8 640 | 10 800 | 12 960 | 16 200 | 19 440 |
| JD6 | 25 | 460 | 610 | 760 | 920 | 1 220 | 1 530 | 1 830 | 2 290 | 2 750 |
| | 35 | 640 | 850 | 1 070 | 1 280 | 1 710 | 2 140 | 2 560 | 3 200 | 3 840 |
| | 40 | 730 | 980 | 1 220 | 1 460 | 1 950 | 2 440 | 2 930 | 3 660 | 4 390 |
| | 45 | 820 | 1 100 | 1 370 | 1 650 | 2 200 | 2 750 | 3 290 | 4 120 | 4 940 |
| | 70 | 1 280 | 1 710 | 2 140 | 2 560 | 3 420 | 4 270 | 5 120 | 6 410 | 7 690 |
| | 90 | 1 650 | 2 200 | 2 750 | 3 290 | 4 390 | 5 490 | 6 590 | 8 240 | 9 880 |
| | 105 | 1 920 | 2 560 | 3 200 | 3 840 | 5 120 | 6 410 | 7 690 | 9 610 | 11 530 |
| | 120 | 1 970 | 2 930 | 3 660 | 4 390 | 5 860 | 7 320 | 8 780 | 10 980 | 13 180 |

#### 4.4.2.5 *Maximum tensile load capacity*

The maximum tensile load capacity for a bolt subject to direct axial loading shall not exceed the value appropriate to the diameter and metal from which the bolt is manufactured as given in Table 4.11.

### TABLE 4.11
### MINIMUM REQUIRED SIZE OF WASHERS FOR STRUCTURAL BOLTED JOINTS

| Bolt* | Washer size, mm | | | Effective area† ($A_w$) for bearing | Axial design capacity of bolt‡ ($N_{d,tb}$) |
|---|---|---|---|---|---|
| | Thickness | Minimum diameter for round washers | Minimum side length for square washers | $mm^2$ | N |
| M6 | 1.6 | 30 | 25 | 200 | 6 400 |
| M8 | 2.0 | 36 | 32 | 350 | 11 700 |
| M10 | 2.5 | 45 | 40 | 570 | 18 600 |
| M12 | 3.0 | 55 | 50 | 750 | 27 000 |
| M16 | 4.0 | 65 | 57 | 1 330 | 50 200 |
| M20 | 5.0 | 75 | 65 | 1 960 | 78 400 |
| M24 | — | — | — | 2 830 | 113 000 |
| M30 | — | — | — | 3 740 | 180 000 |
| M36 | — | — | — | 4 780 | 312 000 |

* Bolts to be of grade 4.6, as given in AS 1111.1.

† The effective area ($A_w$) is less than the actual area because it includes an allowance for bending of the washer.

‡ Bolt capacity in accordance with AS 4100.

### 4.4.3 Design capacity for bolted connections

#### 4.4.3.1 *General*

The design capacity for Type 1 and Type 2 bolted connections shall be calculated in accordance with Clauses 4.4.3.2 to 4.4.3.4.

#### 4.4.3.2 *Type 1 joint*

The design capacity ($N_{d,j}$) for a Type 1 joint containing $n$ bolts in shear to resist lateral loads, as illustrated in Table 4.9 and Table 4.10, shall satisfy the following:

$$N_{d,j} \geq N^* \qquad \ldots 4.4(2)$$

where

$$N_{d,j} = \phi\, k_1\, k_{16}\, k_{17}\, n\, Q_{sk} \qquad \ldots 4.4(3)$$

and

$N^*$ = design action effect in shear

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for fasteners (see Clause 2.4.1.1)

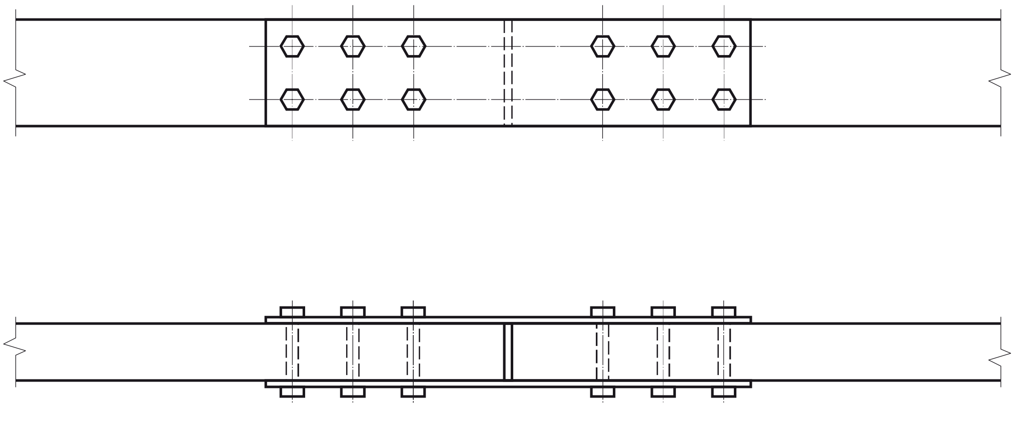
$k_{16}$ = 1.2 for bolts that transfer load through metal side plates (see Figure 4.7) of adequate strength, and the bolts are a close fit to the holes in these plates provided that $b_{eff}/D > 5$ for loads acting parallel to the grain and $b_{eff}/D > 10$ for loads acting perpendicular to the grain (where $b_{eff}$ denotes the effective timber thickness and $D$ is the bolt diameter)

= 1.0 otherwise

$k_{17}$ = factor for multiple bolted joint given in Table 4.12

$n$ = number of bolts resisting design action effect in shear

$Q_{sk}$ = characteristic capacities as derived in Clause 4.4.2.4. See also Clauses 4.4.4 and 4.4.5



$n_a = 3$, $n = 6$

FIGURE 4.7 METAL PLATE BOLTED JOINTS

## TABLE 4.12

## VALUE OF $k_{17}$ FOR USE IN THE DESIGN OF MULTIPLE FASTENER JOINTS FOR BOLTS, COACH SCREWS, SPLIT-RINGS AND SHEAR-PLATES

| Joint configuration | Values of $k_{17}$ | | | | |
|---|---|---|---|---|---|
| | $n_a \leq 4$† | $n_a = 5$ | $n_a = 10$ | $n_a = 15$ | $n_a \geq 16$ |
| Seasoned timber | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Unseasoned timber (no transverse restraint*) | 1.0 | 0.95 | 0.80 | 0.55 | 0.50 |
| Unseasoned timber (transverse restraint*) | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 |

* The term 'transverse restraint' refers to the possibility of restraint to timber shrinkage due to the joint detail.

† Where a connection consists of a single fastener, $k_{17}$ is taken as 1.0 for all timbers.

LEGEND:
$n_a$ = Total number of rows of fasteners per interface.

### 4.4.3.3 *Type 2 joints*

The design capacity $N_{d,j}$ for a Type 2 joint in which bolts are loaded in direct tension shall satisfy:

$$N_{d,j} \geq N^* \quad \text{. . . 4.4(4)}$$

where $N_{d,j}$ is the lesser of—

$$N_{d,j} = n\, N_{d,tb} \quad \text{. . . 4.4(5)}$$

or where crushing under the washer poses a limit to the strength—

$$N_{d,j} = \phi\, k_1\, k_7\, n\, f'_{pj}\, A_w \quad \text{. . . 4.4(6)}$$

and

$N^*$ = design action effect in direct tension

$n$ = number of bolts in the joint

$N_{d,tb}$ = design capacity of bolt in tension (see Table 4.11). See also Clause 4.4.5

$\phi$ = capacity factor for bolted joints (see Clause 2.3)

$k_1$ = duration of load factor for fasteners (see Clause 2.4.1.1)

$k_7$ = length of bearing factor (see Table 2.6) where the length of bearing is taken as the diameter or side length of the washer (see Table 4.11)

$f'_{pj}$ = characteristic bearing capacity for timber in joints (see Table C6)

$A_w$ = effective area of washer for bearing (see Table 4.11)

**4.4.3.4** *Bolted joints with loads at an angle to the bolt axis*

The design capacity ($N_{dj}$) for a joint in which bolts are loaded at an angle to the axis of the bolts (see Figure 4.8) shall satisfy—

(a) Clause 4.4.3.2 for that component of the load normal to the bolt axis; and

(b) Clause 4.4.3.3 for that component parallel to the bolt axis.

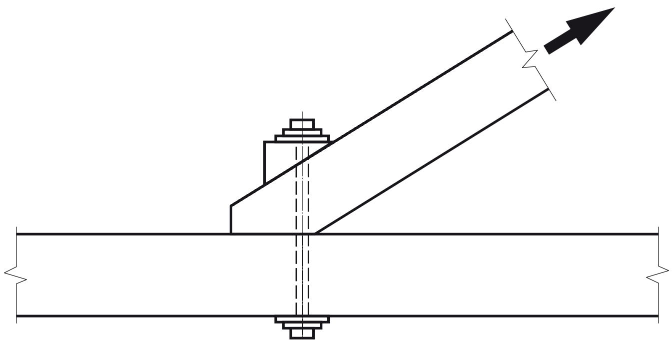


FIGURE 4.8 LOAD AT ANGLE TO BOLT AXIS

### 4.4.4 Spacings, edge and end distances for bolts

**4.4.4.1** *General*

Bolt spacings, edge and end distances shall comply with the requirements given in Clauses 4.4.4.2 to 4.4.4.4.

NOTE: Bolt spacings are measured from centres of bolts.

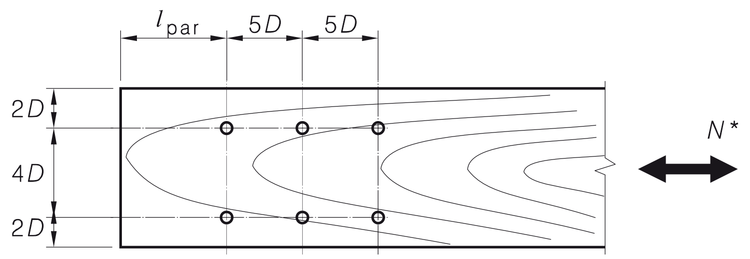
**4.4.4.2** *Loads parallel to grain*

The characteristic capacities given in Tables 4.10(A), 4.10(B), and 4.10(C) apply to joints in which the edge, end and between-fastener spacings are not less than those shown in Figure 4.9(a).

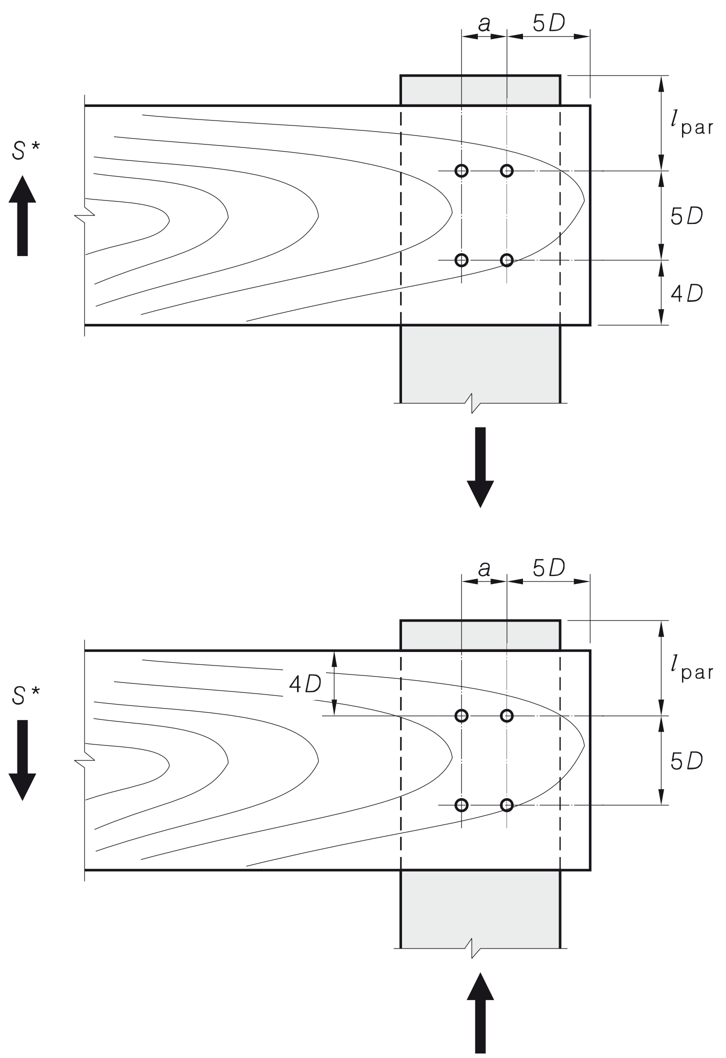
Similarly, the required end distance ($l_{par}$) shall be at least 8$D$ in tension joints in unseasoned timber, 7$D$ in tension joints in seasoned timber, and 5$D$ in compression joints and in joints subject to bending moment for both moisture conditions. It is appropriate to use lesser end distances in tension joints provided the characteristic capacity is reduced in proportion to the reduction in end distance. In no case shall the end distance for tension joints be less than 6$D$ for unseasoned timber and 5$D$ for seasoned timber.

**4.4.4.3** *Loads perpendicular to grain*

The minimum edge, end and between-fastener spacings shall be not less than those shown in Figure 4.9(b). The distance *a* shall be at least 2.5*D* for a *b*/*D* ratio of 2, and it shall be increased proportionately so that it is at least 5*D* for a *b*/*D* ratio of 6 or more, where *b* is the effective thickness of the member loaded perpendicular to the grain.




(a) Load applied parallel to grain




(b) Load applied perpendicular to grain

NOTE: *D* = Bolt diameter.

FIGURE 4.9 SPACINGS, EDGE AND END DISTANCES FOR BOLTED JOINTS

**4.4.4.4** *Loads acting at an angle to the grain*

For loads acting at an angle 0° to 30° to the grain, the spacings, edge and end distances shall be taken as for loads parallel to the grain. For loads acting at an angle of 30° to 90° to the grain, the spacings, edge and end distances shall be taken as for loads acting perpendicular to the grain.

### 4.4.5 Washers

In all timber-to-timber bolted structural joints, every bolt shall be fitted with a washer at each end, of a size not less than that given in Table 4.11. If smaller washers are used, then the characteristic capacities for laterally loaded bolts given in Tables 4.9 to 4.10 shall be reduced in proportion to the dimension of the washer diameter or side length.

### 4.4.6 Eccentric joints

When it is impracticable to ensure that all the members meeting at a joint (see Figure 4.9) are arranged symmetrically, that is, the members' centroidal axes intersecting on a common axis which is also the axis of resistance of the bolt or group of bolts, then the combination of primary stresses (induced by axial loads) and secondary stresses (induced by bending moment resulting from bolt eccentricities) shall be checked to ensure that no member or fastener is excessively stressed. In addition, the design capacity in transverse shear at an eccentric joint ($V_{d,sj}$) shall satisfy the following:

$$V_{d,sj} \geq V_{sj}^{*} \qquad \ldots 4.4(7)$$

where

$$V_{d,sj} = \phi\, k_1\, k_4\, k_6\, f'_{sj}\, A_{sj} \qquad \ldots 4.4(6)$$

and

$V_{sj}^{*}$ = design action effect in shear on the joint

$\phi$ = capacity factor (see Clause 2.3)

$k_1, k_4, k_6$ = modification factors given in Section 2 with $k_1$ appropriate for the member

$f'_{sj}$ = characteristic value in shear at joint details appropriate to species strength group

$A_{sj}$ = transverse shear plane area at joint section

$= \frac{2}{3} b\, d_s$ (see Figure 4.10 where $b$ is the thickness of the member)

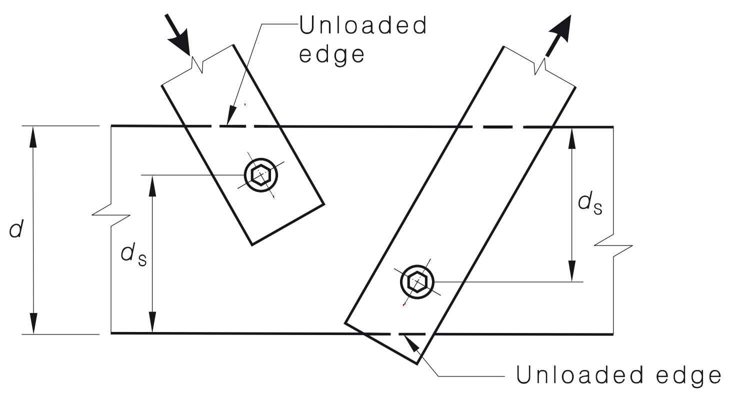



FIGURE 4.10 ECCENTRIC JOINTS

## 4.5 DESIGN OF COACH SCREWED JOINTS

### 4.5.1 General

The characteristic capacities given in Clause 4.5.2 are applicable to steel coach screws as specified in AS/NZS 1393, subject to lateral and withdrawal loads.

### 4.5.2 Characteristic capacities for coach screws

**4.5.2.1** *Type 1 joints*

Characteristic capacities for Type 1 joints for coach screws are as follows:

(a) *Lateral loads in side grain (see Figure* 4.2*(a))* The characteristic capacities for coach screws laterally loaded in shear shall be taken as for bolts as specified in Clause 4.4.2, subject to the following conditions:

(i) For the purpose of Clause 4.4.2, a coach screw shall be considered to be a bolt of diameter equal to the shank diameter of the screw.

(ii) The coach screws shall be fitted with washers as specified in Clause 4.4.5.

(iii) In a two-member joint, the thinner member shall have a minimum thickness of three times the shank diameter of the coach screw.

(iv) The diameter of the hole for the shank shall be not less than the shank diameter of the screw nor exceed it by more than 1 mm or 10% of the shank diameter, whichever is the lesser. The diameter of the hole for the threaded portion of the screw shall not exceed the root diameter of the screw.

The depth of the hole shall exceed the intended depth to which the screw is to be driven.

The screw shall be turned into place and not hammered.

(v) Timber thicknesses and coach screw lengths as shown in Figure 4.6(a) shall be such that—

(A) the thickness of outermost member, $t_1 > 3D$; and

(B) the depth of penetration into the second member ($t_p$), for species groups is—

(1) J1, JD1, J2, JD2, JD3 .......... $t_p > 7D$;

(2) J3, JD4.......... $t_p > 8D$;

(3) J4, JD5.......... $t_p > 10D$; and

(4) J5, J6, JD6 ..........$t_p > 12D$.

For lesser values of $t_p$, the basic load shall be reduced in proportion to the decrease in $t_p$ and the coach screw shall be considered as non-loadbearing if $t_p$ is less than $4D$.

(b) *Lateral loads in end grain (see Figure* 4.2*(b))* The characteristic capacities for coach screws, laterally loaded in the end grain shall not exceed 60% of the values determined in accordance with Clause 4.5.2.1(a).

NOTE: The modification factor $k_{13}$ accounts for end grain effects in Equation 4.5(2) (see Clause 4.5.3.1).

**4.5.2.2** *Type 2 joints*

Characteristic capacities for Type 2 joints shall be as follows:

(a) *Withdrawal loads from side grain (see Figure* 4.4*(a))* The characteristic capacities for coach screws, driven by hand or machine, axially loaded in withdrawal from the side grain of unseasoned timber are given in Table 4.13(A) and of seasoned timber are given in Table 4.13(B).

(b) *Withdrawal loads from end grain (see Figure 4.4(b))* The characteristic capacities for coach screws, driven by hand or machine, axially loaded in withdrawal from the end grain of unseasoned timber and seasoned timber shall not exceed 60% of the values given in Table 4.13(A) and Table 4.13(B).

NOTE: The modification factor $k_{13}$ accounts for end grain effects in Equation 4.5(5) (see Clause 4.5.3.2).

## TABLE 4.13(A)

## CHARACTERISTIC CAPACITY FOR A SINGLE COACH SCREW AXIALLY LOADED IN WITHDRAWAL FROM SIDE GRAIN—UNSEASONED TIMBER

| Joint group | Characteristic capacity ($Q_k$), N per mm penetration of thread | | | | | |
|---|---|---|---|---|---|---|
| | Shank diameter, mm | | | | | |
| | 6 | 8 | 10 | 12 | 16 | 20 |
| J1 | 149 | 168 | 189 | 208 | 241 | 270 |
| J2 | 118 | 133 | 152 | 168 | 193 | 218 |
| J3 | 83 | 98 | 112 | 124 | 143 | 162 |
| J4 | 66 | 69 | 77 | 83 | 98 | 112 |
| J5 | 50 | 52 | 58 | 68 | 77 | 83 |
| J6 | 39 | 39 | 42 | 48 | 58 | 64 |

## TABLE 4.13(B)

## CHARACTERISTIC CAPACITY FOR A SINGLE COACH SCREW AXIALLY LOADED IN WITHDRAWAL FROM SIDE GRAIN— SEASONED TIMBER

| Joint group | Characteristic capacity ($Q_k$), N per mm penetration of thread | | | | | |
|---|---|---|---|---|---|---|
| | Shank diameter, mm | | | | | |
| | 6 | 8 | 10 | 12 | 16 | 20 |
| JD1 | 185 | 210 | 232 | 261 | 301 | 338 |
| JD2 | 147 | 166 | 191 | 210 | 241 | 272 |
| JD3 | 104 | 122 | 141 | 154 | 179 | 205 |
| JD4 | 83 | 87 | 97 | 104 | 124 | 139 |
| JD5 | 62 | 66 | 73 | 85 | 97 | 104 |
| JD6 | 48 | 48 | 54 | 60 | 73 | 79 |

**4.5.2.3** *Maximum tensile capacity*

The maximum tensile capacity for a coach screw subject to direct axial loading shall not exceed the value appropriate to the diameter, as given in Table 4.14.

**TABLE 4.14**

**MAXIMUM CAPACITY IN TENSION PER COACH SCREW SUBJECT TO AXIAL LOADS**

| Nominal diameter of coach screw mm | Maximum tensile load ($N_{d,tc}$) N |
|---|---|
| 6 | 3 900 |
| 8 | 7 700 |
| 10 | 11 600 |
| 12 | 17 400 |
| 16 | 39 000 |
| 20 | 61 000 |

### 4.5.3 Design capacity of coach screwed joints

**4.5.3.1** *Type 1 joints*

The design capacity ($N_{d,j}$) for a joint containing $n$ coach screws to resist shear loads, similar to that illustrated in Figure 4.1(a) for Type 1 joints, shall satisfy the following:

$$N_{d,j} \geq N^* \quad \ldots 4.5(1)$$

where

$$N_{d,j} = \phi\, k_1\, k_{13}\, k_{16}\, k_{17}\, n\, Q_{sk} \quad \ldots 4.5(2)$$

and

$N^*$ = design action effect in shear

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for fasteners (see Clause 2.4.1.1)

$k_{13}$ = 1.0 for coach screws in side grain
= 0.6 for coach screws in end grain

$k_{16}$ = 1.2 where the load is applied through metal side plates of adequate strength to transfer the load and the coach screws are a close fit to the holes in these plates
= 1.0 otherwise

$k_{17}$ = factor for multiple coach screwed joints given in Table 4.12

$n$ = number of coach screws in the connection

$Q_{sk}$ = characteristic capacity defined in Clause 4.4.2.4 taking the thickness of the innermost member as equal to $t_p$, the depth of penetration of the coach screw into that member (see Figure 4.6)

**4.5.3.2** *Type 2 joints*

The design capacity ($N_{d,j}$) for coach screw joints axially loaded in withdrawal shall satisfy the following:

$$N_{d,j} \geq N^* \quad \ldots 4.5(3)$$

where $N_{d,j}$ is the lesser of—

$$N_{d,j} = n\, N_{d,tc} \qquad \ldots 4.5(4)$$

$$N_{d,j} = \phi\, k_{13}\, l_p\, n\, Q_k \qquad \ldots 4.5(5)$$

or where crushing under the head poses a limit to the strength

$$N_{d,j} = \phi\, k_1\, k_7\, n\, f'_{pj} A_w \qquad \ldots 4.5(6)$$

and

$N^*$ = design load action effect on the joint produced by strength limit states design loads (tension across the joint)

$n$ = number of coach screws in the connection

$N_{d,tc}$ = design maximum tensile capacity of a single coach screw given in Table 4.14

$\phi$ = capacity factor (see Clause 2.3)

$k_{13}$ = 1.0 for withdrawal from side grain
= 0.60 for withdrawal from end grain

$l_p$ = depth of penetration of the threaded portion of the coach screw into the innermost member

$Q_k$ = characteristic capacity given in Tables 4.13(A) and 4.13(B) (see also Clause 4.4.5)

$k_1$ = duration of load factor for fasteners (see Clause 2.4.1.1)

$k_7$ = length of bearing factor (see Table 2.6) where the length of bearing is taken as the diameter or side length of washer (see Table 4.11)

$f'_{pj}$ = characteristic bearing capacity for timber in joints (see Table C6, Appendix C)

$A_w$ = effective area of washer for bearing (see Table 4.11)

NOTE: The duration of load factor $k_1$ does not apply to withdrawal capacity for coach screws.

### 4.5.4 Spacings, edge and end distances for coach screws

The spacings, edge and end distances for coach screws shall comply with the requirements for bolts given in Clause 4.4.4.

## 4.6 DESIGN OF SPLIT-RING FASTENER JOINTS

### 4.6.1 General

The requirements of this Clause apply to split-ring fasteners of nominal size 64 mm and 102 mm.

An M12 bolt is used in conjunction with 64 mm fasteners and an M20 bolt is used with 102 mm fasteners. The bolts shall be fitted with washers in accordance with Clause 4.4.5.

NOTE: In computations for the design of the timber members, the projected area of the groove to receive the fastener in one member may be taken as 710 $mm^2$ for the 64 mm fastener and 1450 $mm^2$ for the 102 mm fastener.

### 4.6.2 Characteristic capacities for split-ring fasteners

The characteristic capacities for split-ring fasteners in unseasoned timber ($Q_k$) are given in Table 4.17. The capacities apply to a connector unit comprising one split-ring in the contact faces of a timber-to-timber joint with the bolt and split-ring in single shear.

### 4.6.3 Design capacity for split-ring fastener joints

The design capacity ($N_{d,j}$) for a joint containing $n$ split-ring fasteners to resist shear loads shall satisfy the following:

$$N_{d,j} \geq N^{*} \quad \ldots 4.6(1)$$

where

$$N_{d,j} = \phi\, k_1\, k_{15}\, k_{17}\, k_{18}\, n\, Q_k \quad \ldots 4.6(2)$$

and

$N^{*}$ = design action effect on joint in shear

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for joints (see Clause 2.4.1.1)

$k_{15}$ = 1.0 for unseasoned timber
= factor for seasoned timber given in Table 4.15

$k_{17}$ = factor for multiple joints given in Table 4.12

$k_{18}$ = 1.0 for loads applied in compression along the grain, and for loads applied perpendicular to the grain
= factor for tension loads applied along the grain given in Table 4.16. It is appropriate to determine values of $k_{18}$ for intermediate directions using Hankinson's formula (see Clause 4.4.2.4(c))

$n$ = number of split-ring fasteners in the joint resisting design action effect in shear

$Q_k$ = characteristic capacity of a split-ring fastener given in Table 4.17

**TABLE 4.15**

**FACTOR $k_{15}$ FOR SPLIT-RING AND SHEAR-PLATE FASTENERS IN SEASONED TIMBER**

| **Species group** | JD1, JD2, JD3 | | | | | | | JD4, JD5, JD6 |
|---|---|---|---|---|---|---|---|---|
| **Angle of load to grain** | 0° | 15° | 30° | 45° | 60° | 75° | 90° | any |
| **Factor $k_{15}$** | 1.25 | 1.29 | 1.33 | 1.38 | 1.42 | 1.46 | 1.50 | 1.25 |

**TABLE 4.16**

**FACTOR $k_{18}$ FOR SPLIT-RING AND SHEAR-PLATE FASTENERS USED IN JOINTS SUBJECT TO TENSION LOADS**

| **Size of split-ring or shear-plate mm** | **Factor $k_{18}$** | | |
|---|---|---|---|
| | **Fastener remote from ends of members*** | **Fastener at ends of members** | |
| | | **Seasoned timber** | **Unseasoned timber†** |
| 64, 67 | 1.0 | 1.0 | 0.5 |
| 102 | 0.8 | 0.6 | 0.3 |

* A fastener may be taken to occur in the middle of a member if the distance from the fastener to the end of the timber is greater than 10$D$, where $D$ is the fastener diameter.

† Factors for seasoned timber may be used if the timber has negligible tendency to split (see Clause 4.1.5).

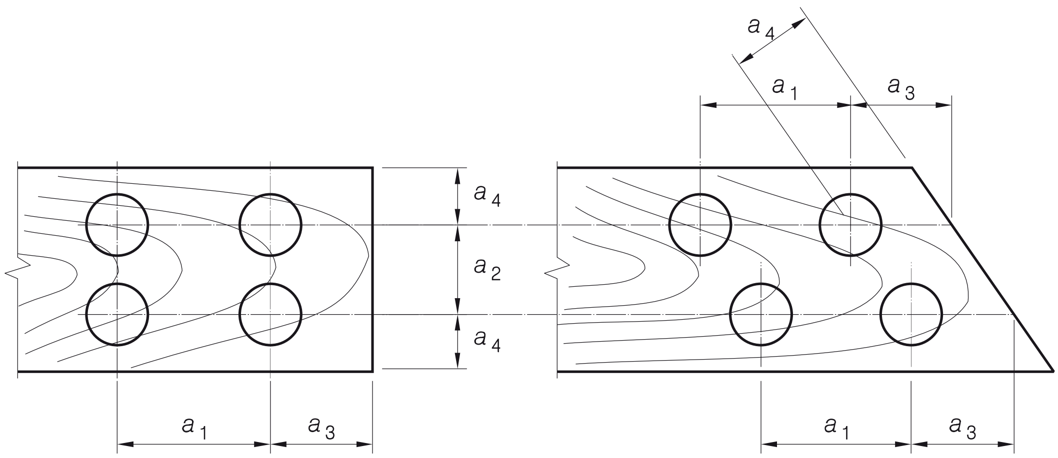
## TABLE 4.17

## CHARACTERISTIC CAPACITY FOR A SINGLE SPLIT-RING FASTENER IN UNSEASONED TIMBER

| Joint group | Internal diameter of ring | Minimal nominal thickness of timber, mm | | Characteristic capacity per fastener in single shear, N | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Angle of load to grain | | | | | | |
| | mm | Split-ring fastener in one side only | Split-ring fasteners opposite in two sides | 0° | 15° | 30° | 45° | 60° | 75° | 90° |
| J1 | 64 | — | 38 | 40 000 | 38 000 | 33 000 | 28 000 | 25 000 | 23 000 | 22 000 |
| | | 25 | 50 | 60 000 | 57 000 | 50 000 | 43 000 | 37 000 | 37 000 | 33 000 |
| | 102 | — | 50 | 77 000 | 75 000 | 69 000 | 63 000 | 58 000 | 54 000 | 53 000 |
| | | 38 | 75 | 105 000 | 102 000 | 94 000 | 86 000 | 78 000 | 74 000 | 72 000 |
| J2 | 64 | — | 38 | 32 000 | 30 000 | 26 000 | 22 000 | 19 000 | 17 000 | 17 000 |
| | | 25 | 50 | 48 000 | 45 000 | 39 000 | 33 000 | 29 000 | 26 000 | 26 000 |
| | 102 | — | 50 | 66 000 | 63 000 | 57 000 | 50 000 | 44 000 | 41 000 | 40 000 |
| | | 38 | 75 | 90 000 | 86 000 | 77 000 | 67 000 | 60 000 | 56 000 | 56 000 |
| J3 | 64 | — | 38 | 30 000 | 28 000 | 23 000 | 19 000 | 16 000 | 14 000 | 14 000 |
| | | 25 | 50 | 45 000 | 42 000 | 35 000 | 29 000 | 24 000 | 22 000 | 21 000 |
| | 102 | — | 50 | 61 000 | 57 000 | 48 000 | 40 000 | 34 000 | 31 000 | 30 000 |
| | | 38 | 75 | 78 000 | 73 000 | 63 000 | 53 000 | 46 000 | 42 000 | 40 000 |
| J4 | 64 | 25 | 38 | 21 000 | 21 000 | 19 000 | 18 000 | 16 000 | 15 000 | 15 000 |
| | | — | 50 | 29 000 | 29 000 | 27 000 | 24 000 | 22 000 | 21 000 | 21 000 |
| | | 38 | 64 | 32 000 | 31 000 | 29 000 | 27 000 | 21 000 | 23 000 | 23 000 |
| | 102 | — | 50 | 47 000 | 45 000 | 39 000 | 34 000 | 30 000 | 27 000 | 26 000 |
| | | 38 | — | 50 000 | 49 000 | 45 000 | 41 000 | 38 000 | 35 000 | 35 000 |
| | | 50 | 75 | 62 000 | 59 000 | 53 000 | 46 000 | 41 000 | 37 000 | 36 000 |
| J5 | 64 | 25 | 38 | 17 000 | 16 000 | 14 000 | 12 000 | 10 000 | 9 400 | 9 100 |
| | | — | 50 | 25 000 | 24 000 | 21 000 | 18 000 | 16 000 | 14 000 | 14 000 |
| | | 38 | 64 | 26 000 | 22 000 | 23 000 | 21 000 | 20 000 | 19 000 | 18 000 |
| | 102 | — | 50 | 37 000 | 35 000 | 31 000 | 27 000 | 24 000 | 22 000 | 21 000 |
| | | 38 | — | 40 000 | 39 000 | 36 000 | 33 000 | 30 000 | 28 000 | 28 000 |
| | | 50 | 75 | 46 000 | 44 000 | 40 000 | 36 000 | 33 000 | 30 000 | 30 000 |
| J6 | 64 | 25 | 38 | 14 000 | 13 000 | 11 000 | 9 600 | 8 400 | 7 700 | 7 400 |
| | | — | 50 | 20 000 | 19 000 | 17 000 | 15 000 | 13 000 | 12 000 | 11 000 |
| | | 38 | 64 | 21 000 | 20 000 | 19 000 | 17 000 | 16 000 | 15 000 | 14 000 |
| | 102 | — | 50 | 31 000 | 29 000 | 26 000 | 22 000 | 19 000 | 18 000 | 17 000 |
| | | 38 | — | 32 000 | 31 000 | 29 000 | 26 000 | 24 000 | 23 000 | 22 000 |
| | | 50 | 75 | 37 000 | 36 000 | 33 000 | 29 000 | 26 000 | 25 000 | 24 000 |

### 4.6.4 Spacings, edge and end distances for split-ring fasteners

For split-ring fasteners, Table 4.18 provides recommended minimum values of spacings, edge and end distances, which are defined and illustrated in Figure 4.11.




LEGEND:

$a_1$ = spacing parallel to grain
$a_2$ = spacing perpendicular to grain
$a_3$ = end distance
$a_4$ = edge distance

FIGURE 4.11 SPACING, EDGE AND END DISTANCES FOR SPLIT-RING AND SHEAR-PLATE FASTENERS

**TABLE 4.18**

**MINIMUM SPACINGS, EDGE AND END DISTANCES FOR SPLIT-RINGS AND SHEAR-PLATES**

| Spacing type | | Minimum distance mm | |
|---|---|---|---|
| | | *D** = 64 mm or 67 mm | *D** = 102 mm |
| End distance | Tension numbers | 150 | 180 |
| | Compression members | 100 | 140 |
| Edge distance | 0° to 30° angle of load to grain | 45 | 70 |
| | 30° to 90° angle of load to grain:<br>—Compression side<br>—Opposite compression side | <br>70<br>45 | <br>95<br>70 |
| Between fasteners | 0° to 30° angle of load to grain:<br>—Spacing parallel to grain<br>—Spacing perpendicular to grain | <br>180<br>90 | <br>230<br>140 |
| | 30° to 90° angle to load to grain:<br>—Spacing parallel to grain<br>—Spacing perpendicular to grain | <br>90<br>115 | <br>140<br>165 |

* *D* is the nominal size of the split-ring or shear-plate fastener.

## 4.7 DESIGN OF SHEAR-PLATE FASTENER JOINTS

### 4.7.1 General

The requirements of this Clause relate to shear-plate fasteners of nominal diameter 67 mm and 102 mm.

An M20 bolt is used in conjunction with a 67 mm diameter shear-plate fastener and an M24 bolt with a 102 mm diameter shear-plate fastener. Bolts shall be fitted with washers as specified in Clause 4.4.5.

NOTE: In computations for the design of the timber members, the projected area of the groove in the timber to receive the shear-plate may be taken as 632 $mm^2$ for the 67 mm shear-plate and 1600 $mm^2$ for the 102 mm shear-plate.

### 4.7.2 Characteristic capacities for shear-plate fasteners

The characteristic capacities for shear-plate fasteners in unseasoned timber ($Q_k$) are given in Table 4.19. These capacities apply to a fastener unit comprising one shear-plate in the contact face of a timber-to-steel joint with its bolt in single shear.

### 4.7.3 Design capacity for joints with shear-plates

The design capacity ($N_{d,j}$) for a joint containing $n$ shear-plate fasteners resisting shear load, shall satisfy the following:

$$N_{d,j} \geq N^* \qquad \text{. . . 4.7(1)}$$

where $N_{d,j}$ is the lesser of—

$$= n\, V_{d,f}\text{; or} \qquad \text{. . . 4.7(2)}$$

$$= \phi\, k_1\, k_{15}\, k_{17}\, k_{18}\, n\, Q_k \qquad \text{. . . 4.7(3)}$$

and

$N^*$ = design action effect on joint

$n$ = number of shear-plate fasteners in the joint

$V_{d,f}$ = fastener capacity in shear (see Table 4.19)

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load for joints (see Clause 2.4.1.1)

$k_{15}$ = 1.0 for unseasoned timber

= factor for seasoned timber given in Table 4.15

$k_{17}$ = factor for multiple fastener joints given in Table 4.12

$k_{18}$ = 1.0 for loads applied in compression along the grain, and for loads applied perpendicular to the grain

= factor for tension loads applied along the grain, given in Table 4.16 (it is appropriate to determine values of $k_{18}$ for intermediate directions using Hankinson's formula, see Clause 4.4.2.4(c))

$Q_k$ = characteristic capacity of shear-plate fasteners given in Table 4.19

NOTE: Some characteristic capacities in Table 4.19 exceed the values given in Table 4.20 but are included for the purposes of interpolation.

### 4.7.4 Spacings, edge and end distances for shear-plate fasteners

For shear-plate fasteners, Table 4.18 gives recommended minimum values of spacings, edge and end distances which are defined and illustrated in Figure 4.11.

## TABLE 4.19

## CHARACTERISTIC CAPACITY FOR A SINGLE SHEAR-PLATE FASTENER IN UNSEASONED TIMBER

| Joint group | External diameter of shear-plate | Bolt diameter | Minimum nominal thickness of timber mm | | Characteristic capacity per fastener (with bolt) in single shear N | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Angle of load to grain | | | | | | |
| | | | Shear-plate fastener in one side only | Shear-plate fastener opposite in two sides | 0° | 15° | 30° | 45° | 60° | 75° | 90° |
| J1 | 67 | M20 | 38 | 50 | 48 000 | 45 000 | 39 000 | 34 000 | 30 000 | 27 000 | 26 000 |
| | 102 | M24 | — | 50 | 77 000 | 75 000 | 69 000 | 63 000 | 58 000 | 54 000 | 53 000 |
| | | | 50 | 75 | 105 000 | 102 000 | 94 000 | 86 000 | 78 000 | 74 000 | 72 000 |
| J2 | 67 | M20 | 38 | 50 | 38 000 | 37 000 | 32 000 | 27 000 | 23 000 | 21 000 | 20 000 |
| | 102 | M24 | — | 50 | 66 000 | 63 000 | 57 000 | 50 000 | 44 000 | 41 000 | 40 000 |
| | | | 50 | 75 | 90 000 | 86 000 | 77 000 | 67 000 | 60 000 | 56 000 | 54 000 |
| J3 | 67 | M20 | 38 | 50 | 36 000 | 34 000 | 28 000 | 23 000 | 19 000 | 17 000 | 17 000 |
| | 102 | M24 | — | 50 | 61 000 | 57 000 | 48 000 | 40 000 | 34 000 | 31 000 | 30 000 |
| | | | 50 | 75 | 83 000 | 78 000 | 66 000 | 54 000 | 46 000 | 42 000 | 40 000 |
| J4 | 67 | M20 | 38 | 50 | 29 000 | 27 000 | 23 000 | 18 000 | 16 000 | 14 000 | 13 000 |
| | 102 | M24 | — | 50 | 36 000 | 35 000 | 33 000 | 30 000 | 27 000 | 26 000 | 25 000 |
| | | | 50 | 75 | 48 000 | 47 000 | 43 000 | 39 000 | 36 000 | 34 000 | 33 000 |
| J5 | 67 | M20 | 38 | 50 | 20 000 | 19 000 | 17 000 | 14 000 | 12 000 | 11 000 | 11 000 |
| | 102 | M24 | — | 50 | 32 000 | 31 000 | 29 000 | 26 000 | 23 000 | 22 000 | 21 000 |
| | | | 50 | 75 | 44 000 | 42 000 | 39 000 | 34 000 | 31 000 | 29 000 | 28 000 |
| J6 | 67 | M20 | 38 | 50 | 17 000 | 16 000 | 14 000 | 11 000 | 9 900 | 9 000 | 8 700 |
| | 102 | M24 | — | 50 | 26 000 | 25 000 | 23 000 | 21 000 | 19 000 | 18 000 | 17 000 |
| | | | 50 | 75 | 35 000 | 34 000 | 31 000 | 28 000 | 25 000 | 23 000 | 23 000 |

## TABLE 4.20

## BOLT CAPACITIES IN SHEAR

| Bolt size | Shear values (single shear), N* | |
|---|---|---|
| | Threads included in shear plane ($V_{d,fN}$) | Threads excluded from shear plane ($V_{d,fX}$) |
| M20 | 44 600 | 62 300 |
| M24 | 64 300 | 89 700 |

* The values apply for Grade 4.6 bolts and have been derived in accordance with AS 4100.

# SECTION 5 PLYWOOD

## 5.1 GENERAL

The requirements of this Section apply for plywood which has been manufactured and stress graded in accordance with AS/NZS 2269.0.

All plywood manufactured in accordance with AS/NZS 2269.0 is Type A bonded.

Section modulus ($Z$) and second moment of area ($I$) shall be determined in accordance with AS/NZS 2269.0 and Appendix I.

NOTES:

1 The section modulus ($Z$) and the second moment of area ($I$) determined in accordance with AS/NZS 2269.0 are based on the application of parallel ply theory.

2 Values of section modulus ($Z$) and second moment of area ($I$) are given in Appendix I for standard constructions, together with a calibrated method of calculation.

## 5.2 DESIGN PROPERTIES

### 5.2.1 Characteristic values for structural design

The characteristic values for structural design for various F-grade classifications of structural plywood are given in Table 5.1.

A3

**TABLE 5.1**

**STRUCTURAL PLYWOOD—CHARACTERISTIC VALUES FOR F-GRADES (Moisture content 15% or less)**

| Stress grade | Characteristic values, MPa | | | | | | |
|---|---|---|---|---|---|---|---|
| | Bending ($f'_b$) | Tension ($f'_t$) | Panel shear ($f'_s$) | Compression in the plane of the sheet ($f'_c$) | Bearing normal to the plane of the sheet ($f'_p$) | Short duration average modulus of elasticity ($E$) | Short duration average modulus of rigidity ($G$) |
| F34 | 90 | 54 | 6.0 | 68 | 31 | 21 500 | 1 075 |
| F27 | 70 | 45 | 6.0 | 55 | 27 | 18 500 | 925 |
| F22 | 60 | 36 | 5.5 | 45 | 23 | 16 000 | 800 |
| F17 | 45 | 27 | 5.1 | 36 | 20 | 14 000 | 700 |
| F14 | 36 | 22 | 4.8 | 27 | 15 | 12 000 | 625 |
| F11 | 31 | 18 | 4.5 | 22 | 12 | 10 500 | 525 |
| F8 | 25 | 15 | 4.2 | 20 | 9.7 | 9 100 | 455 |
| F7 | 20 | 12 | 3.9 | 15 | 7.7 | 7 900 | 395 |

### 5.2.2 Capacity factor

The capacity factor ($\phi$) to be used in the computation of the design capacities for plywood structural elements, shall comply with the values given in Table 2.1.

## 5.3 MODIFICATION FACTORS

### 5.3.1 General

Modification factors for strength and stiffness shall be taken from the relevant clauses in Section 2 and from the factors given in Tables 5.2 to 5.4.

### 5.3.2 Duration of load

The modification factors for duration of load ($k_1$), stiffness ($j_2$ and $j_3$) given in Clause 2.4.1 shall be used where appropriate.

### 5.3.3 Moisture condition

Where plywood is subjected to conditions such that the average moisture content for a 12 month period will exceed 15%, the modification factors $k_{19}$ and $j_6$, given in Tables 5.2(A) and 5.2(B), respectively, shall be used.

**TABLE 5.2(A)**

**MOISTURE CONTENT FACTORS FOR PLYWOOD—STRENGTH FACTOR $k_{19}$**

| Type of stress | Factor $k_{19}$ | |
|---|---|---|
| | Moisture content* 15% or less | Moisture content* 25% or more |
| Bending | 1.0 | 0.6 |
| Tension in plane of sheet | 1.0 | 0.7 |
| Shear | 1.0 | 0.6 |
| Compression in plane of sheet | 1.0 | 0.4 |
| Compression normal to plane of sheet | 1.0 | 0.45 |

* For moisture contents between 15 and 25%, use linear interpolation to obtain $k_{19}$ factor.

**TABLE 5.2(B)**

**MOISTURE CONTENT FACTORS FOR PLYWOOD—STIFFNESS FACTOR $j_6$**

| Type of stiffness | Factor $j_6$ | |
|---|---|---|
| | Moisture content* 15% or less | Moisture content* 25% or more |
| Modulus of elasticity | 1.0 | 0.8 |
| Modulus of rigidity | 1.0 | 0.6 |

* For moisture contents between 15 and 25%, use linear interpolation to obtain $j_6$ factor.

### 5.3.4 Temperature

The provisions of Clause 2.4.3 for seasoned timber shall apply to structural plywood in a similar manner.

### 5.3.5 Plywood assembly factor

The characteristic values for strength and elastic moduli given in Table 5.1, other than for shear, shall apply for individual plies in the direction of the grain of the veneer. In order to use these values to derive the properties of plywood formed from several layers of veneer, computations shall be based on the section properties determined in accordance with AS/NZS 2269.0 and relevant sections of this Standard. The characteristic value for panel shear $(f'_s)$ shall apply for the full thickness of the plywood irrespective of grain direction.

Additionally, the characteristic values for strength and stiffness shall be modified by the plywood assembly factors ($g_{19}$) given in Table 5.3, 5.4 or 5.5, as appropriate.

## 5.4 LOADING NORMAL TO THE PLANE OF THE PLYWOOD PANEL

### 5.4.1 General

The design capacity and deflection for a plywood panel when subject to actions that act normal to the plane of the plywood shall satisfy the requirements of Clauses 5.4.2 to 5.4.5.

### 5.4.2 Bending strength

The design capacity of plywood in bending ($M_{d,p}$) for strength limit state, shall satisfy the following:

$$M_{d,p} \geq M_p^* \quad \ldots 5(1)$$

where

$$M_{d,p} = \phi\, k_1\, k_{19}\, g_{19}\, f'_b\, Z_p \quad \ldots 5(2)$$

and

$M_p^*$ = design action effect for flatwise bending of plywood as shown in Figure 5.1

$\phi$ = capacity factor of plywood (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.3)

$f'_b$ = characteristic value in bending

$Z_p$ = section modulus of plywood = $I_p/y_p$

where

$I_p$ = second moment of area of parallel plies whose grain direction is parallel to the span

$y_p$ = distance from the neutral axis to the extreme fibre of the outermost parallel ply

NOTES:

1 The values of $f'_b$ for plywood assigned an F-grade classification are given in Table 5.1

2 Values of $I_p$ are given in Appendix I for standard structural plywood constructions.

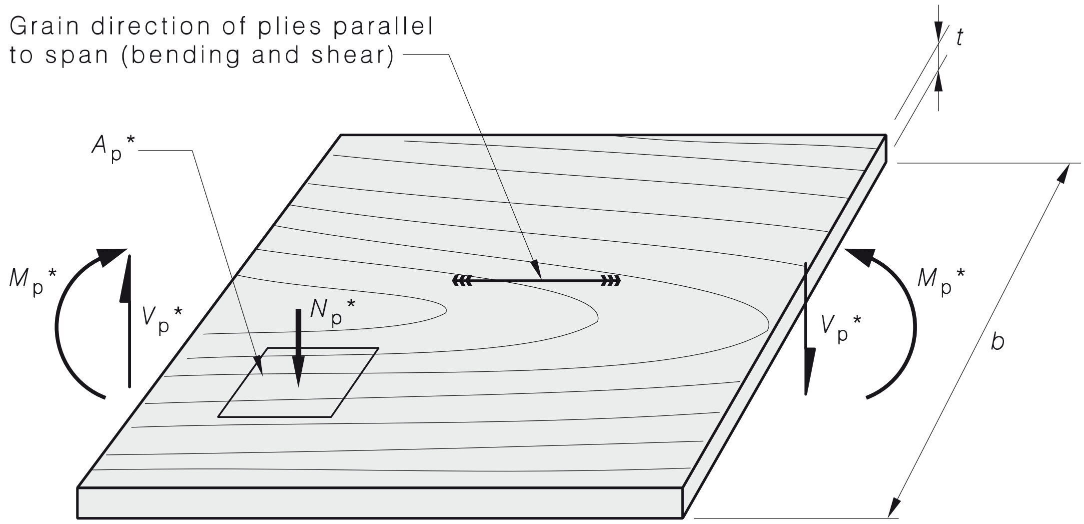



FIGURE 5.1 NOTATION FOR BEARING AND SHEAR NORMAL TO THE FACE OF THE PLYWOOD PANEL AND FOR FLATWISE BENDING

**TABLE 5.3**

**ASSEMBLY FACTORS $g_{19}$ FOR BEARING AND SHEAR NORMAL TO THE FACE OF THE PLYWOOD PANEL AND IN FLATWISE BENDING**

| Property | Direction of face plies | Assembly factor ($g_{19}$) |
|---|---|---|
| Bending strength | Perpendicular to span<br>—3 ply<br>—5 ply or more<br>Parallel to span | <br>1.2<br>1.0<br>1.0 |
| Shear strength | Any orientation | 0.4 |
| Bearing strength | Any orientation | 1.0 |
| Bending deflection | Parallel or perpendicular to span | 1.0 |
| Shear deformation | Parallel or perpendicular to span | 1.0 |

### 5.4.3 Shear strength (interlamina shear)

The design capacity of plywood in beam shear ($V_{d,p}$) for strength limit state shall satisfy the following:

$$V_{d,p} \geq V_p^* \qquad \text{. . . 5(3)}$$

where

$$V_{d,p} = \phi\, k_1\, k_{19}\, g_{19}\, f_s'\, A_s \qquad \text{. . . 5(4)}$$

and

$V_p^*$ = design action effect for shear normal to the face of the plywood panel as shown in Figure 5.1

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.3)

$f_s'$ = characteristic value in panel shear (in-plane)

$A_s$ = $\dfrac{2}{3}(b\,t)$ (see Figure 5.1), for shear in beams; or

= full shear area, for local (punching) shear

where

$b$ = breadth of plywood

$t$ = full thickness of plywood

NOTE: The values of $f_s'$ for plywood assigned an F-grade classification are given in Table 5.1

### 5.4.4 Bearing strength

The design capacity of plywood in bearing ($N_{d,p}$) for strength limit state shall satisfy the following:

$$N_{d,p} \geq N_p^* \qquad \text{. . . 5(5)}$$

where

$$N_{d,p} = \phi\, k_1\, k_7\, k_{19}\, g_{19}\, f_p'\, A_p \qquad \text{. . . 5(6)}$$

and

$N_p^*$ = design action effect for bearing normal to the face of the plywood panel as shown in Figure 5.1

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_7$ = modification factor for length of bearing (see Table 2.6)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.3)

$f'_p$ = characteristic value in compression normal to the plane of the panel

$A_p$ = bearing area under the design loads

NOTE: The values of $f'_p$ for plywood assigned an F-grade classification are given in Table 5.1.

### 5.4.5 Deflection in bending

In calculating deflection, the cross-section stiffness shall be based on the second moment of area ($I$) determined in accordance with AS/NZS 2269.0 (see Appendix I), the characteristic modulus of elasticity values given in Table 5.1, the moisture content stiffness factor $j_6$ given in Table 5.2(B) and the assembly factor $g_{19}$ given in Table 5.3. The duration of load factor $j_2$ given in Table 2.4 shall be used in computing deflections.

## 5.5 LOADING IN THE PLANE OF THE PLYWOOD PANEL

### 5.5.1 General

The design capacity and deformation for a plywood panel when subject to design loads that act in the plane of the plywood, as shown in Figure 5.2, shall satisfy the requirements of Clauses 5.5.2 to 5.5.8.

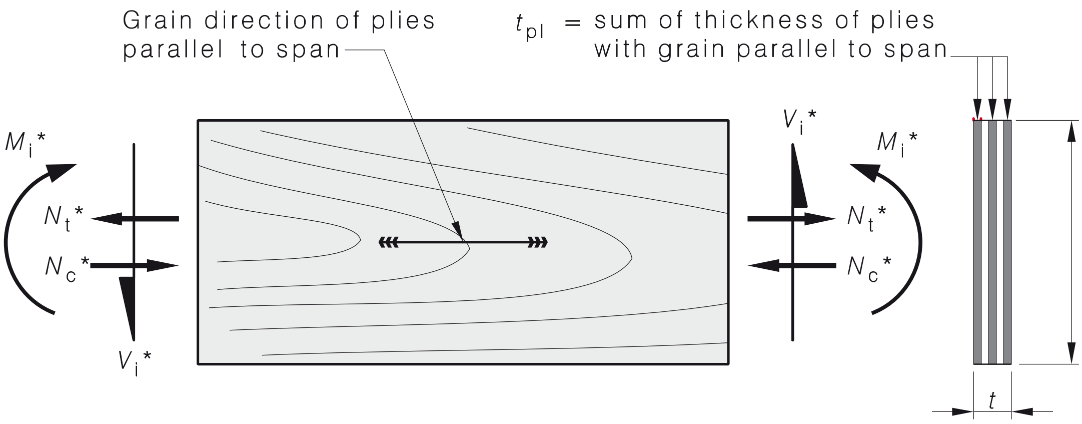



FIGURE 5.2 NOTATION FOR SHEAR, COMPRESSION AND TENSION ACTING IN THE PLANE OF A PLYWOOD PANEL AND FOR EDGEWISE BENDING

### 5.5.2 Bending strength

The design capacity of plywood in bending, for strength limit state, shall satisfy the following:

$$M_{d,i} \geq M_i^* \qquad \ldots 5(7)$$

where

$$M_{d,i} = \phi k_1 k_{12} k_{19} g_{19} f'_b Z_i \qquad \ldots 5(8)$$

and

$M_i^*$ = design action effect for edgewise bending of the plywood panel as shown in Figure 5.2

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{12}$ = modification factor for stability (see Appendix I)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.4)

$f'_b$ = characteristic value in bending

$Z_i$ = section modulus of plywood
= $(t_{pl} d^2)/6$

where

$t_{pl}$ = sum of thicknesses of veneers having their grain parallel to the span (see Figure 5.2)

$d$ = overall depth

NOTE: The values of $f'_b$ for plywood assigned an F-grade classification are given in Table 5.1.

## TABLE 5.4

## ASSEMBLY FACTORS ($g_{19}$) FOR LOADING IN THE PLANE OF THE SHEET OF PLYWOOD AND IN EDGEWISE BENDING

| Property | Direction of grain of plies with respect to stress direction or span | Portion of cross-section considered (effective cross-section area) | Assembly factor ($g_{19}$) |
|---|---|---|---|
| Compression and bending strength | Parallel or perpendicular | Parallel plies only | 1.0 |
| | ±45° | Full cross-section | 0.34 |
| Tension strength | Parallel or perpendicular | Parallel plies only | 1.0 |
| | ±45° | Full cross-section | 0.17 |
| Shear strength | Parallel or perpendicular | Full cross-section | 1.0 |
| | ±45° | Full cross-section | 1.5 |
| Shear deformation | Parallel or perpendicular | Full cross-section | 1.0 |
| Bending deflection | Parallel or perpendicular | Parallel plies only | 1.0 |
| Deformation in compression or tension | Parallel or perpendicular | Parallel plies only | 1.0 |
| | ±45° | Full cross-section | 0.17 |

### 5.5.3 Shear strength

The design capacity of plywood in shear, for strength limit state, shall satisfy the following:

$$V_{d,i} \geq V_i^* \quad \ldots 5(9)$$

where

$$V_{d,i} = \phi\, k_1\, k_{12}\, k_{19}\, g_{19}\, f_s'\, A_s \quad \ldots 5(10)$$

$V_i^*$ = design action effect for shear in the plane of the plywood panel as shown in Figure 5.2

$\phi$ = capacity factor of plywood (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{12}$ = modification factor for stability (see Appendix I)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.4)

$f_s'$ = characteristic value in panel shear

$A_s$ = $\frac{2}{3}(d\,t)$ (see Figure 5.2), for shear in beams; or

= $(d\,t)$ for localized shear

where

$d$ = overall depth of plywood

$t$ = full thickness of plywood

NOTE: The values of $f_s'$ for plywood assigned an F-grade classification are given in Table 5.1.

### 5.5.4 Tension strength

The design capacity of plywood in tension for strength limit state shall satisfy the following:

$$N_{d,t} \geq N_t^* \quad \ldots 5(11)$$

where

$$N_{d,t} = \phi\, k_1\, k_{19}\, g_{19}\, f_t'\, A_t \quad \ldots 5(12)$$

and

$N_t^*$ = design action effect for tension in the plane of the plywood as shown in Figure 5.2

$\phi$ = capacity factor of plywood (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.4)

$f_t'$ = characteristic value in tension

$A_t$ = effective cross-sectional area defined in Table 5.4 which for parallel plies only is $d$ times the sum of thicknesses of veneers having their grain parallel to the direction of load

NOTE: The values of $f_t'$ for plywood assigned an F-grade classification are given in Table 5.1.

### 5.5.5 Compression strength

The design capacity of plywood in compression ($N_{d,c}$) for strength limit state shall satisfy the following:

$$N_{d,c} \geq N_c^* \qquad \text{. . . 5(13)}$$

where

$$N_{d,c} = \phi k_1 k_{12} k_{19} g_{19} f'_c A_c \qquad \text{. . . 5(14)}$$

and

$N_c^*$ = design action effect for compression in the plane of the plywood panel as shown in Figure 5.2

$\phi$ = capacity factor of plywood (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{12}$ = modification factor for stability (see Appendix I)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for plywood assembly (see Table 5.4)

$f'_c$ = characteristic value in compression

$A_c$ = effective cross-sectional area as defined in Table 5.4, which for parallel plies only is $d$ times the sum of thicknesses of veneers having their grain parallel to direction of load

NOTE: The values of $f'_c$ for plywood assigned an F-grade classification are given in Table 5.1.

### 5.5.6 Combined loads

For the case of combined loads, the following interaction criteria shall be satisfied:

(a) For combined compression, bending and shear:

$$\frac{N_c^*}{N_{d,c}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leq 1.0 \qquad \text{. . . 5(15)}$$

(b) For combined tension, bending and shear:

$$\frac{N_t^*}{N_{d,t}} + \left(\frac{M_i^*}{M_{d,i}}\right)^2 + \frac{V_i^*}{V_{d,i}} \leq 1.0 \qquad \text{. . . 5(16)}$$

### 5.5.7 Deflection in bending

In calculating deflection, the cross-section stiffness shall be based on the second moment of area ($I$) of the plies parallel to the direction of span, the characteristic modulus of elasticity values given in Table 5.1, the moisture content stiffness factor $j_6$ given in Table 5.2(B) and the assembly factor $g_{19}$ given in Table 5.4. The duration of load factor $j_2$ given in Table 2.4 shall be used in calculating deflections.

### 5.5.8 Shear deformation

In calculating shear deformations, the cross-section stiffness shall be based on the gross cross-section, using the characteristic moduli values given in Table 5.1, the moisture factor $j_6$ given in Table 5.2(B) and the assembly factor $g_{19}$ given in Table 5.4. The duration of load factor $j_2$ given in Table 2.4 shall be applied.

## 5.6 JOINTS IN COMPOSITE PLYWOOD TO TIMBER CONSTRUCTION

### 5.6.1 Nailed and screwed joints

The design capacity and stiffness of nailed and screwed joints between plywood and solid timber are given in Appendix C.

### 5.6.2 Shear strength at glued interfaces

The design capacity of a glued interface in shear between a plywood section and timber section for strength limit state ($V_{d,sj}$) shall satisfy the following:

$$V_{d,sj} \geq V^*_{sj} \qquad \ldots 5(17)$$

where

$$V_{d,sj} = \phi\, k_1\, k_{19}\, g_{19}\, f'_s\, A_{sj} \qquad \ldots 5(18)$$

$V^*_{sj}$ = design action effect in shear at the glued interface of the plywood and timber sections

$\phi$ = capacity factor of plywood (see Clause 2.3)

$k_1$ = modification factor for duration of load (see Clause 2.4.1.1)

$k_{19}$ = modification factor for moisture condition (see Table 5.2(A))

$g_{19}$ = modification factor for assembly appropriate to the type of construction (see Table 5.5)

$f'_s$ = characteristic value in shear

$A_{sj}$ = area at glued interface as shown in Figures 5.3, 5.4 and 5.5 and defined in Table 5.5

NOTE: The values of $f'_s$ for plywood assigned an F-grade classification are given in Table 5.1.

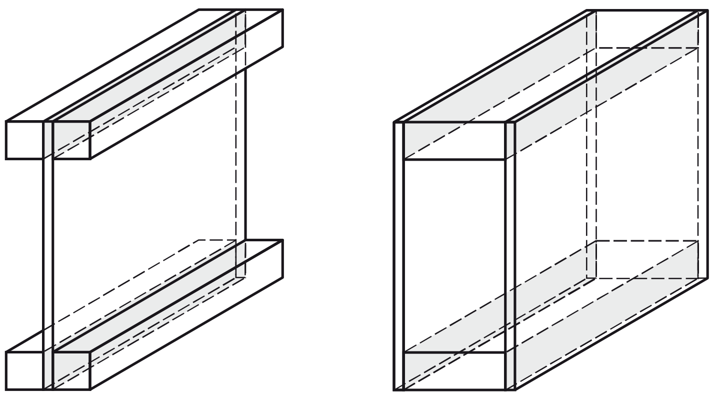


FIGURE 5.3 BEAMS WITH FULL DEPTH WEB

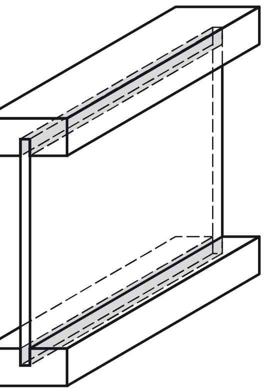


FIGURE 5.4 BEAMS WITH WEBS PARTIALLY EMBEDDED INTO FLANGES

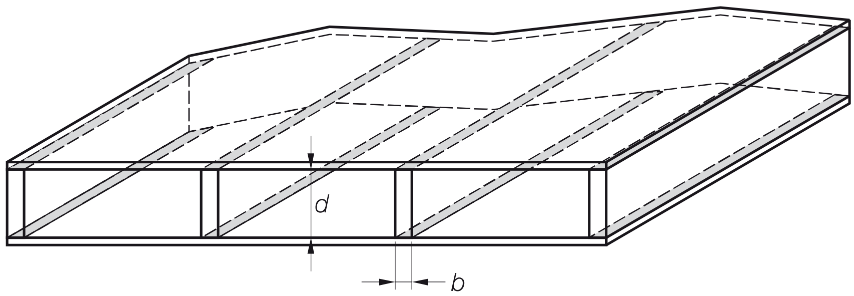



FIGURE 5.5 STRESSED SKIN PANELS

## TABLE 5.5

## ASSEMBLY FACTOR $g_{19}$ FOR GLUED INTERFACES

| Type of construction | Position of shear | Stress direction with respect to grain direction in face plies | Assembly factor | |
|---|---|---|---|---|
| | | | Area to be considered | Modification factor ($g_{19}$) |
| Box beam and I-beams with plywood webs | Full depth web-shear between web and flanges (see Figure 5.3) | Parallel or perpendicular | Full area of contact between plywood and flange | 0.2 × characteristic value |
| | | ±45° (see Table 5.4) | Full area of contact between plywood and flange | 0.2 × characteristic value |
| | Webs partially embedded in flanges—shear between web and flanges (see Figure 5.4) | Parallel or perpendicular | Full area of contact between plywood and flange | 0.4 × characteristic value |
| Stressed skin panels | Shear between plywood and framing members (see Figure 5.5). End nogging shall be used where the depth of framing members exceeds twice their breadth | Parallel or perpendicular | Full area of contact between plywood and members | 0.4 × characteristic value in shear for interior framing members |
| | | | | 0.2 × characteristic value in shear strength for edge framing members |
| | | ±45° (see Table 5.4) | Full area of contact between plywood and framing member | 0.4 × characteristic value in shear for interior framing members |
| | | | | 0.2 × characteristic value in shear for edge framing members |

# SECTION 6 ROUND TIMBERS

## 6.1 GENERAL

Whether naturally round timbers are used as simple structural members (that is, as poles or piles or as elements of a composite structure) the design procedures shall be in accordance with the procedure given in Section 3, subject to the provisions of Clauses 6.2, 6.3, 6.4 and 6.5.

## 6.2 CHARACTERISTIC VALUES FOR STRUCTURAL DESIGN

The characteristic values for structural design for untrimmed logs, poles or piles which conform in quality to the grade requirements specified in AS 3818.3 or AS 3818.11, as appropriate, are determined by the species. Species strength groups are listed for common species in Tables H2.3 and H2.4, Appendix H; these strength groups are used to determine the applicable F-grade in accordance with Table 6.1. For round timbers, the unseasoned condition shall be assumed. Design properties are taken as for F-graded timber in accordance with Appendix H.

Alternatively, characteristic values can be determined by testing and evaluation in accordance with the AS/NZS 4063 series.

**TABLE 6.1**

**ROUND TIMBERS GRADED TO AS 3818.3 or AS 3818.11— RELATIONSHIP BETWEEN STRENGTH GROUPS AND F-GRADES**

| Strength group | Stress grade |
|---|---|
| S1 | F34 |
| S2 | F27 |
| S3 | F22 |
| S4 | F17 |
| S5 | F14 |
| S6 | F11 |
| S7 | F8 |

NOTE: The equivalence expressed is based on the assumption that all poles or logs are cut from mature trees. Factors for immaturity are given in Clause 6.4.1.

## 6.3 DESIGN

### 6.3.1 Bending strength

The design capacity in bending of round timbers ($M_d$) for the strength limit state shall satisfy the following:

$$M_d \geq M^* \qquad \text{. . . 6(1)}$$

where

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_{20}\, k_{21}\, k_{22}\, f'_b Z \qquad \text{. . . 6(2)}$$

and

$M^*$ = design action effect in bending produced by strength limit states design loads

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ to $k_9$ = strength modification factors given in Section 2

| | | |
|---|---|---|
| $k_{12}$ | = | stability factor |
| | = | 1 for round timbers |
| $k_{20}$ | = | immaturity factor as per Clause 6.4.1 |
| $k_{21}$ | = | shaving factor as per Clause 6.4.2 |
| $k_{22}$ | = | processing factor (0.85 if poles are steamed; otherwise 1.0) |
| $f'_b$ | = | characteristic value in bending |
| $Z$ | = | section modulus of a round timber |
| | = | $\frac{\pi d_p^3}{32}$, where $d_p$ = pole diameter at the relevant section |

NOTE: The characteristic value ($f'_b$) may be evaluated by in-grade testing.

### 6.3.2 Shear strength

The design capacity in shear of a round timber ($V_d$) shall satisfy the following:

$$V_d = V^* \quad \text{. . . 6(3)}$$

where

$$V_d = \phi k_1 k_4 k_6 k_{20} f'_s A_s \quad \text{. . . 6(4)}$$

and

| | | |
|---|---|---|
| $V^*$ | = | design action effect in shear |
| $\phi$ | = | capacity factor (see Clause 2.3) |
| $k_1$, $k_4$, $k_6$ | = | modification factors given in Section 2 |
| $k_{20}$ | = | immaturity factor (Table 6.2) |
| $f'_s$ | = | characteristic value in shear |
| $A_s$ | = | shear plane area |
| | = | $\frac{3\pi d_p^2}{16}$, where $d_p$ = smallest end pole diameter |

### 6.3.3 Compressive strength

The design capacity in compression of round timber columns ($N_{d,c}$) for the strength limit state shall satisfy the following:

$$N_{d,c} \geq N^* \quad \text{. . . 6(5)}$$

where

A1 $$N_{d,c} = \phi k_1 k_4 k_6 k_{12} k_{20} k_{21} f'_c A_c \quad \text{. . . 6(6)}$$

and

| | | |
|---|---|---|
| $N^*$ | = | design action effect in compression |
| $\phi$ | = | capacity factor (see Clause 2.3) |
| A1 $k_1$, $k_4$ and $k_6$ | = | modification factors given in Section 2 |
| $k_{12}$ | = | stability factor, determined in accordance with Clause 3.3.3, except that— |

$S = 1.15\ L/d_p$

where

= slenderness coefficient

= distance between effective restraints in any plane and $d_p$ = nominal mid-length diameter between points of lateral restraint

$k_{20}$ = immaturity factor as per Clause 6.4.1

$k_{21}$ = shaving factor as per Clause 6.4.2

$f'_c$ = characteristic value in compression parallel to the grain

A1 $A_c$ = section area of a round timber column

$= \dfrac{\pi\, d_p^2}{4}$, where $d_p$ is the nominal mid-length diameter

NOTE: The nominal mid-length diameter may be calculated from the small end diameter and the taper.

### 6.3.4 Deflections

Deflection calculations shall take into account the modification factors in Clause 2.4.1.2.

## 6.4 ADDITIONAL MODIFICATION FACTORS

### 6.4.1 Factor for immaturity

For poles having mid-length diameters less than 250 mm, due allowance shall be made for the properties of immature timber. For eucalypt and corymbia species and softwoods, the factors $k_{20}$ and $j_9$ given in Tables 6.2(A) and 6.2(B) shall be used to determine capacity and rigidity, respectively.

NOTE: For hardwood species other than eucalypt and corymbia species, conservative assumptions with respect to $k_{20}$, $j_9$, and $k_{21}$ should be used in design unless special investigations have been undertaken to derive accurate values.

**TABLE 6.2(A)**

**IMMATURITY FACTORS—**
**IMMATURITY FACTOR $k_{20}$ FOR DESIGN CAPACITY**

| Species | Factor $k_{20}$ for capacities | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Pole diameter at mid-length ($d_p$), mm | | | | | | | |
| | 75 | 100 | 125 | 150 | 175 | 200 | 225 | 250 |
| Eucalypt and corymbia | 0.80 | 0.90 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Softwoods | 0.70 | 0.75 | 0.80 | 0.85 | 0.90 | 0.95 | 1.00 | 1.00 |

**TABLE 6.2(B)**

**IMMATURITY FACTORS—**
**IMMATURITY FACTOR ($j_9$) FOR STIFFNESS**

| Species | Factor $j_9$ for stiffness | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Pole diameter at mid-length ($d_p$), mm | | | | | | | |
| | 75 | 100 | 125 | 150 | 175 | 200 | 225 | 250 |
| Eucalypt and corymbia | 0.80 | 0.90 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Softwoods | 0.70 | 0.75 | 0.80 | 0.85 | 0.90 | 0.95 | 1.00 | 1.00 |

### 6.4.2 Shaving factor

For timber members in natural pole form, the design capacity shall be reduced if the poles have been shaved. For poles that have been shaved to a smooth cylindrical form, the shaving factor $k_{21}$ shall be taken as specified in Table 6.3. In addition, it shall be assumed that the effect of shaving will be to reduce the modulus of elasticity by 5% (see Note to Clause 6.4.1).

**TABLE 6.3**

**SHAVING FACTOR ($k_{21}$)**

| Stress | Factor $k_{21}$ | |
|---|---|---|
| | Eucalypt and corymbia species | Softwoods |
| Bending | 0.85 | 0.75 |
| Compression parallel to grain | 0.95 | 0.90 |
| Compression perpendicular to grain and shear | 1.00 | 1.00 |
| Tension | 0.85 | 0.75 |

## 6.5 DESIGN DETAILS

### 6.5.1 Effective cross-section of untreated timber

Unless subjected to adequate preservative treatment in accordance with an approved Standard, the sapwood of all timbers shall be disregarded in assessing the effective structural cross-section of poles at or above the ground-line where exposed to the weather or when used as piles above permanent water level.

### 6.5.2 Moisture content of timbers in ground contact

Poles embedded in the ground and up to 1 m above the ground shall be designed for the unseasoned condition.

### 6.5.3 Joints

Design of joints shall be in accordance with Section 4. The joint group shall be determined as for F-graded timber, in accordance with Appendix H.

# SECTION 7 GLUED-LAMINATED TIMBER CONSTRUCTION

## 7.1 GENERAL

This Section shall be applied in conjunction with Sections 2 and 3. The provisions of this Section apply specifically to glued-laminated timber members manufactured in accordance with AS/NZS 1328.1.

Additional design methods, including methods for taper-cut and curved beams, are contained in Appendix E.

Where any glued-laminated timber is likely to be exposed to water or to damp conditions, the glued laminated timber shall be bonded with Type 1 adhesive specified in accordance with AS/NZS 1328.1.

## 7.2 STRUCTURAL DESIGN

The procedures given in Section 3 and Appendix E shall generally apply for design with glued-laminated timber except that values of properties and factors given in this Section shall apply.

Design of joints using glued-laminated timber shall be in accordance with the requirements of Section 4.

## 7.3 DESIGN PROPERTIES

### 7.3.1 Characteristic values for strength and stiffness properties

The characteristic values for strength properties and moduli of elasticity and rigidity for various GL-grade classifications of glued-laminated timber are given in Table 7.1.

Characteristic values related to the strength groups that apply to the computation of timber member design capacities in bearing (both perpendicular and parallel to the grain, in tension parallel to the grain, and in shear at joint details) shall be taken as for F-graded timber in Appendix H.

**TABLE 7.1**

**CHARACTERISTIC VALUES FOR STRUCTURAL DESIGN—GL-GRADES**

| Stress grade | Characteristic values, MPa | | | | | |
|---|---|---|---|---|---|---|
| | Bending ($f'_b$) | Tension parallel to grain ($f'_t$) | Shear in beam ($f'_s$) | Compression parallel to grain ($f'_c$) | Short duration average modulus of elasticity parallel to the grain ($E$) | Short duration average modulus of rigidity for beams ($G$) |
| GL18 | 45 | 25 | 5.0 | 45 | 18500 | 1230 |
| GL17 | 40 | 20 | 4.2 | 33 | 16700 | 1110 |
| GL13 | 33 | 16 | 4.2 | 26 | 13300 | 900 |
| GL12 | 25 | 11 | 4.2 | 22 | 11500 | 770 |
| GL10 | 22 | 8 | 3.7 | 18 | 10000 | 670 |
| GL8 | 19 | 6 | 3.7 | 14 | 8000 | 530 |

NOTE: The characteristic values for tension for GL grades apply for tension members with the larger cross-sectional dimension not greater than 150 mm. For tension members with a cross-sectional dimension greater than 150 mm, the characteristic values are determined by multiplying the value in the table by $(150/d)^{0.167}$, where $d$ is the larger cross-sectional dimension of the section.

### 7.3.2 Capacity factor

The capacity factor ($\phi$) to be used in the computation of the design capacities for glue-laminated timber structural elements shall be as given in Table 2.1.

### 7.3.3 Non-GL-grade properties

The use of other characteristic properties for glued-laminated timber is acceptable provided such characteristic properties are determined by testing and evaluation using procedures consistent with those given in the AS/NZS 4063 series.

## 7.4 MODIFICATION FACTORS

### 7.4.1 General

Modification factors for strength and stiffness, other than those specified in Clause 7.4, shall comply with Section 2.

### 7.4.2 Duration of load

The modification factors for duration of load ($k_1$) and for stiffness ($j_2$ and $j_3$) given in Clause 2.4.1 shall apply where appropriate.

NOTE: Long-term creep is dependent upon size, grade, environmental conditions and surface coatings.

### 7.4.3 Strength sharing between parallel members

The strength-sharing factor ($k_9$) for glued-laminated timber used in parallel systems shall be taken as unity (i.e., $k_9$ = 1.0).

### 7.4.4 Stability factor

The procedures given in Section 3 for calculation of the stability factor ($k_{12}$) shall generally apply for design with glued-laminated timber except that the material constant ($\rho_b$ or $\rho_c$), for beams and columns shall be as given in Tables 7.2(A) and 7.2(B).

**TABLE 7.2(A)**

**MATERIAL CONSTANT, $\rho_b$, FOR BEAMS**

| Stress grade | Ratio temporary design action effect/total design action effect ($r$)* | | | | |
|---|---|---|---|---|---|
| | 0 | 0.25 | 0.5 | 0.75 | 1.0 |
| | Material constant ($\rho$) for beams† | | | | |
| GL18 | 0.89 | 0.89 | 0.85 | 0.83 | 0.82 |
| GL17 | 0.88 | 0.88 | 0.85 | 0.83 | 0.81 |
| GL13 | 0.90 | 0.90 | 0.86 | 0.84 | 0.83 |
| GL12 | 0.84 | 0.84 | 0.81 | 0.79 | 0.78 |
| GL10 | 0.85 | 0.85 | 0.81 | 0.79 | 0.78 |
| GL8 | 0.88 | 0.88 | 0.84 | 0.82 | 0.81 |

* See Paragraph E2.

† These values are derived from Equation E2(1), Appendix E using values of $E$ and $f'_b$ given in Table 7.1.

A1
A2

## TABLE 7.2(B)

## MATERIAL CONSTANT, $\rho_c$, FOR COLUMNS

| Stress grade | Ratio temporary design action effect/total design action effect ($r$)* | | | | |
|---|---|---|---|---|---|
| | 0 | 0.25 | 0.5 | 0.75 | 1.0 |
| | Material constant ($\rho$) for columns† | | | | |
| GL18 | 1.08 | 1.08 | 1.03 | 1.00 | 0.98 |
| GL17 | 0.99 | 0.99 | 0.95 | 0.92 | 0.90 |
| GL13 | 0.99 | 0.99 | 0.94 | 0.91 | 0.89 |
| GL12 | 0.98 | 0.98 | 0.93 | 0.91 | 0.89 |
| GL10 | 0.96 | 0.96 | 0.91 | 0.88 | 0.86 |
| GL8 | 0.95 | 0.95 | 0.90 | 0.87 | 0.85 |

* See Paragraph E2, Appendix E.

† These values are derived from Equation E2(3), Appendix E, using values of $E$ and $f'_c$ given in Table 7.1.

# SECTION 8 STRUCTURAL LAMINATED VENEER LUMBER

## 8.1 GENERAL

The provisions of this Section are appropriate for use with structural laminated veneer lumber (LVL) manufactured and having structural properties determined in accordance with AS/NZS 4357.0.

Generally, structural design with LVL is similar to that given for sawn timber and hence this Section includes only those aspects of design that differ from those given for sawn timber elsewhere in this Standard.

Structural design of taper cut LVL shall be performed in accordance with Appendix E.

All structural LVL manufactured in accordance with AS/NZS 4357.0 is Type A bonded.

## 8.2 STRUCTURAL DESIGN

The procedures given in Section 3 and Appendix E shall generally apply for design with structural LVL, except that values of properties and factors given in this Section shall apply.

For on-flat bending and shear applications involving LVL containing cross bands, design shall be in accordance with the requirements for plywood in Section 5.

## 8.3 DESIGN PROPERTIES

### 8.3.1 Characteristic values for strength properties and elastic moduli

Characteristic values for structural LVL shall be obtained from the manufacturer. Characteristic values for LVL shall include consideration of the section sizes to which they are intended to apply.

Unless otherwise specified by the manufacturer, the characteristic values for LVL for bending and tension shall be modified as follows:

(a) For beam sections of depth 300 mm or less—no adjustment.

(b) For beams with depth exceeding 300 mm—multiply the published characteristic value for bending by $(300/d)^{0.167}$, where $d$ is the depth of the beam.

(c) For tension members with width 150 mm or less—no adjustment.

(d) For tension members with the larger cross-sectional dimension exceeding 150 mm—multiply the published characteristic value for tension by $(150/d)^{0.167}$, where $d$ is the larger cross-sectional dimension of the tension member.

Characteristic values for LVL shall not be assumed on the basis of the strength group of the species of manufacture.

### 8.3.2 Modulus of rigidity

The modulus of rigidity for LVL shall be taken as one-twentieth of the modulus of elasticity, except where otherwise determined by testing.

### 8.3.3 Section properties

Structural LVL is usually manufactured with the grain of all veneers orientated in the longitudinal direction; however, in some instances special constructions may incorporate cross-band veneers. For LVL not containing cross-band veneers, section properties shall be calculated using the actual cross-section dimensions.

For LVL containing cross-band veneers, the thickness of an individual ply shall be assumed to be in proportion to its nominal thickness, as the finished minimum LVL thickness is to the total of the nominal veneer thicknesses. Section properties for cross-banded LVL shall be determined as follows:

(a) Any veneers with nominal grain direction at right angles to the direction of stress shall be ignored for the calculation of area, first moment of area and second moment of area when assessing the edgewise bending, tension and compressive capacity and edgewise flexural rigidity.

(b) For on-flat bending and shear applications, section properties shall be determined in accordance with Paragraph I3 of Appendix I.

It is appropriate to assume the full sectional area is effective in resisting in-plane shear.

### 8.3.4 Capacity factors

The capacity factors to be used for the computation of design capacities for structural LVL elements shall be as given in Table 2.1.

## 8.4 MODIFICATION FACTORS

### 8.4.1 General

The modification factors for strength and stiffness given in Section 2 shall generally apply for design with LVL except for those factors specified in Clause 8.4.

### 8.4.2 Duration of load

The modification factors for duration of load for strength ($k_1$) and stiffness ($j_2$ and $j_3$) given in Clause 2.4.1 shall be used as appropriate.

### 8.4.3 Moisture condition

Where LVL is subjected to conditions, such that the average moisture content for a 12 month period will exceed 15%, the modification factors for strength ($k_4$) and for stiffness ($j_6$) given in Table 8.1 shall be used, except where different values have been determined by testing.

**TABLE 8.1**

**MOISTURE CONTENT FACTORS FOR LVL**

| Property | Equilibrium moisture content (*EMC*) | | |
|---|---|---|---|
| | ≤15% | 15% to 25% | ≥25% |
| Bending and compression | $k_4 = 1.0$ | $k_4 = 1.45 - 0.03\,EMC$ | $k_4 = 0.7$ |
| Tension and shear | $k_4 = 1.0$ | $k_4 = 1.30 - 0.02\,EMC$ | $k_4 = 0.8$ |
| Modulus of elasticity | $j_6 = 1.0$ | $j_6 = 1.30 - 0.02\,EMC$ | $j_6 = 0.8$ |

### 8.4.4 Temperature

The provisions of Clause 2.4.3 for seasoned timber shall apply to structural LVL in a similar manner.

### 8.4.5 Length and position of bearing

Where rectangular bearing areas are located 75 mm or more from the end of a piece of LVL, it is permissible to increase the characteristic capacity in bearing perpendicular to the grain (refer to Clause 3.2.6) by the appropriate value of factor $k_7$ in Table 2.6. The length of bearing shall be measured parallel to the grain of the loaded member. For all other conditions, $k_7 = 1.0$.

### 8.4.6 Strength sharing between parallel members

The strength-sharing factor ($k_9$) for LVL used in parallel systems shall be taken as unity (i.e., $k_9 = 1$).

### 8.4.7 Stability factor

The procedures given in Section 3 for calculation of the stability factor ($k_{12}$) shall generally apply for design with LVL except that the material constant ($\rho_b$ or $\rho_c$) for beams and columns shall be determined as follows:

(a) *Beams*:

$$\rho_b = 14.71 \left(\frac{E}{f'_b}\right)^{-0.480} r^{-0.061} \quad \ldots 8(1)$$

(b) *Columns*:

A2
A3

$$\rho_c = 11.39 \left(\frac{E}{f'_c}\right)^{-0.408} r^{-0.074} \quad \ldots 8(2)$$

The maximum value of $\rho_b$ or $\rho_c$ used need not exceed the value computed for the case $r = 0.25$. In the case of beams where a temporary load causes a stress reversal, the value of $\rho_b$ or $\rho_c$ to be used is that corresponding to $r = 1.0$.

NOTES:

1 These equations are the same as those given for seasoned timber in Appendix E.

2 When a member is normally subjected to axial tension effect, but may act in compression due to temporary design loads such as wind loads, the material constant ($\rho$) may be calculated as for beams (see Equation 8(1)) for the case of $r = 1.0$.

## 8.5 JOINT DESIGN

### 8.5.1 General

Design of joints using LVL shall be performed in accordance with the requirements of Section 4, except where otherwise specified in this Section.

### 8.5.2 Joint group

The joint group as provided by the manufacturer for a particular LVL product and determined in accordance with AS/NZS 4357 shall be taken to apply for nails, screws, bolts and coach screws only. For other fastener types, the use of the joint group may not be appropriate.

### 8.5.3 Characteristic fastener capacities

Characteristic capacities for nails, screws, bolts and coach screws shall be determined on the basis of joint group as tabulated in Section 4 or obtained from the manufacturer appropriate to the particular LVL product, fastener type and size, fastener location (face or edge of LVL) and orientation, and load orientation.

For other fastener types, characteristic capacities determined in accordance with AS/NZS 4357 for the particular LVL product, fastener type and dimensions, fastener location and orientation, and load orientation shall be obtained either from the manufacturer of the LVL or of the fastener.

# APPENDIX A

## NORMATIVE REFERENCES

(Normative)

The following are the normative documents referenced in this Standard.

NOTE: Documents referenced for informative purposes are listed in the Bibliography.

AS

1111 ISO metric hexagon commercial bolts and screws
1111.1 Part 1: Product grade C—Bolts

1442 Carbon steels and carbon-manganese steels—Hot rolled bars and semi-finished products

2082 Timber—Hardwood—Visually stress-graded for structural purposes

2334 Steel nails—Metric series

2858 Timber—Softwood—Visually stress-graded for structural purposes

3519 Timber—Machine proof-grading

3566 Self-drilling screws for the building and construction industries (all Parts)

3818 Timber—Heavy Structural Products—Visually graded
3818.3 Part 3: Piles
3818.11 Part 11: Utility poles

4100 Steel structures

AS/NZS

1170 Structural design actions
1170.0 Part 0: General principles
1170.2 Part 2: Wind actions

1328 Glued-laminated structural timber
1328.1 Part 1: Performance requirements and minimum production requirements

1393 Coach screws—Metric series with ISO hexagon heads

1748 Timber—Mechanically stress-graded for structural purposes

2269 Plywood—Structural
2269.0 Part 0: Specifications

4063 Characterization of structural timber
4063.1 Part 1: Test methods
4063.2 Part 2: Determination of design characteristic values

4357 Structural laminated veneer lumber
4357.0 Part 0: Specifications

4491 Timber—Glossary of terms in timber related Standards

ASTM

D143 Standard Test Methods for Small Clear Specimens of Timber

BS

373 Methods of testing small clear specimens of timber

# APPENDIX B

# GUIDELINES FOR SERVICEABILITY

(Informative)

## B1 GENERAL

Except where absolute limits are required, it is generally best to deal with deflection design in terms of the individual actions being considered; for example, it is usually preferable to deal with the effects of permanent actions separately from the deflection effects of transient or short-term action. Unlikely combinations of actions need not be considered and total deflection usually needs to be considered only where absolute clearance limits must be maintained.

## B2 SERVICEABILITY CONSIDERATIONS

Guidance for the design of members for serviceability is given in Table B1 for roof members, wall members and floor members or systems.

### TABLE B1

### GUIDELINES FOR SERVICEABILITY DESIGN

| Criteria | Long-term effects | Short-term effects |
|---|---|---|
| TIMBER ROOF SYSTEMS AND MEMBERS | | |
| Loads/ Actions | (a) Loads and load combinations<br>$G + \psi_l Q$<br>where values of $\psi_l$ and $Q$ are as per AS/NZS 1170.0 | (a) Loads and load combinations<br>(i) $W_s$<br>(ii) $\psi_s Q$<br>Where $W_s$ is as given in AS/NZS 1170.0 and $Q$ and $\psi_s$ are as given in AS/NZS 1170.0<br>(b) For non-trafficable roofs, live load allowance for 'stacked materials used in repair and maintenance' should not be considered to act in combination with live loads for 'occasional loading on roof trusses or structure'<br>(c) Other load combinations given in AS/NZS 1170.0 need be considered only where there are specific requirements (e.g., a need to maintain clearances)<br>(d) Where there is a possibility of ponding, loading should include an allowance for this |
| Appearance | Deformation limited according to occupancy<br><br>Deflection limited relative to span and relative to proximity of reference lines (e.g., face brickwork) and available lines of sight<br><br>End rotations need to be considered (see Note 1)<br>Camber may be used to improve appearance (see Note 2) | Generally limits need only apply where deformation is visible and may cause alarm to occupants<br>*Both wind and roof live loads are infrequent in occurrence and of short-term duration and this, together with the type of occupancy, should be taken into consideration in assessing acceptable deformation* |
| Functionality | Minimum slope for drainage should consider the effects of long-term deflection. Camber may also be used to offset the effect of long-term deflection (see Note 2) | Where ponding is possible, the additional short-term deflection due to this load should be considered and slope and/or camber increased to ameliorate this (see Notes 2 and 3) |

*(continued)*

**TABLE B1** (*continued*)

| Criteria | Long-term effects | Short-term effects |
|---|---|---|
| Damage | Cracking/damage to roof cladding | Cracking/damage to roof cladding |
| | Cracking/damage to ceilings | Cracking/damage to ceilings |
| | Cracking of glazing (e.g., clerestory windows) due to 'in-plane' movements | Cracking of glazing (e.g., clerestory windows) due to 'in-plane' movements |
| | Excessive end rotations may result in unintended load transfer and damage to other building elements including damage to supporting connections (see Note 1) | Damage to partitions —unintended load transfer to partitions caused by excessive deformations, may cause damage. To avoid this, an upper bound estimate of deflection may need to be calculated and appropriate clearances specified for construction (see Note 3) |
| | Damage to partitions—unintended load transfer to partitions caused by excessive deformations, may result in damage. To avoid this, an upper bound estimate of deflection may need to be calculated and appropriate clearances specified for construction (see Note 3) | |
| | 'In-plane' racking of roofs may distort or damage roof claddings and/or ceiling linings | 'In-plane' racking of roofs may distort or damage roof claddings and/or ceiling linings |
| | *In evaluating the tendency for cracking or damage under long-term loading, considerations should be given to the sequence of construction and the creep deformation characteristics of the materials involved* | *Some damage or distortion of materials may be acceptable under extreme event loads* |
| WALL SYSTEMS AND MEMBERS | | |
| Loads/ Actions | (a) Loads and load combinations<br>$G + \psi_l Q$<br>where values of $G$, $\psi_l$ and $Q$ are as given in AS/NZS 1170.0 except, for floor live loads for houses, take $\psi_l = 0.33$ | (a) Loads and combinations<br>(i) $W_s$<br>(ii) $\psi_s Q$<br>(iii) $G + \psi_s Q$<br>where $W_s$ is as given in AS/NZS 1170.0, and $Q$ and $\psi_s$ are as given in AS/NZS 1170.0.<br>$Q$ may include combinations of floor and roof loads. For houses where load is from one floor only, take $\psi_s = 1.0$<br>Load combination (iii) usually needs to be considered only where clearance must be maintained |
| | (b) Other considerations<br>Effects of differential settlement. Effects of loads applied eccentric to the axes of columns or studs. Loads arising from deflections of other elements (e.g., deflection of a ridge beam causing lateral thrust on walls) | (b) Other considerations<br>Effects of loads applied eccentric to the axes of columns or studs |
| Appearance | Sway or lean of walls or columns and deflection relative to supports or relative to rigid cross walls limited according to occupancy | Sway or lean of walls or columns and deflection relative to supports or relative to rigid cross walls limited according to occupancy |
| | Unresolved thrusts from the roof structure and/or moment transfer from roof or floor elements to columns, may cause sway, bending deformation or rotation of wall elements (e.g., excessive deflection of ridge beams) (see Note 2) | Unresolved thrusts from the roof structure and/or moment transfer from roof or floor elements to columns, may cause sway or lean or deformation (e.g., rotation) of wall elements |
| | Limits on vertical deflections of lintels should include consideration of their floor and/or roof supporting functions | Limits on vertical deflections of lintels should include consideration of their floor and/or roof supporting function |
| | | Excessive lateral movement of wall members, including lintels, may cause alarm to occupants |

(*continued*)

**TABLE B1** (*continued*)

| Criteria | Long-term effects | Short-term effects |
|---|---|---|
| Functionality | Differential deflections resulting in gaps at wall junctions can lead to leakage of air, light, sound, water, etc.<br>Excessive lateral deformations may result in unintended load transfer to adjacent buildings and/or encroachment on title boundaries (see Note 4) | Differential deflections resulting in gaps at wall junctions can lead to leakage of air, light, sound, water, etc.<br>Sway or horizontal movement under short- term loads may shake, rattle or cause dislodgment of fittings or furnishings attached to, or adjacent to walls<br>Excessive sway or horizontal movement may result in closely spaced but otherwise structurally independent buildings transferring load to, or impacting upon, adjacent buildings and/or encroachment on title boundaries (see Note 4) |
| Damage | Cracking of linings, claddings or glazing, due to excessive deformation both 'in-plane' and 'out of plane'. (With many materials, e.g., unreinforced masonry or glass, tolerance to 'in-plane' deformation is significantly less than tolerance to 'out of plane' deformations)<br>Excessive vertical deflection of lintels may result in load transfer to joinery items and consequent damage. An upper bound estimate of deflection should be calculated and appropriate clearance specified for construction (see Note 3)<br>Excessive sway or horizontal deflection may cause damage to adjacent buildings | Cracking of linings, claddings or glazing, due to excessive deformation both 'in plane' and 'out of plane'. (With many materials, e.g., unreinforced masonry or glass, tolerance to 'in plane' deformation is significantly less than tolerance to 'out of plane' deformations)<br>Excessive vertical deflection of lintels may result in load transfer to joinery items and consequent damage. An upper bound estimate of deflection should be calculated and appropriate clearance specified for construction (see Note 3)<br>Excessive sway or horizontal deflection may cause damage to adjacent buildings |
| TIMBER FLOOR SYSTEMS AND MEMBERS | | |
| Loads/ Actions | Loads and load combinations<br>$G + \psi_l Q$<br>where values of $\psi_l$ and $Q$ are as specified in AS/NZS 1170.0, except for housing take $\psi_l = 0.33$ | Loads and load combinations<br>(i) $\psi_s Q$<br>Values of $\psi_s$ and $Q$ as given in AS/NZS 1170.0 except for housing take $\psi_s = 1.0$<br>(ii) $G + \psi_s Q$<br>Values of $Q$ and $\psi_s$ are as given in AS/NZS 1170.0. This combination is normally considered only where clearance is a consideration |
| Appearance | Deflection limited according to occupancy and expectation<br>Deflection limited relative to span and relative to proximity to reference lines (e.g., face brickwork) and dependent on available lines of sight<br>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 high/narrow furniture to lean noticeably (see Note 5)<br>Excessive end rotations can cause furniture to lean noticeably<br>*Camber may be used to improve the appearance (see Note 2)* | Function or amenity may be adversely affected by unsatisfactory dynamic behaviour, and excessive deflections, or slope resulting in shaking, rattling, horizontal movement of furniture, etc.<br>*The dynamic response of floor systems, including frequency of vibration, should be considered. For lightly loaded floors in particular, the application of static live load deflection limits does not necessarily ensure satisfactory dynamic performance*<br>Differential deflection of longer span joists in close proximity to relatively rigid parallel walls or excessive rotations can result in excessive visible vibration or horizontal movements of furniture |

(*continued*)

**TABLE B1** (*continued*)

| Criteria | Long-term effects | Short-term effects |
|---|---|---|
| Functionality | Excessive floor slope may adversely affect occupancy requirements | Function or amenity may be adversely affected by unsatisfactory dynamic behaviour, and excessive deflections, or slope resulting in shaking, rattling, horizontal movement of furniture, etc. |
| | Differential deformations resulting from concentrated loads may cause localized depressions in the floor | *The dynamic response of floor systems, including frequency of vibration, should be considered. For lightly loaded floors in particular, the application of static live load deflection limits does not necessarily ensure satisfactory dynamic performance* |
| Damage | Cracking/damage to ceilings and linings | Cracking/damage to ceilings and linings. In particular the differential deformation of joists running parallel and in close proximity to supports or relatively rigid walls may result in localized damage |
| | Differential deformation of longer span floor joists in close proximity to relatively rigid walls either above or below the floor | |
| | Differential deformations resulting from concentrated loads may cause localized depressions in the floor, resulting in damage to roofs, partitions and floors above and partitions and floors below | Differential deformations resulting from concentrated loads may cause localized depressions in the floor, resulting in damage to roofs, partitions and floors above and partitions and floors below |
| | | The ability of the ceiling materials to resist cracking as a consequence of deformations needs to be considered in respect of the duration of load and appropriate limits applied |
| | Damage to partitions—unintended load transfer to partitions caused by excessive deformations, may cause damage. To avoid this, an upper bound estimate of deflection may need to be calculated and appropriate clearances specified for construction (see Note 3) | Damage to partitions—unintended load transfer to partitions caused by excessive deformations, may cause damage. To avoid this, an upper bound estimate of deflection may need to be calculated and appropriate clearances specified for construction (see Note 3) |
| | *In evaluating the tendency for cracking or damage under long-term loading, considerations should be given to the sequence of construction and the creep deformation characteristics of the materials involved* | |

NOTES TO TABLE B1:

1 Deformation includes both deflection and rotation. For a simply supported uniformly loaded beam, a deflection of span/300 corresponds to an end rotation of 0.01 radians. For a 500 mm deep beam, this corresponds to 5 mm relative horizontal displacement at the ends, which could be significant in respect of appearance, function or damage.

2 Camber may be used in certain circumstances, to reduce the visual effects of deflection or to prevent ponding; however, camber does not reduce the actual amount of deflection due to load effects and its use in design to allow for larger deflections (or smaller sizes) may lead to other undesirable effects, such as larger than acceptable end rotations or significant misalignment of other building elements. The inappropriate use of camber to facilitate larger than normal deflections may also result in construction complexities.

3 Where clearances between building elements must be maintained (for example, where undesirable load paths could lead to damage or loss of function, such as doors or windows jamming or being damaged) then it is recommended that an upper-bound estimate of deflection be calculated and an appropriate clearance specified. An upper-bound estimate of deflection may be obtained using a lower fifth-percentIle estimate of modulus of elasticity rather than the average values given elsewhere in this Standard. In the absence of more precise information, a rough estimate of the lower fifth-percentile values of modulus of elasticity may be calculated as a proportion of the average modulus of elasticity, as follows:

(a) For F-grades of timber graded in accordance with AS 2082, AS 2858 ................................ $E_{0.05} \approx 0.5\ E_{average}$

(b) For MGP grades and F-grades of timber graded in accordance with AS/NZS 1748 .............. $E_{0.05} \approx 0.7\ E_{average}$

(c) For F-grades of plywood to AS/NZS 2269, GL grades for glulam to AS/NZS 1328.1 ......... $E_{0.05} \approx 0.75\ E_{average}$

(d) For LVL to AS/NZS 4357.0.............................................................................................. $E_{0.05} \approx 0.85\ E_{average}$

(e) For A17 grade of timber graded in accordance with AS 2082 ............................................ $E_{0.05} \approx 0.7\ E_{average}$

4 Interaction with adjacent buildings may be a legal issue.

5 Relative deflection is deflection relative to span and relative to reference lines.

6 Further information may be found in the following documents:

(a) KING, A.B., Serviceability Limit State Criteria for New Zealand Buildings. *Building Technology Ltd.*

(b) ISO 4356 Bases for the design of structures—Deformations of buildings at the serviceability limit states.

# APPENDIX C

# JOINTS IN TIMBER STRUCTURES

(Normative)

## C1 GENERAL

This Appendix sets out design methods for the use of nail and screw fasteners in plywood, a method for calculating deformation of joints under load and equations and tables for characteristic capacity for bolts.

## C2 FASTENERS FOR PLYWOOD

### C2.1 General

Requirements for the use of nail and screw fasteners to join plywood to solid timber are set out in Paragraph C2.2.

### C2.2 Strength of joints with plywood

**C2.2.1** *Joint strength grouping*

The grouping of common timber species for joint design is given in Tables H2.3 and H2.4, Appendix H.

**C2.2.2** *Design capacity for laterally loaded nails and screws in plywood*

The design capacity load for a laterally loaded plywood-to-timber joint fastened with nails or screws shall be taken as up to 10% greater than the values given for timber-to-timber joints in Section 4, except fastener diameter and length and plywood thickness (see Figure C1) shall be such that—

$$t_o / D > 1.5$$

$$t_p / D > 10$$

$$t_w / D > 10$$

where

$D$ = nail diameter, in millimetres

$t_o$ = thickness of plywood as indicated in Figure C1, in millimetres

$t_p$ = penetration of nail as indicated in Figure C1, in millimetres

$t_w$ = thickness of solid timber as indicated in Figure C1, in millimetres

For values of $(t_o/D) < 1.5$, the design capacity of the joint shall be reduced linearly with respect to $(t_o/D)$ so as to reach zero when $(t_o/D) = 0$. For values of $t_p/D$ and $t_w/D$ less than 10, the design capacity of the joint shall also be reduced linearly with respect to $t_p/D$ and $t_w/D$ but in addition the fastener shall be considered as non-loadbearing if either $t_p/D$ or $t_w/D$ is less than 5. These requirements shall apply whether the fastener is in single or double shear.

In the case of shear joints, such as occurs in the nailing of plywood webs to the solid timber flanges of box beams, the multiple nail factor $k_{17} = 1.0$ shall be used in Equations 4.2(2) and 4.2(4).

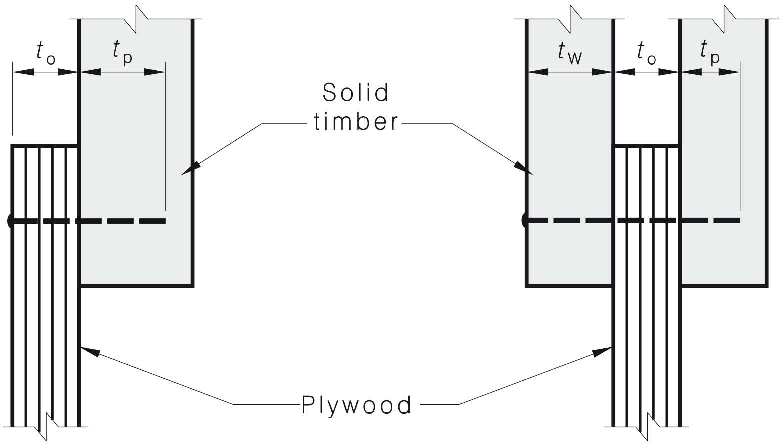



FIGURE C1 PLYWOOD THICKNESS AND NAIL LENGTH

## C3 DEFORMATION OF JOINTS

### C3.1 General

The load-displacement characteristics of a joint are highly nonlinear; however, where a linear joint stiffness is required for design purposes, assumption of a secant stiffness relationship is appropriate. A suitable definition of the secant stiffness ($K_{sec}$) is given by the following equation:

$$K_{sec} = Q^*/\Delta_o \qquad \text{. . . C1}$$

where $\Delta_o$ is the deformation of the joint (including duration effects) when the design action effect ($Q^*$) is applied.

In Paragraphs C3.2 and C3.3, the equations of deformation are good estimates for the deformation due to the first application of a load; for repeated loads, due allowance should be made for incremental slip and changes in joint stiffness.

Where both a long duration design action effect ($Q_L^*$) and a short duration design action effect ($Q_S^*$) act on a fastener, the non-linear load-deformation characteristics shall be considered in evaluating the maximum deformation. In the absence of further information, it is appropriate to take the maximum deformation equal to the long duration deformation due to $Q_L^*$ minus the short duration deformation due to $Q_S^*$ plus the short duration deformation due to design action effect $Q_L^* + Q_S^*$.

### C3.2 Displacement of nailed and screwed joints in solid timber

The displacement ($\Delta$), in millimetres, of nailed or screwed joints in single shear for solid-wood to solid-wood joints is estimated as follows:

(a) For a fastener load ($Q^*$) where $Q^* \leq Q_a$:

$$\Delta = 0.5\left(\frac{Q^*}{Q_a}\right)^{2.17} \qquad \text{. . . C2}$$

(b) For a fastener load ($Q$*) where $Q_a \leq Q^* \leq Q_b$:

$$\Delta = 2\left(\frac{Q^* - Q_a}{Q_b - Q_a}\right) + 0.5 \qquad \text{. . . C3}$$

where

$Q^*$ = lateral shear force, for which displacement is being calculated, in newtons

$Q_a$ = $0.165\, D^{1.75}\, j_{12}\, h_{32}$

$Q_b$ = $0.165\, D^{1.75}\, j_{13}\, h_{32}$

$D$ = diameter of nail or screw, in millimetres

$j_{12}, j_{13}$ = duration factors given in Table C2

$h_{32}$ = stiffness factor given in Table C1

NOTE: For the case of metal and plywood side plates, Equations C2 and C3 lead to conservative overestimates of joint slip.

## C3.3 Displacement of solid timber joints fabricated with bolts, split-rings and shear-plates

### C3.3.1 *General*

Where relevant specific test information is not available, it is appropriate to estimate the displacement of joints fabricated with bolts, split-ring fasteners and shear-plate fasteners, as given in Paragraphs C3.3.2 and C3.3.3.

### C3.3.2 *For loads acting parallel to the grain*

For loads acting parallel to the grain, the displacement ($\Delta$) is given by the following equation:

$$\Delta = \Delta_i + \left(\frac{j_{14}}{h_{33}}\right)\left(\frac{Q^*}{Q_k}\right) \qquad \text{. . . C4}$$

where

$\Delta$ = total displacement, in millimetres

$\Delta_i$ = initial displacement due to oversize holes, in millimetres

= 0, for a load superimposed on an existing load, otherwise

= $\dfrac{1}{\sqrt{n_{con}}}$ for bolted joints

= $\dfrac{1}{2\sqrt{n_{con}}}$ for split-ring fasteners or shear-plate fasteners

$n_{con}$ = number of bolts or split-ring or shear-plate fastener sets in the joint

$j_{14}$ = duration factor given in Table C3

$h_{33}$ = stiffness factor given in Table C4

$Q^*$ = force for which displacement is being calculated, per fastener, in newtons

$Q_k$ = characteristic capacity of a fastener as defined in Section 4, in newtons

Equation C4 is a good approximation for serviceability load effects.

NOTE: For the case of metal and plywood side plates, Equations C2 and C3 lead to conservative overestimates of joint slip.

### C3.3.3 *For loadings acting perpendicular to the grain*

For this case, the displacement ($\Delta$) is given by the following equation:

$$\Delta = \Delta_i + \left(\frac{j_{14}}{h_{33}\, h_{35}}\right)\left(\frac{Q^*}{Q_k}\right) \qquad \text{. . . C5}$$

where

$\Delta_i$ = initial displacement due to oversize holes, in millimetres

= 0, for a load superimposed on an existing load, otherwise

= $\dfrac{1}{\sqrt{n_{con}}}$ for bolted joints

= $\dfrac{1}{2\sqrt{n_{con}}}$ for split-ring fasteners or shear-plate fasteners

$j_{14}$ = duration factor given in Table C3

$h_{33}$ = stiffness factor given in Table C4

$h_{35}$ = 1.5 for two or three member-bolted joints as shown in Table 4.10(A)

= 2.5 for multiple-member bolted joints as shown in Table 4.10(A)

= 1.0 for split-ring and shear-plate fasteners

$Q^*$ = action effect for which displacement is being calculated, per fastener, in newtons

$Q_k$ = characteristic capacity per fastener as defined in Section 4, in newtons

Equation C5 is a good approximation for serviceability load effects.

## TABLE C1

## STIFFNESS FACTOR ($h_{32}$) FOR NAILED AND SCREWED JOINTS IN SOLID TIMBER

| Initial moisture condition | Species joint group | Factor $h_{32}$ |
|---|---|---|
| Unseasoned | J1 | 1450 |
| | J2 | 1050 |
| | J3 | 750 |
| | J4 | 550 |
| | J5 | 410 |
| | J6 | 300 |
| Seasoned | JD1 | 1600 |
| | JD2 | 1250 |
| | JD3 | 990 |
| | JD4 | 750 |
| | JD5 | 590 |
| | JD6 | 470 |

## TABLE C2

## DURATION FACTORS $j_{12}$ AND $j_{13}$

| Initial moisture condition | Duration of load | Factor $j_{12}$ | Factor $j_{13}$ |
|---|---|---|---|
| Unseasoned | More than 3 years | 0.24 | 0.5 |
| | 5 months | 0.34 | 0.7 |
| | Less than 2 weeks | 0.65 | 1.0 |
| Seasoned | More than 3 years | 0.24 | 0.5 |
| | Less than 2 years | 0.65 | 1.0 |

NOTE: If required, intermediate values of $j_{12}$ and $j_{13}$ may be obtained by linear interpolation with log-time.

**TABLE C3**

**DURATION FACTOR $j_{14}$**

| Initial moisture condition | Duration of load | Factor $j_{14}$ |
|---|---|---|
| Unseasoned | More than 3 years<br>5 months<br>2 weeks<br>Less than 5 minutes | 4<br>2<br>1.5<br>1 |
| Seasoned | More than 3 years<br>5 months<br>2 weeks<br>Less than 5 minutes | 3<br>2<br>1.5<br>1 |

NOTE: Intermediate values of $j_{14}$ may be obtained by linear interpolation with log-time.

**TABLE C4**

**STIFFNESS FACTOR $h_{33}$**

| Initial moisture condition | Factor | | |
|---|---|---|---|
| | Bolted joints | | Split-rings and shear-plates |
| | Without metal side plates | With metal side plates | |
| Unseasoned | 0.60 | 0.90 | 0.35 |
| Seasoned | 0.75 | 1.15 | 0.45 |

## C4 EQUATIONS AND TABLES FOR CHARACTERISTIC CAPACITIES FOR BOLTS

### C4.1 Load $Q_{kl}$

The characteristic capacity ($Q_{kl}$) for a single bolt bearing parallel to the grain and acting in single shear shall be the lesser of—

$$b_{eff}\, f'_{cj}\, D/2$$

$1.65\, f'_{cj}\, D^2$ in timber of groups J1 and JD1

$1.75\, f'_{cj}\, D^2$ in timber of groups J2 and JD2

$2.0\, f'_{cj}\, D^2$ in timber of groups J3, J4, JD3 and JD4

$2.2\, f'_{cj}\, D^2$ in timber in groups J5 and JD5

$2.4\, f'_{cj}\, D^2$ in timber of groups J6 and JD6

where

$D$ = bolt diameter, in millimetres

$b_{eff}$ = effective timber thickness as defined in Clause 4.4.2.2

$f'_{cj}$ = characteristic value for the species group and seasoning condition as given in Table C5

Characteristic capacities computed in accordance with the above expressions are given in Tables 4.9(B) and 4.9(C).

## C4.2 Load $Q_{kp}$

The characteristic capacity ($Q_{kp}$) for a single bolt bearing perpendicular to the grain and acting in single shear shall not exceed the lesser of—

$b_{eff} f'_{pj} D/2$

$10 f'_{pj} \sqrt{D^3}$ in timber of groups J1 and JD1

$12 f'_{pj} \sqrt{D^3}$ in timber of groups J2 and JD2

$15 f'_{pj} \sqrt{D^3}$ in timber of groups J3 and JD3

$17 f'_{pj} \sqrt{D^3}$ in timber of groups J4 and JD4

$19 f'_{pj} \sqrt{D^3}$ in timber of groups J5 and JD5

$22 f'_{pj} \sqrt{D^3}$ in timber of groups J6 and JD6

where

$D$ = bolt diameter, in millimetres

$b_{eff}$ = effective timber thickness as defined in Clause 4.4.2.3

$f'_{pj}$ = appropriate characteristic value for the species group and seasoning as given in Table C6

Characteristic capacities computed in accordance with the above expressions are given in Tables 4.10(B) and 4.10(C).

### TABLE C5
### VALUES OF $f'_{cj}$ FOR BOLTED JOINTS

A1

| Joint group | J1 | J2 | J3 | J4 | J5 | J6 |
|---|---|---|---|---|---|---|
| $f'_{cj}$ MPa | 55.5 | 44.0 | 35.5 | 28.0 | 22.0 | 18.0 |
| **Joint group** | JD1 | JD2 | JD3 | JD4 | JD5 | JD6 |
| $f'_{cj}$ MPa | 69.0 | 55.5 | 44.0 | 35.5 | 28.0 | 22.0 |

### TABLE C6
### VALUES OF $f'_{pj}$ FOR BOLTED JOINTS

| Joint group | J1 | J2 | J3 | J4 | J5 | J6 |
|---|---|---|---|---|---|---|
| $f'_{pj}$ MPa | 22.0 | 17.5 | 11.0 | 7.1 | 4.7 | 2.4 |
| **Joint group** | JD1 | JD2 | JD3 | JD4 | JD5 | JD6 |
| $f'_{pj}$ MPa | 29.5 | 22.5 | 17.0 | 12.5 | 9.0 | 6.1 |

# APPENDIX D

# ACCEPTANCE TESTING OF TIMBER STRUCTURES AND ELEMENTS

(Normative)

## D1 GENERAL

### D1.1 Limitations of acceptance testing

The methods of test given in this Appendix are applicable to structures or structural elements and are not appropriate for the testing of structural models or the establishment of general design data for timber or connections.

Reports containing complete information on the design and testing methods and the resulting data shall be made available.

Two types of load test are considered. One is a ‘proof’ load test, which shall be applied to every structure of a population of structures for them to be accepted. The other is a ‘prototype’ load test, which need be applied only to a portion of a population of structures for all structures of that population to be accepted. To carry a given load, a different structure will be necessary depending on whether the design is based on proof testing, prototype testing or computation. Partly, this arises from the necessity to use a load factor to provide for the effect of variability in structural strength. Design by proof testing implies that every structure of a population is tested, while design by computation is usually based on the results of laboratory tests on large samples of structural elements, typically 100. Hence, in general, acceptance based on proof testing will lead to the smallest overall load factor and that based on prototype testing of a few structural elements will lead to the largest load factors.

NOTE: Where a structure comprises several types of components, the specified ratio between test loads and working loads can vary considerably from one component to another. Since the test load to be used is the largest one required, it may be most economical to subdivide a structure into various groups of components and to test each such group individually. This may be effected by temporarily strengthening those parts of the structures that are not under test; however, if such temporary strengthening is carried out, care should be taken to ensure that the components under test receive their correct loading, and do not receive artificial restraints or other forms of strengthening that would not exist in the real structure.

When unseasoned timber is to be used, assessment shall be made by inspection or otherwise of the likelihood of any potential loss of strength or serviceability as a result of members shrinking on drying in the environment in which the material is to be used. Particular attention shall be paid to the effects of differential shrinkage and checking or splitting of members at joints.

NOTE: Conformity with the acceptance requirements of Paragraphs D4 and D5 is a necessary condition but may not be sufficient for total acceptability of a structure or element. The engineer or approving authority, or both, may require that other criteria, apart from strength and stiffness, be satisfied having regard to the particular service conditions of the structure or element.

### D1.2 Circumstances requiring tests

Structures or parts of structures designed in accordance with this Standard are not required to be tested unless by agreement between the parties concerned. Tests are an alternative to calculation or are appropriate in circumstances where—

(a) a structure or part of a structure is not amenable to sufficiently accurate calculation;

(b) materials or design methods are used other than those for which there is a relevant specification or code of practice; or

(c) there is doubt or disagreement as to whether the structure or some part of it complies with design rules, or as to whether the quality of the materials used is to the required Standard.

### D1.3 Information required

A copy of the detailed drawings and the specification, together with any other data or information that might be required for the purpose of the test, shall be deposited with the testing authority before the tests are commenced.

## D2 DEFINITIONS

For the purpose of this Appendix, the definitions below apply.

NOTE: These definitions also appear in Clause 1.7.

### D2.1 Prototype testing

Application of test loads to a structure or element to ascertain the structural characteristics of structures or elements that are nominally identical to the unit or units tested.

### D2.2 Proof testing

Application of test loads to a structure or element to ascertain the structural characteristics of only that one unit under test.

## D3 METHOD OF TESTING

### D3.1 General

The method by which the loading is applied to the unit to be tested and the positions at which deflections are measured can only be decided with special reference to the particular structure or element and to the particular loading conditions to be investigated.

### D3.2 Test load

The test load shall be applied and resisted in a manner that reasonably approximates the actual service conditions.

Lateral support to the unit as a whole or to individual members of the unit shall represent as closely as possible actual service conditions.

NOTE: Although both proof and prototype testing are most likely to involve symmetrical loading in a vertical plane, either additionally or alternatively, asymmetric loading of a structure or element may be required to simulate, for example, the effect of wind loading.

### D3.3 Eccentricities

Any eccentricities not inherent in the design of the structure or element, or not resulting from typical loading in service, shall be avoided at points of loading and reaction, and care shall also be taken to ensure that no inadvertent restraints are present. Where it is clear that the method of test involves a significant or appreciable divergence from service conditions, either in loading or restraint, due allowance shall be made to compensate for this. All likely combinations of permanent loads and imposed loads of shorter duration, including those due to wind and, where applicable, those due to impact, shall be taken into account when determining the worst loading conditions. The latter shall be converted in accordance with Paragraph D4 or D5, as appropriate, into an equivalent test load.

### D3.4 Load-deflection curve

A load-deflection curve shall be plotted during each test on each unit. Such a curve will serve not only as a check against observational errors, but also to indicate any irregularities in the behaviour under load of the structure or element and so enable a particular weakness to be investigated as the test progresses. It is desirable that a minimum of six points, not including the zero load point, be obtained to define the shape of the load-deflection curve if the latter is predominantly linear, and a minimum of 10 points if the curve is significantly non-linear.

## D4 PROOF TESTING

### D4.1 Equivalent test load

For the purpose of establishing an equivalence between the service loading for which the structure or element has been designed and the loading to be applied for test purposes, the following procedure shall be adopted:

(a) For each element of a structure, ascertain the critical combination of design action from either the engineer responsible for the design or from the information supplied in accordance with Paragraph D1.3.

(b) For each element of a structure, calculate the equivalent total test action or action effect ($Q_E$) that includes any loading already on the structure before the test commences, as follows:

$$Q_E = k_{26}\, k_{27} \frac{1.1 Q^*}{k_1} \qquad \text{. . . D1}$$

where

$k_{26}$ = 1.00 for structural elements in which the effect of duration of load on strength is similar to that of simple beams (values of $k_{26}$ for some other special cases are given in Table D1)

$k_{27}$ = factor obtained from Table D2 to compensate for the fact that test load is not of 15 min duration

$Q^*$ = design action or design action effect

$k_1$ = the factor from Table 2.3 appropriate to the design action of shortest duration included in the critical combination

(c) Select the largest $Q_E$ thus obtained.

**TABLE D1**

**COMPENSATION FACTOR $k_{26}$**

| Structural component | Factor $k_{26}$ |
|---|---|
| Beams with slenderness coefficients greater than 10, and all columns— | |
| timber initially dry | 1.1 |
| timber initially green | 1.4 |
| Metal fasteners— | |
| failure in timber that is initially green | 1.2 |
| failure in timber that is initially dry | 1.0 |
| for failure of steel | ($k_1/k_{27}$) |

**TABLE D2**

**COMPENSATION FACTOR $k_{27}$**

| Time to reach $Q_E$ | 15 min | 1 h | 6 h |
|---|---|---|---|
| Factor $k_{27}$ for bending and tension strength | 1.00 | 1.00 | 0.95 |
| Factor $k_{27}$ for compression strength, and for strength of metal fasteners | 1.00 | 0.95 | 0.90 |

### D4.2 Loading

The equivalent test load ($Q_E$) shall be applied to the unit at a rate as uniform as is practicable. The equivalent test load shall not remain on the unit for longer than 15 min before it is removed. If circumstances do not permit the removal of the whole of the test load within a reasonably short period, then at least 25% of the $Q_E$ shall be removed within 15 min subsequent to completion of the test, and 50% within the following hour.

### D4.3 Acceptance for strength

At no stage shall the unit show any sign of distress, or excessive distortion of any part or member. If the load-deflection curve shows any discontinuities or a considerable departure from linearity, a repeat of the test shall be performed.

### D4.4 Acceptance of deflection

A check as to whether the deflection characteristics of a structure are acceptable shall be made from the deflections measured for loads up to the total design load. It should be noted that for long duration components of the load, the effect of creep is to produce long-term deflections that are two and three times the short-term deflections measured for structures made from timbers initially seasoned and unseasoned respectively (see Clause 2.4.1.2).

If the residual deflection on unloading the structure exceeds 30% of the deflection at $Q_E$, the structure shall not be accepted unless no serious permanent damage has been done to the structure. It is appropriate to check by reloading the structure again to the $Q_E$.

## D5 PROTOTYPE TESTING

### D5.1 General

For prototype testing, provisions of Paragraphs D5.2 to D5.6 shall apply in addition to those specified in Paragraphs D1 and D3.

### D5.2 Materials

The timber used in the prototype shall contain material only of the stress grade which is being, or will be, used in manufacture. No material of a higher stress grade shall be incorporated in the unit to be tested.

### D5.3 Manufacture

The manufacture and assembly of the prototype shall comply with the design specifications, and the method of fabrication used shall simulate, as closely as possible, that which would be used in production.

### D5.4 Equivalent test load

For the purpose of establishing an equivalence between the service loading for which the structure or element has been designed and the loading to be applied for test purposes, the following procedure shall be adopted:

(a) For each element of a structure, ascertain the critical design action or action effect arising from the critical combination of action from either the engineer responsible for the design or from the information supplied in accordance with Paragraph D1.3.

(b) For each element of a structure, calculate an equivalent test action or action effect ($Q_E$) as follows:

$$Q_E = \frac{k_2\, k_{26}\, k_{27}\, k_{28}}{k_1}\, Q^* \qquad \text{. . . D2}$$

where

$k_2$ = 0.8 for prototype tests for domestic construction when failure occurs at the connectors

= 1.0 for all other cases

$k_{26}$, $k_{27}$, $k_{28}$ = factors obtained from Tables D1, D2 and D3

$k_1$ = the factor from Table 2.3 appropriate to the design action of shortest duration included in the critical combination

$Q^*$ = critical design action or action effect

Factor $k_{28}$ depends on the number of units to be tested and on the estimated coefficient of variation of strength for the total population from which the test units are selected. For guidance in making an assessment of the coefficient of variation, a likely range of values is provided in Table D4.

(c) Select the largest $Q_E$ thus obtained.

## TABLE D3

## SAMPLING FACTOR $k_{28}$

| Number of similar units to be tested ($n$) | Value of $k_{28}$ for estimated coefficient of variation (percent of strength of individual units) of— | | | |
|---|---|---|---|---|
| | 10 | 20 | 30 | 40 |
| For failure modes related to rupture of solid timber: | | | | |
| 1 | 1.6 | 2.4 | 3.5 | 5.2 |
| 2 | 1.5 | 2.1 | 2.9 | 3.9 |
| 5 | 1.4 | 1.7 | 2.2 | 2.7 |
| 10 | 1.3 | 1.5 | 1.8 | 2.1 |
| 20 | 1.2 | 1.3 | 1.4 | 1.6 |
| For failure modes related to failure of fasteners: | | | | |
| 1 | 1.8 | 2.9 | 4.8 | — |
| 2 | 1.7 | 2.6 | 3.9 | — |
| 5 | 1.6 | 2.1 | 2.9 | — |
| 10 | 1.5 | 1.9 | 2.4 | — |
| 20 | 1.4 | 1.6 | 1.9 | — |

NOTE: For intermediate coefficients of variation, use linear interpolation on a plot of coefficient of variation versus log $k_{28}$.

## TABLE D4

## LIKELY VALUES OF COEFFICIENTS OF VARIATION

| Structural element | Likely range of coefficients of variation of strength of individual unit, % |
|---|---|
| Scantlings | |
| bending strength | 25 to 40 |
| tensile strength | 30 to 50 |
| compression strength (as short column) | 15 to 25 |
| Finger-jointed elements | |
| bending strength | 15 to 20 |
| Connections | |
| nailed joints | 15 |
| toothed plate and other mechanical fasteners | 10 to 15 |
| Plywood and laminated veneer lumber | 10 to 15 |

### D5.5 Test procedure

#### D5.5.1 *Preloads*

A load equal to the design load shall be applied to the unit, maintained for 5 min and then removed. Deflections need not be measured during this preloading unless specifically requested. This load sequence shall be repeated and during this the maximum deflection, residual deflection and any other deflections shall be recorded.

#### D5.5.2 *Test loading*

Each prototype shall be loaded at a rate as uniform as practicable to failure or the $Q_E$, whichever occurs first.

### D5.6 Acceptance of prototype

#### D5.6.1 *For strength*

At no stage in its testing shall a unit have shown any failure of any part or member up to a load equal to the $Q_E$.

#### D5.6.2 *For deflection*

Each unit shall meet the requirements of Paragraph D4.4 and, in addition, the residual deflection or deformation resulting from second preloading of any part or member of the unit shall not exceed 5% of the acceptable deflection or deformation under short-duration loading.

#### D5.6.3 *Acceptance of production units*

Production-run units similar in all respects to the unit or units tested shall be deemed to be structurally acceptable if the results of the tested unit or units comply fully with the requirements of Paragraph D5.6.

## D6 REPORT OF TESTS

The report of the test on each unit, whether a proof test or prototype test, shall contain, in addition to the test results, a clear statement of the conditions of testing including the method of loading and of measuring deflection, together with any other relevant data. The nature and size of defects in the timber, especially at the points of failure, if any, and its moisture content shall be recorded. The report shall also contain a statement as to whether or not the structure or part tested satisfies the acceptance conditions and any other requirements specified.

# APPENDIX E

# FURTHER DESIGN METHODS FOR MEMBERS

(Normative)

## E1 SCOPE

This Appendix extends the design methods given in Section 3, and covers the following topics:

(a) The material constant ($\rho$) (see Paragraph E2).

(b) Slenderness coefficients for columns (see Paragraph E3).

(c) Spaced columns (see Paragraph E4).

(d) Beam-column bent about both axes (see Paragraph E5).

(e) Slenderness coefficients for beams (see Paragraph E6).

(f) Buckling restraints (see Paragraph E7).

(g) Concentrated loads and partial area loads on grid systems (see Paragraph E8).

(h) Notched beams (see Paragraph E9).

(i) Notched columns (see Paragraph E10).

(j) Notched tension members (see Paragraph E11).

(k) Single-tapered straight beams (see Paragraph E12).

(l) Double-tapered, curved and pitched-cambered beams (see Paragraph E13).

## E2 THE MATERIAL CONSTANT

Values of the material constant ($\rho$) are obtained from the following equations:

(a) *Beams of seasoned timber*:

$$\rho_b = 14.71\left(\frac{E}{f'_b}\right)^{-0.480} r^{-0.061} \quad \text{. . . E2(1)}$$

(b) *Beams of unseasoned timber*:

$$\rho_b = 11.63\left(\frac{E}{f'_b}\right)^{-0.435} r^{-0.110} \quad \text{. . . E2(2)}$$

(c) *Columns of seasoned timber*:

$$\rho_c = 11.39\left(\frac{E}{f'_c}\right)^{-0.408} r^{-0.074} \quad \text{. . . E2(3)}$$

(d) *Columns of unseasoned timber*:

$$\rho_c = 9.29\left(\frac{E}{f'_c}\right)^{-0.367} r^{-0.146} \quad \text{. . . E2(4)}$$

where

$E$ = characteristic value of average modulus of elasticity corresponding to relevant stress grade

$f'_b$ = characteristic value for bending

$f'_c$ = characteristic value for compression

$r$ = ratio (temporary design action effect)/(total design action effect)

where 'temporary design action effect' is that resulting from design actions having an effective cumulative duration of less than 12 months (see Clause 2.4.1.1).

The maximum value of $\rho_b$ or $\rho_c$ used need not exceed the value computed for the case $r = 0.25$. In the case of beams where a temporary load causes a stress reversal, the value of $\rho_b$ or $\rho_c$ to be used is that corresponding to $r = 1.0$.

Values of the material constant $\rho_b$ or $\rho_c$ computed from Equations E2(1) to E2(4) for F-grade, MGP grade and A17 grade timber are given in Tables E1 to E4.

## TABLE E1

## MATERIAL CONSTANT ($\rho_b$)—SEASONED TIMBER BEAMS

| Stress grade | Material constant ($\rho_b$) | | | | |
|---|---|---|---|---|---|
| | $r = 0$ | $r = 0.25$ | $r = 0.50$ | $r = 0.75$ | $r = 1.0$ |
| F34 | 1.12 | 1.12 | 1.07 | 1.05 | 1.03 |
| F27 | 1.08 | 1.08 | 1.03 | 1.01 | 0.99 |
| F22 | 1.05 | 1.05 | 1.01 | 0.98 | 0.97 |
| F17 | 0.98 | 0.98 | 0.94 | 0.92 | 0.90 |
| F14 | 0.98 | 0.98 | 0.94 | 0.92 | 0.90 |
| F11 | 0.98 | 0.98 | 0.94 | 0.91 | 0.90 |
| F8 | 0.89 | 0.89 | 0.85 | 0.83 | 0.82 |
| F7 | 0.86 | 0.86 | 0.83 | 0.81 | 0.79 |
| F5 | 0.82 | 0.82 | 0.78 | 0.76 | 0.75 |
| F4 | 0.80 | 0.80 | 0.77 | 0.75 | 0.74 |
| MGP 15 | 0.91 | 0.91 | 0.88 | 0.85 | 0.84 |
| MGP 12 | 0.85 | 0.85 | 0.81 | 0.79 | 0.78 |
| MGP 10 | 0.75 | 0.75 | 0.72 | 0.70 | 0.69 |
| A17 | 0.95 | 0.95 | 0.92 | 0.89 | 0.88 |

NOTE: The material constants for A17 and MGP grades listed in the table are calculated using the design characteristic bending strength values listed for the smallest sized sections in Table H3.1, Appendix H, and may be conservative for larger sizes.

## TABLE E2

## MATERIAL CONSTANT ($\rho_b$)—UNSEASONED TIMBER BEAMS

| Stress grade | Material constant ($\rho_b$) | | | | |
|---|---|---|---|---|---|
| | $r = 0$ | $r = 0.25$ | $r = 0.50$ | $r = 0.75$ | $r = 1.0$ |
| F34 | 1.21 | 1.21 | 1.12 | 1.08 | 1.04 |
| F27 | 1.17 | 1.17 | 1.09 | 1.04 | 1.01 |
| F22 | 1.15 | 1.15 | 1.06 | 1.02 | 0.99 |
| F17 | 1.08 | 1.08 | 1.00 | 0.96 | 0.93 |
| F14 | 1.08 | 1.08 | 1.00 | 0.96 | 0.93 |
| F11 | 1.07 | 1.07 | 1.00 | 0.95 | 0.92 |
| F8 | 0.99 | 0.99 | 0.91 | 0.87 | 0.85 |
| F7 | 0.96 | 0.96 | 0.89 | 0.85 | 0.82 |
| F5 | 0.91 | 0.91 | 0.85 | 0.81 | 0.78 |
| F4 | 0.90 | 0.90 | 0.83 | 0.80 | 0.77 |

NOTE: The material constants for A17 and MGP grades listed in the table are calculated using the design characteristic bending strength values listed for the smallest sized sections in Table H3.1, Appendix H, and may be conservative for larger sizes.

## TABLE E3

## MATERIAL CONSTANT ($\rho_c$)—SEASONED TIMBER COLUMNS

| Stress grade | Material constant ($\rho_c$) | | | | |
|---|---|---|---|---|---|
| | $r$ = 0 | $r$ = 0.25 | $r$ = 0.50 | $r$ = 0.75 | $r$ = 1.0 |
| F34 | 1.17 | 1.17 | 1.11 | 1.08 | 1.05 |
| F27 | 1.14 | 1.14 | 1.08 | 1.05 | 1.03 |
| F22 | 1.12 | 1.12 | 1.06 | 1.03 | 1.01 |
| F17 | 1.08 | 1.08 | 1.03 | 1.00 | 0.98 |
| F14 | 1.05 | 1.05 | 1.00 | 0.97 | 0.95 |
| F11 | 1.02 | 1.02 | 0.97 | 0.94 | 0.92 |
| F8 | 1.00 | 1.00 | 0.95 | 0.92 | 0.90 |
| F7 | 0.92 | 0.92 | 0.88 | 0.85 | 0.83 |
| F5 | 0.91 | 0.91 | 0.87 | 0.84 | 0.82 |
| F4 | 0.87 | 0.87 | 0.82 | 0.80 | 0.78 |
| MGP 15 | 0.99 | 0.99 | 0.94 | 0.92 | 0.90 |
| MGP 12 | 0.98 | 0.98 | 0.93 | 0.90 | 0.88 |
| MGP 10 | 0.96 | 0.96 | 0.91 | 0.88 | 0.86 |
| A17 | 1.10 | 1.10 | 1.04 | 1.01 | 0.99 |

NOTE: The material constants for A17 and MGP grades listed in the table are calculated using the design characteristic bending strength values listed for the smallest sized sections in Table H3.1, Appendix H, and may be conservative for larger sizes.

## TABLE E4

## MATERIAL CONSTANT ($\rho_c$)—UNSEASONED TIMBER COLUMNS

| Stress grade | Material constant ($\rho_c$) | | | | |
|---|---|---|---|---|---|
| | $r$ = 0 | $r$ = 0.25 | $r$ = 0.50 | $r$ = 0.75 | $r$ = 1.0 |
| F34 | 1.34 | 1.34 | 1.21 | 1.14 | 1.09 |
| F27 | 1.31 | 1.31 | 1.18 | 1.11 | 1.07 |
| F22 | 1.28 | 1.28 | 1.16 | 1.09 | 1.05 |
| F17 | 1.25 | 1.25 | 1.13 | 1.06 | 1.02 |
| F14 | 1.21 | 1.21 | 1.10 | 1.03 | 0.99 |
| F11 | 1.18 | 1.18 | 1.07 | 1.01 | 0.97 |
| F8 | 1.16 | 1.16 | 1.05 | 0.99 | 0.95 |
| F7 | 1.08 | 1.08 | 0.98 | 0.92 | 0.88 |
| F5 | 1.07 | 1.07 | 0.97 | 0.91 | 0.87 |
| F4 | 1.02 | 1.02 | 0.92 | 0.87 | 0.84 |

NOTE: The material constants for A17 and MGP grades listed in the table are calculated using the design characteristic bending strength values listed for the smallest sized sections in Table H3.1, Appendix H, and may be conservative for larger sizes.

## E3 SLENDERNESS COEFFICIENTS FOR COLUMNS

### E3.1 End supported columns

An evaluation of the slenderness coefficient of a column, denoted by $S_3$ for bending only about the major axis and $S_4$ for bending only about the minor axis is required in order to evaluate the stability factor $k_{12}$ referred to in Clauses 2.4.6 and 3.3.3. The value of the slenderness coefficient is obtained from the following equation:

$$S = [0.823\ (EA)/N_{cr}]^{1/2} \qquad \text{. . . E3(1)}$$

where

$(EA)$ = effective axial rigidity

$N_{cr}$ = critical elastic axial buckling load of a column

Both $(EA)$ and $N_{cr}$ are referenced to the appropriate axis.

## E3.2 Continuously restrained columns

For a bisymmetrical column, continuously restrained against lateral displacement at a distance $y_o$ from the neutral axis (see Figure E1), the slenderness coefficient with respect to lateral buckling are obtained from the following equations:

$$S = [0.823\,(EA)/N_{cr}]^{1/2} \qquad \text{. . . E3(2)}$$

$$N_{cr} = \frac{(EI)_y \times \left(\dfrac{\pi}{L_{a\phi}}\right)^2 \left(\dfrac{d^2}{4} + y_o^2\right) + (GJ)}{y_o\,(y_o + 2\,y_e) + \dfrac{(EI)_x}{(EA)} + \dfrac{(EI)_y}{(EA)}} \qquad \text{. . . E3(3)}$$

where

| | | |
|---|---|---|
| $(EA)$ | = | effective axial rigidity |
| $(EI)_x$, $(EI)_y$ | = | effective bending rigidity about major and minor axes respectively |
| $(GJ)$ | = | effective torsional rigidity |
| $y_e$ | = | distance from centroid to the point of load application, Figure E1 |
| $L_{a\phi}$ | = | distance between points of effectively rigid rotational restraints |

A1 NOTE: The parameter $y_e$ may take on negative values if the load point is on the same side of the column centroid as the effectively continuous restraint. If Equation E3(3) leads to a negative value of $N_{cr}$, then a value of $N_{cr} = \infty$ may be used in computing the slenderness coefficient $S$, i.e., $S = 0.0$.

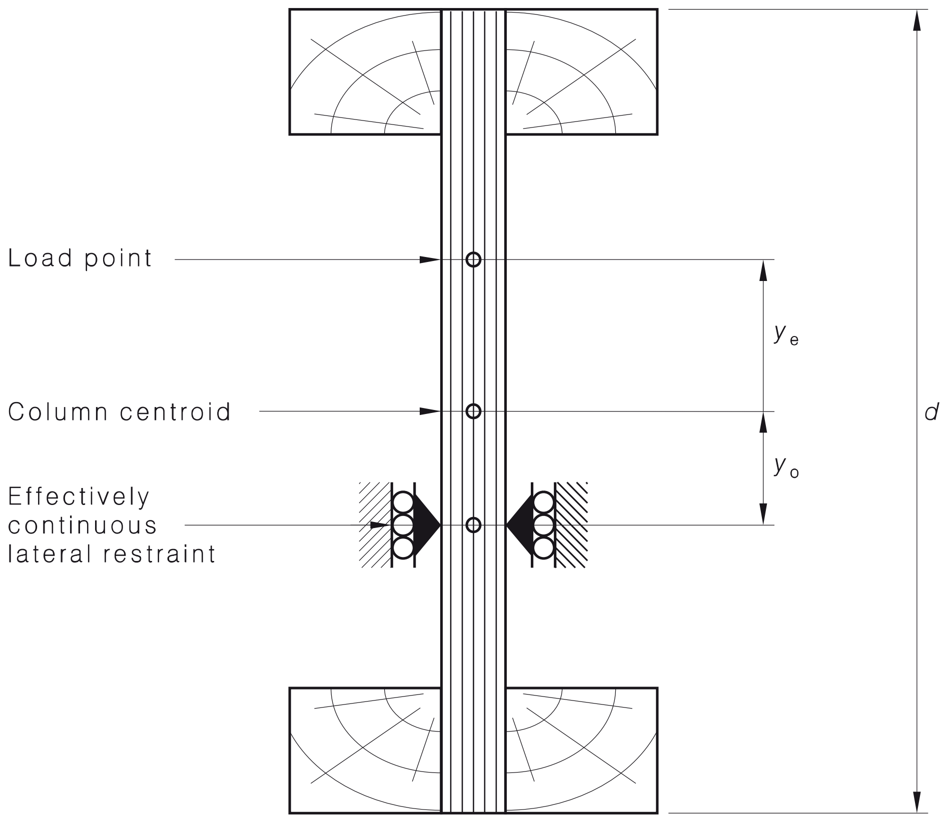


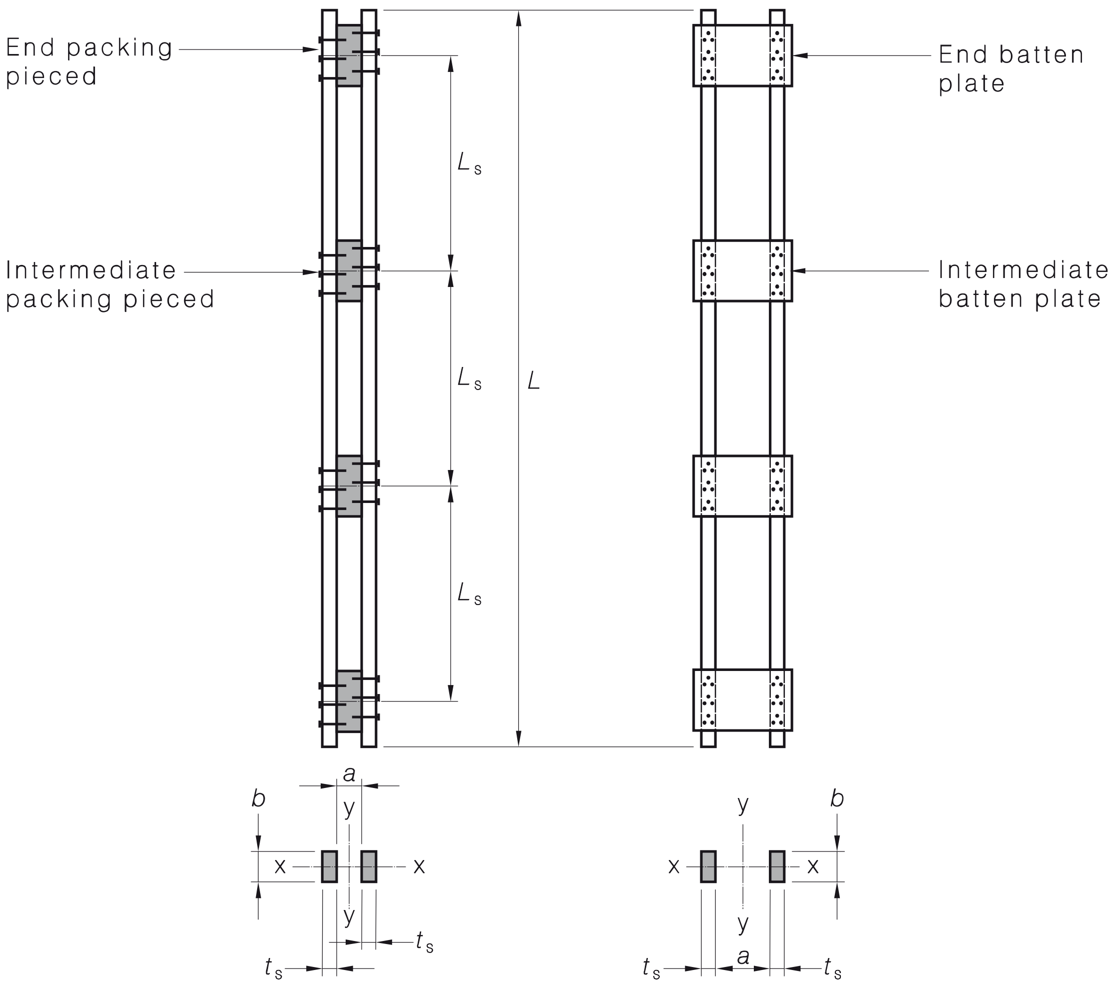

FIGURE E1 NOTATION FOR CONTINUOUSLY RESTRAINED COLUMN

## E4 SPACED COLUMNS

### E4.1 Geometry of spaced columns

Spaced columns have the individual shafts spaced apart by end and intermediate packing pieces or batten plates (see Note). These packing pieces and batten plates shall be fastened by glue, nails, screws, bolts, or split-ring or shear-plate fasteners. The notation used for spaced columns is shown in Figure E2.

NOTE: Paragraph E4.2 provides a design procedure for a particular set of spaced columns. For spaced columns with other parameters and geometry, design information may be obtained from overseas Standards.




(a) Column fabricated with packing piece (b) Column fabricated with batten plate

FIGURE E2 SPACED COLUMNS

### E4.2 Special requirements for spaced columns

**E4.2.1** *Size of connecting pieces*

Packing pieces and batten plates shall be large enough to accommodate the required number of fasteners.

**E4.2.2** *Bolted connections*

Bolts shall not be used with unseasoned timber unless it is practicable to ensure that they are tightened periodically as the timber dries out and shrinks.

**E4.2.3** *Glued connections*

Where batten plates are glued to the shafts, sufficient nails or other mechanical fasteners shall also be employed to transmit the shearing force. This provision does not apply to glued packing pieces, but if nails, screws or bolts are used to obtain clamping pressure then they shall be used in sufficient number and at suitable spacing to obtain adequate pressure over the full area of each piece.

**E4.2.4** *Spacing of intermediate packing pieces and batten plates*

The centre-to-centre distance between packing pieces or batten plates shall not exceed the least of the following:

(a) One-third of the distance between centres of the end packing pieces or end batten plates.

(b) Thirty times the thickness of the thinnest shaft.

(c) The value such that the slenderness coefficient of the portion of an individual shaft between any pair of packing pieces or batten plates is not greater than 0.7 times the maximum slenderness coefficient of the whole column, where the effective length of the individual shaft is taken as equal to the distance $L_s$ (see Figure E2) between centroids of the fasteners or glued areas in the adjacent packing pieces or batten plates.

**E4.2.5** *Distance between shafts*

The clear space between individual shafts shall not exceed 3 times the thickness of the thinnest shaft measured in the same plane.

**E4.2.6** *Battened columns*

Batten plates shall not be made from unseasoned timber.

## E4.3 Shear between components

**E4.3.1** *Design shear force*

The connections between the packing pieces or batten plates of spaced columns shall be designed to transmit the stresses resulting from the design action effect produced by strength limit states design loads in lateral shear—

$$V^* = V_1^* + V_2^* \qquad \text{. . . E4(1)}$$

where

$V_1^*$ = shear action effect

$V_2^*$ = shear action effect due to curvature of the column

= $0.075\ N^*$ for end packing piece or batten plates

= $0.001\ N^*\ (L_{ay}/d)$ for intermediate packing pieces or batten plates

$N^*$ = design action effect in axial compression

$L_{ay}$ = distance between points of lateral restraint on the spaced columns. (An end packing piece or batten plate shall be required at each point of lateral restraint.)

$d$ = $a + 2\,t_s$ (see Figure E2)

where

$a$ = distance between shafts

$t_s$ = shaft thickness

**E4.3.2** *Force effects on packing pieces*

The interface of each packing piece and its connection shall be designed to transmit a shear force $V^*_{pack}$ equal to—

$$V^*_{pack} = V^* L_s/a \qquad \ldots \text{E4(2)}$$

where

$V^*$ = resulting lateral force as defined in Paragraph E4.3.1

$L_s$ = the centre-to-centre distance of packing pieces (see Figure E2)

$a$ = distance between shafts (see Figure E2)

**E4.3.3** *Force effects on batten plates*

Each batten plate and its connection shall be designed to transmit simultaneously a longitudinal shear force ($V^*_{bat}$) and bending moment ($M^*_{bat}$) given by the following equations:

$$V^*_{bat} = \frac{0.5\, V^* L_s}{(a + t_s)} \qquad \ldots \text{E4(3)}$$

$$M^*_{bat} = 0.25\, V^* L_s \qquad \ldots \text{E4(4)}$$

where

$V^*$ = resulting lateral force as defined in Paragraph E4.3.1

$L_s$ = centre-to-centre distance between batten plates as defined in Paragraph E4.2.4

$a$ = distance between shafts (see Figure E2)

$t_s$ = shaft thickness (see Figure E2)

## E4.4 Design capacity

**E4.4.1** *Slenderness coefficients*

### E4.4.1.1 Slenderness coefficients of individual shafts

The effective length ($L_s$) of individual shafts of spaced columns shall be taken as the distance measured along the column axis between centroids of the fastener groups or glued areas in adjacent packing pieces or batten plates. From this effective length, the slenderness coefficients of the individual shafts shall be obtained in accordance with Clause 3.3.2.

### E4.4.1.2 Slenderness coefficient of composite cross-sections

For spaced columns with packing pieces, composed of two shafts of timber, the slenderness coefficient for bending about the y-axis ($S_5$) is given by the following equation:

$$S_5 = 0.3\, g_{13}\, g_{28}\, L\, (A / I)^{1/2} \qquad \ldots \text{E4(5)}$$

where

$g_{13}$, $g_{28}$ = modification factors as given in Table 3.2 and Table E5, respectively

$L$ = length of composite column

$A$ = net cross-sectional area of the shafts

$I$ = second moment of area (moment of inertia) of the net composite cross-section about the y-axis

The slenderness coefficient for bending about the x-axis shall be taken to be that of a solid timber column having the cross-sections shown in Figure E2.

**TABLE E5**

**MODIFICATION FACTOR ($g_{28}$) FOR THE EFFECTIVE LENGTH OF SPACED COLUMNS**

| Space | Value of $g_{28}$ | |
|---|---|---|
| Shaft spacing $\frac{a}{t_s}$ (see Figure E2) | Glued packing pieces and batten plates | Packing pieces and batten plates fastened by metal connectors |
| 0 | 1.0 | 1.6 |
| 1 | 1.1 | 2.2 |
| 2 | 1.3 | 2.7 |
| 3 | 1.4 | 3.0 |

**E4.4.2** *Design procedure*

The design capacity shall be taken as the least of—

(a) the design capacity for a solid column whose area is that of the sum of the cross-sectional areas of the shafts, bending about the x-axis;

(b) the design capacity for a column bending about the y-axis, whose geometrical properties of cross-section are those of the composite column, but whose slenderness coefficient is as given in Equation E4(5); and

(c) the sum of the design capacity for the individual shafts where the design capacity for each shaft is equal to that for a solid column, the effective length of which is equal to the values of $L_s$ defined in Paragraph E4.4.1.1.

NOTE: For calculation of capacity, see Clause 3.3.1.1.

## E5 BEAM-COLUMN BENT ABOUT BOTH AXES

For the case of a beam-column of rectangular cross-section, subjected to an axial compression load and bent about both axes, the following conservative criteria for strength shall be used:

$$\left(\frac{M_x^*}{M_{d,x}}\right)^2 + \frac{M_y^*}{M_{d,y}} + \frac{N_c^*}{N_{d,cy}} \leq 1 \qquad \ldots \text{E5(1)}$$

$$\frac{M_x^*}{M_{d,x}} + \left(\frac{M_y^*}{M_{d,y}}\right)^2 + \frac{N_c^*}{N_{d,cx}} \leq 1 \qquad \ldots \text{E5(2)}$$

NOTE: Equations E5(1) and E5(2) include an allowance for the effect of bending moment amplification due to the axial load.

## E6 SLENDERNESS COEFFICIENTS FOR BEAMS

### E6.1 General

To evaluate the stability factor $k_{12}$ referred to in Clause 3.2.3, the slenderness coefficient ($S_1$) of a beam is defined by the following equation:

$$S_1 = \left[\frac{1.1\,(EI)_x}{M_{cr}\, y_{max.}}\right]^{1/2} \qquad \ldots \text{E6(1)}$$

where

$(EI)_x$ = rigidity in bending about the major axis as defined in Figure 3.1

$y_{max.}$ = distance from the neutral axis to the extreme fibre

$M_{cr}$ = critical elastic buckling moment of the beam, applies about the major axis

NOTE: In some odd cases, the evaluation of the above equation for a solid beam of rectangular section can lead to a value of $S_1$ greater than given by the equations in Clause 3.2.3. In such cases, the value as given by Clause 3.2.3 may be used for obtaining $k_{12}$.

The evaluation of the slenderness coefficient requires a knowledge of $M_{cr}$, the critical elastic buckling moment. Values of the critical elastic moment for particular structural situations can be obtained from standard texts on structural analysis; however, as an aid to design, some values of the critical elastic moment are presented in Paragraphs E6.2 to E6.4.

## E6.2 End-supported beams

### E6.2.1 *General*

Paragraph E6.2 is applicable to end-supported beams of bisymmetrical cross-section for which it is appropriate to ignore the contribution of warping stiffness on buckling strength.

The ends at supports are assumed to be effectively restrained against twisting. This condition will be satisfied if the supports possess a torsional stiffness in excess of $20(GJ)/L$, where $GJ$ is the torsional rigidity of the beam and $L$ is its length.

NOTE: Information on more general sections, including the effects of warping stiffness, may be obtained from the following document:

NETHERCOT, D.A. and ROCKEY, K.C., *Unified Approach to the Elastic Lateral Buckling of Beams*, The Structural Engineer, Vol. 49, No. 7, July 1971, pp 321-330. (For erratum see vol. 51, No. 4, April 1973, pp 138-139).

### E6.2.2 *Beams with intermediate buckling restraints*

The critical elastic value of the maximum moment between two buckling restraints is given by the following equation:

$$M_{cr} = \left(\frac{h_1}{L_{ay}}\right)[(EI)_y\ (GJ)]^{1/2} \qquad \text{. . . E6(2)}$$

where

$h_1$ = moment factor given in Table E6

$L_{ay}$ = distance between effectively rigid buckling restraints

$(EI)_y$ = effective rigidity for bending about the minor axes

$(GJ)$ = effective torsional rigidity

NOTES:

1 In computing the effective torsional rigidity $(GJ)$ of beams of solid rectangular cross-section, the value of $G$ may be obtained from the following:

(a) For F-grades, MGP grades and A17 grade timber refer to Table H2.2, Appendix H.

(b) For GL grades refer to Table 7.1.

(c) For LVL refer to manufacturers specifcations.

2 The value of St Venant torsion constant $(J)$ may be taken to be given by—

$$J = \frac{d\,b^3}{3}\left(1 - 0.63\frac{b}{d}\right)$$

For rectangular sections of solid wood, a conservative approximation to the value of slenderness coefficient obtained from Equations E6(1) and E6(2) is—

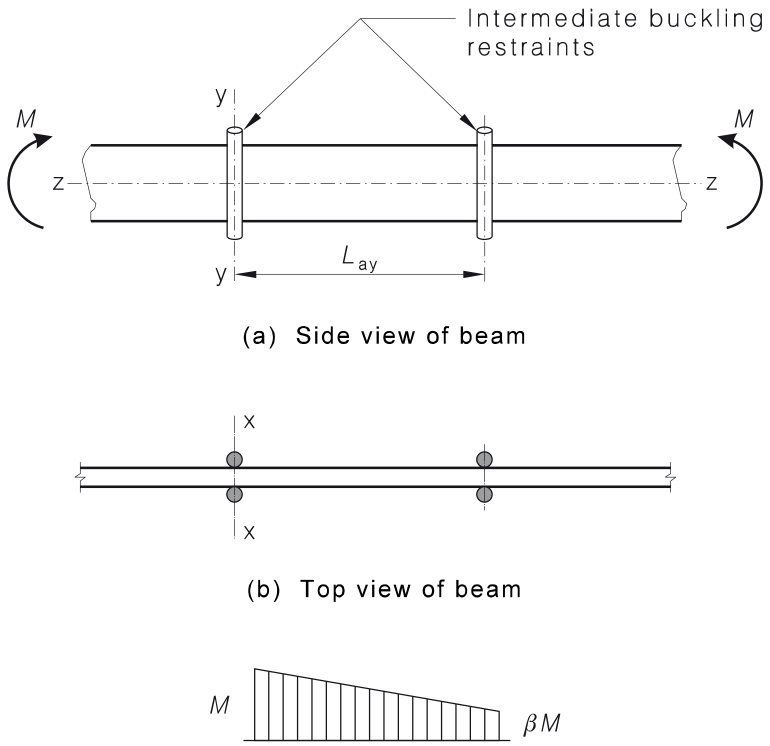
$$S_1 = \left(\frac{4.8\,d\,L_{ay}}{b^2\,h_1}\right)^{1/2} \qquad \text{. . . E6(3)}$$

## TABLE E6

## MOMENT FACTORS OF BISYMMETRICAL BEAMS WITH INTERMEDIATE BUCKLING RESTRAINTS

| Moment parameter ($\beta$) (see Figure E3(c)) | Slenderness factor ($h_1$) | |
|---|---|---|
| | Free restraint condition (see Note) | Fixed restraint condition (see Note) |
| 1.0 | 3.1 | 6.3 |
| 0.5 | 4.1 | 8.2 |
| 0.0 | 5.5 | 11.1 |
| −0.5 | 7.3 | 14.0 |
| −1.0 | 8.0 | 14.0 |

NOTE: The buckling restraints shall prevent rotation of the beam about the z-axis. The terms 'free' and 'fixed' restraint condition refer to the possibility for rotation of the beam about the y-y axis at the restraint locations, as shown in Figure E3.




(c) Diagram of bending moment between buckling restraints

FIGURE E3 NOTATION FOR BEAMS WITH INTERMEDIATE BUCKLING RESTRAINTS

## E6.3 Beams with no intermediate buckling restraints but with torsional restraints at the ends

For beams with no intermediate buckling restraints but with torsional restraints at the ends, the critical elastic value of maximum moment is given by the following equation:

$$M_{cr} = \frac{h_2}{L_{ay}}[(EI)_y \times (GJ)]^{1/2}\left\{1 - h_3 \frac{y_h}{L_{ay}}\left[\frac{(EI)_y}{(GJ)}\right]^{1/2}\right\} \quad \text{. . . E6(4)}$$

where

$(EI)_y$ = rigidity in bending about the minor axis, (y-axis) (Figure E3(a))

$(GJ)$ = torsional rigidity

$y_h$ = height above centroid of the point of load application

$h_2$, $h_3$ = moment factors, (see Table E7)

$L_{ay}$ = $L$ = span of beam

For beams loaded only by end moments, it is appropriate to use Equation E6(4) with $h_3 = y_h = 0$ and the coefficient $h_2$ equal to $h_1$ and taken from Table E6.

For rectangular sections of solid wood, a conservative approximation of the value of the slenderness coefficient is obtained from the following equation:

$$S = \left(\frac{\dfrac{4.8\, d\, L_{ay}}{b^2}}{h_2\,[1 - 2.4\, h_3\,(y_h / L_{ay})]}\right)^{1/2} \quad \text{. . . E6(5)}$$

Equations E6(3) and E6(5) are good approximations for $b \leq 0.5d$.

NOTE: In Table E7, the values of the coefficients $h_2$ and $h_3$ apply to beams with lateral restraints only at their end points. These coefficients may be used for any other beam load system that has a similar shape of bending moment diagram between points of lateral restraint.

## E6.4 Continuously restrained beams

For beams of bisymmetrical cross-section, continuously restrained against lateral displacement at a distance $y_o$ on the tension side of the neutral axis (see Figure E4), the critical elastic moment ($M_{cr}$) is given by the following equation:

$$M_{cr} = \frac{\left(\dfrac{\pi}{L_{a\phi}}\right)^2 (EI)_y \left(\dfrac{d^2}{4} + y_o^2\right) + (GJ)}{2\, y_o + y_h} \quad \text{. . . E6(6)}$$

where

$L_{a\phi}$ = distance between points of effectively rigid rotational restraints

$y_o$ = distance to the lateral restraint from the neutral axis on the tension side

$y_h$ = location above the neutral axis of the loading point (see Figure E4)

NOTE: The parameter $y_h$ may take on negative values. If Equation E6(6) leads to a negative value
A1 of $M_{cr}$, then a value of $M_{cr} = \infty$ may be used to compute the slenderness coefficient ($S$), i.e., $S = 0.0$. A rotational restraint may be obtained by the use of diagonal fly braces.

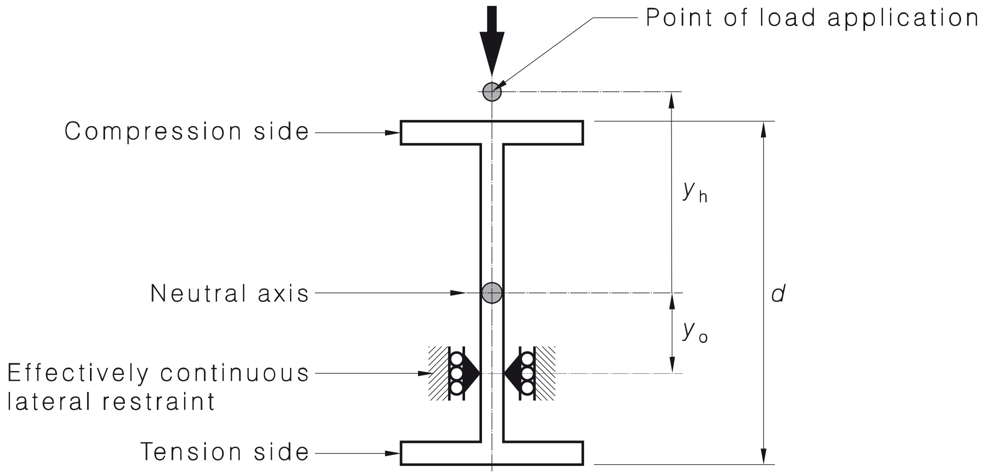
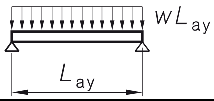
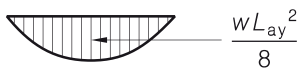
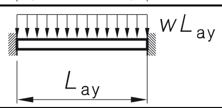
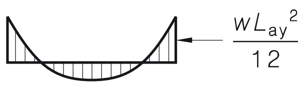
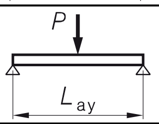
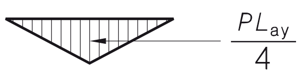
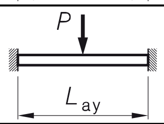
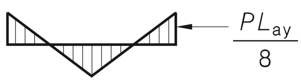
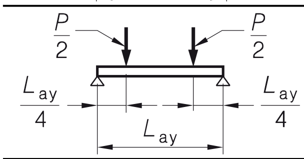
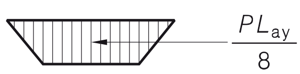
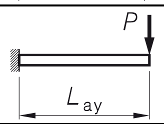
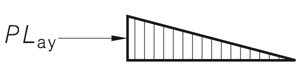
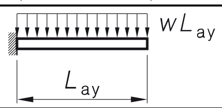
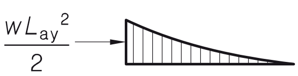



FIGURE E4 CONTINUOUSLY RESTRAINED BEAM

**TABLE E7**

**MOMENT FACTORS OF BISYMMETRICAL BEAMS WITH NO INTERMEDIATE BUCKLING RESTRAINTS**

| Loading | Bending moment (*M*) | Condition of end restraint against rotation about y-y axis* | Slenderness factors $h_2$ | Slenderness factors $h_3$ |
|---|---|---|---|---|
|  $wL_{ay}$, $L_{ay}$ |  $\frac{wL_{ay}^2}{8}$ | Free<br>Fixed | 3.6<br>6.1 | 1.4<br>1.8 |
|  $wL_{ay}$, $L_{ay}$ |  $\frac{wL_{ay}^2}{12}$ | Free<br>Fixed | 4.1<br>7.6 | 4.9<br>5.2 |
|  $P$, $L_{ay}$ |  $\frac{PL_{ay}}{4}$ | Free<br>Fixed | 4.2<br>6.7 | 1.7<br>2.6 |
|  $P$, $L_{ay}$ |  $\frac{PL_{ay}}{8}$ | Free<br>Fixed | 5.3<br>6.5 | 4.5<br>5.3 |
|  $\frac{P}{2}$, $\frac{P}{2}$, $\frac{L_{ay}}{4}$, $L_{ay}$, $\frac{L_{ay}}{4}$ |  $\frac{PL_{ay}}{8}$ | Free<br>Fixed | 3.3<br>— | 1.3<br>— |
|  $P$, $L_{ay}$ |  $PL_{ay}$ | Fixed | 4.0 | 2.0 |
|  $wL_{ay}$, $L_{ay}$ |  $\frac{wL_{ay}^2}{2}$ | Fixed | 6.4 | 2.0 |

* For direction of y-y axis (minor axis), see Figure E3(a) (free ends of cantilevers excepted).

## E7 BUCKLING RESTRAINTS

### E7.1 Effectiveness of buckling restraints

For most design situations, the effectiveness of buckling restraints need not be checked. Where a check is needed, it is appropriate to assess the capacity of the restraint system using the following equations:

$$N_R^* = K_A \Delta_A \qquad \text{. . . E7(1)}$$

$$T_R^* = K_B \theta_B \qquad \text{. . . E7(2)}$$

where $N_R^*$ and $T_R^*$ are the restraint force and torque respectively that occur when the point of attachment of the restraint to the beam undergoes a displacement ($\Delta_A$) and rotation ($\theta_B$).

It is assumed that the ends of beams are effectively restrained against torsional rotation (see Paragraph E6.2.1).

### E7.2 Notation

Notation used in Paragraph E7 is as follows:

$h_{26}$ = 1.0 when loads are live loads only
= 1.5 when loads are dead loads only and timber is initially seasoned
= 2.0 when loads are dead loads only and timber is initially unseasoned (values of $h_{26}$ for other conditions are obtained by linear interpolation)

$h_{27}$ = 1.0 for sawn timbers
= 0.4 for laminated and other carefully fabricated timber members

$g_{38}$ = lesser of $(m + 1)/2$ and 5

$m$ = number of members supported by restraint system

$n$ = number of equally spaced intermediate restraints

$S_{max.}$ = slenderness coefficient if there are no restraints

$S_{min.}$ = slenderness coefficient if the restraints are effectively rigid

### E7.3 Columns

**E7.3.1** *Load capacity*

In computing the load capacity of a column of length ($L$) with $n$ intermediate lateral restraints as shown in Figure E5(a), the slenderness coefficient ($S_4$) for buckling about the minor axis is given by the following equation:

$$S_4 = S_{max.}/(\alpha_1)^{1/4} \qquad \text{. . . E7(3)}$$

but not less than $S_{min.}$ and not more than $S_{max.}$ and where—

$$\alpha_1 = (n + 1)\, K_A\, S_{max.}^2\, L\, g_{38} / (2\, E\, A) \qquad \text{. . . E7(4)}$$

It is appropriate to use a similar method to compute the effect of restraints against buckling about the major axis.

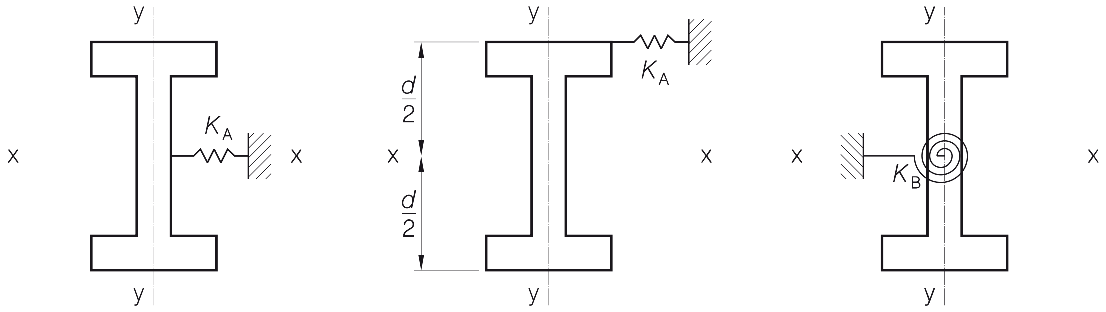
**E7.3.2** *Force on lateral restraints*

The design force ($N_R^*$) on each lateral restraint is estimated by the following equation:

$$N_R^* = \frac{0.1\, N_c^*}{n+1} h_{26}\, h_{27}\, g_{38} \qquad \text{. . . E7(5)}$$

where

$N_c^*$ = design action effect in compression




(a) Column lateral restraint (b) Beam lateral restraint (c) Beam torsional restraint

FIGURE E5 INTERMEDIATE RESTRAINTS

## E7.4 Beam with lateral restraints

### E7.4.1 *Load capacity*

In computing the load capacity of a beam of length ($L$) with $n$ intermediate lateral restraints as shown in Figure E5(b), the slenderness coefficient ($S_1$) for buckling about the minor axis is given by the following equation:

$$S_1 = \frac{S_{\text{max.}}}{(\alpha_2)^{1/6}} \qquad \text{. . . E7(6)}$$

but not less than $S_{\min}$ and not more than $S_{\max}$ and where—

$$\alpha_2 = \frac{(n+1) K_A d\, L\, S^2_{\text{max.}}}{E\, Z_x}\, g_{38} \qquad \text{. . . E7(7)}$$

It is appropriate to use a similar method to compute the effect of restraints against buckling about the major axis.

### E7.4.2 *Force on lateral restraints*

The design force $N^*_R$ on each lateral restraint is estimated by the following equation:

(a) For members of rectangular section and for box beams:

$$N^*_R = \frac{0.1\, M^*_a}{d\,(n+1)}\, h_{26}\, h_{27}\, g_{38} \qquad \text{. . . E7(8)}$$

(b) For I-beams:

$$N^*_R = \frac{0.05\, M^*_a}{d\,(n+1)}\, h_{26}\, h_{27}\, g_{38} \qquad \text{. . . E7(9)}$$

where

$M^*_a$ = applied bending moment on the beam

## E7.5 Beam with torsional restraints

### E7.5.1 *Load capacity*

In computing the load capacity of a beam of length ($L$) with $n$ intermediate lateral restraints as shown in Figure E5(c), the slenderness coefficient ($S_1$) for buckling about the minor axis is given by the following equation:

$$S_1 = \frac{S_{\text{max.}}}{(\alpha_3)^{1/4}} \qquad \text{. . . E7(10)}$$

but not less than $S_{min.}$ and not more than $S_{max.}$ and where—

$$\alpha_3 = \frac{(n+1)\, I_y\, K_B\, S_{max.}^4}{Z_x^2\, L\, E}\, g_{38} \qquad \text{. . . E7(11)}$$

**E7.5.2** *Torque on torsional restraints*

The design torque ($T_R^*$) on each restraint for members of rectangular section and for box beams is estimated by the following equation:

$$T_R^* = \frac{0.2\, M_a^*}{n+1}\, h_{26}\, h_{27}\, g_{38} \qquad \text{. . . E7(12)}$$

The design torque on each restraint for I-beams is estimated by the following equation:

$$T_R^* = \frac{0.10\, M_a^*}{n+1}\, h_{26}\, h_{27}\, g_{38} \qquad \text{. . . E7(13)}$$

where

$M_a^*$ = applied bending moment on the beam

## E8 CONCENTRATED LOADS AND PARTIAL AREA LOADS ON GRID SYSTEMS

### E8.1 General

Paragraph E8 provides a method for assessing the lateral distribution effects of a beam grid system with respect to concentrated and partial area loads. It is appropriate to use the strength-sharing factor $k_9$ specified in Clause 2.4.5.3 in combination with the lateral distribution effects given in Paragraphs E8.2 and E8.3.

### E8.2 Concentrated action

For a beam located within a grid system and subjected to a concentrated action $P^*$ as shown in Figure E6, then the maximum bending and shear design action effects, and also the maximum deflection, may be taken to be equal to that of an isolated beam loaded by a concentrated action ($P_{eff}^*$) as defined by the following equation:

$$P_{eff}^* = g_{42}\, P^* \qquad \text{. . . E8(1)}$$

where $g_{42}$ is bounded by the range $0.2 \le g_{42} \le 1.0$, and in this range it is given by the following equation:

$$g_{42} = 0.20\, \log\left(\frac{h_B}{n_C\, h_C}\right) + 0.95 \qquad \text{. . . E8(2)}$$

$P^*$ = concentrated action

where

$h_B$ = $E_B\, I_B / L^3$

$h_C$ = $E_C\, I_C / S^3$

$E_B\, I_B$, $E_C\, I_C$ = rigidity of a single beam and a single crossing member, respectively

$n_C$ = number of crossing members

$L$, $S$ = span and spacing of beams (see Figure E6)

NOTE: The expressions for rigidity, $E_B I_B$ or $E_C I_C$, given above are 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 on overall rigidity may be significant, the expressions are not valid. A suitable alternative is to calculate an effective overall rigidity, taking appropriate account of the effect of shear, for the particular span and load case.

Equation E8(2) applies when the centroid of the loads lies within the middle half of the beam, and the loaded beam is at least two beams in from the edge. For loads outside these limits, $g_{42}$ is obtained by interpolating between the above value of $g_{42}$ and 1.0.

Values of $g_{42}$ derived according to Equation E8(2) are shown in Figure E7.

NOTE: If the point load ($P^*$) shown in Figure E6(a) is located somewhere between two main beams, then a conservative load distribution factor may be obtained by using the value $g_{43}$ given in Paragraph E8.3 for the case of a partial area load of width equal to $s$.

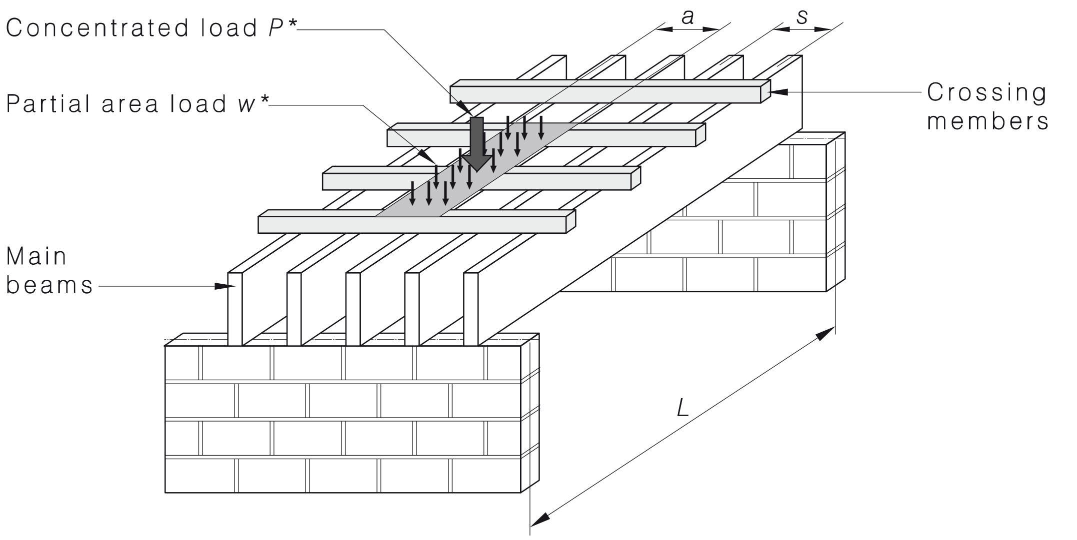



FIGURE E6 NOTATION FOR BEAM-GRID SYSTEM

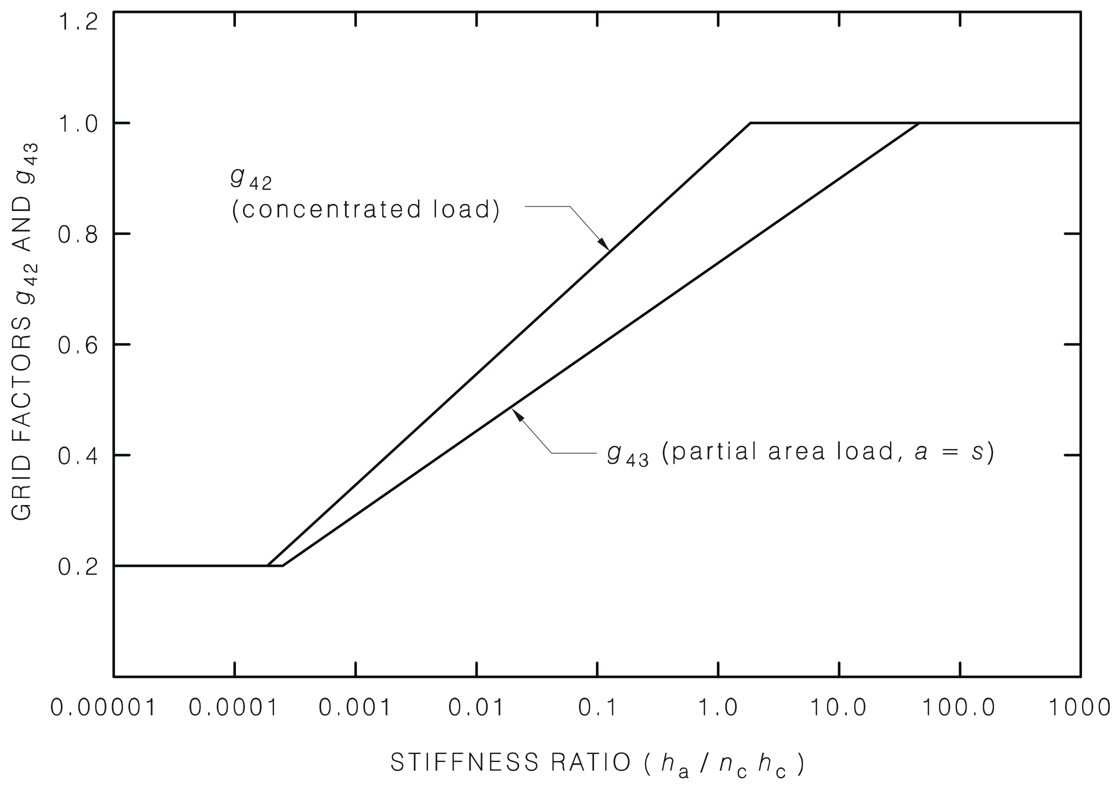



FIGURE E7 GRID FACTORS $g_{42}$ AND $g_{43}$

For the special case where the load is in the centre of the span of the member, the maximum deflection may be taken to be equal to that of an isolated beam loaded with a point load ($P^*_{\text{eff}}$) as defined by the following equation:

$$P^*_{\text{eff}} = g_{41}\, P^* \qquad \text{. . . E8(3)}$$

where

$g_{41}$ is bounded by the range $0.2 \leq g_{41} \leq 1.0$ and, within this range, is given by the following equation:

$$g_{41} = 0.883 - 0.34 \log\left(\frac{n_{\text{C}}\, h_{\text{C}}}{h_{\text{B}}} + 0.44\right) \qquad \text{. . . E8(4)}$$

### E8.3 Partial area load

For a beam located within a grid system and subjected to a uniformly distributed load ($w^*$) over a load width ($a$) equal to or less than the beam spacing ($s$) as shown in Figure E6(b), the maximum action effect in bending, shear, and deformation may be taken as equal to that of an isolated beam loaded by a load of intensity ($w^*_{\text{eff}}$) as defined by the following equation:

$$w^*_{\text{eff}} = g_{43}\, w^* \qquad \text{. . . E8(5)}$$

where the value of $g_{43}$ is bounded by $0.2 \leq g_{43} \leq 1.0$.

For the case of $a = s$, where $s$ is the centre-to-centre spacing of support members in a grid system, the value of $g_{43}$ is given by the following equation:

$$g_{43} = 0.15\ \log_{10}\left(\frac{h_{\text{B}}}{n_{\text{C}}\, h_{\text{C}}}\right) + 0.75 \qquad \text{. . . E8(6)}$$

For the case of $a = 0$ (i.e., a line load) the value of $g_{43}$ is given by $g_{42}$ as in Equation E8(2).

For the case of $0 < a < s$, the value $g_{43}$ is obtained by linear interpolation between the two above cases.

Equation E8(6) applies when the centroid of the action effects lies within the middle half of the beam, and the loaded beam is to be at least two beams in from the edge. For action effects outside these limits, $g_{43}$ is obtained by interpolating between the above value and 1.0.

Values of $g_{43}$ derived according to Equation E8(6) are shown in Figure E7.

## E9 NOTCHED BEAMS

For a rectangular beam of depth ($d$) notched as shown in Figure E8, 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:

$$\frac{6\, M^*}{b\, d_{\text{n}}^2} + \frac{6\, V^*}{b\, d_{\text{n}}} \leq \phi\, g_{40}\, k_1\, k_4\, k_6\, k_{12}\, f'_{\text{sj}} \qquad \text{. . . E9(1)}$$

where the factors $k_1$ to $k_{12}$ are given in Section 2 and coefficient $g_{40}$ is given in Table E8.

The stability factor $k_{12}$ need not be considered in checking the fracture strength of notched beams, provided the notch is not located within the middle third of the beam span.

Strength-reducing characteristics (for grading purposes, i.e., knots and similar) shall not be permitted within 150 mm of the notched roots of critical beams (i.e., non-load-sharing beams).

If, according to the sign convention shown in Figure E8, $M^*$ is negative, it is appropriate to take $M^*$ equal to zero in the application of Equation E9(1). Similarly, if $V^*$ is negative, it is appropriate to take $V^*$ equal to zero in the application of Equation E9(1).

NOTE: In addition to the check on fracture strength according to Equation E9(1), the net section of depth ($d_n$) should also be checked for its un-notched strength according to Clause 3.2.1. Moreover, it should be noted that in calculating the shear force ($V^*$) for use in Equation E9(1), all loads on the beam are to be taken into consideration, including those loads lying within a distance of 1.5 times the height of the beam from the inside face of the support.

Notching creates a significant reduction in the strength of a beam unless the notching is limited to the vicinity of support points. The adverse effects of notching may be minimized by increasing the opening angle of the notch.

A typical example of a beam notched on the compression edge would be that of a continuous member notched over a support across which it rests. In this case $M^*$ may be neglected, but an effective value of $V^*$ still occurs for use in Equation E9(1).

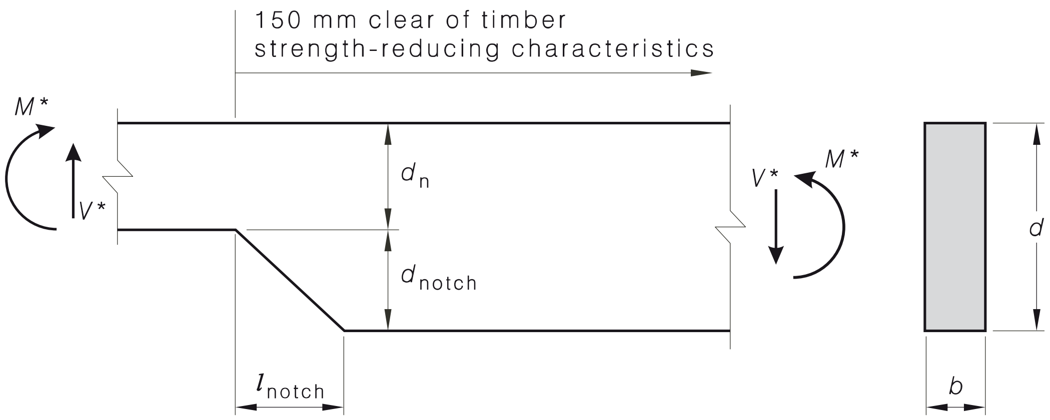



FIGURE E8 NOTATION FOR NOTCH

**TABLE E8**

**COEFFICIENT $g_{40}$ FOR SAWN NOTCH ON BEAM EDGE**

| Notch angle slope (see Figure E8) | $g_{40}$ | |
|---|---|---|
| | $d_{notch} \geq 0.1d$ | $d_{notch} < 0.1d$ |
| $l_{notch}/d_{notch} = 0$ | $9.0/d^{0.45}$ | $3.2/d_{notch}^{0.45}$ |
| $l_{notch}/d_{notch} = 2$ | $9.0/d^{0.33}$ | $4.2/d_{notch}^{0.33}$ |
| $l_{notch}/d_{notch} = 4$ | $9.0/d^{0.24}$ | $5.2/d_{notch}^{0.24}$ |

NOTE: $l_{notch}$, $d_{notch}$ and $d$ are to be stated in millimetres (see Figure E8).

## E10 NOTCHED COLUMNS

For a column supporting an axial load ($N^*$) notched in the middle third and with a stability factor $k_{12}$ <0.5, a check shall be made that the fracture strength is adequate. In other cases, no check need be made for the effect of notching on the strength of a column.

The fracture strength is considered to be adequate if the notched beam is capable of sustaining an applied moment $M^* = (d/6)\, k_1\, N^* (1 - 2\, k_{12})$ when a check is made in accordance with Paragraph E9.

## E11 NOTCHED TENSION MEMBERS

In the absence of other information, the characteristic tension strength applied to the net section of a notched member shall be taken to be equal to that of the characteristic bending strength of a similar notched member. This bending strength shall be computed according to Equation E9(1) except that the factor $k_{12}$ shall be taken as unity.

In computing the characteristic tension strength, any stresses induced by bending due to notching and any other geometric asymmetries shall be taken into account.

## E12 SINGLE-TAPERED STRAIGHT BEAMS

### E12.1 Geometry

Single-tapered straight beams are beams with a uniform taper from one end to the other. The provisions of this Paragraph shall apply only if the angle ($\alpha$) of the taper is less than or equal to 10 degrees. The angle ($\alpha$) is the angle between the tapered surface and the grain direction (see Figure E9).

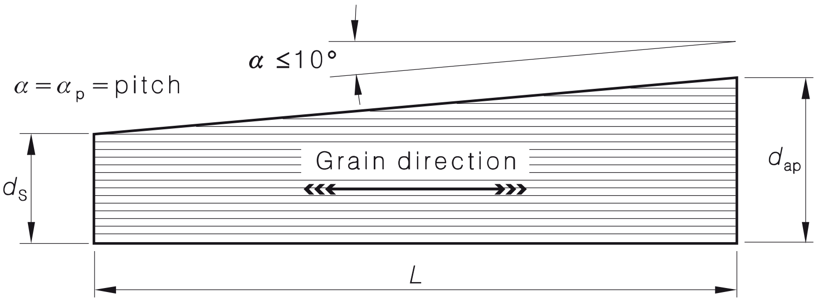



FIGURE E9 NOTATION—SINGLE TAPERED STRAIGHT BEAM

### E12.2 Design bending capacity of single-tapered beams

The design capacity in bending of single-tapered laminated beams, $M_d$, shall satisfy the following:

$$M_d \geq M^* \qquad \text{. . . E12(1)}$$

where

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_{tg}\, k_{tb}\, f'_b Z \qquad \text{. . . E12(2)}$$

and

the limiting value of $M_d$ is the minimum of $M_d$ calculated for the top edge of the beam and $M_d$ calculated for the bottom edge of the beam

$M^*$ = design action effect in bending

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load (see Clause 2.4.1.1)

$k_4$ = factor for partial seasoning (see Clause 2.4.2 for sawn rectangular and glued-laminated beams, and Clause 8.4.3 for structural LVL)

$k_6$ = factor for temperature (see Clause 2.4.3)

$k_9$ = factor for strength sharing (see Clause appropriate to member being designed, Clause 2.4.5 for rectangular sawn members, Clause 7.4.3 for glued-laminated beams and Clause 8.4.6 for LVL members)

$k_{12}$ = stability factor (see Clauses 3.2.3, 7.4.4 and 8.4.7)

$k_{tg}$ = grain orientation factor for single tapered straight beams (as defined in Paragraph E12.4)

$k_{tb}$ = taper angle factor for single tapered straight beams (as defined in Paragraph E12.5)

$f'_b$ = characteristic value in bending (given in Table H2.1 and H3.1, Appendix H for rectangular sawn members, Clause 7.3.1 for glued laminated beams and Clause 8.3.1 for LVL members)

$Z$ = section modulus about the bending axis (given in Clause 3.2.1.1 for rectangular beams) at the 'critical' section

Different values of $k_{tg}$ $k_{tb}$ apply for the top of the beam and the bottom of the beam.

### E12.3 Critical section for bending

The critical section for bending is the cross-section in the length of the beam that has the highest elastic bending stress at the outside fibre.

For simply supported single-tapered straight beams under uniformly distributed loading, the critical section occurs at a distance $z$ from the smallest end of the beam (see Figure E9), where $z$ is given by the following equation:

$$z = L\left(1 + \frac{d_{ap}}{d_s}\right)^{-1} \quad \ldots \text{E12(3)}$$

where

$L$ = span of the beam

$d_{ap}$ = depth at the deepest end (see Figure E9)

$d_s$ = depth at the smallest end (see Figure E9)

### E12.4 Grain orientation modification factor ($k_{tg}$)

The grain orientation modification factor for single-tapered straight beams is given in Table E9.

**TABLE E9**

A1 **VALUES OF GRAIN ORIENTATION MODIFICATION FACTOR ($k_{tg}$) FOR α ≤10°**

| Angle of taper (α), degrees | For edge at angle to the grain | For edge parallel to grain |
|---|---|---|
| 0° | 1.00 | 1.00 |
| 2.5° | 0.99 | 1.01 |
| 5° | 0.97 | 1.03 |
| 7.5° | 0.94 | 1.07 |
| 10° | 0.89 | 1.14 |

NOTE: For the single-tapered beam shown in Figure E9, the lower edge will have grain parallel to the edge, and the upper edge will have grain at an angle α to the edge.

### E12.5 Taper angle factor ($k_{tb}$)

The taper angle factor for single-taper beams is given by the following equations:

(a) For tension edges:

$$k_{tb} = \left(\frac{f'_b}{f'_{tp}} \times \sin^2\alpha_t + \cos^2\alpha_t\right)^{-1} \quad \ldots \text{E12(4)}$$

(b) For compression edges:

$$k_{tb} = \left(\frac{f'_b}{f'_p} \times \sin^2\alpha_c + \cos^2\alpha_c\right)^{-1} \quad \ldots \text{E12(5)}$$

where

$f'_b$ = characteristic value in bending (given in Table H2.1 and H3.1, Appendix H for rectangular sawn members, Clause 7.3.1 for glued laminated beams and Clause 8.3.1 for LVL members)

$f'_{tp}$ = characteristic tensile strength perpendicular to grain for the member (given in Table H2.2, Appendix H, for rectangular sawn members and glued laminated beams and Clause 8.3.1 for LVL)

$f'_p$ = characteristic compression strength perpendicular to grain for the member (given in Table H2.2, Appendix H for rectangular sawn members and glued laminated beams and Clause 8.3.1 for LVL)

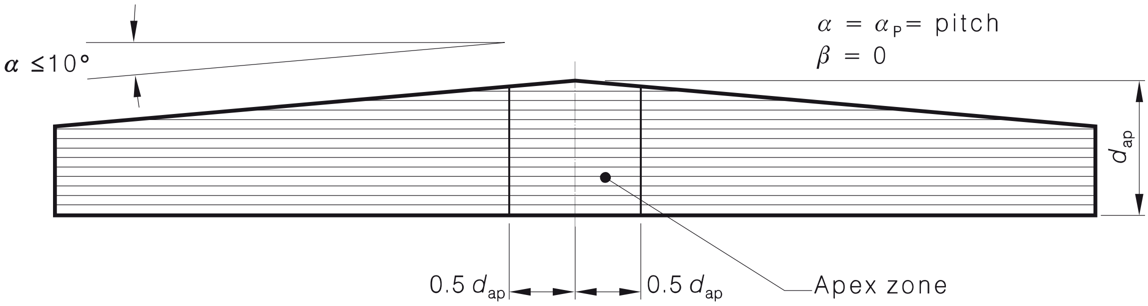
$\alpha_c$ = angle of taper at the compression edge

$\alpha_t$ = angle of taper at the tension edge

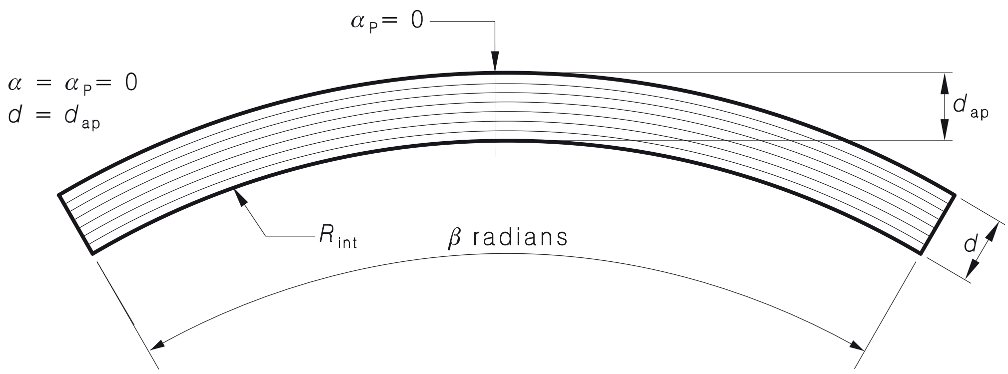
## E13 DOUBLE-TAPERED, CURVED AND PITCHED-CAMBERED BEAMS

### E13.1 Geometry

Double-tapered, curved and pitched-cambered beams are beams with varying cross-section and vertical alignment, with geometrical parameters as defined in Figure E10.

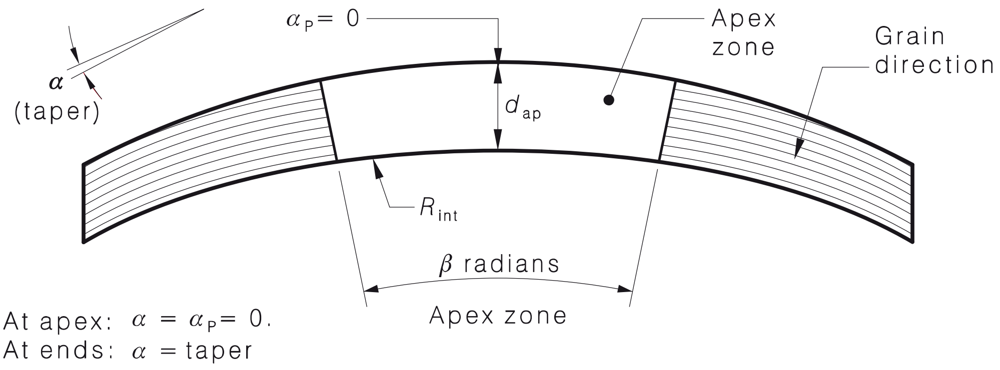



(a) Double-tapered straight beam

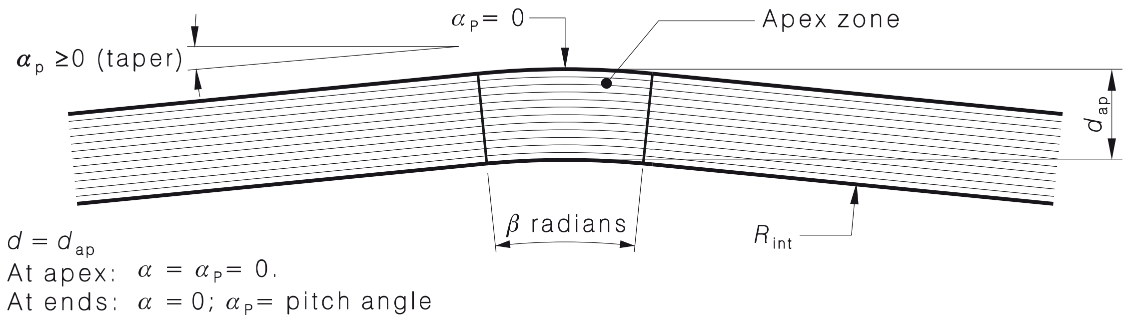



(b) Constant radius curved beam

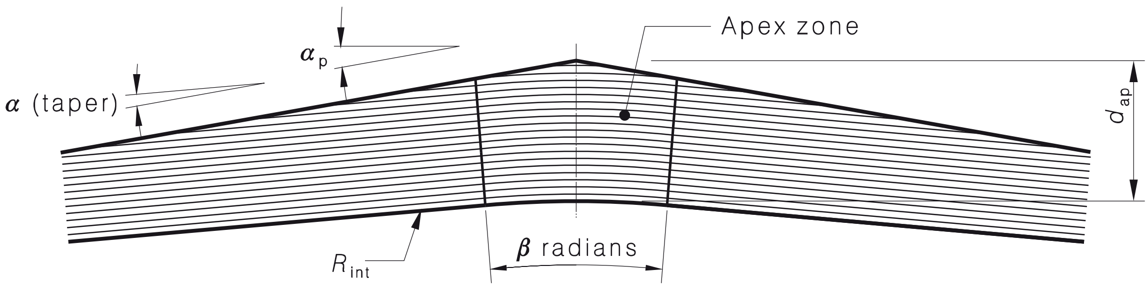
FIGURE E10 (in part) GEOMETRY OF APEX ZONE IN CURVED AND TAPERED BEAMS




(c) Constant radius curved-tapered beam




(d) Curved apex cambered beam




(e) Pitched-cambered beam

NOTES:

1 The pitch angle ($\alpha_p$) is the angle between a horizontal plane and the upper surface of the beam measured at the apex.

2 For beams with no curvature (such as double-pitched beams), the ratio of depth at the apex to radius of curvature of the timber surface is equal to zero.

3 The enclosed angle defining the apex zone is $\beta$ radians, measured between the points of tangency.

FIGURE E10 (in part) GEOMETRY OF APEX ZONE IN CURVED AND TAPERED BEAMS

## E13.2 Double-tapered, curved and pitched-cambered beams

### E13.2.1 *Design capacity*

The design capacity in bending of double-tapered, curved and pitched cambered beams ($M_d$) shall satisfy the following:

$$M_d \geq M^* \qquad \text{. . . E13(1)}$$

where

$M_d$ is the lesser of—

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, k_{sh}\, k_r\, f'_b Z \qquad \text{. . . E13(2)}$$

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_v\, f'_{tp} \frac{2A_{ap}\, R_{cl}}{3} \qquad \text{. . . E13(3)}$$

$$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_v\, k_{tp}\, f'_{tp} Z \qquad \text{. . . E13(4)}$$

and

$\phi$ = capacity factor (see Clause 2.3)

$k_1$ = factor for duration of load (see Clause 2.4.1.1)

$k_4$ = factor for partial seasoning (see Clause 2.4.2 for sawn rectangular and glued-laminated beams, and Clause 8.4.3 for structural LVL)

$k_6$ = factor for temperature (see Clause 2.4.3)

$k_9$ = factor for strength sharing (see Clause appropriate to member being designed, Clause 2.4.5 for rectangular sawn members, Clause 7.4.3 for glued-laminated beams and Clause 8.4.6 for LVL members)

$k_{12}$ = stability factor (see Clauses 3.2.3, 7.4.4 and 8.4.7)

$k_{sh}$ = beam shape factor (as defined in Paragraph E13.2.3)

$k_r$ = radius of curvature factor (as defined in Paragraph E13.2.4)

$k_v$ = size/volume factor (as defined in Paragraph E13.2.5)

$k_{tp}$ = radial stress factor (as defined in Paragraph E13.2.6)

$f'_b$ = characteristic value in bending (given in Table H2.1 and H3.1, Appendix H, for rectangular sawn members, Clause 7.3.1 for glued laminated beams and Clause 8.3.1 for LVL members)

$f'_{tp}$ = characteristic tensile strength perpendicular to grain (or the member given in Table H2.2, Appendix H, for rectangular sawn members and glued laminated beams and Clause 8.3.1 for LVL)

$Z$ = section modulus about the bending (axis as given in Clause 3.2.1.1 for rectangular beams) at the critical section

$A_{ap}$ = cross-sectional area at the critical section

$R_{cl}$ = radius of curvature of the centre-line of the beam at the critical section

### E13.2.2 *Critical section for bending*

The critical section for bending is the cross-section in the length of the beam that has the highest bending moment. In general, this will be the apex (i.e., depth = $d_{ap}$)

**E13.2.3** *Shape factor ($k_{sh}$)*

Values of $k_{sh}$ for various radii and pitch angles shall be determined from Figure E11.

**E13.2.4** *Radius of curvature factor ($k_r$)*

Values of $k_r$ for various radii and pitch angles shall be determined from the following equation:

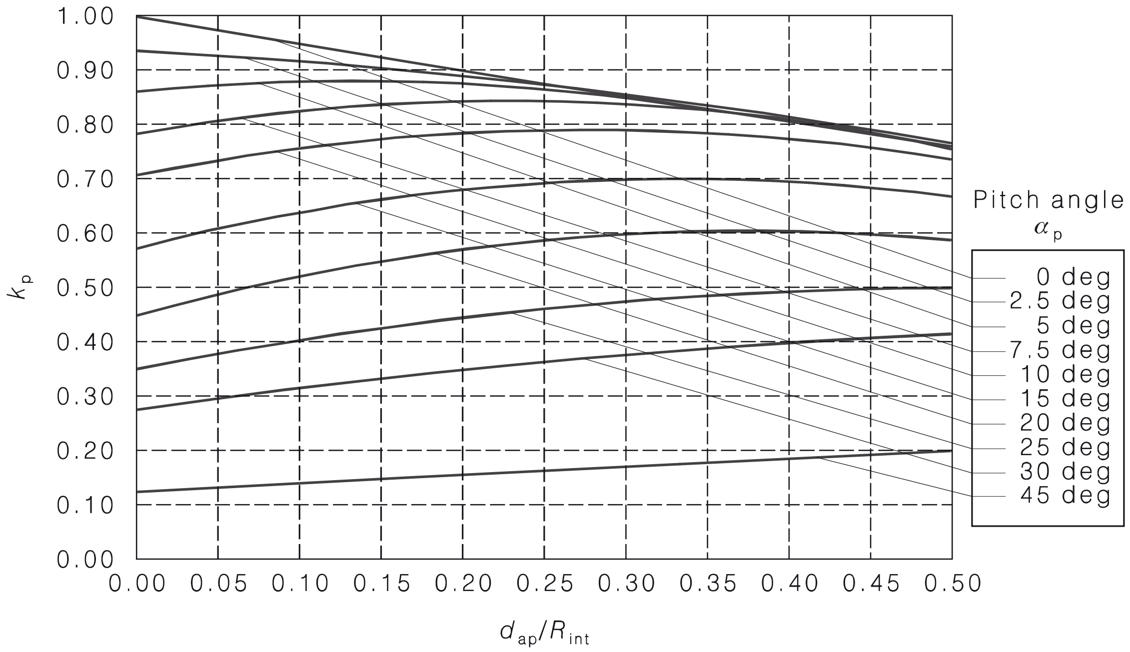
$$k_r = 0.76 + 0.001\left(\frac{R_{int}}{t}\right) \text{ for } 120 < \left(\frac{R_{int}}{t}\right) < 240 \quad \text{. . . E13(5)}$$

$$k_r = 1.0 \qquad \text{for } \left(\frac{R_{int}}{t}\right) > 240 \quad \text{. . . E13(6)}$$

where

$R_{int}$ = radius of bending at fabrication of the inside face of the beam at the critical section

$t$ = for glued laminated beams, the thickness of the innermost lamination; or
= for other members, the depth of the member at the critical section




NOTES:

1 For double-tapered beams, $k_{sh}$ is determined for the case where $\frac{d_{ap}}{R_{int}} = 0$.

2 For constant radius curved beams, $k_{sh}$ is determined for the case where $\alpha_p = 0$.

3 The relevant radius to be checked is that of the inner surface of the curved beams.

FIGURE E11 SHAPE FACTOR $k_{sh}$

**E13.2.5** *Volume/size factor ($k_v$)*

The value of $k_v$ applicable to design is given by the expressions in Table E10.

**TABLE E10**

**VOLUME/SIZE FACTOR ($k_v$)**
**FOR TENSION PERPENDICULAR TO GRAIN**

| Member type | Uniformly distributed loads | All other loads |
|---|---|---|
| Constant depth, curved | $\frac{24}{(A_{ap} R_{cl} \beta)^{0.2}}$ | $\frac{20}{(A_{ap} R_{cl} \beta)^{0.2}}$ |
| Double tapered, curved (including pitched cambered beams) | $\frac{35}{(A_{ap} R_{cl} \beta)^{0.2}}$ | $\frac{22}{(A_{ap} R_{cl} \beta)^{0.2}}$ |
| Double tapered, straight | $\frac{36}{(A_{ap} d_{ap})^{0.2}}$ | $\frac{23}{(A_{ap} d_{ap})^{0.2}}$ |

LEGEND:
$\beta$ = angle of sweep of the beam in radians (see Figure E10)
$A_{ap}$ = maximum cross-section at the apex = $d_{ap}\, b$
$d_{ap}$ = depth of the beam at the apex (see Figure E10)
$R_{cl}$ = centre-line radius of curvature of the beams at the critical section

**E13.2.6** *Radial stress factor ($k_{tp}$)*

Values of $k_{tp}$ for various centre-line radii and pitch angles shall be determined from Figure E12.

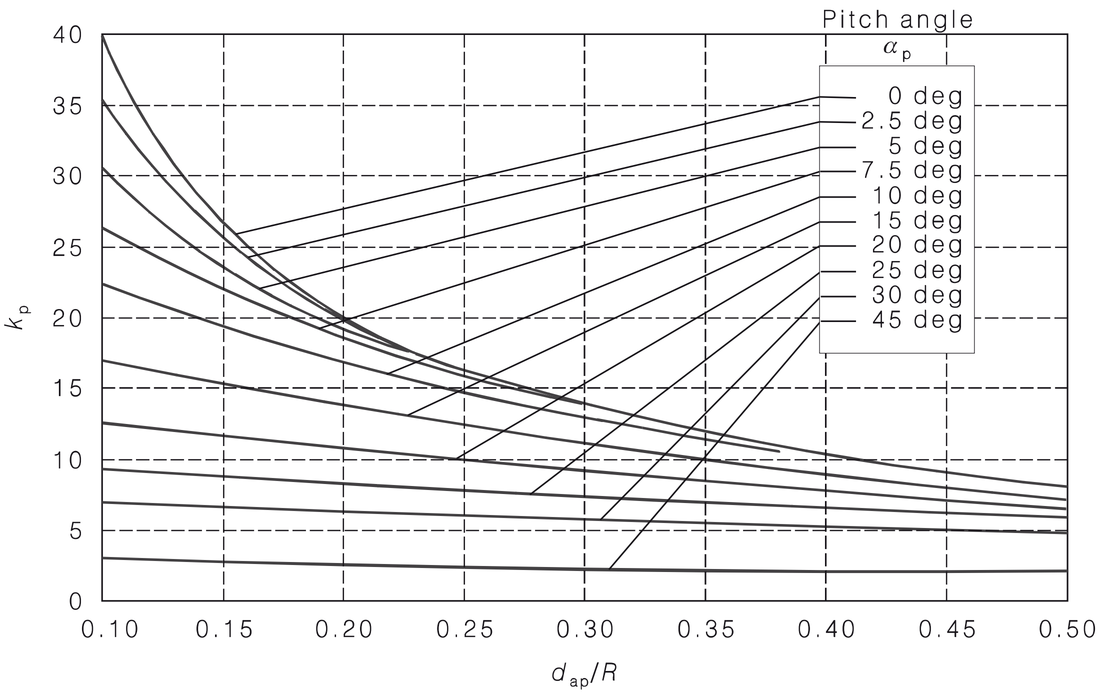



FIGURE E12 RADIAL STRESS FACTOR ($k_{tp}$)

# APPENDIX F

# NOTATION AND FACTORS

(Normative)

This Appendix sets out the notation used in this Standard, lists the modification factors for stiffness ($j$) (see Table F1) and for strength ($k$) (see Table F2). Figure F1 illustrates examples of dimensional symbols used.

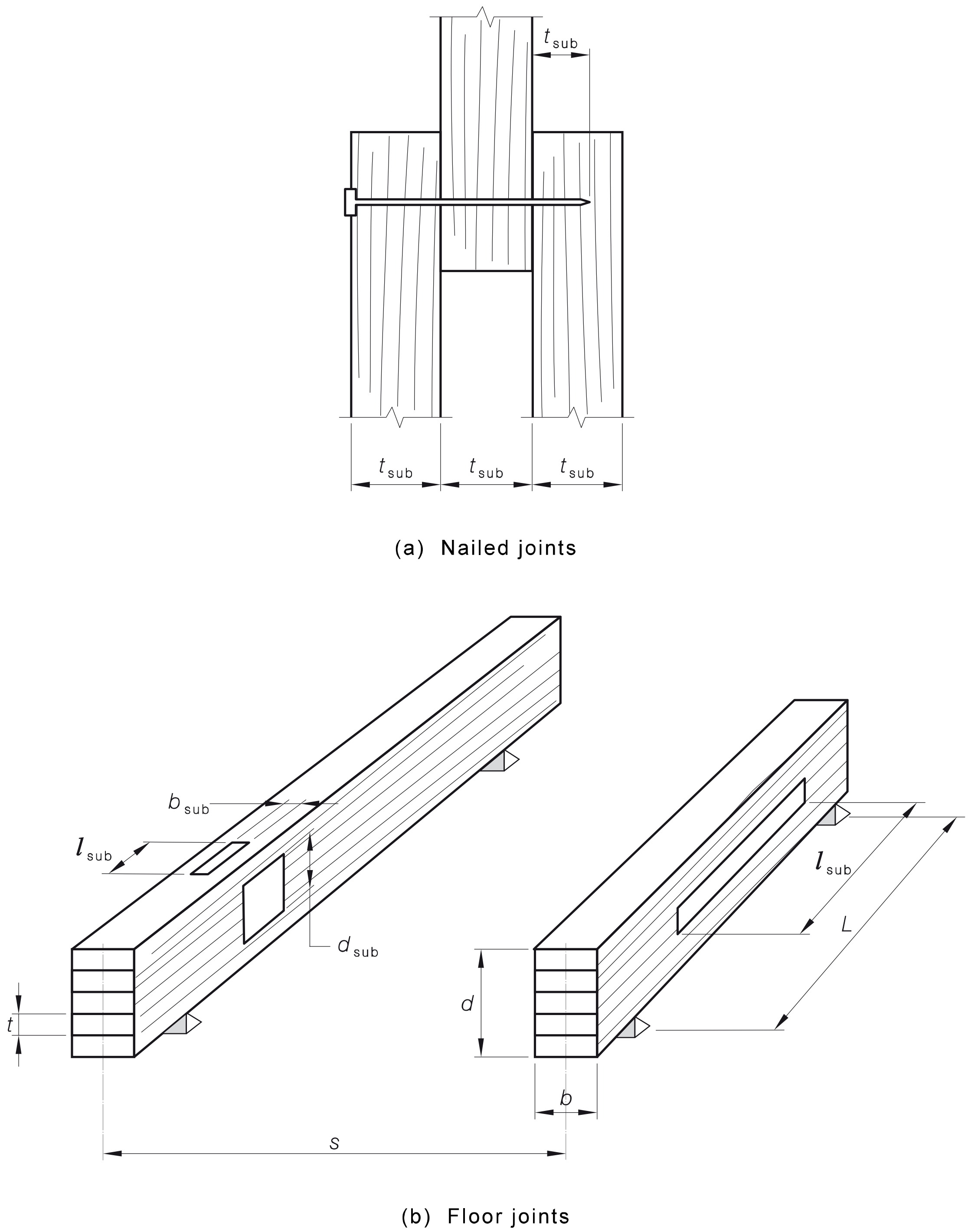



FIGURE F1 ILLUSTRATED EXAMPLES OF DIMENSIONAL SYMBOLS USED

Quantity symbols used in this Standard are listed below.

| | | |
|---|---|---|
| $A$ | = | area of net cross-section |
| $A_{ap}$ | = | cross-sectional area at the critical section of curved tapered beam (Paragraph E13.2 and Table E10, Appendix E) |
| $A_c$ | = | cross-sectional area of column (Clauses 3.3.1.1 and 6.3.3) |
| | = | effective cross-sectional area for plywood (Clause 5.5.5) |
| $A_\ell$ | = | bearing area for loading parallel to grain (Clause 3.2.6.2) |
| $A_p$ | = | bearing area for loading perpendicular to grain (Clause 3.2.6.1) |
| | = | bearing area for plywood (Clause 5.4.4) |
| $A_s$ | = | shear plane area (Clauses 3.2.5 and 6.3.2) |
| | = | interlamina shear area or punching shear area, for plywood (Clause 5.4.3) |
| | = | shear area in bending or localized shear area, for plywood (Clause 5.5.3) |
| $A_{sj}$ | = | transverse shear plane area at joint section (Clause 4.4.6) |
| | = | area at glued interface for plywood (Clause 5.6.2) |
| $A_t$ | = | net cross-sectional area of tension member (Clause3.4.1) |
| | = | effective cross-sectional area for plywood (Clause5.5.4) |
| $A_{tp}$ | = | member width by effective length stressed in tension for tension perpendicular to grain (Clause 3.4.2) |
| $A_w$ | = | effective area of washer for bearing (Clauses 4.4.3.3, 4.5.3.2 and Table 4.11) |
| $a$ | = | general dimension |
| $a_1, a_2, a_3, a_4$ | = | spacings, edge and end distances (Figure 4.11) |
| $b$ | = | breadth or horizontal dimension of the member (Figure F1, Appendix F) |
| | = | breath of veneer for plywood (Figures I4 and I5, Appendix I) |
| $b_{eff}$ | = | effective thickness (breadth) of member in joint assembly (Tables 4.9 and 4.10) |
| $D$ | = | diameter of a metal fastener (Section 4) |
| $d$ | = | depth or vertical dimension of the member (Figure F1, Appendix F) |
| $d_{ap}$ | = | depth of tapered beam at the deepest section usually at the apex (Figure E9, Appendix E) |
| $d_n$ | = | net depth of notched beam (Paragraph E9 and Figure E8, Appendix E) |
| $d_{notch}$ | = | notch depth (Figure E8, Appendix E) |
| $d_p$ | = | mid-length diameter of a pole (Clause 6.3, Table 6.2) |
| $d_s$ | = | distance from loaded edge (Clause 4.4.6, Figure 4.10; or |
| | = | depth of tapered beam at the smallest end (Paragraph E12 and Figure E9, Appendix E) |
| $d_w$ | = | depth of web of I-beam or box beam (Figure I3, Appendix I) |
| $d_1, d_2, d_3$ | = | thickness of plies (Figures I4 and I5, Appendix I) |
| $E$ | = | modulus of elasticity (Tables 5.1, 7.1, H2.1, Appendix H, and Paragraphs E2 and E7.5, Appendix E) |
| $(EA)$ | = | effective axial rigidity (Paragraph E3.1, Appendix E) |
| $E_B I_B$ | = | flexural rigidity of a single beam (Paragraph E8.2, Appendix E) |

$E_C I_C$ = flexural rigidity of a single crossing member (Paragraph E8.2, Appendix E)

$(EI)_x$ = rigidity in bending about the major axis, i.e., x-axis in Figure 3.1, (Paragraphs E3.2 and E6, Appendix E)

$(EI)_y$ = rigidity in bending about the minor axis, i.e., y-axis in Figure 3.1, (Paragraphs E3.2 and E6, Appendix E)

$EMC$ = equilibrium moisture content

$f'_b$ = characteristic value in bending (Tables 5.1, 7.1, H2.1 and H3.1, Appendix H, and Clause 3.2.1.1)

$f'_c$ = characteristic value in compression (Tables 5.1, 7.1, H2.1 and H3.1, Appendix H, and Clause 3.3.1)

$f'_{cj}$ = characteristic value for bolts in bearing parallel to the grain (Paragraph C4.1, Appendix C)

$f'_\ell$ = characteristic value in bearing parallel to grain (Tables H2.2 and H3.1, Appendix H, and Clause 3.2.6.2)

$f'_o$ = characteristic value of material (Clause 2.1.2)

$f'_p$ = characteristic value in bearing perpendicular to grain (Tables H2.2 and H3.1, Appendix H, Clause 3.2.6.1 and Paragraph E12.5, Appendix E); or
= for plywood, characteristic value in compression normal to the plane of the plywood panel (Clause 5.4.4)

$f'_{pj}$ = characteristic value for bolts bearing perpendicular to grain (Paragraph C4.2, Appendix C)

$f'_s$ = characteristic value in shear parallel to grain (Tables H2.1 and H3.1, Appendix H, and Clause 3.2.5); or
= for plywood, characteristic value in panel shear (in-plane shear) (Clause 5.3.5)

$f'_{sj}$ = characteristic value in shear at joint details appropriate to species strength group (Clause 4.4.6, Tables H2.2 and H3.1, Appendix H, and Paragraph E9, Appendix E)

$f'_t$ = characteristic value in tension parallel to grain (Tables 5.1, 7.1, H2.1, and H3.1, Appendix H, and Clause 3.4.1)

$f'_{tp}$ = characteristic value in tension perpendicular to the grain (Tables H2.2 and H3.1, Appendix H, Clause 3.4.2 and Paragraph E12.5, Appendix E)

$[f'_o X]$ = characteristic capacity (see Clause 2.1.2)

$G$ = permanent action (Table B1, Appendix B, and Table G1, Appendix G, see also AS/NZS 1170.0)
= modulus of rigidity (Tables 5.1, 7.1, and Tables H2.1 and H3.1, Appendix H)

$(GJ)$ = torsional rigidity (Paragraphs E3.2 and E6, Appendix E)

$g_{13}$ = modification factor for effective length of columns (Clause 3.3.2.2)

$g_{19}$ = modification factor for plywood assembly (Clause 5.4.4)

$g_{28}$ = modification factor for effective length of spaced columns (Paragraph E4 and Table E5, Appendix E)

$g_{31}$, $g_{32}$ = geometric factors for calculating $k_9$ (Table 2.7)

$g_{38}$ = modification factor for buckling restraints (Paragraph E7.2, Appendix E)

$g_{40}$ = modification factor for notched beams (Paragraph E9, Appendix E)

$g_{41}$, $g_{42}$ = modification factors for concentrated loads on grid systems (Paragraph E8, Appendix E)

$g_{43}$ = modification factor for partial area load on a grid system (Paragraph E8, Appendix E)

$g_{60}$ = slenderness coefficient for plywood diaphragms (Paragraph I2.2, Appendix I)

$g_{61}$ = slenderness coefficient for plywood diaphragms (Paragraph I2.3, Appendix I)

$g_{62}$ = modification factor for effective buckling width of plywood for concentrated loads (Paragraph I2.4.2, Appendix I)

$h_1$ = moment factors (Paragraph E6.2.2, Appendix E)

$h_2$ = moment factor of bisymmetrical beams (Paragraph E6.3, Appendix E)

$h_3$ = moment factor of bisymmetrical beams (Paragraph E6.3, Appendix E)

$h_{26}$ = stiffness factor for buckling for loading duration (Paragraph E7.2, Appendix E)

$h_{27}$ = stiffness factor of buckling for materials type (Paragraph E7.2, Appendix E)

$h_{32}$ = stiffness factor given in Table C1 (Paragraph C3.2, Appendix C)

$h_{33}$ = stiffness factor given in Table C4 (Paragraph C3.3.2, Appendix C)

$h_{35}$ = stiffness factor (Paragraph C3.3.3, Appendix C)

$h_B$, $h_C$ = $E_B I_B / L^3$, $E_C I_C / s^3$ (Paragraph E8.2, Appendix E)

$I$ = second moment of area (moment of inertia) (Paragraph E4.4.1, Appendix E)

$I_C$ = second moment of area about the neutral axis for bending capacity calculations (Paragraphs I3.3.1(b) and I3.3.2(b))

$I_p$ = second moment of area of parallel plies whose grain direction is parallel to the span (Clause 5.4.2)

$I_R$ = second moment of area about the neutral axis for bending rigidity calculations (Paragraphs I3.3.1(a) and I3.3.2(a) , Appendix I)

$J$ = St Venant torsion constant (see Note to Paragraph E6.2.2, Appendix E)

$j$ = stiffness modification factors (Table F1, Appendix F)

$K_A$, $K_B$ = stiffness coefficients (Paragraph E7, Appendix E)

$K_{sec}$ = secant modulus of a joint (Paragraph C3.1, Appendix C)

$k_{mod}$ = product of the relevant modification factors (Clause 2.1 and Table F2, Appendix F)

$L$ = length of column or span of beam

$L_{a\phi}$ = distance between points of effectively rigid rotational restraints (Clause 3.2.3.2, Paragraphs E3.2 and E6.4, Appendix E)

$L_{ax}$ = distance between points of effectively rigid restraints against lateral movement in the direction of the y-axis (Clause 3.3.2.2)

$L_{ay}$ = distance between points of effectively rigid restraints against lateral movement in the direction of the x-axis (Clause 3.2.3.2, Paragraphs E4.3.1, E6.2.2 and E6.3, Appendix E)

$L_s$ = centre-to-centre distance of packing pieces (Figure E2 and Paragraph E4.3, Appendix E)

$l_b$ = length of bearing (Figure I3, Appendix I)

$L_{ch}$ = characteristic length for buckling of plywood webs (Paragraph I2.2.1, Appendix I)

$l_{eff}$ = effective width in compression (Figure I3, Appendix I)

$l_{notch}$ = notch length (Figure E8, Appendix E)

$l_p$ = depth of nail or screw penetration into supporting member (Figure 4.4(a), Clauses 4.2.3.4 and 4.5.3.2)

$l_{par}$ = end distance (Clause 4.4.4.2 and Figure 4.9)

$M^*$ = design action effect in bending produced by strength limit states design loads about the axis of bending (Clauses 3.2, 3.5.2, 4.2.3.3, Paragraphs E9, E10, E12 and E13, Appendix E)

$M_d$ = design capacity of a member in bending (Clauses 3.2, 3.5.2 and 6.3.1, Paragraphs E12 and E13, Appendix E

$M_a^*$ = design action effect produced by strength limit states design loads as an applied moment (Paragraphs E7.4 and E7.5, Appendix E)

$M_{bat}^*$ = moment on batten plate (Paragraph E4.3.3, Appendix E)

$M_{cr}$ = critical elastic buckling moment (Paragraph E6, Appendix E)

*MGP* = a stress grade

$M_i^*$ = design action effect for edgewise bending of the plywood panel (Figure 5.2 and Clause 5.5.2)

$M_{d,i}$ = design capacity in bending of plywood (Clause 5.5.2)

$M_{d,j}$ = design capacity for joints under bending (Clauses 4.2.3.3 and 4.3.3.3)

$M_p^*$ = design action effect in bending produced by strength limit states design loads acting normal to the plane of the plywood (flatwise bending) (Clause 5.4.2)

$M_{d,p}$ = design capacity in bending of plywood (Clause 5.4.2)

$M_x$ = bending moment, about x-axis

$M_x^*$ = design action effect in bending about the major principal x-axis, produced by strength limit states design loads (Clauses 3.2.1.2, 3.5.1 and Paragraph E5, Appendix E)

$M_{d,x}$ = design capacity in bending about the major principal x-axis (Clauses 3.2.1.2, 3.5.1, 3.5.2 and Paragraph E5, Appendix E)

$M_y^*$ = design action effect in bending about the minor principal y-axis, produced by strength limit states design loads (Clause 3.2.1.2 and Paragraph E5, Appendix E)

$M_{d,y}$ = design capacity in bending about the minor principal y-axis (Clause 3.2.1.2 and Paragraph E5, Appendix E)

$m$ = number of members supported by restraint system (Paragraph E7.2, Appendix E)

$N^*$ = design action effect produced by strength limit state design loads (Clause 6.3.3, Paragraph E10, Appendix E, and Section 4)

$N_c^*$ = design action effect produced by strength limit states design loads acting in compression (Clauses 3.3.1.1, 3.5.1, and 5.5.5, and Paragraphs E5 and E7.3, Appendix E)

$N_{d,c}$ = design capacity of a member in compression (Clauses 3.3.1.1, 5.5.5, 6.3.3)

$N_{cr}$ = critical elastic buckling load (Paragraph E3.1, Appendix E)

$N_{d,cx}$ = design capacity in compression for buckling about the major x-axis (Clause 3.3.1.2 and Paragraph E5, Appendix E)

$N_{d,cy}$ = design capacity in compression for buckling about the minor y-axis (Clause 3.3.1.2 and Paragraph E5, Appendix E)

$N_{dj}$ = design capacity for joints under direct load (Section 4)

$N_{\ell}^*$ = design action effect produced by strength limit states design loads acting in bearing parallel to the grain (Clause 3.2.6.2)

$N_{d,\ell}$ = design capacity in bearing parallel to grain (Clause 3.2.6.2)

$N_p^*$ = design action effect in bearing produced by strength limit states design loads acting normal to the plane of the plywood (Figure 5.1, Clause 5.4.4)

$N_{d,p}$ = design capacity in bearing perpendicular to grain (Clauses 3.2.6.1, 5.4.4)

$N_R^*$ = force generated in a restraint under a displacement (Paragraph E7, Appendix E)

$N_t^*$ = design action effect produced by strength limit states design loads acting in tension (Clauses 3.4.1, 3.5.2, 5.5.4)

$N_{d,t}$ = design capacity of a member in tension (Clauses 3.4.1, 3.5.2)

$N_{d,tb}$ = design capacity in tension for a bolt (Clause 4.4.3.3)

$N_{d,tc}$ = maximum capacity in tension of a simple coach screw (Table 4.14, Clauses 4.5.2.3 and 4.5.3.2)

$N_{tp}^*$ = design action effect produced by strength limit states design loads acting in tension perpendicular to grain (Clause 3.4.2)

$N_{d,tp}$ = design capacity in tension perpendicular to grain (Clause 3.4.2)

$N_{d,\theta}$ = design bearing capacity at an angle to the grain of wood (Clause 3.2.6.3)

$n$ = number of items, e.g., screws, nails, lateral restraints etc. (Table 4.3(B))

$n_a$ = number of fasteners or rows of fasteners (Tables 4.3(A) and 4.12)

$n_C$ = number of crossing members in a grid system (Paragraph E8.2, Appendix E)

$n_{com}$ = number of elements (Clause 2.4.5.2)

$n_{mem}$ = total number of members in a load-sharing system (Clause 2.4.5.2))

$P^*$ = design point action effect produced by strength limit states design loads (Paragraph E8.2, Appendix E)

$P_{eff}^*$ = effective point load on a grid system (Paragraph E8.2, Appendix E)

| | | |
|---|---|---|
| $Q$ | = | imposed action (see AS/NZS 1170.0, Tables B1, Appendix B, and G1, Appendix G) |
| $Q^*$ | = | design action effect (Appendices C and D) |
| | = | design action effect per nail or screw (Paragraph C3, Appendix C) |
| $Q_D^*$ | = | long duration design action effect (Paragraph C3.1, Appendix C) |
| $Q_E$ | = | equivalent total test load (Appendix D) |
| $Q_L^*$ | = | short duration design action effect (Paragraph C3.1, Appendix C) |
| $Q_k$ | = | characteristic capacity related to the type of fastener (Clauses 4.2.3, 4.3.3 and Paragraph C3.3.2, Appendix C) |
| $Q_{kl}$ | = | characteristic capacity for a laterally loaded single bolt bearing parallel to the grain (Paragraph C4.1, Appendix C and Table 4.9) |
| $Q_{kp}$ | = | characteristic capacity for a laterally loaded single bolt bearing perpendicular to the grain (Paragraph C4.2, Appendix C and Table 4.10) |
| $Q_{sk}$ | = | characteristic capacity for a laterally loaded single bolt in a joint system (Clauses 4.4.2.4, 4.4.3.2 and 4.5.3) |
| $Q_{skl}$ | = | system capacity for systems loaded parallel to the grain (Clause 4.4.2.4) |
| $Q_{skp}$ | = | system capacity for systems loaded perpendicular to the grain (Clause 4.4.2.4) |
| $R_{cl}$ | = | centre-line radius of curvature for curved and pitched cambered beams (Paragraph E13.2.1 and Table E10, Appendix E) |
| $R_{int}$ | = | internal radius of curvature for curved and pitched cambered beams (Figure E10 and Paragraph E13.2.4, Appendix E) |
| $R_d$ | = | design capacity (Clauses 1.4.2.2, 2.1.2 and 2.1.4) |
| $r$ | = | ratio of design action effect for actions of duration less than or equal to 12 months, divided by the total action effect (Paragraph E2, Appendix E) |
| $r_i$ | = | distance to *i*-th nail (Clauses 4.2.3.3 and 4.3.3.3) |
| $r_{max.}$ | = | distance to the farthest nail (Clauses 4.2.3.3 and 4.3.3.3) |
| $S$ | = | slenderness coefficient |
| $S^*$ | = | design action effect, such as bending moment, axial force or shear force resulting from the combinations of actions for the strength limit states (Clauses 1.4.2.2, Appendix I) |
| $S_1$ | = | slenderness coefficient, major axis (Clause 3.2.3.2 and Paragraph E6.1, Appendix E) |
| $S_2$ | = | slenderness coefficient, minor axis (Clause 3.2.3.2(c)) |
| $S_3$ | = | slenderness coefficient about major axis for discrete restraint systems (Clause 3.3.2.2(a)) |
| $S_4$ | = | slenderness coefficient about minor axis for discrete restraint systems (Clause 3.3.2.2(b), Paragraph E7.3, Appendix E) |
| $S_5$ | = | slenderness coefficient for an individual shaft about the y-axis (Paragraph E4.4.1, Appendix E) |
| $s$ | = | centre-to-centre spacing of supporting members in a grid system (Paragraph E8.2, Appendix E) |

$T_R^*$ = torque generated in a restraint under a rotation (Paragraph E7.5, Appendix E)

$t$ = thickness of member (Figure F1, Appendix F), or thickness of plywood (Figures I4 and I5, Appendix I)
= depth of the critical section (Paragraph E13.2.4, Appendix E)
= thickness of the innermost lamination for glued laminated beams (Paragraph E13.2.4, Appendix E)

$t_1$ = thickness of first member of a two-member joint (Figure 4.6, and Clauses 4.2.5, 4.3.5)

$t_1$, $t_2$, $t_3$, $t_4$ = thickness of timber in bolted joint (Tables 4.9(A) and 4.10(A))

$t_f$ = flange thickness of I-beam or box beam (Figure I3, Appendix I)

$t_m$ = thickness of member of a nailed joint (Figure 4.6 and Clause 4.2.5)

$t_o$ = thickness of member of a nailed joint (Clauses 4.2.5, 4.3.5, and Figures 4.6 and C1, Appendix C)

$t_p$ = depth of nail penetration into second member of a two-member joint or third member of a three-member joint (Clauses 4.2.5, 4.3.5, 4.5.2.1 and Figures 4.6 and C1, Appendix C)

$t_{pi}$ = thickness of an inner ply of plywood (Tables I1 and I2, Appendix I)

$t_{pl}$ = sum of thicknesses of plies with grain parallel to span (Clause 5.5.2)

$t_{po}$ = thickness of outermost ply of plywood (Tables I1 and I2, Appendix I)

$t_s$ = thickness of spaced column member (Figure E2 and Paragraph E4, Appendix E)

$t_w$ = thickness of timber member (Figure C1, Appendix C)
= thickness of web of I-beam or box beam (Paragraph I2, Appendix I)

$V$ = nominal capacity of a member in shear

$V^*$ = design action effect in shear produced by strength limit states design loads (Clauses 3.2.5 and 6.3.2 and Paragraphs E4.3 and E9, Appendix E)

$V_{bat}^*$ = longitudinal shear force on batten plate (Paragraph E4.3.3, Appendix E)

$V_d$ = design capacity of a member in shear (Clauses 3.2.5, 6.3.2)

$V_{d,f}$ = fastener capacity in shear (Clause 4.7.3)

$V_{d,fN}$ = bolt capacity in shear, threads included (Table 4.20)

$V_{d,fX}$ = bolt capacity in shear, threads excluded (Table 4.20)

$V_i^*$ = design action effect for shear in the plane of the plywood panel (Figure 5.2 and Clause 5.5.3)

$V_{d,i}$ = design capacity of plywood in shear (Clause 5.5.3)

$V_p^*$ = design action effect for shear normal to the face of the plywood panel produced by strength limit states design loads (Clause 5.4.3, Figure 5.1)

$V_{d,p}$ = design capacity in planar shear of plywood (Clause 5.4.3)

$V_{pack}^*$ = shear action effect on packing pieces (Paragraph E4.3.2)

| | | |
|---|---|---|
| $V_{sj}^*$ | = | design action effect in shear on the joint (Clause 4.4.6) |
| | = | design action effect in shear at the glued interface of the plywood and timber sections (Clause 5.6.2) |
| $V_{d,sj}$ | = | design capacity in transverse shear at eccentric joint (Clause 4.4.6) |
| | = | design capacity of a glued interface in shear between a plywood section and timber section (Clause 5.6.2) |
| $V_1^*$ | = | shear action effect produced by strength limit states design loads (Paragraph E4.3.1, Appendix E) |
| $V_2^*$ | = | shear action effect due to curvature of the column (Paragraph E4.3.1, Appendix E) |
| $w^*$ | = | load, uniformly distributed over a partial area or length (Paragraph E8.3, and Figure E6, Appendix E) |
| $w_{eff}^*$ | = | effective load, uniformly distributed over a partial area or length (Paragraph E8.3, Appendix E) |
| $X$ | = | geometric section property, e.g., section modulus $Z$ (Clause 2.1.2) |
| $x$ | = | cartesian coordinate or distance (Figures 3.1, E2, E3 and E5, Appendix E) |
| $y$ | = | cartesian coordinate (Figures 3.1, E2, E3 and E5, Appendix E) |
| $y_1, y_2$ | = | distance from the neutral axis of a plywood panel to the extreme fibre of each ply (Figures I4 and I5, Appendix I) |
| $\bar{y}_1, \bar{y}_2$ | = | distance from the neutral axis of a plywood panel to the neutral axis of each individual veneer (Figure I4, Appendix I) |
| $y_e$ | = | distance from column centroid to point of load application (Figure E1 and Paragraph E3.2, Appendix E) |
| $y_h$ | = | height above beam centroid of the point of load application (Figure E4 and Paragraphs E6.3 and E6.4, Appendix E) |
| $y_{max.}$ | = | distance from the neutral axis to the extreme fibre (Paragraph E6.1, Appendix E) |
| $y_o$ | = | distance of lateral restraint below the neutral axis (Figures E1 and E4, Appendix E) |
| $y_p$ | = | distance from the neutral axis to the extreme fibre of the outermost parallel ply |
| $z$ | = | cartesian coordinate, in direction of the longitudinal or neutral axis of members (Figure E3, Appendix E) |
| | = | distance along the longitudinal axis of a member (Paragraph E12.3, Appendix E) |
| $Z$ | = | section modulus about the axis of bending, $mm^4$ (Clauses 3.2.1.1 and 3.5.2) |
| $Z_i$ | = | section modulus of plywood in plane (Clause 5.5.2) |
| $Z_p$ | = | section modulus of plywood = $I_p/y_p$ (Clause 5.4.2) |
| $Z_x$ | = | section modulus about x-axis (Clause 3.2, Paragraph E7.5, Appendix E) |
| $Z_y$ | = | section modulus about y-axis (Clause 3.2) |
| $\alpha$ | = | parameter related to tendency to split (Clause 4.1.5); or |
| | = | angle of taper of double tapered beams at the apex (Figures E9 and E10, Appendix E) |

$\alpha_1, \alpha_2, \alpha_3$ = parameters defined in Paragraph E7, Appendix E

$\alpha_c$ = angle of taper in tapered beams at the compression edge (Paragraph E12.5, Appendix E)

$\alpha_p$ = pitch angle of tapered beams at the apex (Figure E9, Appendix E)

$\alpha_t$ = angle of taper in tapered beams at the tension edge (Paragraph E12.5, Appendix E)

$\beta$ = moment diagram parameter (Figure E3(c), Appendix E); or
= angle (radians) of apex zone of curved and pitch-cambered beams (Figure E10, Table E10, Appendix E)

$\gamma$ = tangential cleavage, in newtons per millimetre (Clause 4.1.5)

$\Delta$ = deflection or joint displacement

$\Delta_A$ = displacement resulting in a resisting force (Paragraph E7.1, Appendix E)

$\Delta_I$ = initial displacement of fastener (Paragraph C3.3.3)

$\Delta_o$ = deformation of the joint including long duration effects (Paragraph C3.1, Appendix C)

$\varepsilon$ = tangential shrinkage,% (Clause 4.1.5)

$\theta$ = angle between the direction of the load and the direction of the grain (Clause 4.4.2.4(c))
= direction of face grain of plywood, as defined in Figures I1, I2 and I3, Appendix I

$\theta_B$ = torsional rotation (Paragraph E7.1, Appendix E)

$\rho_b, \rho_c$ = material constants for beams and columns, respectively (Clauses 3.2, 3.3, 7.4.4, 8.4.7 and Paragraphs E2, Appendix E)

$\phi$ = capacity factor (see Clause 2.3)

$\psi$ = load factor (Table B1, Appendix B, and Table G1, Appendix G, see also AS/NZS 1170.0)

## TABLE F1

## MODIFICATION FACTORS FOR STIFFNESS (*j*)

| Factor | Definition | Text reference |
|---|---|---|
| | | Section 2 |
| $j_2$ | Factor for duration of load for bending, compression and shear | 2.4.1.2 |
| $j_3$ | Factor for duration of load for tension | 2.4.1.2 |
| | | Section 5 |
| $j_6$ | Factor for moisture content for plywood and LVL | 5.3.3<br>Section 8<br>8.4.3 |
| | | Section 6 |
| $j_9$ | Factor for immaturity of round timbers | 6.4.1 |
| | | Appendix C |
| $j_{12}$ | Factor for duration of load on nailed and screwed joints | C3.2 |
| $j_{13}$ | Factor for duration of load on nailed and screwed joints | C3.2 |
| $j_{14}$ | Factor for duration of load on split-ring and shear-plate fasteners | C3.3 |

## TABLE F2

## MODIFICATION FACTORS FOR STRENGTH ($k$)

| Factor | Definition | Text reference |
|---|---|---|
| | | Section 2 |
| $k_1$ | Factor for load duration (see Appendix G) | 2.4.1.1 |
| $k_4$ | Factor for in-service absorption or desorption of moisture by timber | 2.4.2 |
| $k_6$ | Factor for temperature/humidity effect | 2.4.3 |
| $k_7$ | Factor for bearing length | 2.4.4 |
| $k_9$ | Factor for load sharing in grid systems | 2.4.5 |
| $k_{11}$ | Factor for effect of volume, in tension perpendicular to grain | 3.4.2 |
| $k_{12}$ | Factor for stability | 2.4.6 |
| | | Section 4 |
| $k_{13}$ | Factor for end grain effects | 4.2.3, 4.3.3, 4.5.3 |
| $k_{14}$ | Factor for effect of double shear | 4.2.3 |
| $k_{15}$ | Factor for effect of seasoning of timber | 4.6.3, 4.7.3 |
| $k_{16}$ | Factor for plywood or metal side plates | 4.2.3, 4.3.3, 4.4.3, 4.5.3 |
| $k_{17}$ | Factor for multiple fastener effect | 4.2.3, 4.3.3, 4.4.3, 4.5.3, 4.6.3, 4.7.3 |
| $k_{18}$ | Factor for effect of tension loads | 4.6.3, 4.7.3 |
| | | Section 5 |
| $k_{19}$ | Factor for moisture content of plywood | 5.3.3 |
| | | Section 6 |
| $k_{20}$ | Factor for timber immaturity | 6.4.1 |
| $k_{21}$ | Factor for effect of shaving | 6.4.2 |
| | | Appendix D |
| $k_{26}$ | Factor related to design load duration | D4.1 |
| $k_{27}$ | Factor for duration of test | D4.1 |
| $k_2$ | Factor for prototype testing | D5.4 |
| $k_{28}$ | Factor for effect of sample size and coefficient of variation | D5.4 |
| | | Appendix E |
| $k_{tg}$ | Factor for grain orientation for single-tapered straight beams | E12.2, E12.4 |
| $k_{tb}$ | Factor for taper angle for single-tapered straight beams | E12.2, E12.5 |
| $k_{sh}$ | Factor for depth and curvature of curved and pitched cambered beams | E13.2.3 |
| $k_r$ | Factor for radius of curvature | E13.2.4 |
| $k_v$ | Factor for volume/size | E13.2.5 |
| $k_{tp}$ | Factor for radial stress | E13.2.6 |

# APPENDIX G

# MISCELLANEOUS DESIGN INFORMATION

(Informative)

## G1 ADDITIONAL INFORMATION

Basic design properties and additional design information on roughly 40 species commonly used in Australia are given in AS/NZS 1720.2.

NOTE: Design information, including strength grouping, on hundreds of other species from Australia and overseas can be found in AS 2878 and in the following publications:

BERNI, C., BOLZA, E. and CHRISTENSEN, F.J. *South American timber—The characteristics, properties and uses of 190 species*. CSIRO Division of Building Research, Melbourne, 1979.

BOLZA, E. and KEATING, *W. African timbers—The properties, uses and characteristics of 700 species.* CSIRO Division of Building Research, Melbourne, 1972.

BOLZA, E. and KLOOT, N.H. *The mechanical properties of 174 Australian Timbers.* Technological Paper No. 25. CSIRO Division of Forest Products, 1963.

KEATING, W.G. and BOLZA, E. *Timbers of commerce*, Vol. 1. South East Asia, Northern Australia and Pacific. Incata Press, Melbourne, 1982.

## G2 EXAMPLES OF LOAD DURATION FACTORS FOR TYPICAL LOAD COMBINATIONS

Table G1 gives examples of load duration factor $k_1$ for load combinations given in AS/NZS 1170.0.

### TABLE G1

### LOAD DURATION FACTORS FOR TYPICAL LOAD COMBINATIONS FOR STRENGTH LIMIT STATE

| Type of load (action) | AS/NZS 1170.0 specified load combination* | Load duration factor | |
|---|---|---|---|
| | | Solid timber | Joints |
| Permanent action (dead load) | $1.35\ G$ | 0.57 | 0.57 |
| Permanent and short term imposed actions | | | |
| (a) Roof live load—Distributed | $1.2\ G + 1.5\ Q$ | 0.94 | 0.77 |
| (b) Roof live load—Concentrated | | 0.97 | 0.86 |
| (c) Floor live loads— Distributed | | 0.80 | 0.69 |
| (d) Floor live loads— Concentrated | | 0.94 | 0.77 |
| Permanent and long-term† imposed action | $1.2\ G + 1.5\ \psi_l Q$ | 0.57 | 0.57 |
| Permanent, wind and imposed action | $1.2\ G + W_u + \psi_c Q$ | 1.00 | 1.14 |
| Permanent and wind action reversal | $0.9\ G + W_u$ | 1.00 | 1.14 |
| Permanent, earthquake and imposed action | $G + E_u + \psi_c Q$ | 1.00 | 1.14 |
| Fire | $G + \psi_\ell\ Q$ | 0.94 | 0.77 |

* The notation used in this Table is drawn from AS/NZS 1170.0.

† Long-term in this context is the terminology in AS/NZS 1170.0 for the quasi-permanent component of imposed action.

# APPENDIX H

# DESIGN PROPERTIES FOR STRUCTURAL GRADED TIMBER

(Normative)

## H1 GENERAL

The design properties of structural timber are given for the standardized stress grades of graded sawn timber. For each stress grade, the determination of some design properties involves consideration of the species or species mixtures and the moisture condition (seasoned or unseasoned).

## H2 DESIGN PROPERTIES FOR F-GRADES

### H2.1 General

F-grades are applicable for both seasoned and unseasoned hardwood and softwood timber. The characteristic values for design for bending and shear in beams and tension and compression parallel to grain relative to F-grade are listed in Table H2.1.

Other design properties for F-graded timber are determined on the basis of species and moisture condition and are listed in Table H2.2.

### H2.2 Characteristic values for bending and shear of beams, tension and compression and elastic moduli parallel to the grain

Characteristic values for bending, beam shear, tension and compression and elastic moduli parallel to the grain are given in Table H2.1.

## TABLE H2.1

## CHARACTERISTIC VALUES FOR DESIGN—F-GRADES—BENDING AND SHEAR FOR BEAMS, TENSION, COMPRESSION AND ELASTIC MODULI PARALLEL TO GRAIN

| Stress grade | Characteristic values, MPa | | | | | | |
|---|---|---|---|---|---|---|---|
| | Bending | Tension parallel to grain | | Shear in beam | Compression parallel to grain | Short duration average modulus of elasticity* parallel to the grain, MPa | Short duration average modulus of rigidity, MPa |
| | | Hardwood | Softwood | | | | |
| | ($f'_b$) | ($f'_t$) | | ($f'_s$) | ($f'_c$) | (*E*) | (*G*) |
| F34 | 84 | 51 | 42 | 6.1 | 63 | 21 500 | 1 430 |
| F27 | 67 | 42 | 34 | 5.1 | 51 | 18 500 | 1 230 |
| F22 | 55 | 34 | 29 | 4.2 | 42 | 16 000 | 1 070 |
| F17 | 42 | 25 | 22 | 3.6 | 34 | 14 000 | 930 |
| F14 | 36 | 22 | 19 | 3.3 | 27 | 12 000 | 800 |
| F11 | 31 | 18 | 15 | 2.8 | 22 | 10 500 | 700 |
| F8 | 22 | 13 | 12 | 2.2 | 18 | 9 100 | 610 |
| F7 | 18 | 11 | 8.9 | 1.9 | 13 | 7 900 | 530 |
| F5 | 14 | 9 | 7.3 | 1.6 | 11 | 6 900 | 460 |
| F4 | 12 | 7 | 5.8 | 1.3 | 8.6 | 6 100 | 410 |

NOTES:

A1 1 The characteristic values for bending listed in this Table for F-grades apply for naturally round timbers and for rectangular beam sections not greater than 300 mm in depth. For rectangular beams greater than 300 mm in depth, the characteristic values are obtained by multiplying the value in this Table by $(300/d)^{0.167}$, where *d* is the depth of the section.

2 The characteristic values for tension listed in this Table for F-grades apply for tension members with width not greater than 150 mm. For tension members with a cross-sectional dimension greater than 150 mm, the characteristic values are obtained by multiplying the value in this Table by $(150/d)^{0.167}$, where *d* is the width or largest dimension of the cross-section.

3 The modulus of elasticity (*E*) is an average value and includes an allowance of about 5 percent for shear deformation. For estimating a fifth percentile value, expressions are given in Appendix B.

### H2.3 Characteristic values for bearing, shear at joint details, tension perpendicular to the grain and design density

The characteristic values for bearing, shear at joint details, tension perpendicular to the grain and design density for F-graded timber are determined on the basis of the strength group classification for the timber species or species mixture and moisture condition. The characteristic values for design properties related to strength group are listed in Table H2.2.

**TABLE H2.2**

**CHARACTERISTIC VALUES FOR DESIGN RELATED TO STRENGTH GROUP**

| Strength Group | | Characteristic values, MPa | | | |
|---|---|---|---|---|---|
| | | Bearing | | Shear at joint details | Tension perpendicular to grain |
| Unseasoned | Seasoned | Perpendicular to grain ($f'_p$) | Parallel to grain ($f'_\ell$) | ($f'_{sj}$) | ($f'_{tp}$) |
| | SD1 | 26 | 76 | 10 | 0.8 |
| | SD2 | 23 | 67 | 8.4 | 0.8 |
| | SD3 | 19 | 59 | 7.3 | 0.6 |
| | SD4 | 17 | 51 | 6.1 | 0.6 |
| | SD5 | 13 | 40 | 5.4 | 0.5 |
| | SD6 | 10 | 30 | 4.2 | 0.5 |
| | SD7 | 8.6 | 23 | 3.8 | 0.4 |
| | SD8 | 6.8 | 20 | 3.3 | 0.4 |
| S1 | | 17 | 51 | 6.1 | 0.8 |
| S2 | | 13 | 40 | 5.4 | 0.8 |
| S3 | | 10 | 30 | 4.2 | 0.6 |
| S4 | | 8.6 | 23 | 3.8 | 0.6 |
| S5 | | 6.8 | 20 | 3.3 | 0.5 |
| S6 | | 5.5 | 17 | 2.8 | 0.5 |
| S7 | | 4.4 | 13 | 2.2 | 0.4 |

## H2.4 Strength group and joint group classifications for seasoned and unseasoned, hardwood and softwood timber

The strength group and joint group classifications and design densities for seasoned and unseasoned timber of commonly encountered hardwood and softwood species are listed in Table H2.3 and Table H2.4.

### TABLE H2.3

### STRENGTH GROUP AND JOINT GROUP CLASSIFICATIONS AND DESIGN DENSITIES FOR SOME COMMON HARDWOOD SPECIES

| Species | Moisture condition | Strength group (see Note 1) | Joint group | Design density (see Note 2) kg/m$^3$ |
|---|---|---|---|---|
| Mixed Australian hardwoods (excluding rainforest species) from S.A. and southern N.S.W. | Unseasoned | S4 | J3 | 1 050 |
| | Seasoned | SD4 | JD3 | 650 |
| Ash-type eucalypts from N.S.W. Highlands, Victoria and Tasmania | Unseasoned | S4 | J3 | 1 050 |
| | Seasoned | SD4 | JD3 | 650 |
| Non-ash-type eucalypts and corymbias from Qld and N.S.W. | Unseasoned | S3 | J2 | 1 150 |
| | Seasoned | SD3 | JD2 | 750 |
| Rainforest species | Unseasoned | S7 | J4 | 800 |
| | Seasoned | SD7 | JD4 | 500 |
| Ash, alpine | Unseasoned | S4 | J3 | 1 050 |
| | Seasoned | SD4 | JD3 | 650 |
| Ash, mountain | Unseasoned | S4 | J3 | 1 050 |
| | Seasoned | SD3 | JD3 | 650 |
| Ash, silver-top | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD3 | JD2 | 850 |
| Balau | Unseasoned | S2 | J2 | 1 150 |
| | Seasoned | SD3 | JD2 | 900 |
| Blackbutt | Unseasoned | S2 | J2 | 1 150 |
| | Seasoned | SD2 | JD2 | 900 |
| Box, brush | Unseasoned | S3 | J2 | 1 150 |
| | Seasoned | SD3 | JD2 | 900 |
| Box, grey, coast | Unseasoned | S1 | J1 | 1 200 |
| | Seasoned | SD1 | JD1 | 1 100 |
| Brown barrel | Unseasoned | S4 | J3 | 1 100 |
| | Seasoned | SD4 | JD3 | 750 |
| Chengal | Unseasoned | S1 | J2 | 1 150 |
| | Seasoned | SD2 | JD2 | 950 |
| Gum, blue, southern | Unseasoned | S3 | J2 | 1 150 |
| | Seasoned | SD2 | JD2 | 1 000 |
| Gum, blue, Sydney | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD3 | JD2 | 850 |
| Gum, red, river | Unseasoned | S5 | J2 | 1 150 |
| | Seasoned | SD5 | JD2 | 900 |
| Gum, rose | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD4 | JD2 | 750 |
| Gum, spotted | Unseasoned | S2 | J1 | 1 200 |
| | Seasoned | SD2 | JD1 | 1 100 |

(*continued*)

**TABLE H2.3** (*continued*)

| Species | Moisture condition | Strength group (see Note 1) | Joint group | Design density (see Note 2) kg/m$^3$ |
|---|---|---|---|---|
| Hardwood, Johnstone River | Unseasoned | S2 | J1 | 1 150 |
| | Seasoned | SD3 | JD1 | 950 |
| Ironbark, grey | Unseasoned | S1 | J1 | 1 250 |
| | Seasoned | SD1 | JD1 | 1 100 |
| Ironbark, red, narrow-leaved | Unseasoned | S2 | J1 | 1 250 |
| | Seasoned | SD3 | JD1 | 1050 |
| Jarrah | Unseasoned | S4 | J2 | 1 100 |
| | Seasoned | SD4 | JD2 | 800 |
| Kapur | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD4 | JD2 | 750 |
| Karri | Unseasoned | S3 | J2 | 1 150 |
| | Seasoned | SD2 | JD2 | 900 |
| Kempas | Unseasoned | S2 | J1 | 1 100 |
| | Seasoned | SD2 | JD2 | 900 |
| Kwila (Merbau) | Unseasoned | S2 | J2 | 1 150 |
| | Seasoned | SD3 | JD2 | 850 |
| Lumbayau, Chengkulang | Unseasoned | S5 | J3 | 1 100 |
| | Seasoned | SD5 | JD3 | 750 |
| Mahogany, red | Unseasoned | S2 | J1 | 1 200 |
| | Seasoned | SD3 | JD1 | 950 |
| Marri | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD3 | JD2 | 850 |
| Meranti, dark red | Unseasoned | S5 | J4 | 1 100 |
| | Seasoned | SD6 | JD4 | 600 to 750 |
| Mersawa | Unseasoned | S6 | J3 | 1 050 |
| | Seasoned | SD6 | JD3 | 700 |
| Messmate | Unseasoned | S3 | J3 | 1 100 |
| | Seasoned | SD3 | JD3 | 750 |
| Oak, tulip, brown | Unseasoned | S2 | J2 | 1 150 |
| | Seasoned | SD2 | JD2 | 900 |
| Stringybark, brown | Unseasoned | S3 | J2 | 1 100 |
| | Seasoned | SD3 | JD2 | 850 |
| Stringybark, yellow | Unseasoned | S3 | J2 | 1 150 |
| | Seasoned | SD3 | JD2 | 900 |
| Tallowwood | Unseasoned | S2 | J1 | 1 200 |
| | Seasoned | SD2 | JD2 | 1 000 |
| Turpentine | Unseasoned | S3 | J2 | 1 050 |
| | Seasoned | SD3 | JD2 | 950 |
| Wandoo | Unseasoned | S2 | J1 | 1 250 |
| | Seasoned | SD3 | JD1 | 1 100 |

NOTES:

1 For classification into strength groups—see AS 2878.

2 For use only in computing dead load due to mass of timber.

## TABLE H2.4

## STRENGTH GROUP AND JOINT GROUP CLASSIFICATIONS AND DESIGN DENSITIES FOR SOME COMMON SOFTWOOD SPECIES

| Softwood Species | Moisture condition | Strength group(1) | Joint group | Design density(2) kg/m$^3$ |
|---|---|---|---|---|
| Mixed *Pinus* species (Australian grown) | Unseasoned | — | — | 850 |
| | Seasoned | SD7 | JD4 | 550 |
| Mixed softwood species (excl. *Pinus* species) | Unseasoned | — | — | 850 |
| | Seasoned | SD8 | JD4 | 500 |
| Imported softwoods (unidentified) | Unseasoned | S7 | J6 | 850 |
| | Seasoned | SD8 | JD6 | 400 |
| Fir, Douglas, North America | Unseasoned | S5 | J4 | 710 |
| | Seasoned | SD5 | JD4 | 550 |
| Fir, Douglas, elsewhere | Unseasoned | S6 | J5 | 710 |
| | Seasoned | SD6 | JD5 | 550 |
| Hemlock western | Unseasoned | S6 | J4 | 750 |
| | Seasoned | SD6 | JD4 | 500 |
| Hem-fir (species mixture) | Unseasoned | S7 | J5 | 750 |
| | Seasoned | SD7 | JD5 | 550 |
| Pine, cypress, white | Unseasoned | S5 | J3 | 850 |
| | Seasoned | SD6 | JD3 | 700 |
| Pine, hoop | Unseasoned | S6 | J4 | 800 |
| | Seasoned | SD5 | JD4 | 550 |
| Pine, radiata (Australia) | Unseasoned | S6 | J4 | 800 |
| Pine, radiata (New Zealand) | Unseasoned | S7 | J4 | 800 |
| Pine, radiata (Australia and New Zealand) | Seasoned | SD6 | JD4(3) | 550 |
| Pine, slash | Unseasoned | S5 | J3 | 850 |
| | Seasoned | SD5 | JD3 | 650 |
| Spruce-pine-fir (species mixture) | Unseasoned | — | — | 700 |
| | Seasoned | SD7 | JD5 | 500 |

NOTES:

1 For classification into strength groups—see AS 2878.

2 For use only in computing dead load due to mass of timber.

3 JD5 shall be used where heart-in material is included.

A1

## H3 DESIGN PROPERTIES FOR MGP10, MGP12 AND MGP15 AND A17 STRESS GRADES

The design characteristic values and joint group for MGP10, MGP12 MGP15 and A17 stress grades are listed in Table H3.1. The properties are only applicable for seasoned timber.

NOTES:

1 For MGP grades, the tabulated properties for bearing, shear at joints, tension perpendicular to grain, design density and joint group are based upon the species being strength group SD6 (or better).

2 For A17 grade, the tabulated values for bearing, shear at joints, tension perpendicular to grain, design density and joint group are based upon the species being strength group SD4 (or better).

3 Alternatively, the applicability of the properties may be confirmed on the basis of authoritative test data.

## TABLE H3.1

## CHARACTERISTIC VALUES FOR DESIGN—MGP10, MGP12, MGP15 & A17 STRESS GRADES

| Stress grade | Section size | | Characteristic values, MPa | | | | | | | | | | Design density | Joint group |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Depth | Breadth | Bending | Tension parallel to grain | Compression parallel to grain | Shear in beams | Average modulus of elasticity (see Note1) parallel to grain | Average modulus of rigidity | Bearing: Perpendicular to grain | Bearing: Parallel to grain | Shear at joint details | Tension perpendicular to grain | | |
| | mm | mm | ($f'_b$) | ($f'_t$) | ($f'_c$) | ($f'_s$) | ($E$) | ($G$) | ($f'_p$) | ($f'_\ell$) | ($f'_{sj}$) | ($f'_{tp}$) | (kg/m$^3$) | |
| MGP 10 | 70 to 140 | 35 and 45 | 17 | 7.7 | 18 | 2.6 | 10 000 | 670 | 10 | 30 | 4.2 | 0.5 | 500 | JD5 (see Note 2) |
| | 190 | | 16 | 7.1 | 18 | 2.5 | | | | | | | | |
| | 240 | | 15 | 6.6 | 17 | 2.4 | | | | | | | | |
| | 290 | | 14 | 6.1 | 16 | 2.3 | | | | | | | | |
| MGP 12 | 70 to 140 | 35 and 45 | 28 | 12 | 24 | 3.5 | 12 700 | 850 | 10 | 30 | 4.2 | 0.5 | 540 | JD4 |
| | 190 | | 25 | 12 | 23 | 3.3 | | | | | | | | |
| | 240 | | 24 | 11 | 22 | 3.2 | | | | | | | | |
| | 290 | | 22 | 9.9 | 22 | 3.1 | | | | | | | | |
| MGP 15 | 70 to 140 | 35 and 45 | 39 | 18 | 30 | 4.3 | 15 200 | 1 010 | 10 | 30 | 4.2 | 0.5 | 570 | JD4 |
| | 190 | | 36 | 17 | 29 | 4.1 | | | | | | | | |
| | 240 | | 33 | 16 | 28 | 4.0 | | | | | | | | |
| | 290 | | 31 | 14 | 27 | 3.8 | | | | | | | | |
| A17 | 70 to 120 | 35 | 45 | 26 | 40 | 5.1 | 16 000 | 930 | 17 | 50 | 6.0 | 0.6 | 650 | JD3 |
| | | 45 | 40 | 24 | 35 | 4.5 | | | | | | | | |
| | 140, 190 | 35 | 45 | 24 | 35 | 4.5 | | | | | | | | |
| | | 45 | 40 | 21 | 32 | 4.0 | | | | | | | | |
| | 240, 290 | 35 | 40 | 18 | 27 | 3.6 | | | | | | | | |
| | | 45 | 40 | 17 | 25 | 3.3 | | | | | | | | |

NOTES:

1 The average modulus of elasticity includes an allowance for shear deformation and is for short duration loading.

2 For MGP 10 grade, JD4 may be used where heart-in material is excluded.

3 The modulus of rigidity (estimated as one-fifteenth of the average modulus of elasticity) is included for the estimation of torsional rigidity.

4 Interpolation may be used to obtain properties for depths not listed.

# APPENDIX I

# BUCKLING STRENGTH OF PLYWOOD DIAPHRAGMS

(Normative)

## I1 SCOPE

Where large sheets of thin plywood are used in composite construction, it is possible for buckling distortions to cause a reduction in the load capacity of the plywood membrane. In this Appendix, strength reductions of this type are stated in terms of a stability factor $k_{12}$ for some typical membranes and plywood lay-ups.

## I2 BUCKLING STRENGTH FOR DIAPHRAGMS LOADED IN-PLANE

### I2.1 Application

The requirements of Paragraph I2 apply to the design of members constructed from continuous sheets of plywood attached to continuous solid timber edge members. Where either of these is discontinuous, these requirements shall apply only if they are spliced so as to develop the strength and stiffness equivalent to that of continuous elements.

### I2.2 Diaphragms with lateral edges supported and subjected to uniformly loaded edge forces

**I2.2.1** *Slenderness coefficient*

Figure I1 illustrates the notation to be used for a typical plywood diaphragm. The diaphragm is of length ($L$), depth ($d_w$) and thickness ($t_w$). The face grain of the plywood is at an angle ($\theta$) to the longitudinal edge as shown. The diaphragm is loaded along its edge by a combination of load effects comprising a shear load effect ($V_i^*$), a direct compression load effect ($N_c^*$), and a compression load effect due to edgewise bending that has a maximum value of $M_i^*$.

Where all edges are simply supported, the slenderness coefficient ($S$) for computation of the stability factor $k_{12}$ is given by the following equation:

$$S = g_{60} \frac{d_w}{t_w} \qquad \text{. . . I1}$$

where the factor $g_{60}$ is given in Table I1.

For short panels in which the length ($L$) is less than the characteristic length ($L_{ch}$) given in Table I2, the slenderness coefficient is given by the following equation:

$$S = g_{60} \left(\frac{L}{L_{ch}}\right)^{1/2} \frac{d_w}{t_w} \qquad \text{. . . I2}$$

If the lateral edges AB and CD shown in Figure I1 are effectively fixed, and AD and BC are simply supported, then it is appropriate to take the slenderness coefficients as 80% of those computed according to Equations I1 and I2.

## TABLE I1

## FACTOR $g_{60}$ FOR SLENDERNESS COEFFICIENTS OF PLYWOOD DIAPHRAGMS WITH LATERAL EDGES SUPPORTED

| Plywood lay-up* | | Factor $g_{60}$ | | | | | |
|---|---|---|---|---|---|---|---|
| | | Compression force ($N_c^*$) | | Moment ($M^*$) | | Shear force ($V^*$) | |
| | | $\theta = 0°$ | $\theta = 90°$ | $\theta = 0°$ | $\theta = 90°$ | $\theta = 0°$ | $\theta = 90°$ |
| $t_{po}/t_{pi} = 0.5$ | 3 ply<br>5 or more plies | 0.71<br>0.63 | 0.71<br>0.63 | 0.28<br>0.24 | 0.28<br>0.24 | 0.46<br>0.34 | 0.30<br>0.31 |
| $t_{po}/t_{pi} = 1.0$ | 3 ply<br>5 or more plies | 0.93<br>0.73 | 0.66<br>0.60 | 0.37<br>0.28 | 0.26<br>0.23 | 0.60<br>0.40 | 0.31<br>0.30 |
| $t_{po}/t_{pi} = 1.5$ | 3 ply<br>5 or more plies | 1.05<br>0.83 | 0.60<br>0.58 | 0.41<br>0.32 | 0.24<br>0.23 | 0.67<br>0.46 | 0.31<br>0.30 |

* All plies assumed to be of the same species; all inner plies assumed to be of equal thickness.

$t_{po}/t_{pi}$ = ratio of thickness of outer to inner plies.

NOTE: For direction of $\theta$, see Figure I1.

## TABLE I2

## CHARACTERISTIC LENGTH OF PANELS

| Plywood lay-up* | | Characteristic side ratio $L_{ch}/d_w$ (see Figure I1) | | | | | |
|---|---|---|---|---|---|---|---|
| | | Compression force ($N_c^*$) | | Moment ($M^*$) | | Shear force ($V^*$) | |
| | | $\theta = 0°$ | $\theta = 90°$ | $\theta = 0°$ | $\theta = 90°$ | $\theta = 0°$ | $\theta = 90°$ |
| $t_{po}/t_{pi} = 0.5$ | 3 ply<br>5 or more plies | 1.54<br>1.09 | 0.65<br>0.92 | 1.09<br>0.77 | 0.46<br>0.65 | 1.65<br>1.15 | 0.69<br>0.96 |
| $t_{po}/t_{pi} = 1.0$ | 3 ply<br>5 or more plies | 1.93<br>1.36 | 0.52<br>0.74 | 1.37<br>0.96 | 0.37<br>0.52 | 2.13<br>1.44 | 0.57<br>0.78 |
| $t_{po}/t_{pi} = 1.5$ | 3 ply<br>5 or more plies | 2.12<br>1.56 | 0.47<br>0.64 | 1.50<br>1.10 | 0.33<br>0.45 | 2.38<br>1.67 | 0.53<br>0.69 |

* All plies assumed to be of the same species; all inner plies assumed to be of equal thickness.

$t_{po}/t_{pi}$ = ratio of thickness of outer to inner plies.

NOTE: For direction of $\theta$, see Figure I1.

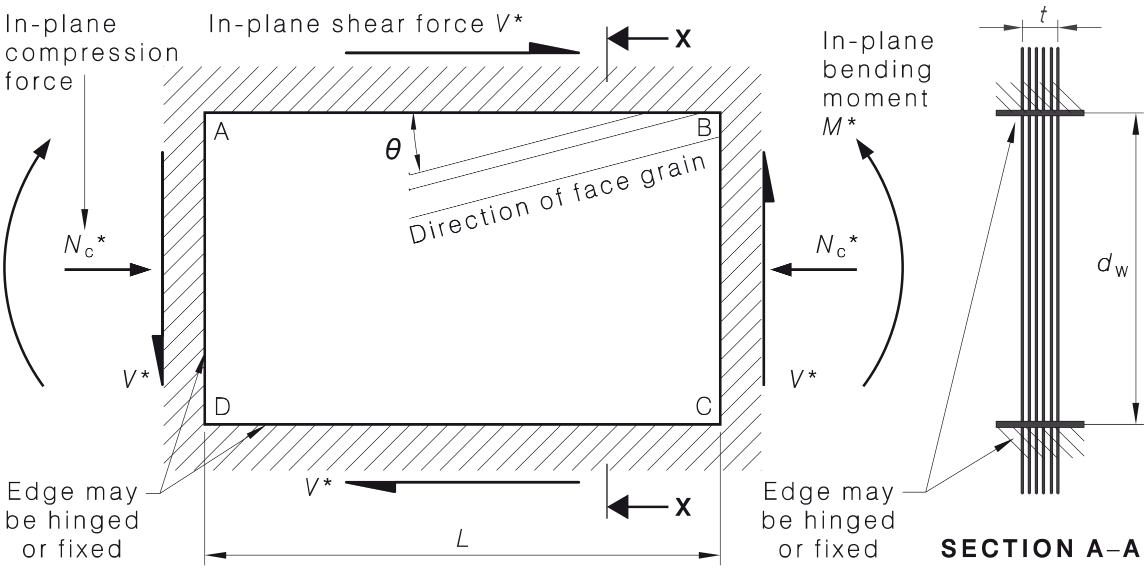



FIGURE I1 IN-PLANE FORCES ON DIAPHRAGM WITH SUPPORTED LATERAL EDGES

**I2.2.2** *Stability factor for edge shear forces*

The stability factor $k_{12}$ for the design capacity of the plywood in shear shall be taken as the lesser of the following:

$$k_{12} = 1.0 \quad \text{... I3(a)}$$

$$k_{12} = \frac{E}{15\, k_1\, f'_s\, S^2} + 0.5 \quad \text{... I3(b)}$$

where

$E$ = modulus of elasticity specified in Table 5.1, in megapascals

$k_1$ = duration factor as specified in Table 2.3

$f'_s$ = characteristic shear stress as specified in Table 5.1, in megapascals

$S$ = slenderness coefficient derived according to Paragraph I2.2.1

NOTE: From the data in Tables 5.1 and I1, it follows that $k_{12}$ =1.0 for all plywoods of stress grade equal to or greater than F7, and web thickness ratio $d_w/t_w \leq 19$.

**I2.2.3** *Stability factor for edge compression and edge bending moment*

The stability factor for the design capacity of the plywood in compression shall be taken to be the lesser of the following:

$$k_{12} = 1.0 \quad \text{... I4(a)}$$

$$k_{12} = \frac{E}{2\, k_1\, f'_c\, S^2} + 0.2 \quad \text{... I4(b)}$$

where

$E$ = modulus of elasticity specified in Table 5.1, in megapascals

$k_1$ = duration of load factor as specified in Table 2.3

$f'_c$ = characteristic compression stress as specified in Table 5.1, in megapascals

$S$ = slenderness coefficient derived according to Paragraph I2.2.1

NOTE: From the data in Tables 5.1 and I1, it follows that $k_{12}$ = 1.0 for all plywoods of stress grade equal to or less than F34, and web thickness ratio $d_w/t_w \leq 10$.

## I2.3 Diaphragms with lateral edges free and subjected to uniformly loaded edge forces

For a diaphragm, as shown in Figure I2, with lateral edges AC and BD free and end edges AB and CD simply supported, the slenderness coefficient $S$ for the design capacity to resist the compression force $N_c^*$ is given by the following equation:

$$S = g_{61} \frac{d_w}{t_w} \quad \text{... I6}$$

where the factor $g_{61}$ is given in Table I3.

If the edges AB and CD are fixed, then it is appropriate to take the slenderness coefficient as 70% of that calculated by Equation I6.

The appropriate stability factor $k_{12}$ to be used for the design capacity of the plywood is that given in Equations 3.3(11a), 3.3(11b) and 3.3(11c) of Clause 3.3.3 for solid timber members and using a material constant $\rho$ equal to 1.

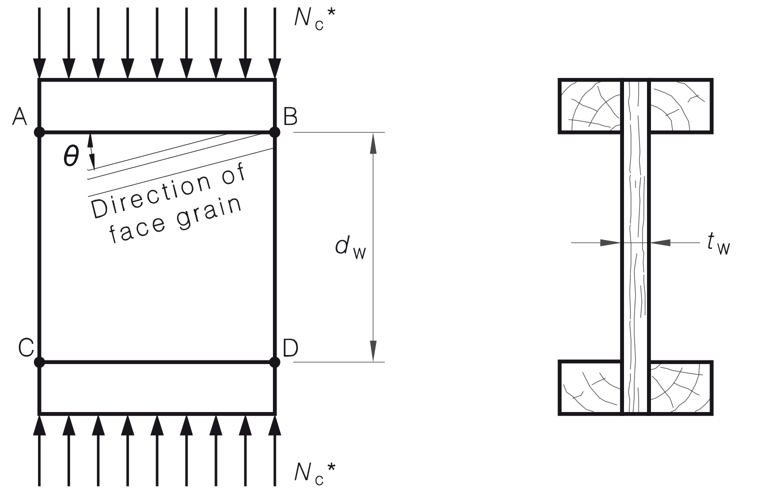



FIGURE I2 NOTATION FOR AXIALLY LOADED DIAPHRAGM WITH FREE LATERAL EDGES AC AND BD

## I2.4 Diaphragms subjected to concentrated edge forces

### I2.4.1 *Effective width*

In assessing the resistance of a plywood web to the concentrated load and support reaction as shown in Figure I3, the effective bearing width of the load, denoted by $l_{eff}$, shall be taken as follows:

(a) For the midspan concentrated load:

$$l_{eff} = l_b + 2\,t_f \quad \text{. . . I7}$$

(b) For the end support reaction:

$$l_{eff} = l_b + t_f \quad \text{. . . I8}$$

### I2.4.2 *Slenderness coefficients*

In assessing the resistance of a plywood web to the concentrated load and the support reaction as shown in Figure I3, the slenderness coefficient of the web shall be defined by the following:

A1 (a) Case A, load at end of beam (e.g., end support):

$$S = g_{61}\,\frac{d_w}{t_w}\left(\frac{l_b + 0.5\,t_f}{0.5\,g_{62}\,d_w + 3\,l_b + 1.5\,t_f}\right)^{1/2} \quad \text{. . . I9}$$

(b) Case B, load not at end of beam (e.g., intermediate support or load):

$$S = g_{61}\,\frac{d_w}{t_w}\left(\frac{l_b + 2\,t_f}{2.5\,g_{62}\,d_w + 2\,l_b + 6\,t_f}\right)^{1/2} \quad \text{. . . I10}$$

where

$g_{61}$ and $g_{62}$ = geometry factors given in Tables I3 and I4

$d_w$, $t_w$, $t_f$ and $l_b$ = dimensions (in millimetres) indicated in Figure I3

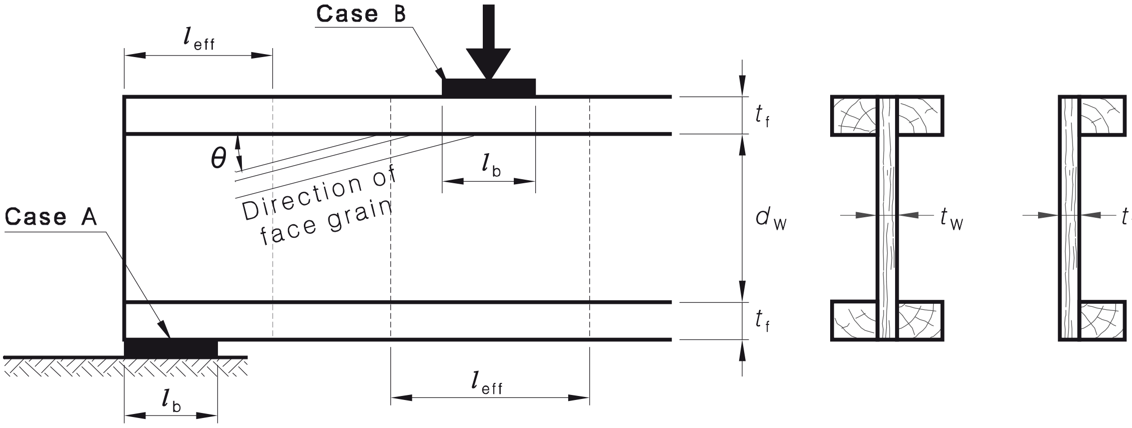



FIGURE I3 NOTATION FOR BEAM WITH UNSTIFFENED PLYWOOD WEB

## TABLE I3

## FACTOR $g_{61}$ FOR SLENDERNESS COEFFICIENTS OF DIAPHRAGMS WITH FREE EDGES

| Plywood lay-up* | | Factor $g_{61}$ | |
|---|---|---|---|
| | | $\theta = 0°$ | $\theta = 90°$ |
| $t_{po}/t_{pi} = 0.5$ | 3 ply | 1.8 | 0.75 |
| | 5 or more plies | 1.1 | 0.91 |
| $t_{po}/t_{pi} = 1.0$ | 3 ply | 2.2 | 0.83 |
| | 5 or more plies | 1.3 | 0.87 |
| $t_{po}/t_{pi} = 1.5$ | 3 ply | 2.3 | 0.87 |
| | 5 or more plies | 1.5 | 0.87 |

* All plies assumed to be of the same species; all inner plies assumed to be of equal thickness.

$t_{po}/t_{pi}$ = ratio of thickness of outer to inner plies.

NOTE: For direction of $\theta$, see Figure I2.

## TABLE I4

## FACTOR $g_{62}$ FOR EFFECTIVE BUCKLING WIDTH OF CONCENTRATED LOADS

| Plywood lay-up* | | Factor $g_{62}$ | |
|---|---|---|---|
| | | $\theta = 0°$ | $\theta = 90°$ |
| $t_{po}/t_{pi} = 0.5$ | 3 ply | 1.54 | 0.65 |
| | 5 or more plies | 1.09 | 0.92 |
| $t_{po}/t_{pi} = 1.0$ | 3 ply | 1.93 | 0.52 |
| | 5 or more plies | 1.36 | 0.74 |
| $t_{po}/t_{pi} = 1.5$ | 3 ply | 2.12 | 0.47 |
| | 5 or more plies | 1.56 | 0.64 |

* All plies assumed to be of the same species; all inner plies assumed to be of equal thickness.

$t_{po}/t_{pi}$ = ratio of thickness of outer to inner plies.

NOTE: For direction of $\theta$, see Figure I2.

**I2.4.3** *Stability factor*

The stability factor $k_{12}$ to be used for the design capacity of the plywood is given by the following equation:

$$k_{12} = 1.0 - 0.3\, S \left( \frac{k_1 f'_c}{E} \right)^{1/2} \qquad \text{. . . I11}$$

where

$S$ = slenderness coefficient

$k_1$ = duration of load factor specified in Table 2.3

$f'_c$ = characteristic compression stress as specified in Table 5.1 , in megapascals

$E$ = modulus of elasticity as specified in Table 5.1, in megapascals.

## I2.5 Stiffeners for beam webs

At supports or load points, where the buckling strength of the webs is inadequate, the webs should be reinforced by vertical stiffeners. The size of the stiffeners will be adequate if they extend the full width of the flanges and have a cross-section equal in area to that of one flange.

For webs in which the slenderness coefficient ($d_w/t_w$), for the shear force, as shown in Figure I3, is greater than 15, it is desirable that vertical stiffeners be placed at intervals along the web in order to reduce shear distortions normal to the web. It is recommended that each of these stiffeners have a cross-sectional area not less than 0.25 times the area of a flange and that they be spaced not further than 1.5 $g_{62}$ $d_w$ apart, where the factor $g_{62}$ is given in Table I4.

All vertical stiffeners should extend from flange to flange.

# I3 METHOD OF CALCULATION OF SECTION PROPERTIES

## I3.1 General

The method of calculation of section properties in AS/NZS 2269, or an equivalent alternative, shall be used to establish the second moment of area (moment of inertia) and section modulus of structural plywood panels.

For the computation of bending capacity, the second moment of area ($I$) shall be computed based only on plies parallel to the direction of span.

For the computation of bending rigidity, the second moment of area ($I$) shall be computed based on parallel plies plus 0.03 times plies perpendicular to the span.

This method satisfies the requirements of AS/NZS 2269.

## I3.2 Definitions for use in calculation of section properties

Definitions for use in calculation of section properties are as follows:

(a) The thickness of individual veneers ($d$) in the plywood assembly shall be taken as the actual value given to the thickness of individual plies through the assembly in Table I5 for standard plywood constructions. In non-standard constructions the value of ($d$) shall be taken as the thickness of the dried veneer less 6% to allow for compression and sanding losses.

(b) The overall thickness of the panel ($t$) is the summation of the actual individual veneer thicknesses as defined in Item (a).

(c) $\bar{y}$ is the distance between the neutral axis of the panel (NA) and the neutral axis of each individual veneer as computed based upon Items (a) and (b).

## I3.3 Calculation method

**I3.3.1** *Face grain parallel to the span*

An illustration and section of face grain parallel to the span is shown in Figure I4.

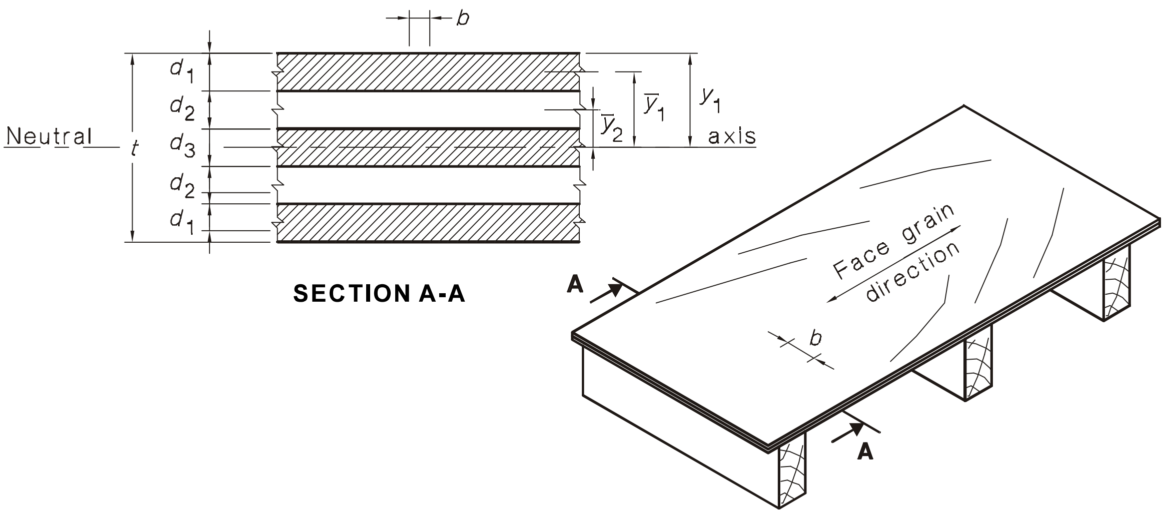



FIGURE I4 SECTION PROPERTIES—FACE GRAIN PARALLEL TO THE SPAN

Using the theory of parallel axes and parallel ply theory, the calculations are as follows:

(a) Second moment of area for calculation of bending rigidity ($EI_R$) for plywood with face grain parallel to span:

$$I_R = 2\left(\frac{1}{12} b\, d_1^3 + A_1\, \bar{y}_1^2\right) + 2 \times 0.03\left(\frac{1}{12} b\, d_2^3 + A_2\, \bar{y}_2^2\right) + \frac{1}{12} b\, d_3^3 \quad \text{. . . I12}$$

where

$I_R$ = second moment of area about the neutral axis for bending rigidity calculations for a width $b$ = 1 mm, in $mm^4/mm$

$A_1$ = $d_1 b$

$A_2$ = $d_2 b$

0.03 = factor applied for plies at right angles to the span (for estimation of rigidity only)

(b) Section modulus for calculation of bending capacity for plywood with face grain parallel to span:

$$I_C = 2\left(\frac{1}{12} b\, d_1^3 + A_1\, \bar{y}_1^2\right) + \frac{1}{12} b\, d_3^3 \quad \text{. . . I13}$$

neglecting cross-directional veneers as required by AS 1720.1

$$Z = \frac{I_C}{y_1}$$

where

$I_C$ = second moment of area about the neutral axis for bending capacity calculations for a width $b$ = 1 mm, in $mm^4/mm$

$Z$ = section modulus for a width $b$ = 1 mm, in $mm^3/mm$

$y_1$ = distance from neutral axis (NA), which is at the centre-line of plywood with balanced construction, to the farthest veneer that is parallel to the span (see Figure I4)

**I13.3.2** *Face grain perpendicular to the span*

An illustration and section of face grain perpendicular to the span is shown in Figure I5.

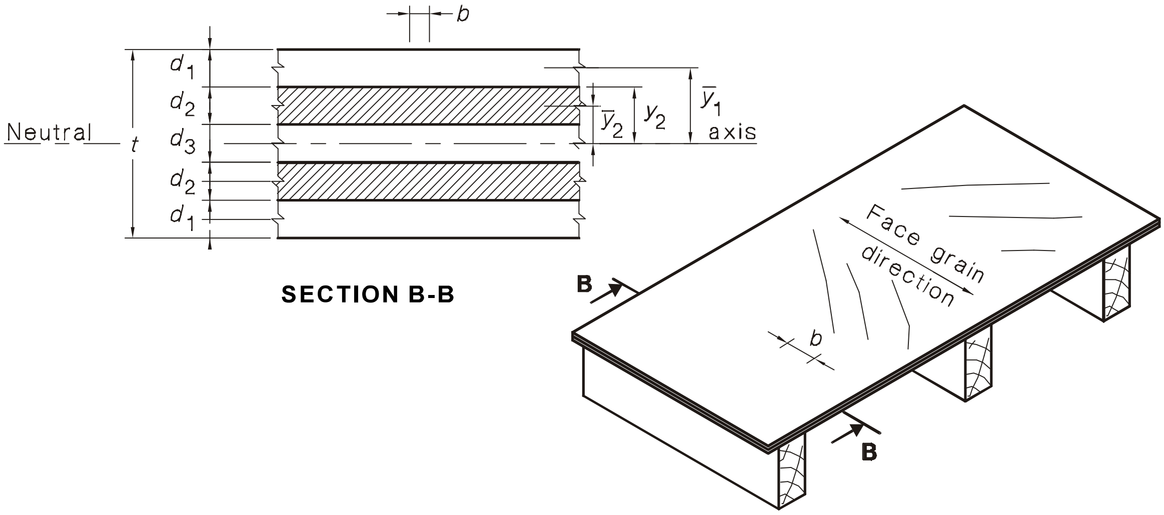



FIGURE I5 FACE GRAIN PERPENDICULAR TO SPAN

The calculations are as follows:

(a) Second moment of area for calculation of bending rigidity ($EI_R$) for plywood with face grain perpendicular to span:

$$I_R = 2 \times 0.03\left(\frac{1}{12} b\, d_1^3 + A_1\, \bar{y}_1^2\right) + 2\left(\frac{1}{12} b\, d_2^3 + A_2\, \bar{y}_2^2\right) + 0.03\frac{1}{12} b\, d_3^3 \qquad \ldots \text{I14}$$

where

$I_R$ = second moment of area about the neutral axis for bending rigidity calculations for a width $b$ = 1 mm, in $mm^4/mm$

$A_1$ = $d_1 b$

$A_2$ = $d_2 b$

0.03 = factor applied for plies at right angles to the span (for estimation of rigidity only)

(b) Section modulus for calculation of bending capacity for plywood with face grain perpendicular to span:

$$I_C = 2\left(\frac{1}{12} b\, d_2{}^3 + A_2\,(\bar{y}_2)^2\right) \qquad \ldots \text{I15}$$

$$Z = \frac{I_C}{y_2}$$

where

$I_C$ = second moment of area about the neutral axis for bending capacity calculations for a width $b$ = 1 mm, in $mm^4/mm$

$Z$ = section modulus for a width $b$ = 1 mm, in $mm^3/mm$

$y_2$ = distance from neutral axis to the outside of the farthest veneer parallel to the span (see Figure I5)

## I4 SECTION PROPERTIES FOR STANDARD CONSTRUCTIONS

The second moments of area (moments of inertia) and section moduli for the standard structural plywood constructions in AS/NZS 2269, for directions parallel and perpendicular to the grain, are detailed in Table I5.

### TABLE I5

### SECOND MOMENTS OF AREA (MOMENTS OF INERTIA) AND SECTION MODULI FOR STANDARD STRUCTURAL PLYWOOD

| Nominal thickness mm | Identification code | Face grain parallel to span | | Face grain perpendicular to span | |
|---|---|---|---|---|---|
| | | Moment of inertia ($I$), $mm^4/mm$ | Section modulus ($Z$), $mm^3/mm$ | Moment of inertia ($I$), $mm^4/mm$ | Section modulus ($Z$), $mm^3/mm$ |
| 4.0 | 4-14-3 | 6.0 | 2.8 | 0.4 | 0.3 |
| 4.0 | 4-09-3 | 5.4 | 2.5 | 1.5 | 1.0 |
| 4.5 | 4.5-15-3 | 7.3 | 3.3 | 0.5 | 0.4 |
| 5.0 | 5-09-3 | 7.0 | 2.9 | 2.5 | 1.5 |
| 5.0 | 5-14-3 | 11.0 | 4.2 | 1.6 | 1.0 |
| 6.0 | 6-15-3 | 16.0 | 5.3 | 2.7 | 1.5 |
| 7.0 | 7-24-3 | 30.0 | 8.3 | 2.1 | 1.0 |
| 7.0 | 7-14-5 | 23.0 | 6.5 | 6.6 | 2.8 |
| 7.5 | 7.5-15-5 | 28.0 | 7.4 | 8.1 | 3.3 |
| 7.5 | 7.5-25-3 | 34.0 | 9.0 | 2.3 | 1.0 |
| 7.5 | 7.5-24-3 | 37.0 | 9.6 | 3.4 | 1.5 |
| 9.0 | 9-15-5(a) | 45.0 | 10.0 | 17.0 | 5.3 |
| 9.0 | 9-15-5(b) | 47.0 | 10.0 | 22.0 | 6.5 |
| 9.0 | 9-30-3 | 60.0 | 13.0 | 4.0 | 1.5 |
| 9.0 | 9-09-5 | 28.0 | 6.3 | 28.0 | 7.9 |
| 9.0 | 9-24-5 | 60.0 | 13.0 | 9.1 | 3.3 |
| 9.5 | 9-15-5 | 60.0 | 11.5 | 32.0 | 8.3 |
| 9.5 | 9.5-14-7 | 55.0 | 11.5 | 24.0 | 6.5 |
| 11.0 | 11-09-5 | 48.0 | 8.6 | 60.0 | 13.0 |
| 12.0 | 12-15-5 | 85.0 | 14.5 | 60.0 | 13.0 |
| 12.0 | 12-24-5 | 115.0 | 19.0 | 33.0 | 8.3 |
| 12.0 | 12-30-4 | 125.0 | 21.0 | 22.0 | 6 |
| 12.0 | 12-30-5 | 130.0 | 21.5 | 20.0 | 5.3 |
| 12.0 | 12-40-3 | 140.0 | 23.0 | 9.5 | 2.7 |
| 12.5 | 12.5-25-5 | 130.0 | 20.5 | 38.0 | 9.0 |
| 12.5 | 12.5-24-5 | 130.0 | 20.5 | 41.0 | 9.6 |
| 12.5 | 12.5-14-7 | 105.0 | 16.0 | 75.0 | 14.5 |
| 12.5 | 12.5-14-9 | 115.0 | 17.5 | 60.0 | 11.5 |
| 12.5 | 12.5-15-7 | 125.0 | 18.0 | 90.0 | 16.0 |
| 13.0 | 13-24-5 | 145.0 | 21.5 | 55.0 | 11.5 |
| 13.0 | 13-30-5 | 165.0 | 24.5 | 35.0 | 8.3 |
| 14.0 | 14-24-5 | 160.0 | 23.5 | 65.0 | 13.0 |
| 14.0 | 14-30-5 | 185.0 | 26.5 | 43.0 | 9.6 |
| 15.0 | 15-15-7 | 170.0 | 22.5 | 120.0 | 19.0 |
| 15.0 | 15-24-7 | 205.0 | 27.5 | 85.0 | 15.0 |
| 15.0 | 15-30-5 | 225.0 | 29.5 | 65.0 | 13.0 |
| 15.0 | 15-14-7 | 155.0 | 19.5 | 150.0 | 23.0 |
| 15.0 | 15-14-11 | 195.0 | 25.0 | 115.0 | 17.5 |
| 17.0 | 17-15-7 | 220.0 | 25.5 | 190.0 | 26.5 |
| 17.0 | 17-24-6 | 273.0 | 32.0 | 134.0 | 21.0 |
| 17.0 | 17-24-7 | 285.0 | 33.5 | 120.0 | 19.0 |
| 17.0 | 17-10-7 | 195.0 | 22.0 | 230.0 | 29.5 |
| 17.0 | 17-30-5 | 305.0 | 35.5 | 120.0 | 20.0 |
| 17.0 | 17-30-6 | 300.0 | 35.5 | 105.0 | 17.5 |

*(continued)*

## TABLE I5 (*continued*)

| Nominal thickness mm | Identification code | Face grain parallel to span | | Face grain perpendicular to span | |
|---|---|---|---|---|---|
| | | Moment of inertia ($I$), mm$^4$/mm | Section modulus ($Z$), mm$^3$/mm | Moment of inertia ($I$), mm$^4$/mm | Section modulus ($Z$), mm$^3$/mm |
| 17.5 | 17.5-25-7 | 320.0 | 36.5 | 140.0 | 20.5 |
| 17.5 | 17.5-24-7 | 345.0 | 38.0 | 155.0 | 21.5 |
| 18.0 | 18-15-7 | 270.0 | 29.5 | 230.0 | 29.5 |
| 18.0 | 18-30-7 | 375.0 | 41.5 | 125.0 | 19.0 |
| 18.0 | 18-14-13 | 315.0 | 34.0 | 205.0 | 25.0 |
| 18.0 | 18-30-6 | 365.0 | 40.0 | 135.0 | 21.0 |
| 18.5 | 18.5-32-7 | 410.0 | 44.0 | 125.0 | 19.0 |
| 19.0 | 19-24-7 | 360.0 | 38.0 | 190.0 | 26.5 |
| 19.0 | 19-24-9 | 380.0 | 39.5 | 200.0 | 26.5 |
| 19.0 | 19-30-7 | 450.0 | 46.5 | 155.0 | 21.5 |
| 19.0 | 19-09-9 | 290.0 | 29.0 | 325.0 | 36.5 |
| 19.0 | 19-14-9 | 355.0 | 35.5 | 290.0 | 33.5 |
| 19.5 | 19.5-15-9 | 375.0 | 37.0 | 290.0 | 33.3 |
| 21.0 | 21-24-9 | 565.0 | 51.5 | 300.0 | 33.5 |
| 21.0 | 21-30-7 | 555.0 | 52.5 | 240.0 | 29.5 |
| 21.0 | 21-30-9 | 625.0 | 59.0 | 170.0 | 20.5 |
| 22.0 | 22-25-9 | 640.0 | 56.0 | 335.0 | 36.5 |
| 22.5 | 22.5-30-9 | 715.0 | 62.0 | 300.0 | 33.5 |
| 24.0 | 24-30-8 | 800.0 | 66.0 | 385.0 | 40.0 |
| 25.0 | 25-30-9 | 900.0 | 70.5.0 | 380.0 | 38.0 |
| 25.0 | 25-14-9 | 685.0 | 53.0 | 685.0 | 59.5 |
| 25.0 | 25-24-9 | 770.0 | 61.0 | 535.0 | 51.5 |
| 26.0 | 26-24-11 | 990.0 | 74.0 | 590.0 | 51.5 |
| 26.0 | 26-32-9 | 1070.0 | 81.0 | 445.0 | 42.0 |
| 27.0 | 27-30-9 | 1110.0 | 81.0 | 580.0 | 52.5 |
| 28.0 | 28-15-13 | 1070.0 | 73.5 | 920.0 | 69.5 |
| 28.0 | 28-30-11 | 1210.0 | 86.5 | 595.0 | 51.5 |
| 28.0 | 28-14-11 | 970.0 | 65.0 | 1080.0 | 81.0 |
| 28.0 | 28-40-7 | 1320.0 | 93.0 | 565.0 | 53.0 |
| 30.0 | 30-14-13 | 1250.0 | 80.5 | 1140.0 | 80.0 |
| 30.0 | 30-30-10 | 1480.0 | 97.0 | 840.0 | 66.0 |
| 31.0 | 31-14-11 | 1340.0 | 82.5 | 1360.0 | 92.0 |
| 31.0 | 31-24-13 | 1590.0 | 100.0 | 1020.0 | 74.0 |
| 31.0 | 31-30-9 | 1510.0 | 95.5 | 1050.0 | 80.0 |
| 32.0 | 32-14-11 | 2330.0 | 120.0 | 2380.0 | 130.0 |
| 32.0 | 32-30-11 | 1570.0 | 105.0 | 750.0 | 59.0 |
| 33.0 | 33-30-11 | 1940.0 | 115.0 | 1150.0 | 81.0 |
| 34.0 | 34-14-13 | 1640.0 | 92.0 | 1880.0 | 115.0 |
| 34.0 | 34-24-13 | 2010.0 | 115.0 | 1600.0 | 105.0 |
| 35.0 | 35-14-15 | 1960.0 | 110.0 | 1820.0 | 110.0 |
| 35.0 | 35-16-13 | 2430.0 | 130.0 | 1940.0 | 115.0 |
| 36.0 | 36-14-15 | 2200.0 | 115.0 | 2040.0 | 115.0 |
| 37.0 | 37-14-13 | 2330.0 | 120.0 | 2380.0 | 130.0 |
| 38.0 | 38-25-13 | 3020.0 | 150.0 | 2270.0 | 125.0 |
| 39.0 | 39-30-13 | 3100.0 | 155.0 | 1990.0 | 115.0 |
| 40.0 | 40-25-15 | 3510.0 | 160.0 | 3030.0 | 155 |
| 40.0 | 40-24-17 | 3400.0 | 165.0 | 2430.0 | 130 |
| 46.0 | 46-24-15 | 4750.0 | 200.0 | 3830.0 | 175 |
| 50.0 | 50-24-17 | 5870.0 | 230.0 | 4730.0 | 200 |

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# AMENDMENT CONTROL SHEET

# AS 1720.1—2010

## Amendment No. 1 (2010)

### CORRECTION

*SUMMARY:* This Amendment applies to Clauses 1.4.4.5 and 6.3.3, Table 7.2(B), and Appendices C, E, H and I.

Published on 7 December 2010.

## Amendment No. 2 (2011)

### CORRECTION

*SUMMARY:* This Amendment applies to Clause 8.4.7 and Table 7.2(B).

Published on 18 August 2011.

## Amendment No. 3 (2015)

### REVISED TEXT

*SUMMARY:* This Amendment applies to the Table 5.1 and Equation 8(2).

Published on 13 August 2015.

NOTES

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