

AS 4100:2020



# Steel structures



AS 4100:2020

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- Australian Chamber of Commerce and Industry
- Australian Industry Group
- Australian Steel Association
- Australian Steel Institute
- Austrroads
- Bureau of Steel Manufacturers of Australia
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# Steel structures

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## Preface

This Standard was prepared by the Standards Australia Committee BD-001, Steel Structures, to supersede AS 4100—1998.

The objective of this Standard is to provide designers of steel structures with specifications for steel structural members used for load-carrying purposes in buildings and other structures.

Major changes to the 1998 edition include the following:

- (a) Reference to the “construction specification” (as the document containing the particular design data and details to be provided) as one deliverable from the design process. A definition of the construction specification consistent with that in AS/NZS 5131 has also been provided ([Clauses 1.3.16](#) and [1.6.2](#)).
- (b) Introduction of the definition of “construction category” and an informative appendix providing guidance on selection of the appropriate construction category, both consistent with AS/NZS 5131 ([Clauses 1.3.15](#) and [1.7.2](#), and [Appendix L](#)).
- (c) Definition and description of “architecturally exposed structural steelwork” (AESS) ([Clauses 1.3.3](#) and [1.7.3](#)).
- (d) Definition and treatment of “lamellar tearing” consistent with AS/NZS 1554.1 ([Clauses 1.3.40](#) and [3.8](#), and [Appendix M](#)).
- (e) Alignment with AS/NZS 5100.6:2017 (various clauses).
- (f) Referencing of AS/NZS 5131:2016 for the majority of requirements in the fabrication and erection sections of this Standard ([Sections 14](#) and [15](#)).
- (g) Alignment with AS/NZS 1252.1:2016, which includes introduction of an “alternative bolt assembly type” to EN 14399-3 System HR for grade 8.8 bolts and an “additional bolt assembly type” to EN 14399-3 System HR for grade 10.9 bolts. The Australian Steel Institute (ASI), Technical Note TN-001, *High strength structural bolt assemblies to AS/NZS 1252:2016*, provides background and basis for the revision to AS/NZS 1252:1996 ([Clauses 9.1.6](#), [9.3](#), [15.2](#)).
- (h) New specification of geometrical tolerances for fabrication and erection aligned with AS/NZS 5131 ([Clauses 14.4](#) and [15.3](#)).
- (i) New [Appendix K](#) “Statistical data”, aligned with AS/NZS 5100.6.
- (j) Inclusion of shear modulus  $G$  at elevated temperature in [Clause 12.4.2](#) and a new [Clause 12.4.3](#), Slenderness at elevated temperature

[Table M.2](#), Criteria affecting the target value of  $Z_{Ed}$ , was adapted with permission from Table 3.2 of EN 1993-1-10. Copyright © 2005. CEN, Belgium. [www.cen.eu](http://www.cen.eu)

The terms “normative” and “informative” are used in Standards to define the application of the appendices 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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# Australian Standard®

## Steel structures

### Section 1 Scope and general

#### 1.1 Scope and exclusions

##### 1.1.1 Scope

This Standard sets out minimum requirements for the design and the engineering aspects of fabrication and erection, and modification of steelwork in structures in accordance with the limit states design method.

This Standard applies to buildings, structures and cranes constructed of steel.

NOTE For design of box and longitudinally stiffened girders, refer to AS/NZS 5100.6.

##### 1.1.2 Exclusions

This Standard does not apply to the following structures and materials:

- (a) Steel elements less than 3 mm thick, with the exception of sections in accordance with AS/NZS 1163 and packers.
- (b) Steel members for which the value of the yield stress used in design ( $f_y$ ) exceeds 690 MPa.
- (c) Cold-formed members, other than those in accordance with AS/NZS 1163, which are designed in accordance with AS/NZS 4600.
- (d) Composite steel-concrete members, which are designed in accordance with AS/NZS 2327.
- (e) Road, railway and pedestrian bridges, which are designed in accordance with AS 5100.1, AS 5100.2 and AS/NZS 5100.6.

NOTE The general principles of design, fabrication, erection, and modification embodied in this Standard may be applied to steel-framed structures or members not specifically mentioned herein.

#### 1.2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document.

NOTE Documents for informative purposes are listed in the Bibliography.

AS 1101.3, *Graphical symbols for general engineering, Part 3: Welding and non-destructive examination*

AS 1110.1, *ISO metric hexagon bolts and screws—Product grades A and B, Part 1: Bolts*

AS 1110.2, *ISO metric hexagon bolts and screws—Product grades A and B, Part 2: Screws*

AS 1111.1, *ISO metric hexagon bolts and screws—Product grade C, Part 1: Bolts*

AS 1111.2, *ISO metric hexagon bolts and screws—Product grade C, Part 2: Screws*

AS 1112.1, *ISO metric hexagon nuts, Part 1: Style 1—Product grades A and B*

AS 1112.2, *ISO metric hexagon nuts, Part 2: Style 2—Product grades A and B*

AS 1112.3, *ISO metric hexagon nuts, Part 3: Product grade C*

AS 1112.4, *ISO metric hexagon nuts, Part 4: Chamfered thin nuts—Product grades A and B*

- AS 1170.4, *Structural design actions, Part 4: Earthquake actions in Australia*
- AS 1237.1, *Plain washers for metric bolts, screws and nuts for general purposes, Part 1: General plan*
- AS 1275, *Metric screw threads for fasteners*
- AS 1391, *Metallic materials—Tensile testing at ambient temperature*
- AS 1418.1, *Cranes, hoists and winches, Part 1: General requirements*
- AS 1418.3, *Cranes, hoists and winches, Part 3: Bridge, gantry and portal (including container cranes) and jib cranes*
- AS 1418.5, *Cranes, hoists and winches, Part 5: Mobile cranes*
- AS 1418.18, *Cranes, hoists and winches, Part 18: Crane runways and monorails*
- AS 1530.4, *Methods for fire tests on building materials, components and structures, Part 4: Fire-resistance test of elements of construction*
- AS 1657, *Fixed platforms, walkways, stairways and ladders—Design, construction and installation*
- AS 1735, *Lifts, escalators and moving walks (series)*
- AS 1858.1, *Electrodes and fluxes for submerged-arc welding, Part 1: Carbon steels and carbon manganese steels*
- AS 1858.2, *Electrodes and fluxes for submerged-arc welding, Part 2: Low and intermediate alloy steels*
- AS 2074, *Cast steels*
- AS 2670, *Evaluation of human exposure to whole-body vibration (series)*
- AS 3597, *Structural and pressure vessel steel—Quenched and tempered plate*
- AS 5100.1, *Bridge design, Part 1: Scope and general principles*
- AS 5100.2, *Bridge design, Part 2: Design loads*
- AS 5216, *Design of post-installed and cast-in fastenings in concrete*
- AS/NZS 1170.0, *Structural design actions, Part 0: General principles*
- AS/NZS 1170.1, *Structural design actions, Part 1: Permanent, imposed and other actions*
- AS/NZS 1170.2, *Structural design actions, Part 2: Wind actions*
- AS/NZS 1170.3, *Structural design actions, Part 3: Snow and ice actions*
- AS/NZS 1252.1, *High strength steel fastener assemblies for structural engineering—Bolts, nuts and washer, Part 1: Technical requirements*
- AS/NZS 1163, *Cold-formed structural steel hollow sections*
- AS/NZS 1554.1, *Structural steel welding, Part 1: Welding of steel structures*
- AS/NZS 1554.2, *Structural steel welding, Part 2: Stud welding (steel studs to steel)*
- AS/NZS 1554.4, *Structural steel welding, Part 4: Welding of high strength quenched and tempered steels*
- AS/NZS 1554.5, *Structural steel welding, Part 5: Welding of steel structures subject to high levels of fatigue loading*
- AS/NZS 1559, *Hot-dip galvanized steel bolts with associated nuts and washers for tower construction*

AS/NZS 1594, *Hot-rolled steel flat products*

AS/NZS 1873, *Powder-actuated (PA) hand-held fastening tools (All parts)*

AS/NZS 2327, *Composite structures—Composite steel-concrete construction in buildings*

AS/NZS 2717.1, *Welding—Electrodes—Gas metal arc, Part 1: Ferritic steel electrodes*

AS/NZS 3678, *Structural steel—Hot-rolled plates, floorplates and slabs*

AS/NZS 3679.1, *Structural steel, Part 1: Hot-rolled bars and sections*

AS/NZS 3679.2, *Structural steel, Part 2: Welded I sections*

AS/NZS 4600, *Cold-formed steel structures*

AS/NZS 4855, *Welding consumables—Covered electrodes for manual metal arc welding of non-alloy and fine grain steels—Classification*

AS/NZS 4857, *Welding consumables—Covered electrodes for manual metal arc welding of high-strength steels—Classification*

AS/NZS 5100.6, *Bridge design, Part 6: Steel and composite construction*

AS/NZS 5131, *Structural steelwork—Fabrication and erection*

AS ISO/IEC 17025, *General requirements for the competence of testing and calibration laboratories*

AS/NZS ISO 17632, *Welding consumables—Tubular cored electrodes for gas shielded and non-gas shielded metal arc welding of non-alloy and fine grain steels—Classification*

AS/NZS ISO 18276, *Welding consumables—Tubular cored electrodes for gas shielded and non-gas-shielded metal arc welding of high-strength steels—Classification*

NZS 1170.5, *Structural design actions—Part 5: Earthquake actions—New Zealand*

NZS 3404, *Steel structures standard—Parts 1 and 2*

ISO 636, *Welding consumables — Rods, wires and deposits for tungsten inert gas welding of non-alloy and fine-grain steels — Classification*

ISO 13918, *Welding — Studs and ceramic ferrules for arc stud welding*

ISO 14341, *Welding consumables — Wire electrodes and weld deposits for gas shielded metal arc welding of non alloy and fine grain steels — Classification*

ISO 16834, *Welding consumables — Wire electrodes, wires, rods and deposits for gas shielded arc welding of high strength steels — Classification*

EN 13381-4, *Test methods for determining the contribution to the fire resistance of structural members — Part 4: Applied passive protection products to steel members*

EN 13381-8, *Test methods for determining the contribution to the fire resistance of structural members — Part 8: Applied reactive protection to steel members*

### **1.3 Terms and definitions**

For the purposes of this document, the following terms and definitions apply. Definitions peculiar to a particular Clause or Section are given in that Clause or Section.

#### **1.3.1 action**

cause of stress or deformations in a structure



**1.3.2****action effect or load effect**

internal force or bending moment due to actions or loads

**1.3.3****architecturally exposed structural steelwork****AESS**

structural steelwork that is exposed to view and considered a fundamental component of the architectural intent of the structure

**1.3.4****authority**

body having statutory powers to control the design and erection of a structure

**1.3.5****bearing-type connection**

connection effected using either snug-tight bolts, or high-strength bolts tightened to induce a specified minimum bolt tension, in which the design action is transferred by shear in the bolts and bearing on the connected parts at the strength limit state

**1.3.6****bearing-wall system**

see AS 1170.4

**1.3.7****braced frame**

see AS 1170.4

**1.3.8****braced frame, concentric**

see AS 1170.4

**1.3.9****braced frame, eccentric**

see AS 1170.4

**1.3.10****braced member**

one for which the transverse displacement of one end of the member relative to the other is effectively prevented

**1.3.11****capacity factor**

factor used to multiply the nominal capacity to obtain the design capacity

**1.3.12****complete penetration butt weld**

butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint

**1.3.13****connection**

joint between components of a structural member or a joint between separate structural members, including splices

**1.3.14****constant stress range fatigue limit**

highest constant stress range for each detail category at which fatigue cracks are not expected to propagate (see [Figure 11.6.1](#))

**1.3.15****construction category**

classified set of requirements specified for construction of the works as a whole, of an individual component or of a detail of a component

Note 1 to entry: Both ISO 2394 and European Standards use the term “execution” to describe all activities performed for the physical completion of the works (i.e. procurement, fabrication, welding, mechanical fastening, transportation, erection, surface treatment and the inspection and documentation) and this term may be assumed to be equivalent to “fabrication and erection”. “Execution Class” (EXC) is the term used in European Standards to refer to a risk-based categorization of a structure, similar in intent to the construction category adopted in this Standard. It should be noted, however, that the two categorizations are not based on the same risk matrix assessment and cannot be used interchangeably.

**1.3.16****construction specification**

set of documents covering technical data and other requirements for a particular steel structure, including those specified to supplement and qualify the provisions of this Standard

**1.3.17****cut-off limit**

for each detail category, the highest variable stress range which does not require consideration when carrying out cumulative damage calculations (see [Figures 11.6.1](#) and [11.6.2](#))

**1.3.18****design action effect****design load effect**

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

**1.3.19****design action****design load**

combination of the nominal actions or loads and the load factors specified in AS/NZS 1170.0, AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3, AS 1170.4 or other standards referenced in [Clause 3.2.1](#)

**1.3.20****design capacity**

product of the nominal capacity and the capacity factor

**1.3.21****design life**

period over which a structure or structural element is expected to fulfil its intended function

**1.3.22****design resistance effect**

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

**1.3.23****design spectrum**

sum of the stress spectra from all of the nominal loading events expected during the design life

**1.3.24****detail category**

designation given to a particular detail to indicate which of the S-N curves is to be used in the fatigue assessment

Note 1 to entry: The detail category takes into consideration the local stress concentration at the detail, the size and shape of the maximum acceptable discontinuity, the loading condition, metallurgical effects, residual stresses, the welding process and any post weld improvement.

Note 2 to entry: The detail category number is defined by the fatigue strength at  $2 \times 10^6$  cycles on the S-N curve (see [Figures 11.6.1](#) and [11.6.2](#)).

**1.3.25  
discontinuity**

absence of material, causing a stress concentration

Note 1 to entry: Typical discontinuities include cracks, scratches, corrosion pits, lack of penetration, slag inclusions, cold laps, porosity and undercut.

**1.3.26  
ductility**

<of structure> see AS 1170.4

**1.3.27  
exposed surface area to mass ratio**

ratio of the surface area exposed to the fire to the mass of steel

**1.3.28  
fatigue**

damage caused by repeated fluctuations of stress leading to gradual cracking of a structural element

**1.3.29  
fatigue loading**

set of nominal loading events described by the distribution of the loads, their magnitudes and the numbers of applications of each nominal loading event

**1.3.30  
fatigue strength**

stress range defined in [Clause 11.6](#) for each detail category (see [Figures 11.6.1](#) and [11.6.2](#)) varying with the number of stress cycles

**1.3.31  
fire exposure condition**

extent of exposure of member surfaces to the fire

**1.3.31.1  
three-sided fire exposure condition**

steel member incorporated in or in contact with a concrete or masonry floor or wall

**1.3.31.2  
four-sided fire exposure condition**

steel member exposed to fire on all sides

**1.3.32  
fire protection system**

fire protection material and its method of attachment to the steel member

**1.3.33  
fire-resistance level**

**FRL**  
fire-resistance grading period for structural adequacy only, in minutes, which is required to be attained in the standard fire test

**1.3.34  
firm contact**

condition that exists between plies in a bolted connection where the plies are solidly seated against each other, but may not be in continuous contact

**1.3.35****friction-type connection**

connection effected using high-strength bolts tightened to induce a specified minimum bolt tension such that the resultant clamping action transfers the design shear forces at the serviceability limit state acting in the plane of the common contact surfaces by the friction developed between the contact surfaces

**1.3.36****full tensioning**

method of installing and tensioning a bolt in accordance with AS/NZS 5131

**1.3.37****geometrical slenderness ratio**

$l_e/r$

ratio taken as the effective length ( $l_e$ ), specified in [Clause 6.3.2](#), divided by the radius of gyration ( $r$ ) computed for the gross section about the relevant axis

**1.3.38****incomplete penetration butt weld**

butt weld in which the depth of penetration is less than the complete depth of the joint

**1.3.39****in-plane loading**

loading for which the design forces and bending moments are in the plane of the connection, so that the design action effects induced in the connection components are shear forces only

**1.3.40****lamellar tearing**

cracking in the parent metal adjacent to the weld and arising from weld stresses in the through thickness direction of the plate. The cracking occurs in a stepped configuration, associated with lamellar non-metallic inclusions in the plate, parallel to the fusion boundary

[SOURCE: AS 2812—2005, 8.043]

**1.3.41****length**

<of a compression member> actual length ( $l$ ) of an axially loaded compression member, taken as the length centre-to-centre of intersections with supporting members, or the cantilevered length in the case of a free-standing member

**1.3.42****limit state**

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

**1.3.43****load**

externally applied force

**1.3.44****miner's summation**

cumulative damage calculation based on the Palmgren-Miner summation or equivalent

**1.3.45****moment-resisting frame**

see AS 1170.4

**1.3.46****moment-resisting frame, intermediate**

see AS 1170.4

**1.3.47****moment-resisting frame, ordinary**

see AS 1170.4

**1.3.48****moment-resisting frame, special**

see AS 1170.4

**1.3.49****nominal action or load**

action or load, as specified in [Clause 3.2.1](#) or [3.2.2](#)

**1.3.50****nominal capacity**

capacity of a member or connection computed using the parameters specified in this Standard

**1.3.51****nominal loading event**

loading sequence for the structure or structural element

Note 1 to entry: One nominal loading event may produce one or more stress cycles depending on the type of load and the point in the structure under consideration.

**1.3.52****non-slip fasteners**

fasteners that do not allow slip to occur between connected plates or members at the serviceability limit state so that the original alignment and relative positions are maintained

**1.3.53****out-of-plane loading**

loading for which the design forces or bending moments result in design action effects normal to the plane of the connection

**1.3.54****period of structural adequacy****PSA**

<fire> time ( $t$ ), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test

**1.3.55****pin**

unthreaded fastener manufactured out of round bar

**1.3.56****plastic hinge**

yielding zone with inelastic rotation which forms in a member when the plastic moment is reached

**1.3.57****ply**

plate that forms part of a bolted connection

**1.3.58****prequalified weld preparation**

joint preparation prequalified in terms of AS/NZS 1554.1

**1.3.59****proof testing**

application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that one unit under test

**1.3.60****prototype**

<fire> test specimen representing a steel member and its fire protection system which is subjected to the standard fire test

**1.3.61****prototype testing**

application of test loads to one or more structures, sub-structures, members or connections to ascertain the structural characteristics of that class of structures, sub-structures, members or connections which are nominally identical to the units tested

**1.3.62****prying force**

additional tensile force developed as a result of the flexing of a connection component in a connection subjected to tensile force. External tension force reduces the contact pressure between the component and the base, and bending in part of the component develops a prying force near the edge of the connection component

**1.3.63****quenched and tempered steel**

high strength steel manufactured by heating, quenching, tempering and levelling steel plate

**1.3.64****segment**

<in a member subjected to bending> length between adjacent cross-sections which are fully or partially restrained, or the length between an unrestrained end and the adjacent cross-section which is fully or partially restrained

**1.3.65****serviceability limit state**

limit state of acceptable in-service condition

**1.3.66****shall**

indicates that a statement is mandatory

**1.3.67****should**

indicates a recommendation

**1.3.68****shear wall**

wall designed to resist lateral forces parallel to the plane of the wall

**1.3.69****S-N curve**

curve defining the limiting relationship between the number of stress cycles and stress range for a detail category

**1.3.70****snug tight**

tightness in the bolts in a bolted connection attained by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner to bring the plies into firm contact

**1.3.71****space frame**

see AS 1170.4

**1.3.72****splice**

connection between two parts of a structural member

**1.3.73****stability limit state**

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

**1.3.74****standard fire test**

fire-resistance test specified in AS 1530.4

**1.3.75****stickability**

ability of the fire protection system to remain in place as the member deflects under load during a fire test

**1.3.76****strength limit state**

limit state of collapse or loss of structural integrity

**1.3.77****stress cycle**

one cycle of stress defined by stress cycle counting

**1.3.78****stress cycle counting method**

any rational method used to identify individual stress cycles from the stress history

**1.3.79****stress range**

algebraic difference between two extremes of stress

**1.3.80****stress spectrum**

histogram of the stress cycles produced by a nominal loading event

**1.3.81****structural adequacy**

<fire> ability of the member exposed to the standard fire test to carry the test load specified in AS 1530.4

**1.3.82****structural ductility factor**

see AS 1170.4

**1.3.83****structural performance factor**

see AS 1170.4

**1.3.84****sway member**

one for which the transverse displacement of one end of the member relative to the other is not effectively prevented

**1.3.85****tensile strength**

minimum ultimate strength in tension specified for the grade of steel in the appropriate Australian Standard

**1.3.86****yield stress**

minimum yield stress in tension specified for the grade of steel in the appropriate Australian Standard

**1.4 Notation**

Symbols used in this Standard are listed below.



Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.

The dimensional units for length and stress in all expressions or equations are to be taken as millimetres (mm) and megapascals (MPa) respectively, unless specifically noted otherwise.

An asterisk "\*" placed after a symbol denotes a design action effect due to the design load for the strength limit state.

$A$	=	area of cross-section
$A_c$	=	minor diameter area of a bolt, as defined in AS 1275
$A_e$	=	effective sectional area of a hollow section in shear; <i>or</i> effective area of a compression member
$A_{ep}$	=	area of an end post
$A_{fc}$	=	flange area at critical cross-section
$A_{fg}$	=	gross area of a flange
$A_{fm}$	=	flange area at minimum cross-section; <i>or</i> lesser of the flange effective areas
$A_{fn}$	=	net area of a flange
$A_g$	=	gross area of a cross-section
$A_{gv}$	=	gross area subject to shear at rupture
$A_n$	=	net area of a cross-section
$A_{nt}$	=	net area subject to tension at rupture
$A_{nv}$	=	net area subject to shear at rupture
$A_o$	=	nominal plain shank area of a bolt
$A_p$	=	cross-sectional area of a pin
$A_s$	=	tensile stress area of a bolt as defined in AS 1275; <i>or</i> area of a stiffener or stiffeners in contact with a flange; <i>or</i> area of an intermediate web stiffener
$A_w$	=	gross sectional area of a web; <i>or</i> effective shear area of a plug or slot weld
$a_e$	=	minimum distance from the edge of a hole to the edge of a ply measured in the direction of the component of a force plus half the bolt diameter
$a_o$	=	length of unthreaded portion of the bolt shank contained within the grip
$a_t$	=	length of threaded portion of the bolt contained within the grip
$a_0, a_1$	=	out-of-square dimensions of flanges

$a_2, a_3$	=	diagonal dimensions of a box section
$b$	=	width; <i>or</i>
	=	lesser dimension of a web panel; <i>or</i>
	=	clear width of an element outstand from the face of a supporting plate element; <i>or</i>
	=	clear width of a supported element between faces of supporting plate elements
$b_b, b_{bf}, b_{bw}, b_o$	=	bearing widths defined in <a href="#">Clause 5.13</a>
$b_d$	=	distance from the stiff bearing to the end of the member
$b_e$	=	effective width of a plate element
$b_{es}$	=	stiffener outstand from the face of a web
$b_f$	=	width of a flange
$b_{fo}$	=	half the clear distance between the webs; <i>or</i>
	=	least of 3 dimensions defined in <a href="#">Clause 5.11.5.2</a>
$b_s$	=	stiff bearing length
$b_w$	=	web depth
$b_1, b_2$	=	greater and lesser leg lengths of an angle section
$C_3, C_4, C_{4r}$	=	factors given in <a href="#">Table H.3</a> and <a href="#">Clause H.5</a>
$c_h$	=	perpendicular distance to centroid of an angle section from the face of the loaded leg of the angle
$c_m$	=	factor for unequal end moments
$d$	=	depth of a section; <i>or</i>
	=	depth of preparation for incomplete penetration butt weld; <i>or</i>
	=	maximum cross-sectional dimension of a built-up compression member
$d_b$	=	lateral distance between centroids of the welds or fasteners connecting battens to main components
$d_c$	=	depth of a section at a critical cross-section
$d_e$	=	effective outside diameter of a circular hollow section; <i>or</i>
	=	factor defined in <a href="#">Appendix I</a>
$d_f$	=	diameter of a fastener (bolt or pin); <i>or</i>
	=	distance between flange centroids
$d_m$	=	depth of a section at minimum cross-section
$d_o$	=	overall section depth including out-of-square dimensions; <i>or</i>
	=	overall section depth of a segment; <i>or</i>

	=	outside diameter of a circular hollow section
$d_p$	=	clear transverse dimension of a web panel; <i>or</i>
	=	depth of deepest web panel in a length
$d_x, d_y$	=	distances of the extreme fibres from the neutral axes
$d_1$	=	clear depth between flanges ignoring fillets or welds
$d_2$	=	twice the clear distance from the neutral axis to the compression flange
$d_3, d_4$	=	depths of preparation for incomplete penetration butt welds
$d_5$	=	flat width of web of hollow sections
$E$	=	Young's modulus of elasticity
$E(T), E(20)$	=	$E$ at temperature $T$ , and 20 °C, respectively
$e$	=	eccentricity; <i>or</i>
	=	web off-centre dimension; <i>or</i>
	=	distance between an end plate and a load-bearing stiffener
$e_c, e_t$	=	eccentricities of compression and tension angles ( <a href="#">Clause 8.4.6</a> )
$F$	=	action in general, force or load
$F^*$	=	total design load on a member between supports
$F_n^*$	=	design force normal to a web panel
$F_p^*$	=	design force parallel to a web panel
$f_c$	=	fatigue strength corrected for thickness of material
$f_f$	=	uncorrected fatigue strength
$f_{rn}$	=	detail category reference fatigue strength at $n_r$ cycles — normal stress
$f_{rnc}$	=	corrected detail category reference fatigue strength — normal stress
$f_{rsc}$	=	corrected detail category reference fatigue strength — shear stress
$f_{rs}$	=	detail category reference fatigue strength at $n_r$ cycles — shear stress
$f_u$	=	tensile strength used in design
$f_{uc}$	=	minimum tensile strength of connection element
$f_{uf}$	=	minimum tensile strength of a bolt
$f_{up}$	=	tensile strength of a ply
$f_{uw}$	=	nominal tensile strength of weld metal
$f_y$	=	yield stress used in design
$f_{yc}$	=	yield stress of connection element

$f_y(T), f_y(20)$	=	yield stresses of steel at temperature $T$ , and 20 °C, respectively
$f_{yp}$	=	yield stress of a pin used in design
$f_{ys}$	=	yield stress of a stiffener used in design
$f_3$	=	detail category fatigue strength at constant amplitude fatigue limit
$f_{3c}$	=	corrected detail category fatigue strength at constant amplitude fatigue limit
$f_5$	=	detail category fatigue strength at cut-off limit
$f_{5c}$	=	corrected detail category fatigue strength at cut-off limit
$f^*$	=	design stress range
$f_i^*$	=	design stress range for loading event $i$
$f_{va}^*$	=	average design shear stress in a web
$f_{vm}^*$	=	maximum design shear stress in a web
$f_w^*$	=	equivalent design stress on a web panel ( <a href="#">Appendix I</a> )
$G$	=	shear modulus of elasticity; <i>or</i>
	=	nominal dead load
$G(T)$	=	shear modulus of steel at $T$ degrees Celsius
$h$	=	rectangular centroidal axis for angle parallel to the loaded leg
$h_b$	=	vertical distance between tops of beams
$h_e$	=	effective thickness of fire protection material
$h_i$	=	thickness of fire protection material
$h_s$	=	storey height
$I$	=	second moment of area of a cross-section
$I_{cy}$	=	second moment of area of compression flange about the section minor principal $y$ -axis
$I_m$	=	$I$ of the member under consideration
$I_r$	=	$I$ of a restraining member
$I_s$	=	$I$ of a pair of stiffeners or a single stiffener
$I_w$	=	warping constant for a cross-section
$I_x$	=	$I$ about the cross-section major principal $x$ -axis
$I_y$	=	$I$ about the cross-section minor principal $y$ -axis
$i$	=	number of loading event ( <a href="#">Section 11</a> )
$J$	=	torsion constant for a cross-section

$K$	=	$\sqrt{\left[ \pi^2 EI_w / (GJL^2) \right]}$
$K_d$	=	deflection amplification factor
$k$	=	coefficient used in <a href="#">Appendix J</a>
$k_b$	=	elastic buckling coefficient for a plate element
$k_{bo}$	=	basic value of $k_b$
$k_{bs}$	=	a factor to account for the effect of eccentricity on the block shear capacity
$k_e$	=	member effective length factor
$k_f$	=	form factor for members subject to axial compression
$k_h$	=	factor for different hole types
$k_l$	=	effective length factor for load height
$k_p$	=	factor for pin rotation
$k_r$	=	effective length factor for restraint against lateral rotation; <i>or</i>
	=	effective length factor for a restraining member; <i>or</i>
	=	reduction factor to account for the length of a bolted or welded lap splice connection
$k_{rd}$	=	reduction factor to account for the reduced ductility of grade 10.9 bolts when subjected to shear where threads intercept the shear plane
$k_s$	=	ratio used to calculate $\alpha_p$ and $\alpha_{pm}$
$k_{sm}$	=	exposed surface area to mass ratio
$k_t$	=	effective length factor for twist restraints; <i>or</i>
	=	correction factor for distribution of forces in a tension member
$k_v$	=	ratio of flat width of web ( $d_5$ ) to thickness ( $t$ ) of hollow section
$k_0$ - $k_6$	=	regression coefficients ( <a href="#">Section 12</a> )
$l$	=	span; <i>or</i>
	=	member length; <i>or</i>
	=	member length from centre to centre of its intersections with supporting members; <i>or</i>
	=	segment or sub-segment length
$l_b$	=	length between points of effective bracing or restraint
$l_c$	=	distance between adjacent column centres
$l_e$	=	effective length of a compression member; <i>or</i>
	=	effective length of a laterally unrestrained member

$\frac{l_e}{r}$	=	geometrical slenderness ratio
$\left(\frac{l_e}{r}\right)_{bn}$	=	slenderness ratio of a battened compression member about the axis normal to the plane of the battens
$\left(\frac{l_e}{r}\right)_{bp}$	=	slenderness ratio of a battened compression member about the axis parallel to the plane of the battens
$\left(\frac{l_e}{r}\right)_c$	=	slenderness ratio of the main component in a laced or battened compression member
$\left(\frac{l_e}{r}\right)_m$	=	slenderness ratio of the whole battened compression member
$l_j$	=	length of a bolted lap splice connection
$l_m$	=	length of the member under consideration
$l_r$	=	length of a restraining member; <i>or</i>
	=	length of a segment over which the cross-section is reduced
$l_s$	=	distance between points of effective lateral support
$l_w$	=	greatest internal dimension of an opening in a web; <i>or</i>
	=	length of a fillet weld in a welded lap splice connection
$l_z$	=	distance between partial or full torsional restraints
$M_b$	=	nominal member moment capacity
$M_{bx}$	=	$M_b$ about major principal x-axis
$M_{bxo}$	=	$M_{bx}$ for a uniform distribution of moment
$M_{cx}$	=	lesser of $M_{ix}$ and $M_{ox}$
$M_f$	=	nominal moment capacity of flanges alone
$M_i$	=	nominal in-plane member moment capacity
$M_{ix}$	=	$M_i$ about major principal x-axis
$M_{iy}$	=	$M_i$ about minor principal y-axis
$M_o$	=	nominal out-of-plane member moment capacity; <i>or</i>
	=	reference elastic buckling moment for a member subject to bending
$M_{oa}$	=	amended elastic buckling moment for a member subject to bending
$M_{ob}$	=	elastic buckling moment determined using an elastic buckling analysis
$M_{obr}$	=	$M_{ob}$ decreased for elastic torsional end restraint
$M_{oo}$	=	reference elastic buckling moment obtained using $l_e = l$

$M_{Os}$	=	$M_{Ob}$ for a segment, fully restrained at both ends, unrestrained against lateral rotation and loaded at shear centre
$M_{Ox}$	=	nominal out-of-plane member moment capacity about major principal $x$ -axis
$M_p$	=	nominal moment capacity of a pin
$M_{pr}$	=	nominal plastic moment capacity reduced for axial force
$M_{prx}$	=	$M_{pr}$ about major principal $x$ -axis
$M_{pry}$	=	$M_{pr}$ about minor principal $y$ -axis
$M_{rx}$	=	$M_s$ about major principal $x$ -axis reduced by axial force
$M_{ry}$	=	$M_s$ about minor principal $y$ -axis reduced by axial force
$M_s$	=	nominal section moment capacity
$M_{sx}$	=	$M_s$ about major principal $x$ -axis
$M_{sy}$	=	$M_s$ about minor principal $y$ -axis
$M_{tx}$	=	lesser of $M_{rx}$ and $M_{ox}$
$M_w$	=	nominal section moment capacity of a web panel
$M^*$	=	design bending moment
$M_e^*$	=	second-order or amplified end bending moment
$M_f^*$	=	design end bending moment
$M_{fb}^*$	=	braced component of $M_f^*$ obtained from a first-order elastic analysis of a frame with sway prevented
$M_{fs}^*$	=	sway component of $M_f^*$ obtained from $(M_f^* - M_{fb}^*)$
$M_h^*$	=	design bending moment on an angle, acting about the rectangular $h$ -axis parallel to the loaded leg
$M_m^*$	=	maximum calculated design bending moment along the length of a member or in a segment
$M_w^*$	=	design bending moment acting on a web panel
$M_x^*$	=	design bending moment about major principal $x$ -axis
$M_y^*$	=	design bending moment about minor principal $y$ -axis
$M_2^*, M_3^*, M_4^*$	=	design bending moments at quarter and mid points of a segment
$N_c$	=	nominal member capacity in axial compression
$N_{ch}$	=	$N_c$ for angle buckling about $h$ -axis, parallel to the loaded leg
$N_{cy}$	=	$N_c$ for member buckling about minor principal $y$ -axis



$N_{0l}$	=	$\frac{\pi^2 EI}{l^2}$
$N_{0lr}$	=	$\frac{\pi^2 EI}{l_r^2}$
$N_{0m}$	=	elastic buckling load
$N_{0mb}$	=	$N_{0m}$ for a braced member
$N_{0ms}$	=	$N_{0m}$ for a sway member
$N_{0z}$	=	nominal elastic torsional buckling capacity of a member
$N_s$	=	nominal section capacity of a compression member; <i>or</i> nominal section capacity for axial load
$N_t$	=	nominal section capacity in tension
$N_{tf}$	=	nominal tension capacity of a bolt
$N_{ti}$	=	minimum bolt tension at installation; <i>or</i> tension induced in a bolt during installation
$N_{wo}$	=	nominal axial load capacity of a web panel
$N^*$	=	design axial force, tensile or compressive
$N_r^*$	=	design axial force in a restraining member
$N_{tf}^*$	=	design tensile force on a bolt
$N_w^*$	=	design axial force acting on a web panel
$n$	=	number of specimens tested
$n_b$	=	number of parallel planes of battens
$n_{ei}$	=	number of effective interfaces
$n_i$	=	number of cycles of nominal loading event $i$
$n_n$	=	number of shear planes with threads intercepting the shear plane — bolted connections
$n_r$	=	reference number of stress cycles
$n_s$	=	number of shear planes
$n_{sc}$	=	number of stress cycles
$n_w$	=	number of webs
$n_x$	=	number of shear planes without threads intercepting the shear plane — bolted connections
$Q$	=	nominal live load

$Q^*$	=	design transverse force; <i>or</i>
	=	design live load
$R_b$	=	nominal bearing capacity of a web
$R_{bb}$	=	nominal bearing buckling capacity of a web
$R_{bs}$	=	nominal design capacity in block shear
$R_{by}$	=	nominal bearing yield capacity of a web
$R_{sb}$	=	nominal buckling capacity of a stiffened web
$R_{sy}$	=	nominal yield capacity of a stiffened web
$R_u$	=	nominal capacity
$R^*$	=	design bearing force; <i>or</i>
	=	design reaction
$R_{bs}^*$	=	design reaction
$R_w^*$	=	design bearing force or reaction on a web panel
$r$	=	radius of gyration
$r_{ext}$	=	outside radius of hollow section
$r_f$	=	ratio of design action on the member under design load for fire to the design capacity of the member at room temperature
$r_T$	=	ratio defined in <a href="#">Clause 5.6.1.1</a>
$r_s$	=	ratio defined in <a href="#">Clause 5.6.1.1</a>
$r_y$	=	radius of gyration about minor principal $y$ -axis
$S$	=	plastic section modulus
$S_p$	=	structural performance factor
$S^*$	=	design action effect
$s$	=	spacing of stiffeners; <i>or</i>
	=	width of a web panel
$s_b$	=	longitudinal centre-to-centre distance between battens
$s_g$	=	gauge of bolts
$s_p$	=	staggered pitch of bolts
$T$	=	steel temperature in degrees Celsius
$T_l$	=	limiting steel temperature in degrees Celsius
$t$	=	thickness; <i>or</i>

	=	element thickness; <i>or</i>
	=	thickness of thinner part joined; <i>or</i>
	=	wall thickness of a circular hollow section; <i>or</i>
	=	thickness of an angle section; <i>or</i>
	=	time
$t_f$	=	thickness of a flange; <i>or</i>
	=	thickness of the critical flange
$t_n$	=	thickness of a nut
$t_p$	=	thickness of a ply; <i>or</i>
	=	thickness of thinner ply connected; <i>or</i>
	=	thickness of a plate; <i>or</i>
	=	connecting plate thickness(es) at a pin
$t_s$	=	thickness of a stiffener
$t_t, t_{t1}, t_{t2}$	=	design throat thickness of a weld
$t_w$	=	thickness of a web or web panel
$t_w, t_{w1}, t_{w2}$	=	leg lengths of a fillet weld used to define the size of a fillet weld
$V_b$	=	nominal bearing capacity of a ply or a pin; <i>or</i>
	=	nominal shear buckling capacity of a web
$V_f$	=	nominal shear capacity of a bolt or pin — strength limit state
$V_{sf}$	=	nominal shear capacity of a bolt — serviceability limit state
$V_{si}$	=	measured slip-load at the <i>i</i> th bolt
$V_u$	=	nominal shear capacity of a web with a uniform shear stress distribution
$V_v$	=	nominal shear capacity of a web
$V_{vm}$	=	nominal web shear capacity in the presence of bending moment
$V_w$	=	nominal shear yield capacity of a web; <i>or</i>
	=	nominal shear capacity of a plug or slot weld
$V^*$	=	design shear force; <i>or</i>
	=	design horizontal storey shear force at column ends; <i>or</i>
	=	design transverse shear force
$V_b^*$	=	design bearing force on a ply at a bolt or pin location
$V_f^*$	=	design shear force on a bolt or a pin — strength limit state

$V_l^*$	=	design longitudinal shear force
$V_{sf}^*$	=	design shear force on a bolt — serviceability limit state
$V_w^*$	=	design shear force acting on a web panel; <i>or</i>
	=	design shear force on a plug or slot weld
$v_w$	=	nominal capacity of a fillet weld per unit length
$v_w^*$	=	design force per unit length on a fillet weld
$x$	=	major principal axis coordinate
$y$	=	minor principal axis coordinate
$y_L$	=	distance of the gravity loading below the centroid
$y_o$	=	coordinate of shear centre
$Z$	=	elastic section modulus
$Z_c$	=	$Z_e$ for a compact section
$Z_e$	=	effective section modulus
$Z_{we}$	=	elastic section modulus of a web panel
$\alpha$	=	angle between $x$ - and $h$ -axes for an angle section
$\alpha_a$	=	compression member factor, as defined in <a href="#">Clause 6.3.3</a>
$\alpha_b$	=	compression member section constant, as defined in <a href="#">Clause 6.3.3</a>
$\alpha_{bc}$	=	moment modification factor for bending and compression
$\alpha_c$	=	compression member slenderness reduction factor
$\alpha_d$	=	tension field coefficient for web shear buckling
$\alpha_f$	=	flange restraint factor for web shear buckling
$\alpha_l$ $\alpha_{lc}$ $\alpha_{mc}$	=	factors for bending defined in <a href="#">Clauses H.2</a> and <a href="#">H.3</a>
$\alpha_m$	=	moment modification factor for bending
$\alpha_p$	=	coefficient used to calculate the nominal bearing yield capacity ( $R_{by}$ ) for square and rectangular hollow sections to AS/NZS 1163
$\alpha_{pm}$	=	coefficient used to calculate $\alpha_p$
$\alpha_{ry}$	=	elastic stiffness of a flexural end restraint
$\alpha_{rz}$	=	elastic stiffness of a torsional end restraint
$\alpha_s$	=	slenderness reduction factor; <i>or</i>
	=	inverse of the slope of the S-N curve for fatigue
$\alpha_{sr}$	=	stability function multiplier

$\alpha_{st}$	=	reduction factor for members of varying cross-section
$\alpha_T$	=	coefficient of thermal expansion for steel
$\alpha_t$	=	factor for torsional end restraint defined in <a href="#">Clause 5.14.5</a>
$\alpha_v$	=	shear buckling coefficient for a web
$\alpha_w$	=	factor defined in <a href="#">Appendix I</a>
$\beta_e$	=	modifying factor to account for conditions at the far ends of beam members
$\beta_m$	=	ratio of smaller to larger bending moment at the ends of a member; <i>or</i> ratio of end moment to fixed end moment
$\beta_t$	=	measure of elastic stiffness of torsional end restraint used in <a href="#">Appendix H</a>
$\beta_{tf}$	=	thickness correction factor for fatigue
$\beta_x$	=	monosymmetry section constant
$\beta_w$	=	factor defined in <a href="#">Appendix I</a>
$\gamma$	=	index used in <a href="#">Clause 8.3.4</a> ; <i>or</i> factor for transverse stiffener arrangement in stiffened web ( <a href="#">Clause 5.15.3</a> )
$\gamma, \gamma_1, \gamma_2,$	=	ratios of compression member stiffness to end restraint stiffness used in <a href="#">Clause 4.6.3.3</a>
$\Delta$	=	deflection; <i>or</i> deviation from nominated dimension; <i>or</i> measured total extension of a bolt when tightened
$\Delta_{ct}$	=	mid-span deflection of a member resulting from transverse loading together with both end bending moments
$\Delta_{cw}$	=	mid-span deflection of a member resulting from transverse loading together with only those end bending moments which produce a mid-span deflection in the same direction as the transverse load
$\Delta_f$	=	out-of-flatness of a flange plate
$\Delta h_b$	=	deviation from $h_b$
$\Delta l_c$	=	deviation from $l_c$
$\Delta_s$	=	translational displacement of the top relative to the bottom for a storey height
$\Delta_v$	=	deviation from verticality of a web at a support
$\Delta_w$	=	out-of-flatness of a web
$\delta$	=	standard deviation
$\delta_b$	=	moment amplification factor for a braced member
$\delta_m$	=	moment amplification factor, taken as the greater of $\delta_b$ and $\delta_s$

$\delta_p$	=	moment amplification factor for plastic design
$\delta_s$	=	moment amplification factor for a sway member
$\zeta$	=	compression member factor defined in <a href="#">Clause 6.3.3</a>
$\eta$	=	compression member imperfection factor defined in <a href="#">Clause 6.3.3</a>
$\theta$	=	angle of preparation of an incomplete penetration butt weld
$\pi$	=	pi (approximately 3.14159)
$\lambda$	=	slenderness ratio; <i>or</i>
	=	elastic buckling load factor
$\lambda_c$	=	elastic buckling load factor
$\lambda_e$	=	plate element slenderness
$\lambda_{ed}$	=	plate element deformation slenderness limit
$\lambda_{ep}$	=	plate element plasticity slenderness limit
$\lambda_{ey}$	=	plate element yield slenderness limit
$\lambda_m$	=	elastic buckling load factor for a member
$\lambda_{ms}$	=	elastic buckling load factor for the storey under consideration
$\lambda_n$	=	modified compression member slenderness
$\lambda_s$	=	section slenderness
$\lambda_{sp}$	=	section plasticity slenderness limit
$\lambda_{sy}$	=	section yield slenderness limit
$\lambda_w, \lambda_{ew}$	=	values of $\lambda_e$ and $\lambda_{ey}$ for the web
$\mu$	=	slip factor
	=	structural ductility factor
$\mu_m$	=	mean value of the slip factor
$\mu_i$	=	individual test result from test for slip factor
$\nu$	=	Poisson's ratio
$\rho$	=	ratio of design axial force in a restraining member to the elastic buckling load for a member of length $l$ ( <a href="#">Appendix G</a> ); <i>or</i>
	=	$I_{cy}/I_y$
$\phi$	=	capacity factor
$\phi R_u$	=	design capacity

## 1.5 Use of alternative materials or methods

### 1.5.1 General

This Standard shall not be interpreted so as to prevent the use of materials or methods of design or construction not specifically referred to herein, provided that the requirements of [Section 3](#) are conformed to.

### 1.5.2 Existing structures

Where the strength or serviceability of an existing structure is to be evaluated, the general principles of this Standard may be applied. The actual properties of the materials in the structure shall be used.

NOTE 1 AS 5104 (ISO 2394) covers design by testing and may be utilized for assessment of existing structures.

NOTE 2 AS 5100.7 and AS 5100.8 cover assessment and rehabilitation of existing structures.

## 1.6 Design

### 1.6.1 Design data

The following design data shall be shown in the drawings:

- (a) The reference number and date of issue of applicable design Standards used.
- (b) The nominal loads.
- (c) The corrosion protection, if applicable.
- (d) The fire-resistance level, if applicable.
- (e) The steel grades used.

### 1.6.2 Design details

The drawings or construction specification, or both, for steel members and structures shall include, as appropriate, the following:

- (a) The size and designation of each member.
- (b) The number, sizes and categories of bolts used in the connections.
- (c) The sizes, types, strength and categories of welds used in the connections, together with the level of visual examination and other non-destructive examination required.
- (d) The sizes of the connection components.
- (e) The locations and details of planned joints, connections and splices.
- (f) Any constraint on construction assumed in the design.
- (g) The camber of any members.
- (h) Details of the construction category or categories required ([Clause 1.7.2](#)).
- (i) Details of the corrosion protection required ([Clause 3.5.6](#)).
- (j) Any requirement for architecturally exposed structural steelwork (AESS) ([Clause 1.7.3](#)).
- (k) Any specific tolerances if different to those specified in AS/NZS 5131 ([Clauses 14.4](#), [15.3](#)), including selection of the functional tolerance class if different to Class 1.



- (l) Any requirement for selection of materials to avoid lamellar tearing ([Clause 2.2.5](#)).
- (m) Any other requirements for fabrication, erection and operation.

## 1.7 Workmanship

### 1.7.1 General

All steel structures, designed in accordance with this Standard, shall be constructed to ensure that all the requirements of the design, as contained in the drawings and construction specification, are satisfied.

Requirements relating to fabrication and erection are a function of the nominated construction category.

### 1.7.2 Construction category

Four construction categories denoted CC1, CC2, CC3, CC4 are nominated in this Standard, for which requirements increase in strictness from CC1 to CC4.

NOTE 1 The requirements for CC4 are additional to CC3. CC4 applies to unusual or special structures for which it is expected that requirements additional to those for CC3 will be defined at a project specific level or by a particular organization, agency or business.

A construction category shall apply to either the whole structure or to parts of the structure or to specific details. A structure may therefore have multiple construction categories, provided that all parts of the structure are categorized. A detail or group of details shall be ascribed one construction category.

NOTE 2 The choice of construction category does not necessarily have to be the same for all components.

The construction category shall be nominated in the construction specification. If no construction category is specified, then CC2 shall apply.

The list of requirements for fabrication and erection and related to construction categories is specified in AS/NZS 5131.

NOTE 3 Guidance on the selection of construction category is given in [Appendix L](#). The choice of construction category is related to the importance factor, the service category and the fabrication category as set out in [Appendix L](#).

### 1.7.3 Architecturally exposed structural steelwork

Any requirement for architecturally exposed structural steelwork (AESS) shall be designated in the construction specification. The requirements for AESS are defined in AS/NZS 5131 based on five categories:

- (a) AESS 1 — Basic elements which require enhanced workmanship.
- (b) AESS 2 — Feature elements viewed at a distance greater than 6 m requiring good fabrication practices with enhanced treatment of weld, connection, fabrication detail, tolerances for gaps and copes.
- (c) AESS 3 — Feature elements viewed at a distance of less than 6 m requiring welds that are generally smooth but visible, some grind marks are acceptable. Tolerances are tighter than normal standards.
- (d) AESS 4 — Showcase or dominant elements used where the form is the only feature showing in an element. All welds which have ground and filled edges are ground smooth and true. All surfaces are sanded and filled to be smooth to the touch. Tolerances of fabricated items are more stringent.
- (e) AESS C — Custom elements.

#### 1.7.4 Fabrication and erection

The minimum requirements for fabrication and erection of structural steelwork shall be in accordance with [Sections 14](#) and [15](#) respectively.

NOTE Minimum required standards of workmanship ensure that the design assumptions remain valid.

## Section 2 Materials

### 2.1 Yield stress and tensile strength used in design

#### 2.1.1 Yield stress

The yield stress used in design ( $f_y$ ) shall not exceed that given in [Table 2.1](#).

#### 2.1.2 Tensile strength

The tensile strength used in design ( $f_u$ ) shall not exceed that given in [Table 2.1](#).

**Table 2.1 — Strengths of steels in accordance with AS/NZS 1163, AS/NZS 1594, AS/NZS 3678, AS/NZS 3679.1, AS/NZS 3679.2 ( see Note) and AS 3597**

Steel Standard	Form	Steel grade	Thickness of material (t) mm	Yield stress ( $f_y$ ) MPa	Tensile strength ( $f_u$ ) MPa
AS/NZS 1163 (See Note)	Hollow sections	C450	All	450	500
		C350	All	350	430
		C250	All	250	320
AS/NZS 1594	Plate, strip, sheet floorplate	HA400	All	380	460
		HW350	All	340	450
		HA350	All	350	430
		HA300/1 HU300/1	All	300	430
		HA300 HU300	All	300	400
		HA250 HU250	All	250	350
		HA200	All	200	300
	Plate and strip	XF500	$t \leq 8$	480	570
		XF400	$t \leq 8$	380	460
		XF300	All	300	440
AS/NZS 3678 (See requirement and Note below)	Plate and floorplate	450	$t \leq 20$	450	520
		450	$20 < t \leq 32$	420	500
		450	$32 < t \leq 50$	400	500
		400	$t \leq 12$	400	480
		400	$12 < t \leq 20$	380	480
		400	$20 < t \leq 80$	360	480
		350	$t \leq 12$	360	450
		350	$12 < t \leq 20$	350	450
AS/NZS 3678 (See Note)	Plate and floorplate	350	$80 < t \leq 150$	330	450
		WR350	$t \leq 50$	340	450
		300	$t \leq 8$	320	430
		300	$8 < t \leq 12$	310	430

Table 2.1 (continued)

Steel Standard	Form	Steel grade	Thickness of material (t) mm	Yield stress (f <sub>y</sub> ) MPa	Tensile strength (f <sub>u</sub> ) MPa	
		300	12 < t ≤ 20	300	430	
		300	20 < t ≤ 50	280	430	
		300	50 < t ≤ 80	270	430	
		300	80 < t ≤ 150	260	430	
		250	t ≤ 8	280	410	
		250	8 < t ≤ 12	260	410	
		250	12 < t ≤ 50	250	410	
		250	50 < t ≤ 80	240	410	
		250	80 < t ≤ 150	230	410	
		200	t ≤ 12	200	300	
AS/NZS 3679.1 (see Note)	Flats and sections	350	t ≤ 11	360	480	
		350	11 < t < 40	340	480	
		350	t ≥ 40	330	480	
		300	t < 11	320	440	
		300	11 ≤ t ≤ 17	300	440	
		300	t > 17	280	440	
	Hexagons, rounds and squares	350	t ≤ 50	340	480	
		350	50 < t < 100	330	480	
			350	t ≥ 100	320	480
			300	t ≤ 50	300	440
			300	50 < t < 100	290	440
			300	t ≥ 100	280	440
AS 3597	Plate	500	5 ≤ t ≤ 110	500	590	
		600	5 ≤ t ≤ 110	600	690	
		700	t ≤ 5	650	750	
		700	5 < t ≤ 65	690	790	
		700	65 < t ≤ 110	620	720	

Welded I-sections in accordance with AS/NZS 3679.2 are manufactured from hot-rolled structural steel plates in accordance with AS/NZS 3678, so the values listed for steel grades to AS/NZS 3678 shall be used for welded I-sections to AS/NZS 3679.2.

NOTE AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 1163 all contain, within each grade, a variety of impact grades not individually listed in the Table. All impact tested grades within the one grade have the same yield stress and tensile strength as the grade listed.

## 2.2 Structural steel

### 2.2.1 Australian Standards

Except as otherwise permitted in [Clause 2.2.3](#), all structural steel covered by the scope of this Standard shall, before fabrication, be in accordance with the requirements of the following Standards, as appropriate:

AS 3597, *Structural and pressure vessel steel—Quenched and tempered plate*

AS/NZS 1163, *Cold-formed structural steel hollow sections*

AS/NZS 1594, *Hot-rolled steel flat products*

AS/NZS 3678, *Structural steel—Hot-rolled plates, floorplates and slabs*

AS/NZS 3679.1, *Structural steel, Part 1: Hot-rolled bars and sections*

AS/NZS 3679.2, *Structural steel, Part 2: Welded I sections*

### 2.2.2 Acceptance of steels

Test reports or test certificates that conform to the minimum requirements of the appropriate Standard listed in [Clause 2.2.1](#) shall constitute sufficient evidence of conformance of the steel to the Standards listed in [Clause 2.2.1](#).

NOTE Requirements for test reports or test certificates are provided in the relevant Standards listed in [Clause 2.2.1](#).

### 2.2.3 Unidentified steel

If unidentified steel is used, it shall be free from surface imperfections, and shall be used only where the particular physical properties of the steel and its weldability shall not adversely affect the strength and serviceability of the structure. Unless a full test in accordance with AS 1391 is made, the yield stress of the steel used in design ( $f_y$ ) shall be taken as not exceeding 170 MPa, and the tensile strength used in design ( $f_u$ ) shall be taken as not exceeding 300 MPa.

### 2.2.4 Properties of steel

The following properties shall be used for all grades of steel for design purposes:

- |     |   |  |
|-----|---|--|
| (a) | Modulus of elasticity ( $E$ )                   | $200 \times 10^3$ MPa.                 |
| (b) | Shear modulus of elasticity ( $G$ )             | $80 \times 10^3$ MPa.                  |
| (c) | Poisson's ratio ( $\nu$ )                       | 0.25.                                  |
| (d) | Coefficient of thermal expansion ( $\alpha_T$ ) | $11.7 \times 10^{-6}/^\circ\text{C}$ . |

### 2.2.5 Through-thickness deformation properties

The specified through-thickness deformation properties are necessary to guarantee adequate deformation capacity perpendicular to the surface to provide ductility and toughness against lamellar tearing. For product in accordance with AS/NZS 3678, the minimum values for reduction of area for the specified quality class should be in accordance with the relevant requirements of AS/NZS 3678 and as given in [Table 2.2.5](#). For flat products, the minimum values for reduction of area should apply to the whole product.

**Table 2.2.5 — Minimum reduction in area**

Grade suffix	%	Range of $Z_{Ed}$ (see Notes)
Z15	15	$11 \leq Z_{Ed} \leq 20$
Z25	25	$21 \leq Z_{Ed} \leq 30$
Z35	35	$> 30$
NOTE 1 See also <a href="#">Clause 3.8</a> and <a href="#">Appendix M</a> .		
NOTE 2 Z quality steel is not required where $Z_{Ed}$ is $\leq 10$ or where steel thickness is 16 mm or less.		

See [Clause 3.8](#) for requirements to address lamellar tearing.

## 2.3 Fasteners

### 2.3.1 Steel bolts, nuts and washers

Steel bolts, nuts and washers shall be in accordance with the following Standards, as appropriate:

AS 1110, *ISO metric hexagon bolts and screws—Product grades A and B (series)*

AS 1111, *ISO metric hexagon bolts and screws—Product grade C (series)*

AS 1112, *ISO metric hexagon nuts (series)*

AS 1237.1, *Plain washers for metric bolts, screws and nuts for general purposes — Part 1: General plan*

AS/NZS 1252.1, *High strength steel fastener assemblies for structural engineering — Bolts, nuts and washers — Part 1: Technical requirements*

AS/NZS 1559, *Hot-dip galvanized steel bolts with associated nuts and washers for tower construction*

NOTE 1 AS/NZS 1559 has been written specifically for tower design and may not be suitable for use in all structures.

Test certificates that state that the bolts, nuts and washers conform to all the provisions of the appropriate Standard listed in this Clause shall constitute sufficient evidence of conformance to the appropriate Standard.

NOTE 2 Requirements for test reports or test certificates are provided in the relevant Standards listed in this Clause.

Where the Standards listed in this Clause do not specify requirements for laboratories, the laboratories that perform tests outlined in this Standard shall meet the requirements of AS ISO/IEC 17025.

NOTE 3 By way of example, Accreditation bodies which are signatories to the International Laboratory Accreditation Cooperation (ILAC) Mutual Recognition Arrangement (MRA) for testing laboratories may be able to offer accreditation against the requirements of AS ISO/IEC 17025. A listing of ILAC signatories is available from the ILAC website ([www.ilac.org](http://www.ilac.org)). In Australia, the National Association of Testing Authorities (NATA) is a signatory to the ILAC MRA.

Acceptable bolts and associated bolting categories are specified in [Table 9.2.1](#).

### 2.3.2 Equivalent high strength fasteners

The use of other high strength fasteners having special features in lieu of bolts to AS/NZS 1252.1 shall be permitted provided that evidence of their equivalence to high strength bolts in accordance with AS/NZS 1252.1 and installation in accordance with this Standard is available.

Equivalent fasteners shall meet the following requirements:

- (a) The chemical composition and mechanical properties of equivalent fasteners shall be in accordance with AS/NZS 1252.1 for the relevant bolt, nut and washer components.
- (b) The body diameter, head or nut bearing areas, or their equivalents, of equivalent fasteners shall not be less than those provided by a bolt and nut in accordance with AS/NZS 1252.1 of the same nominal dimensions. Equivalent fasteners may differ in other dimensions from those specified in AS/NZS 1252.1.
- (c) The method of tensioning and the inspection procedure for equivalent fasteners may differ in detail from those specified in [Clause 15.2.2.2](#), provided that the minimum fastener tension is not less than the minimum bolt tension given in [Table 15.2.2.2](#) and that the tensioning procedure is able to be checked.

### 2.3.3 Welds

Welding shall be undertaken in accordance with AS/NZS 5131.

### 2.3.4 Welded studs

Welded studs shall conform to, and shall be installed in accordance with AS/NZS 1554.2. The weld collars of welded studs in non-prequalified stud applications shall be in accordance with the requirements of ISO 13918.

### 2.3.5 Explosive fasteners

All explosive fasteners shall conform to, and shall be installed in accordance with AS/NZS 1873.

### 2.3.6 Anchor bolts

Anchor bolts shall conform to either the bolt Standards of [Clause 2.3.1](#) or shall be manufactured from rods in accordance with the steel Standards of [Clause 2.2.1](#) provided that the threads are in accordance with AS 1275.

### 2.3.7 Mechanical and chemical anchors

All mechanical and chemical fasteners shall be designed and specified in the construction specification in accordance with AS 5216.

## 2.4 Steel castings

All steel castings shall be in accordance with AS 2074.

## Section 3 General design requirements

### 3.1 Design

#### 3.1.1 Aim

The aim of structural design is to provide a structure which is stable, has adequate strength, is serviceable and durable, and which satisfies other objectives such as economy and ease of construction.

A structure is stable if it does not overturn, tilt or slide throughout its intended life.

A structure has adequate strength and is serviceable if the probabilities of structural failure and of loss of serviceability throughout its intended life are acceptably low.

A structure is durable if it withstands the expected wear and deterioration throughout its intended life without the need for undue maintenance.

#### 3.1.2 Requirements

The structure and its component members and connections shall satisfy the design requirements for stability, strength, serviceability, brittle fracture, lamellar tearing, fatigue, fire and earthquake in accordance with the procedures given in this Standard.

### 3.2 Loads and other actions

#### 3.2.1 Loads

The design of a structure for the stability, strength and serviceability limit states shall account for the action effects directly arising from the following loads:

- (a) Dead, live, wind, snow, ice and earthquake loads specified in AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3 and AS 1170.4.
- (b) For the design of cranes, any relevant loads specified in AS 1418 series.
- (c) For the design of fixed platforms, walkways, stairways and ladders, any relevant loads specified in AS 1657.
- (d) For the design of lifts, any relevant loads specified in AS 1735 series.
- (e) Other specific loads, as required.

NOTE 1 For the design of bridges, loads specified in AS 5100.2 should be used.

NOTE 2 For multi-storey building structures, see also [Clause 3.2.4](#).

#### 3.2.2 Other actions

Any action which may significantly affect the stability, strength or serviceability of the structure, including the following, shall be taken into account:

- (a) Foundation movements.
- (b) Temperature changes and gradients.
- (c) Axial shortening.
- (d) Dynamic effects.
- (e) Construction loading.



### 3.2.3 Design load combinations

The design load combinations for the stability, strength and serviceability limit states shall be those specified in AS/NZS 1170.0.

NOTE For the design of bridges, load combinations specified in AS 5100.2 should be used.

### 3.2.4 Notional horizontal forces

For multi-storey building structures only, notional horizontal forces, each equal to 0.002 times the total design vertical loads applied at a floor level, shall be applied at that floor level. These notional horizontal forces shall act in conjunction with only the design dead and live loads from AS/NZS 1170.1 for the strength and serviceability limit states. These notional horizontal forces shall not be included for the stability limit state.

### 3.2.5 Structural robustness

All steel structures, including members and connection components, shall conform to the structural robustness requirements of AS/NZS 1170.0.

## 3.3 Stability limit state

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

- (a) The loads determined in accordance with [Clause 3.2](#) shall be subdivided into the components tending to cause instability and the components tending to resist instability.
- (b) The design action effect ( $S^*$ ) shall be calculated from the components of the loads tending to cause instability, combined in accordance with the load combinations for the strength limit state specified in AS/NZS 1170.0.
- (c) The design resistance effect shall be calculated as 0.9 times the part of the dead load tending to resist the instability plus the design capacity ( $\phi R_u$ ) of any elements contributing towards resisting the instability, where  $\phi$  is a capacity factor which shall not exceed the appropriate value given in [Table 3.4](#).
- (d) The whole or part of the structure shall be proportioned so that the design resistance effect is not less than the design action effect.

## 3.4 Strength limit state

The structure and its component members and connections shall be designed for the strength limit state as follows:

- (a) The loads and actions shall be determined in accordance with [Clauses 3.2.1](#) and [3.2.2](#), and the strength limit state design loads shall be determined in accordance with [Clauses 3.2.3](#) and [3.2.4](#).
- (b) The design action effects ( $S^*$ ) resulting from the strength limit state design loads shall be determined by an analysis in accordance with [Section 4](#).
- (c) The design capacity ( $\phi R_u$ ) shall be determined from the nominal capacity ( $R_u$ ) determined from [Sections 5](#) to [9](#), where the capacity factor ( $\phi$ ) shall not exceed the appropriate value given in [Table 3.4](#).
- (d) All members and connections shall be proportioned so that the design capacity ( $\phi R_u$ ) is not less than the design action effect ( $S^*$ ), i.e. —

$$S^* \leq \phi R_u$$

**Table 3.4 — Capacity factors ( $\phi$ ) for strength limit states**

Design capacity for	Clauses	Capacity factor ( $\phi$ )	
		SP Category	GP Category
Member subject to bending			
— full lateral support	<a href="#">5.1</a> , <a href="#">5.2</a> and <a href="#">5.3</a>	0.90	
— segment without full lateral support	<a href="#">5.1</a> and <a href="#">5.6</a>	0.90	
— web in shear	<a href="#">5.11</a> and <a href="#">5.12</a>	0.90	
— web in bearing	<a href="#">5.13</a>	0.90	
— stiffener	<a href="#">5.14</a> , <a href="#">5.15</a> and <a href="#">5.16</a>	0.90	
Member subject to axial compression			
— section capacity	<a href="#">6.1</a> and <a href="#">6.2</a>	0.90	
— member capacity	<a href="#">6.1</a> and <a href="#">6.3</a>	0.90	
Member subject to axial tension	<a href="#">7.1</a> and <a href="#">7.2</a>	0.90	
Member subject to combined actions			
— section capacity	<a href="#">8.3</a>	0.90	
— member capacity	<a href="#">8.4</a>	0.90	
Connection component other than a bolt, pin or weld	<a href="#">9.1.9(a)</a> , <a href="#">(b)</a> , <a href="#">(c)</a> , and <a href="#">(d)</a> <a href="#">9.1.9(e)</a>	0.90 0.75	
Bolted connection			
— bolt in shear	<a href="#">9.2.2.1</a>	0.80	
— bolt in tension	<a href="#">9.2.2.2</a>	0.80	
— bolt subject to combined shear and tension	<a href="#">9.2.2.3</a>	0.80	
— ply in bearing	<a href="#">9.2.2.4</a>	0.90	
— bolt group	<a href="#">9.3</a>	0.80	
Pin connection			
— pin in shear	<a href="#">9.4.1</a>	0.80	
— pin in bearing	<a href="#">9.4.2</a>	0.80	
— pin in bending	<a href="#">9.4.3</a>	0.80	
— ply in bearing	<a href="#">9.4.4</a>	0.90	
Welded connection			
— complete penetration butt weld	<a href="#">9.6.2.7</a>	0.90	0.60
— longitudinal fillet weld in RHS ( $t < 3$ mm)	<a href="#">9.6.3.10</a>	0.70	—
— other fillet weld and incomplete penetration butt weld	<a href="#">9.6.3.10</a>	0.80	0.60
— plug or slot weld	<a href="#">9.6.4</a>	0.80	0.60
— weld group	<a href="#">9.7</a>	0.80	0.60

## 3.5 Serviceability limit state

### 3.5.1 General

The structure and its components shall be designed for the serviceability limit state by controlling or limiting deflection, vibration, bolt slip and corrosion, in accordance with the relevant requirements of [Clauses 3.5.2](#) to [3.5.6](#).

### 3.5.2 Method

The structure and its components shall be designed for the serviceability limit state as follows:

- (a) The loads and other actions shall be determined in accordance with [Clauses 3.2.1](#) and [3.2.2](#), and the serviceability limit state design loads shall be determined in accordance with [Clauses 3.2.3](#) and [3.2.4](#).
- (b) Deflections due to the serviceability limit state design loads shall be determined by the first-order elastic analysis method of [Clause 4.4.2.1](#) with all amplification factors taken as unity. Deflections shall be in accordance with [Clause 3.5.3](#).
- (c) Vibration behaviour shall be assessed in accordance with [Clause 3.5.4](#).
- (d) Bolt slip shall be limited, where required, in accordance with [Clause 3.5.5](#).
- (e) Corrosion protection shall be provided in accordance with [Clause 3.5.6](#).

### 3.5.3 Deflection limits

The deflection limits for the serviceability limit state shall be appropriate to the structure and its intended use, the nature of the loading, and the elements supported by it.

NOTE Suggested deflection limits may be found in [Appendix B](#).

### 3.5.4 Vibration of beams

Beams which support floors or machinery shall be checked to ensure that the vibrations induced by machinery, or vehicular or pedestrian traffic do not adversely affect the serviceability of the structure.

Where there is a likelihood of a structure being subjected to vibration from causes such as wind forces or machinery, measures shall be taken to prevent discomfort or alarm, damage to the structure, or interference with its proper function.

NOTE AS 2670 gives guidance for the evaluation of human exposure to whole-body vibrations of the type likely to be transmitted by structures.

### 3.5.5 Bolt serviceability limit state

In a connection, where slip under the serviceability design loads shall be avoided, the fasteners shall be selected in accordance with [Clause 9.1.6](#).

For a friction-type connection which is subject to shear force in the plane of the interfaces, and for which slip under serviceability loads shall be avoided, the capacity factor ( $\phi$ ) shall be taken as 0.7 and the bolts shall be designed in accordance with [Clause 9.2.3](#).

### 3.5.6 Corrosion protection

Where steelwork in a structure is to be exposed to a corrosive environment, the steelwork shall be given protection against corrosion. The degree of protection to be employed shall be determined based on the use of the structure, its maintenance, and the climatic or other local conditions.

NOTE 1 Guidance on selection of corrosion protection requirements is included in [Appendix C](#).

NOTE 2 Requirements for surface preparation and application of corrosion protection are defined in AS/NZS 5131.

NOTE 3 The basis for the design of components contained in this Standard makes no allowance for any loss of material due to corrosion.

NOTE 4 A corrosion allowance is not required for structural steel in corrosivity categories C1 and C2 where the coatings are appropriately specified, applied and maintained to AS 2312.1 or the structural steel is hot dip galvanized to AS/NZS 4680 with design details from AS/NZS 2312.2. For corrosivity categories C3, C4, C5 and CX, the selection of a coating system and the maintenance strategy should be the subject of professional advice.

NOTE 5 An estimate of the durability of a coating system can be obtained from AS 2312.1 or, where the structure or elements are hot dip galvanized, from AS/NZS 2312.2.

NOTE 6 The application to weathering steel is beyond the scope of this Standard. For design requirements for weathering steel refer to AS/NZS 5100.6.

### 3.6 Strength and serviceability limit states by load testing

Notwithstanding the requirements of [Clause 3.4](#) or [3.5](#), a structure or a component member or connection may be designed for the strength or serviceability limit state or both, by load-testing in accordance with [Section 17](#). If this alternative procedure is adopted, the requirements of [Clauses 3.7](#) to [3.11](#) shall also apply.

### 3.7 Brittle fracture

In order to avoid failure by brittle fracture, the selection of the parent material shall be made in accordance with [Section 10](#).

### 3.8 Lamellar tearing

Joints whose function is to transmit stress normal to the surface of the connected part, especially when the thickness of the branch member or the required weld size is 20 mm or greater, require special attention during design, parent material selection and detailing. Joint details shall minimize stress intensity on parent materials subject to stress in the through-thickness direction. Weld sizes that are larger than necessary shall not be specified.

The suitability of material for through-thickness requirements should be based on the through-thickness ductility quality criterion as specified in AS/NZS 3678, which is expressed in terms of quality classes identified by Z-values. The choice of materials depends on requirements affected by the design of welded connections and the fabrication technique. [Appendix M](#) provides guidance for the choice of Z-qualities to avoid lamellar tearing in welded connections subject to tension stresses in the through-thickness direction.

NOTE 1 Z quality steel is not required where  $Z_{Ed} \leq 10$  or where steel thickness is 16 mm or less.

NOTE 2 Refer also to [Clause 2.2.5](#) and [Appendix M](#).

### 3.9 Fatigue

For structures and structural elements subject to loadings which could lead to fatigue, the fatigue strength shall be determined in accordance with [Section 11](#).

### 3.10 Fire

The structure, its component members and connections shall be designed in accordance with [Section 12](#).

### 3.11 Earthquake

The structure, its component members, connections and any non-structural components shall be designed for earthquake loads in accordance with AS 1170.4 and [Section 13](#).

### 3.12 Other design requirements

Requirements other than those listed in [Clause 3.1.2](#), such as differential settlement, progressive collapse and any special performance requirements shall be taken into account in the design of the structure in accordance with the principles of this Standard.

The design of bridges for loads resulting from floods or collision shall be carried out in accordance with AS 5100.2.

### 3.13 Reliability management

For the application of AS/NZS 5131, the selection of construction category should be made in accordance with [Appendix L](#) of this Standard.

If different levels of reliability are specified, these levels should be achieved by an appropriate choice of quality management in design together with fabrication and erection, according to AS/NZS 5131 and AS 5104 (ISO 2394).

## Section 4 Methods of structural analysis

### 4.1 Methods of determining action effects

#### 4.1.1 General

For the purpose of complying with the requirements for the limit states of stability, strength and serviceability specified in [Section 3](#), the design action effects in a structure and its members and connections caused by the design loads shall be determined by structural analysis using the assumptions of [Clauses 4.2](#) and [4.3](#) and one of the methods of —

- (a) elastic analysis, in accordance with [Clause 4.4](#);
- (b) plastic analysis, in accordance with [Clause 4.5](#); or
- (c) advanced analysis, in accordance with [Appendix D](#).

The design action effects for earthquake loads shall be obtained using either the equivalent static analysis of Section 6 of AS 1170.4 or the dynamic analysis of Section 7 of AS 1170.4.

#### 4.1.2 Definitions

For the purpose of this Section, the definitions below apply:

*Braced member* — one for which the transverse displacement of one end of the member relative to the other is effectively prevented. This applies to triangulated frames and trusses or to frames where in-plane stiffness is provided by diagonal bracing, or by shear walls, or by floor slabs or roof decks secured horizontally to walls or to bracing systems parallel to the plane of buckling of the member.

*Sway member* — one for which the transverse displacement of one end of the member relative to the other is not effectively prevented. Such members occur in structures which depend on flexural action to limit the sway.

### 4.2 Forms of construction assumed for structural analysis

#### 4.2.1 General

The distribution of the design action effects throughout the members and connections of a structure shall be determined by assuming one or a combination of the following forms of construction:

- (a) Rigid.
- (b) Semi-rigid.
- (c) Simple.

#### 4.2.2 Rigid construction

For rigid construction, the connections shall be assumed to have sufficient rigidity to hold the original angles between the members unchanged.

#### 4.2.3 Semi-rigid construction

For semi-rigid construction, the connections may not have sufficient rigidity to hold the original angles between the members unchanged, but shall be assumed to have the capacity to furnish a reliable known degree of flexural restraint.

The relationship between the degree of flexural restraint and the level of the load effects shall be established by methods based on test results.

#### 4.2.4 Simple construction

For simple construction, the connections at the ends of members shall be assumed not to develop bending moments.

#### 4.2.5 Design of connections

The design of all connections shall be consistent with the form of construction, and the behaviour of the connections shall not adversely affect any other part of the structure beyond what is allowed for in design. Connections shall be designed in accordance with [Section 9](#).

### 4.3 Assumptions for analysis

#### 4.3.1 General

The structure shall be analysed in its entirety, except as follows:

- (a) Regular building structures may be analysed as a series of parallel two-dimensional substructures, the analysis being carried out in each of two directions at right angles, except when there is significant load redistribution between the substructures.
- (b) For vertical loading in a multi-storey building structure provided with bracing or shear walls to resist all lateral forces, each level thereof together with the columns immediately above and below may be treated as substructures, the columns being assumed fixed at the ends remote from the level under consideration.

Where floor beams in a multi-storey building structure are treated as substructures, the bending moment at a support may be determined on the basis that the floor is fixed at the support one span away, provided that the floor beam continues beyond that point.

#### 4.3.2 Span length

The span length of a flexural member shall be taken as the distance centre-to-centre of the supports.

#### 4.3.3 Arrangements of live loads for buildings

For building structures, the arrangements of live loads considered in the analysis shall include at least the following:

- (a) Where the loading pattern is fixed, the arrangement concerned.
- (b) Where the nominal live load ( $Q$ ) is variable and not greater than three-quarters of the nominal dead load ( $G$ ), the design live load ( $Q^*$ ) on all spans.
- (c) Where the nominal live load ( $Q$ ) is variable and exceeds three-quarters of the nominal dead load ( $G$ ), arrangements for the floor under consideration consisting of —
  - (i) the design live load ( $Q^*$ ) on alternate spans;
  - (ii) the design live load ( $Q^*$ ) on two adjacent spans; and
  - (iii) the design live load ( $Q^*$ ) on all spans.

#### 4.3.4 Simple construction

Bending members may be assumed to have their ends connected for shear only and to be free to rotate. In triangulated structures, axial forces may be determined by assuming that all members are pin connected.

A beam reaction or a similar load on a column shall be taken as acting at a minimum distance of 100 mm from the face of the column towards the span or at the centre of bearing, whichever gives the greater eccentricity, except that for a column cap, the load shall be taken as acting at the face of the column, or edge of packing if used, towards the span.

For a continuous column, the design bending moment ( $M^*$ ) due to eccentricity of loading at any one floor or horizontal frame level shall be taken as —

- (a) ineffective at the floor or frame levels above and below that floor; and
- (b) divided between the column lengths above and below that floor or frame level in proportion to the values of  $l/l$  of the column lengths.

## 4.4 Elastic analysis

### 4.4.1 General

#### 4.4.1.1 Assumptions

Individual members shall be assumed to remain elastic under the action of the design loads for all limit states.

The effect of haunching or any variation of the cross-section along the axis of a member shall be taken into account in the determination of the member stiffness.

#### 4.4.1.2 Second-order effects

The analysis shall allow for the effects of the design loads acting on the structure and its members in their displaced and deformed configuration. These second-order effects shall be taken into account by using either —

- (a) a first-order elastic analysis with moment amplification in accordance with [Clause 4.4.2](#), provided the moment amplification factors ( $\delta_b$ ) or ( $\delta_s$ ) are not greater than 1.4; or
- (b) a second-order elastic analysis in accordance with [Appendix E](#).

### 4.4.2 First-order elastic analysis

#### 4.4.2.1 General

In a first-order elastic analysis, changes in the geometry are not accounted for, and changes in the effective stiffnesses of the members due to axial force are neglected. The effects of these on the first-order bending moments shall be allowed for by using one of the methods of moment amplification of [Clause 4.4.2.2](#) or [Clause 4.4.2.3](#) as appropriate, except that where the moment amplification factor ( $\delta_b$ ) or ( $\delta_s$ ), calculated in accordance with [Clause 4.4.2.2](#) or [Clause 4.4.2.3](#) is greater than 1.4, a second-order elastic analysis in accordance with [Appendix E](#) shall be carried out.

The maximum calculated bending moment ( $M_m^*$ ) shall be taken as the maximum bending moment along the length of a member obtained by superposition of the simple beam bending moments resulting from any transverse loading on the member with the end bending moments determined by the analysis.



#### 4.4.2.2 Moment amplification for a braced member

For a braced member with zero axial force or a braced member subject to axial tension, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = M_m^*$$

For a braced member with a design axial compressive force ( $N^*$ ) as determined by the analysis, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where  $\delta_b$  is a moment amplification factor for a braced member calculated as follows:

$$\delta_b = \frac{c_m}{1 - \left( \frac{N^*}{N_{omb}} \right)} \geq 1$$

and  $N_{omb}$  is the elastic buckling load, determined in accordance with [Clause 4.6.2](#), for the braced member buckling about the same axis as that about which the design bending moment ( $M^*$ ) is applied.

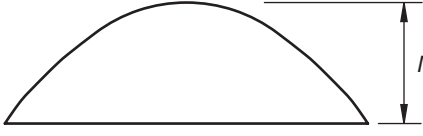
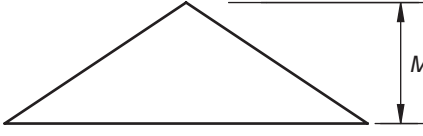
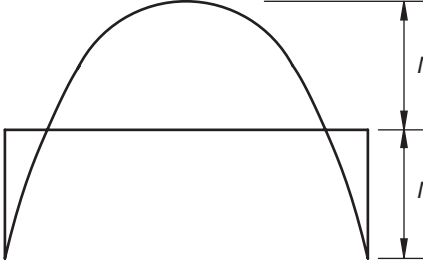
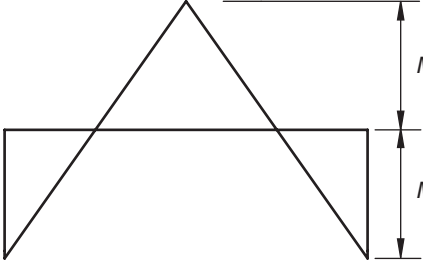
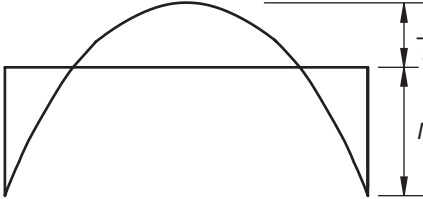
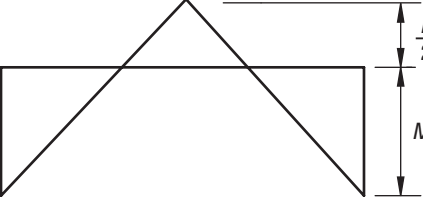
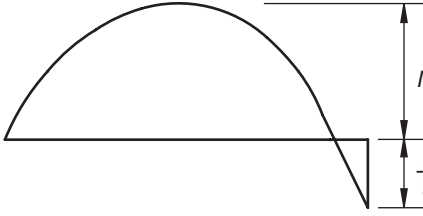
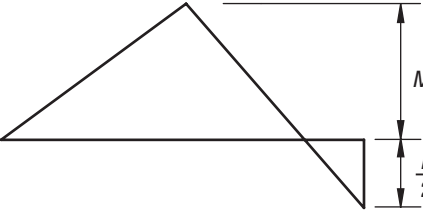
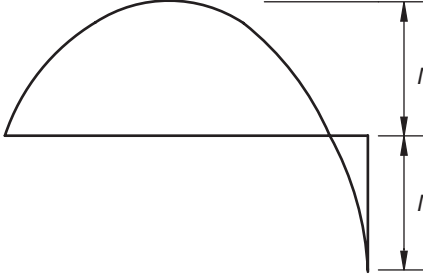
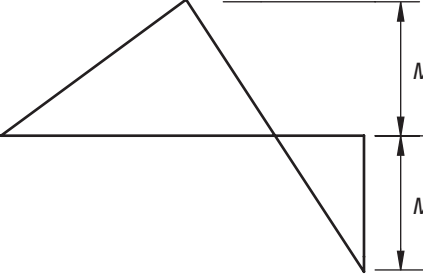
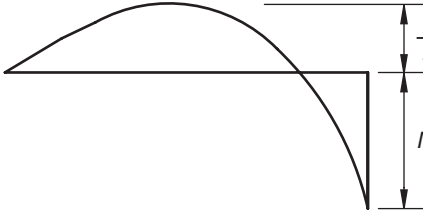
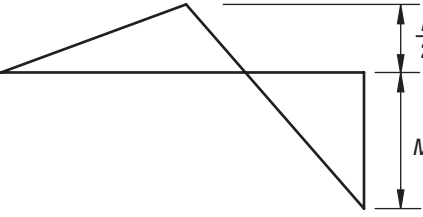
For a braced member subject to end bending moments only, the factor  $c_m$  shall be calculated as follows:

$$c_m = 0.6 - 0.4\beta_m \leq 1.0$$

where  $\beta_m$  is the ratio of the smaller to the larger bending moment at the ends of the member, taken as positive when the member is bent in reverse curvature.

The same expression for  $c_m$  shall be used for a braced member with transverse load applied to it, provided that  $\beta_m$  is determined by one of the following methods:

- (a)  $\beta_m = -1.0$ ;
- (b)  $\beta_m$  is approximated by the value obtained by matching the distribution of bending moment along the member with one of the typical distributions of bending moment shown in [Figures 4.4.2.2\(A\)](#) and [4.4.2.2\(B\)](#); or
- (c)  $\beta_m = 1 - \left( \frac{2\Delta_{ct}}{\Delta_{cw}} \right)$  with  $-1.0 \leq \beta_m \leq 1.0$

Moment distribution	$\beta_m$	Moment distribution	$\beta_m$
	-1.0		-1.0
	+0.2		+0.5
	+0.6		+1.0
	-0.5		+0.4
	+0.2		+0
	+0.2		+0.5

where

- $\Delta_{ct}$  = mid-span deflection of the member resulting from the transverse loading together with both end bending moments, if any, as determined by the analysis
- $\Delta_{cw}$  = mid-span deflection of the member resulting from the transverse loading together with only those end bending moments which produce a mid-span deflection in the same direction as the transverse load

**Figure 4.4.2.2(A) — Values of  $\beta_m$  for various distributions of bending moment — Part A**

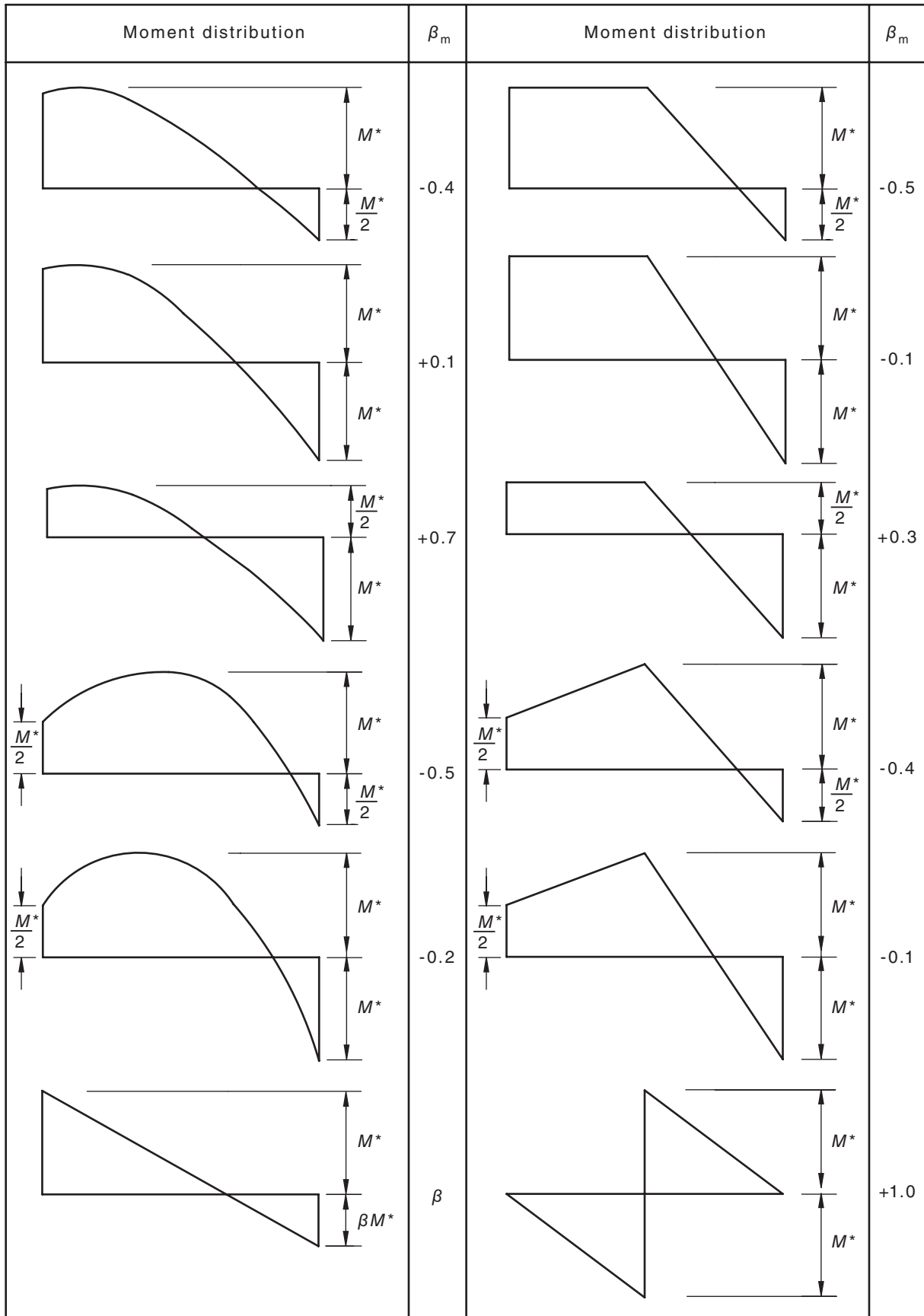


Figure 4.4.2.2(B) — Values of  $\beta_m$  for various distributions of bending moment — Part B

#### 4.4.2.3 Moment amplification for a sway member

For a sway member, the design bending moment ( $M^*$ ) shall be calculated using either the method given in this Clause, or the method given in [Appendix F](#).

For this Clause, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = \delta_m M_m^*$$

The moment amplification factor ( $\delta_m$ ) shall be taken as the greater of —

$\delta_b$  = moment amplification factor for a braced member determined in accordance with [Clause 4.4.2.2](#), and

$\delta_s$  = moment amplification factor for a sway member determined as follows:

(a) *Sway members in rectangular frames:* For all sway columns in a storey of a rectangular frame, the amplification factor ( $\delta_s$ ) shall be calculated from —

$$(i) \quad \delta_s = \frac{1}{1 - \left( \frac{\Delta_s \Sigma N^*}{h_s \Sigma V^*} \right)}$$

where  $\Delta_s$  is the translational displacement of the top relative to the bottom in the storey of height ( $h_s$ ), caused by the design horizontal storey shears ( $V^*$ ) at the column ends,  $N^*$  is the design axial force in a column of the storey, and the summations include all the columns of the storey;

$$(ii) \quad \delta_s = \frac{1}{1 - \left( \frac{1}{\lambda_{ms}} \right)}$$

where the elastic buckling load factor ( $\lambda_{ms}$ ) for the storey under consideration is determined in accordance with [Clause 4.7.2.2](#); or

$$(iii) \quad \delta_s = \frac{1}{1 - \left( \frac{1}{\lambda_c} \right)}$$

where the elastic buckling load factor ( $\lambda_c$ ) is determined from a rational buckling analysis of the whole frame (see [Clause 4.7.2](#)).

(b) *Sway members in non-rectangular frames* — The amplification factor ( $\delta_s$ ) for each sway member shall be taken as the value for the frame calculated as follows:

$$\delta_s = \frac{1}{1 - \left( \frac{1}{\lambda_c} \right)}$$

where the elastic buckling load factor ( $\lambda_c$ ) is determined from a rational buckling analysis of the whole frame (see [Clause 4.7.2](#)).

## 4.5 Plastic analysis

### 4.5.1 Application

The design action effects throughout all or part of a structure may be determined by a plastic analysis provided that the limitations of [Clause 4.5.2](#) are observed. The distribution of design action effects shall satisfy equilibrium and the boundary conditions.

### 4.5.2 Limitations

When a plastic method of analysis is used, all of the following conditions shall be satisfied unless adequate ductility of the structure and plastic rotation capacity of its members and connections are established for the design loading conditions:

- (a) The minimum yield stress specified for the grade of the steel shall not exceed 450 MPa.
- (b) The stress-strain characteristics of the steel shall be in accordance with AS/NZS 3678 or AS/NZS 3679.1, and shall be such as to ensure moment redistribution.

This requirement may be deemed to be satisfied if —

- (i) the stress-strain diagram has a plateau at the yield stress extending for at least six times the yield strain;
  - (ii) the ratio of the tensile strength to the yield stress specified for the grade of the steel (see [Table 2.1](#)) is not less than 1.2;
  - (iii) the elongation on a gauge length in accordance with AS 1391 is not less than 15 %; and
  - (iv) the steel exhibits a strain-hardening capability.
- (c) The members used shall be hot-formed.
  - (d) The members used shall be doubly symmetric I-sections.
  - (e) The geometry of the member sections shall conform to the requirements specified for a compact section in [Clause 5.2.3](#).
  - (f) The members shall not be subject to impact loading or fluctuating loading requiring a fatigue assessment (see [Section 11](#)).

### 4.5.3 Assumptions of analysis

The design action effects shall be determined using a rigid plastic analysis.

It shall be permissible to assume full strength or partial strength connections, provided the capacities of these are used in the analysis, and provided that —

- (a) for a full strength connection, for which the moment capacity of the connection shall be not less than that of the member being connected, the behaviour of the connection shall be such that the rotation capacity at none of the hinges in the collapse mechanism is exceeded; and
- (b) for a partial strength connection, for which the moment capacity of the connection may be less than that of the member being connected, the behaviour of the connection shall be such as to allow all plastic hinges necessary for the collapse mechanism to develop, and shall be such that the rotation capacity at none of the plastic hinges is exceeded.

#### 4.5.4 Second order effects

Any second-order effects of the loads acting on the structure in its deformed configuration may be neglected where the elastic buckling load factor ( $\lambda_c$ ) (see [Clause 4.7](#)) satisfies —

$$10 \leq \lambda_c$$

For  $5 \leq \lambda_c < 10$ , second-order effects may be neglected provided the design load effects are amplified by a factor  $\delta_p$

where

$$\delta_p = \frac{0.9}{1 - \left(\frac{1}{\lambda_c}\right)}$$

For  $\lambda_c < 5$ , a second-order plastic analysis shall be carried out.

### 4.6 Member buckling analysis

#### 4.6.1 General

The elastic buckling load of a member ( $N_{om}$ ) for the particular conditions of end restraint provided by the surrounding frame shall be determined in accordance with [Clause 4.6.2](#).

The member buckling load ( $N_{omb}$ ) is used in the determination of the moment amplification factor for a braced member ( $\delta_b$ ) in [Clause 4.4.2.2](#), and the member buckling load ( $N_{oms}$ ) in the determination of the elastic buckling load factor ( $\lambda_{ms}$ ) in [Clause 4.7.2.2](#) which is used in the determination of the moment amplification factor for a sway member ( $\delta_s$ ) in [Clause 4.4.2.3](#).

#### 4.6.2 Member elastic buckling load

The elastic buckling load of a member ( $N_{om}$ ) shall be determined as follows:

$$N_{om} = \frac{\pi^2 EI}{(k_e l)^2}$$

where

- $k_e$  = member effective length factor, determined in accordance with [Clause 4.6.3](#)
- $l$  = member length from centre to centre of its intersections with supporting members

#### 4.6.3 Member effective length factor

##### 4.6.3.1 General

The value of the member effective length factor ( $k_e$ ) depends on the rotational restraints and the translational restraints at the ends of the member. In [Figure 4.6.3.3\(a\)](#) for a braced member, the translational restraint has been assumed to be infinite. In [Figure 4.6.3.3\(b\)](#) for a sway member, the translational restraint has been assumed to be zero.

The value of the member effective length factor ( $k_e$ ) shall be determined in accordance with the following:

- (a) [Clause 4.6.3.2](#) for members with idealized end restraints.
- (b) [Clause 4.6.3.3](#) or [Appendix G](#) for braced members in frames.

- (c) [Clause 4.6.3.3](#) for sway members in rectangular frames with regular loading and negligible axial forces in the beams.
- (d) [Clause 4.6.3.5](#) for members in triangulated structures.

**4.6.3.2 Members with idealized end restraints**

Values of the member effective length factor ( $k_e$ ) which shall be used for some idealized conditions of end restraint for members are given in [Figure 4.6.3.2](#).

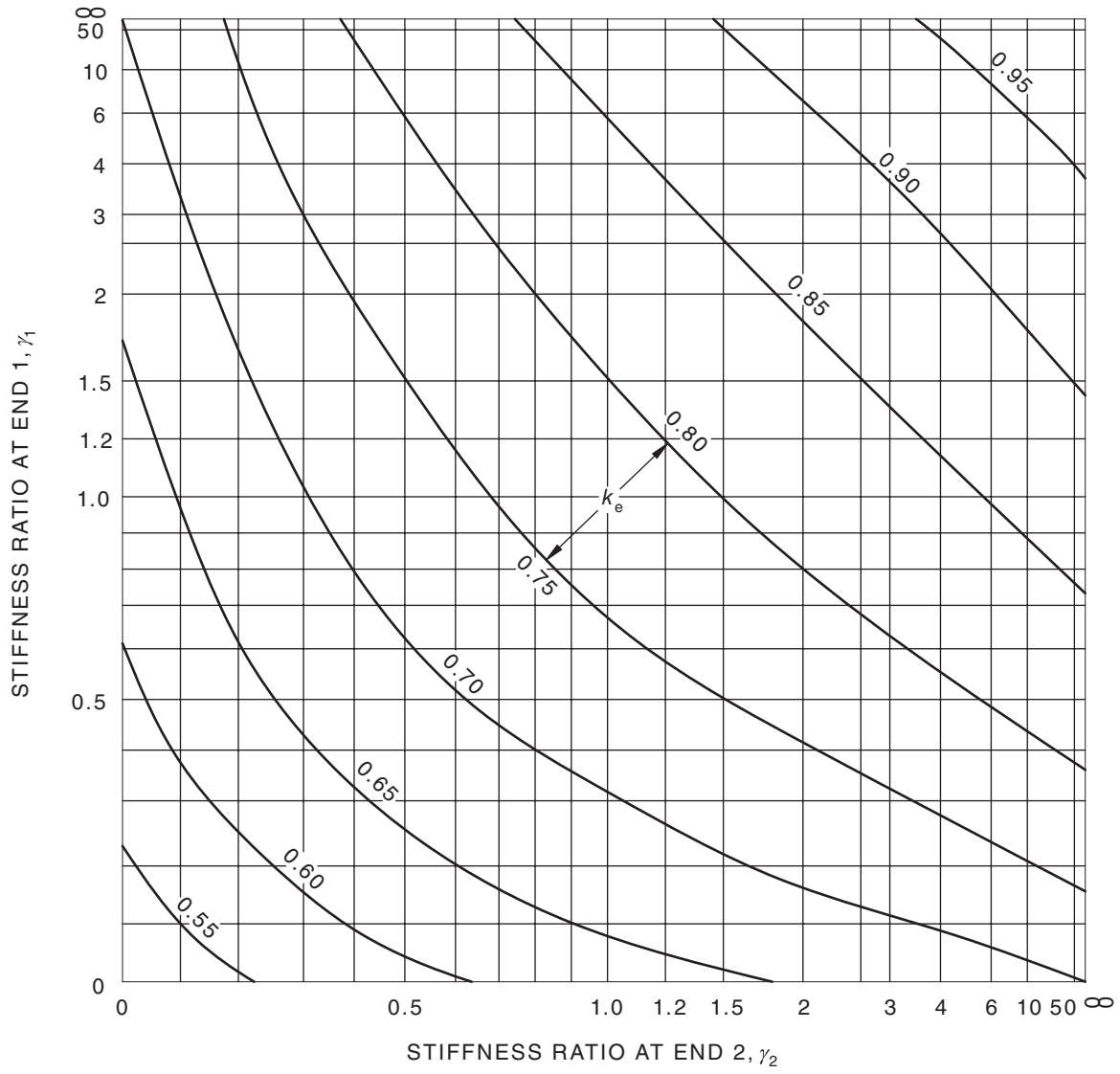
	Braced member			Sway member		
Buckled shape						
Effective length factor ( $k_e$ )	0.7	0.85	1.0	1.2	2.2	2.2
Symbols for end restraint conditions		= Rotation fixed, translation fixed		= Rotation free, translation fixed		= Rotation fixed, translation free
		= Rotation free, translation free		= Rotation free, translation free		= Rotation free, translation free

**Figure 4.6.3.2 — Effective length factors for members for idealized conditions of end restraint**

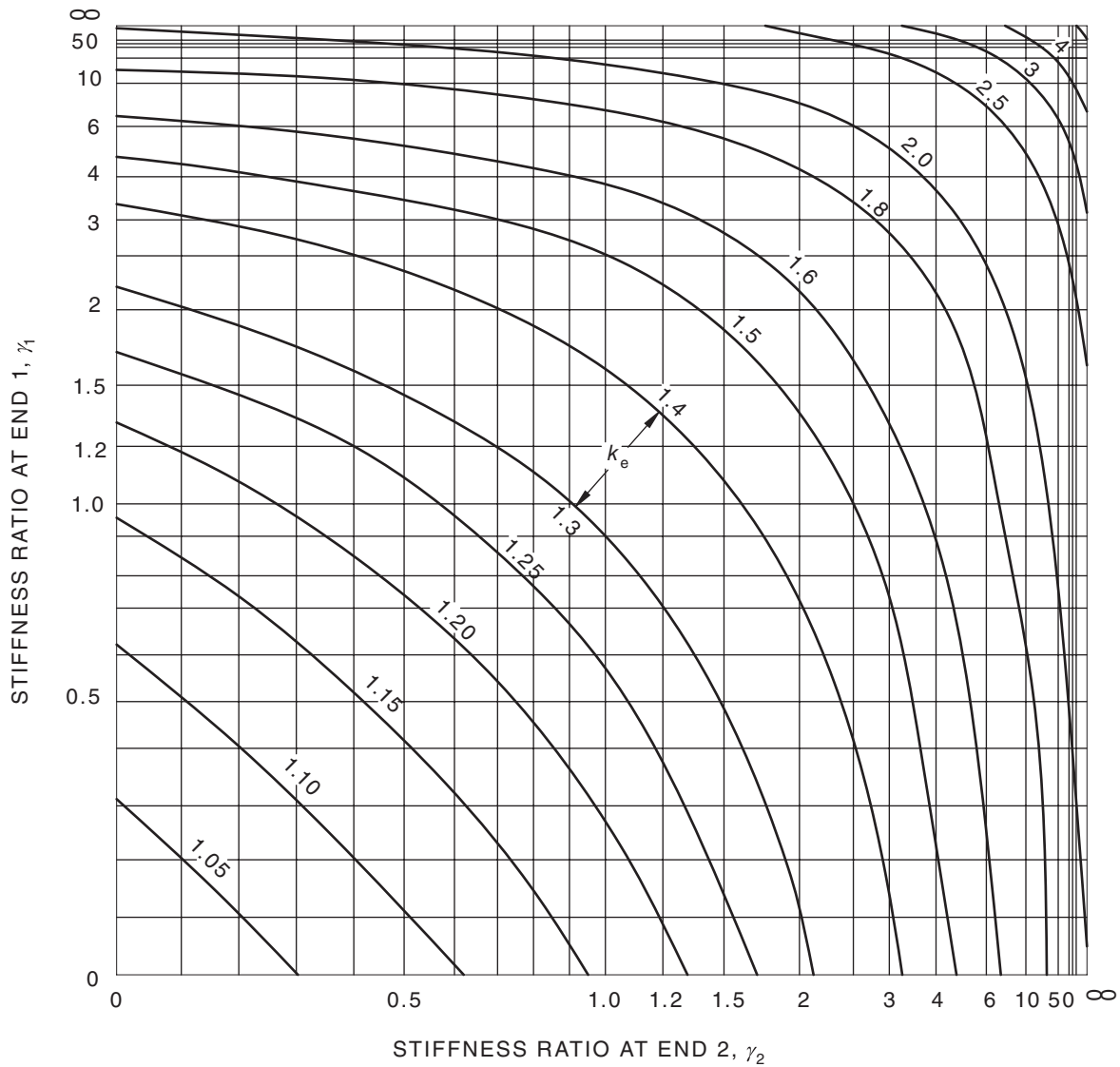
**4.6.3.3 Members in frames**

For a compression member which forms part of a rigid-jointed frame, the member effective length factor ( $k_e$ ) shall be obtained from [Figure 4.6.3.3\(a\)](#) for a braced member and from [Figure 4.6.3.3\(b\)](#) for a sway member. In these figures,  $\gamma_1$  and  $\gamma_2$  are the ratios of the compression member stiffness to the end restraint stiffnesses. The  $\gamma$ -values shall be determined in accordance with [Clause 4.6.3.4](#) or [Appendix G](#), as appropriate.





**(a) For braced members**



(b) For sway members

Figure 4.6.3.3 — Effective length factors

4.6.3.4 Stiffness ratios in rectangular frames

The  $\gamma$ -value of a compression member in a rectangular frame with regular loading and negligible axial forces in the beams shall be calculated as follows:

$$\gamma = \frac{\sum \left( \frac{I}{l} \right)_c}{\sum \beta_e \left( \frac{I}{l} \right)_b}$$

except that —

- (a) for a compression member whose base is not rigidly connected to a footing, the  $\gamma$ -value shall not be taken as less than 10 unless a rational analysis would justify a different value; and
- (b) for a compression member whose end is rigidly connected to a footing, the  $\gamma$ -value shall not be taken as less than 0.6, unless a rational analysis would justify a different value.

The quantity  $\Sigma(I/I)_c$  shall be calculated from the sum of the stiffnesses in the plane of bending of all the compression members rigidly connected at the end of the member under consideration, including the member itself.

The quantity  $\Sigma\beta_e(I/I)_b$  shall be calculated from the sum of the stiffnesses in the plane of bending for all the beams rigidly connected at the end of the member under consideration. The contributions of any beams pin-connected to the member shall be neglected.

The modifying factor ( $\beta_e$ ) which accounts for the conditions at the far ends of the beams, shall be as given in [Table 4.6.3.4](#).

**Table 4.6.3.4 — Modifying factors ( $\beta_e$ )**

Fixity conditions at far end of beam	Beam restraining a braced member	Beam restraining a sway member
Pinned	1.5	0.5
Rigidly connected to a column	1.0	1.0
Fixed	2.0	0.67

#### 4.6.3.5 Members in triangulated structures

The effective length ( $l_e$ ) of a member in a triangulated structure shall be taken as not less than its length ( $l$ ) from centre to centre of intersections with other members, unless shown otherwise by a rational elastic buckling analysis consistent with [Appendix G](#).

### 4.7 Frame buckling analysis

#### 4.7.1 General

The elastic buckling load factor ( $\lambda_c$ ) shall be the ratio of the elastic buckling load set of the frame to the design load set for the frame, and shall be determined in accordance with [Clause 4.7.2](#). The elastic buckling load factor ( $\lambda_c$ ) is used in the determination of the moment amplification factor for a sway member ( $\delta_s$ ) in [Clause 4.4.2.3\(b\)](#) and in establishing limits for the methods of analysis in [Clause 4.5.4](#) and [Appendix E](#).

NOTE The value of  $\lambda_c$  depends on the load set.

#### 4.7.2 In-plane frame buckling

The elastic buckling load factor ( $\lambda_c$ ) of a rigid-jointed frame shall be determined by using —

- one of the approximate methods of [Clauses 4.7.2.1](#) and [4.7.2.2](#); or
- a rational elastic buckling analysis of the whole frame.

NOTE The value of  $\lambda_c$  depends on the load set.

##### 4.7.2.1 Rectangular frames with all members braced

In a rectangular frame with regular loading and negligible axial forces in the beams, the braced member buckling load ( $N_{omb}$ ) for each column shall be determined in accordance with [Clauses 4.6.2](#), [4.6.3.3](#) and [4.6.3.4](#).

The elastic buckling load factor ( $\lambda_m$ ) for each column shall be determined as follows:

$$\lambda_m = \frac{N_{omb}}{N^*}$$

The elastic buckling load factor ( $\lambda_c$ ) for the whole frame shall be taken as the lowest of all the  $\lambda_m$  values.

#### 4.7.2.2 Rectangular frames with sway members

In a rectangular frame with regular loading and negligible axial forces in the beams, the member buckling load ( $N_{oms}$ ) for each column shall be determined in accordance with [Clauses 4.6.2, 4.6.3.3](#) and [4.6.3.4](#).

The elastic buckling load factor ( $\lambda_{ms}$ ) for each storey shall be determined as follows:

$$\lambda_{ms} = \frac{\sum \left( \frac{N_{oms}}{l} \right)}{\sum \left( \frac{N^*}{l} \right)}$$

where

$N^*$  = member design axial force, with tension taken as negative, and the summation includes all columns within a storey

The elastic buckling load factor ( $\lambda_c$ ) for the whole frame shall be taken as the lowest of all the  $\lambda_{ms}$  values.

## Section 5 Members subject to bending

### 5.1 Design for bending moment

A member bent about the section major principal  $x$ -axis which is analysed by the elastic method (see [Clause 4.4](#)) shall satisfy —

$$M_x^* \leq \phi M_{sx}, \text{ and}$$

$$M_x^* \leq \phi M_{bx}$$

where

- $M_x^*$  = design bending moment about the  $x$ -axis determined in accordance with [Clause 4.4](#)
- $\phi$  = capacity factor (see [Table 3.4](#))
- $M_{sx}$  = nominal section moment capacity, as specified in [Clause 5.2](#), for bending about the  $x$ -axis
- $M_{bx}$  = nominal member moment capacity, as specified in [Clause 5.3](#) or [5.6](#), for bending about the  $x$ -axis

A member bent about the section minor principal  $y$ -axis which is analysed by the elastic method (see [Clause 4.4](#)) shall satisfy —

$$M_y^* \leq \phi M_{sy}$$

where

- $M_y^*$  = design bending moment about the  $y$ -axis determined in accordance with [Clause 4.4](#)
- $M_{sy}$  = nominal section moment capacity, as specified in [Clause 5.2](#), for bending about the  $y$ -axis

A member which is analysed by the plastic method (see [Clause 4.5](#)) shall be compact at all sections where plastic hinges may form (see [Clause 5.2.3](#)), shall have full lateral restraint as specified in [Clause 5.3.2](#), and its web shall satisfy [Clause 5.10.6](#). The member shall satisfy —

$$M^* \leq \phi M_s$$

where

- $M^*$  = design bending moment determined in accordance with [Clause 4.5](#)
- $M_s$  = nominal section moment capacity as specified in [Clause 5.2.1](#)

A member whose deflections are constrained to a non-principal plane shall be analysed as specified in [Clause 5.7.1](#), and shall satisfy [Clause 8.3.4](#).

A member which is bent about a non-principal axis and whose deflections are unconstrained shall be analysed as specified in [Clause 5.7.2](#), and shall satisfy [Clauses 8.3.4](#) and [8.4.5](#).

A member subjected to combined bending and shear shall satisfy the requirements of this Clause and [Clause 5.12](#).

A member subjected to combined bending and axial compression or tension shall be in accordance with [Section 8](#).

## 5.2 Section moment capacity for bending about a principal axis

### 5.2.1 General

The nominal section moment capacity ( $M_s$ ) shall be calculated as follows:

$$M_s = f_y Z_e$$

where the effective section modulus ( $Z_e$ ) shall be as specified in [Clauses 5.2.3](#), [5.2.4](#), or [5.2.5](#).

### 5.2.2 Section slenderness

For a section with flat compression plate elements, the section slenderness ( $\lambda_s$ ) shall be taken as the value of the plate element slenderness ( $\lambda_e$ ) for the element of the cross-section which has the greatest value of  $\lambda_e/\lambda_{ey}$  —

where

$$\lambda_e = \left( \frac{b}{t} \right) \sqrt{\left( \frac{f_y}{250} \right)}$$

$\lambda_{ey}$  = plate element yield slenderness limit (see [Table 5.2](#))

$b$  = clear width of the element outstand from the face of the supporting plate element or the clear width of the element between the faces of supporting plate elements

$t$  = element thickness

The section plasticity and yield slenderness limits ( $\lambda_{sp}$ ) and ( $\lambda_{sy}$ ) respectively shall be taken as the values of the element slenderness limits ( $\lambda_{ep}$ ) and ( $\lambda_{ey}$ ) respectively given in [Table 5.2](#) for the element of the cross-section which has the greatest value of  $\lambda_e/\lambda_{ey}$ .

For circular hollow sections, the section slenderness ( $\lambda_s$ ) shall be calculated as follows:

$$\lambda_s = \left( \frac{d_o}{t} \right) \left( \frac{f_y}{250} \right)$$

where  $d_o$  is the outside diameter of the section. The section plasticity and yield slenderness limits ( $\lambda_{sp}$ ) and ( $\lambda_{sy}$ ) respectively shall be taken as the values of the element slenderness limits ( $\lambda_{ep}$ ) and ( $\lambda_{ey}$ ) respectively given in [Table 5.2](#).

**Table 5.2 — Values of plate element slenderness limits**

Plate element type	Longitudinal edges supported	Residual stresses (see Note)	Plasticity limit ( $\lambda_{ep}$ )	Yield limit ( $\lambda_{ey}$ )	Deformation limit ( $\lambda_{ed}$ )
Flat	One	SR	10	16	35
(Uniform compression)		HR	9	16	35
		LW,CF	8	15	35
		HW	8	14	35
Flat	One	SR	10	25	—

Table 5.2 (continued)

Plate element type	Longitudinal edges supported	Residual stresses (see Note)	Plasticity limit ( $\lambda_{ep}$ )	Yield limit ( $\lambda_{ey}$ )	Deformation limit ( $\lambda_{ed}$ )
(Maximum compression at unsupported edge, zero stress or tension at supported edge)		HR	9	25	—
		LW,CF	8	22	—
		HW	8	22	—
Flat	Both	SR	30	45	90
(Uniform compression)		HR	30	45	90
		LW, CF	30	40	90
		HW	30	35	90
Flat	Both	Any	82	115	—
(Compression at one edge, tension at the other)					
Circular hollow sections		SR	50	120	—
		HR, CF	50	120	—
		LW	42	120	—
		HW	42	120	—
<b>Key</b>					
SR = stress relieved					
HR = hot-rolled or hot-finished					
CF = cold formed					
LW = lightly welded longitudinally					
HW = heavily welded longitudinally					
NOTE Welded members whose compressive residual stresses are less than 40 MPa may be assumed to be lightly welded.					

### 5.2.3 Compact sections

For sections which satisfy  $\lambda_s \leq \lambda_{sp}$ , the effective section modulus ( $Z_e$ ) shall be the lesser of  $S$  or  $1.5Z$ , where  $S$  and  $Z$  are the plastic and elastic section moduli respectively, determined in accordance with [Clause 5.2.6](#).

### 5.2.4 Non-compact sections

For sections which satisfy  $\lambda_{sp} < \lambda_s \leq \lambda_{sy}$ , the effective section modulus ( $Z_e$ ) shall be calculated as follows:

$$Z_e = Z + \left[ \left( \frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}} \right) (Z_c - Z) \right]$$

where  $Z_c$  is the effective section modulus ( $Z_e$ ) for a compact section specified in [Clause 5.2.3](#).

### 5.2.5 Slender sections

For sections with flat plate elements in uniform compression which satisfy  $\lambda_s > \lambda_{sy}$ , the effective section modulus ( $Z_e$ ) shall be calculated either as follows:

$$Z_e = Z \left( \frac{\lambda_{sy}}{\lambda_s} \right)$$

or for the effective cross-section determined by omitting from each flat compression element the width in excess of the width corresponding to  $\lambda_{sy}$ .

For a section whose slenderness is determined by the value calculated for a flat plate element with maximum compression at an unsupported edge and zero stress or tension at the other edge and which satisfies  $\lambda_s > \lambda_{sy}$ , the effective section modulus ( $Z_e$ ) shall be calculated as follows:

$$Z_e = Z \left( \frac{\lambda_{sy}}{\lambda_s} \right)^2$$

For circular hollow sections which satisfy  $\lambda_s > \lambda_{sy}$ , the effective section modulus shall be taken as the lesser of —

$$Z_e = Z \sqrt{\left( \frac{\lambda_{sy}}{\lambda_s} \right)} \text{ and}$$

$$Z_e = Z \left( \frac{2\lambda_{sy}}{\lambda_s} \right)^2$$

For elements where  $\lambda_e > \lambda_{ed}$  in which  $\lambda_{ed}$  is the deformation slenderness limit given in [Table 5.2](#), noticeable deformations may occur under service loading.

### 5.2.6 Elastic and plastic section moduli

For sections without holes, or for sections with holes that reduce either of the flange areas by not more than  $100\{1 - [f_y/(0.85f_u)]\}\%$ , the elastic and plastic section moduli may be calculated using the gross section.

For sections with holes that reduce either of the flange areas by more than  $100\{1 - [f_y/(0.85f_u)]\}\%$ , the elastic and plastic section moduli shall be calculated using either —

- (a)  $(A_n/A_g)$  times the value for the gross section, in which  $A_n$  is the sum of the net areas of the flanges and the gross area of the web, and  $A_g$  the gross area of the section; or
- (b) the net section.

When net areas are calculated, any deductions for fastener holes shall be made in accordance with [Clause 9.1.10](#).

## 5.3 Member capacity of segments with full lateral restraint

### 5.3.1 Member capacity

The nominal member moment capacity ( $M_b$ ) of a segment with full lateral restraint shall be taken as the nominal section moment capacity ( $M_s$ ) (see [Clause 5.2](#)) of the critical section (see [Clause 5.3.3](#)).



A segment in a member subjected to bending is the length between adjacent cross-sections which are fully or partially restrained (see [Clauses 5.4.2.1](#) and [5.4.2.2](#)), or the length between an unrestrained end (see [Clause 5.4.1](#)) and the adjacent cross-section which is fully or partially restrained.

## 5.3.2 Segments with full lateral restraint

### 5.3.2.1 General

A segment may be treated as having full lateral restraint if it satisfies one of the following clauses: [Clause 5.3.2.2](#), [5.3.2.3](#) or [Clause 5.3.2.4](#), or if its nominal member moment capacity ( $M_b$ ) calculated in accordance with [Clause 5.6](#) is not less than the nominal section moment capacity ( $M_s$ ) (see [Clause 5.2](#)) at the critical section (see [Clause 5.3.3](#)).

### 5.3.2.2 Segments with continuous lateral restraints

A segment with continuous lateral restraints may be treated as having full lateral restraint, provided that —

- (a) both ends are fully or partially restrained (see [Clauses 5.4.2.1](#), [5.4.2.2](#), [5.4.3.1](#), and [5.4.3.2](#)); and
- (b) the continuous restraints act at the critical flange (see [Clause 5.5](#)), and satisfy [Clause 5.4.3.1](#).

### 5.3.2.3 Segments with intermediate lateral restraints

A segment with intermediate lateral restraints (see [Clauses 5.4.2.4](#) and [5.4.3.1](#)) that divide the segment into a series of sub-segments may be treated as having full lateral restraint, provided that —

- (a) both ends are fully or partially restrained (see [Clauses 5.4.2.1](#), [5.4.2.2](#), [5.4.3.1](#) and [5.4.3.2](#));
- (b) the length ( $l$ ) of each sub-segment satisfies [Clause 5.3.2.4](#); and
- (c) the lateral restraints act at the critical flange (see [Clause 5.5](#)), and satisfy [Clause 5.4.3.1](#).

### 5.3.2.4 Segments with full or partial restraints at both ends

A segment with full or partial restraints at both ends (see [Clauses 5.4.2.1](#), [5.4.2.2](#), [5.4.3.1](#) and [5.4.3.2](#)) may be treated as having full lateral restraint, provided its length ( $l$ ) satisfies —

$$\frac{l}{r_y} \leq (80 + 50\beta_m) \sqrt{\left(\frac{250}{f_y}\right)} \text{ if the segment is of equal flanged I-section;}$$

$$\frac{l}{r_y} \leq (60 + 40\beta_m) \sqrt{\left(\frac{250}{f_y}\right)} \text{ if the segment is an equal flanged channel;}$$

$$\frac{l}{r_y} \leq (80 + 50\beta_m) \left[ \sqrt{\left(\frac{2\rho A d_f}{2.5Z_{ex}}\right)} \right] \sqrt{\left(\frac{250}{f_y}\right)} \text{ if the segment is of I-section with unequal flanges;}$$

$$\frac{l}{r_y} \leq (1800 + 1500\beta_m) \left(\frac{b_f}{b_w}\right) \left(\frac{250}{f_y}\right) \text{ if the segment is of rectangular or square hollow section; or}$$

$$\frac{l}{t} \leq (210 + 175\beta_m) \left[ \sqrt{\left(\frac{b_2}{b_1}\right)} \right] \left(\frac{250}{f_y}\right) \text{ if the segment is of angle section}$$

where

$A$	=	area of cross-section
$b_f, b_w$	=	flange width and web depth, respectively
$b_1, b_2$	=	greater and lesser leg lengths, respectively
$d_f$	=	distance between flange centroids
$I_{cy}$	=	second moment of area of the compression flange about the section minor $y$ -axis
$I_y$	=	second moment of area of the section about the section minor principal $y$ -axis
$r_y$	=	radius of gyration about the minor principal $y$ -axis
$t$	=	thickness of an angle section
$Z_e$	=	effective section modulus (see <a href="#">Clause 5.2</a> )
$\rho$	=	$I_{cy}/I_y$

The ratio  $\beta_m$  shall be taken as one of the following:

- (a) -1.0;
- (b) -0.8 for segments with transverse loads; or
- (c) the ratio of the smaller to the larger end moments in the length ( $l$ ), (positive when the segment is bent in reverse curvature and negative when bent in single curvature) for segments without transverse loads.

### 5.3.3 Critical section

The critical section in a segment shall be taken as the cross-section which has the largest value of the ratio of the design bending moment ( $M^*$ ) to the nominal section capacity in bending ( $M_s$ ) (see [Clause 5.2](#)).

## 5.4 Restraints

### 5.4.1 General

A cross-section may be assumed to be fully, partially, rotationally or laterally restrained if its restraints satisfy the appropriate requirements of [Clause 5.4.2](#).

Restraints against lateral deflection, twist rotation, or lateral rotation may be assumed to be effective if they satisfy the appropriate requirements of [Clause 5.4.3](#).

The members and connections of restraint systems shall be designed to transfer the appropriate forces and bending moments specified in [Clause 5.4.3](#), together with any other forces or bending moments which may act simultaneously, from the points where the forces or bending moments arise to anchorage or reaction points.

Any cross-section of a member which does not satisfy any of [Clauses 5.4.2.1](#) to [5.4.2.4](#) shall be deemed to be unrestrained, as for example in [Figure 5.4.1](#), unless the member capacity in bending is determined by the method of design by buckling analysis (see [Clause 5.6.4](#)).

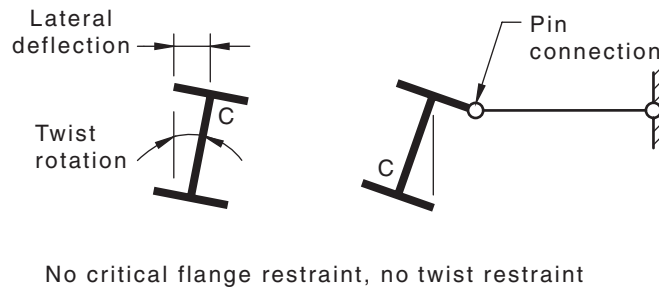


Figure 5.4.1 — Unrestrained cross-sections

5.4.2 Restraints at a cross-section

5.4.2.1 Fully restrained

A cross-section of a member may be assumed to be fully restrained if either —

- (a) the restraint or support effectively prevents lateral deflection of the critical flange (see Clause 5.5), and effectively prevents twist rotation of the section, as, for example, in Figure 5.4.2.1(a); or partially prevents twist rotation of the section, as, for example, in Figure 5.4.2.1(b); or
- (b) the restraint or support effectively prevents lateral deflection of some other point in the cross-section, and effectively prevents twist rotation of the section, as, for example, in Figure 5.4.2.1(c).

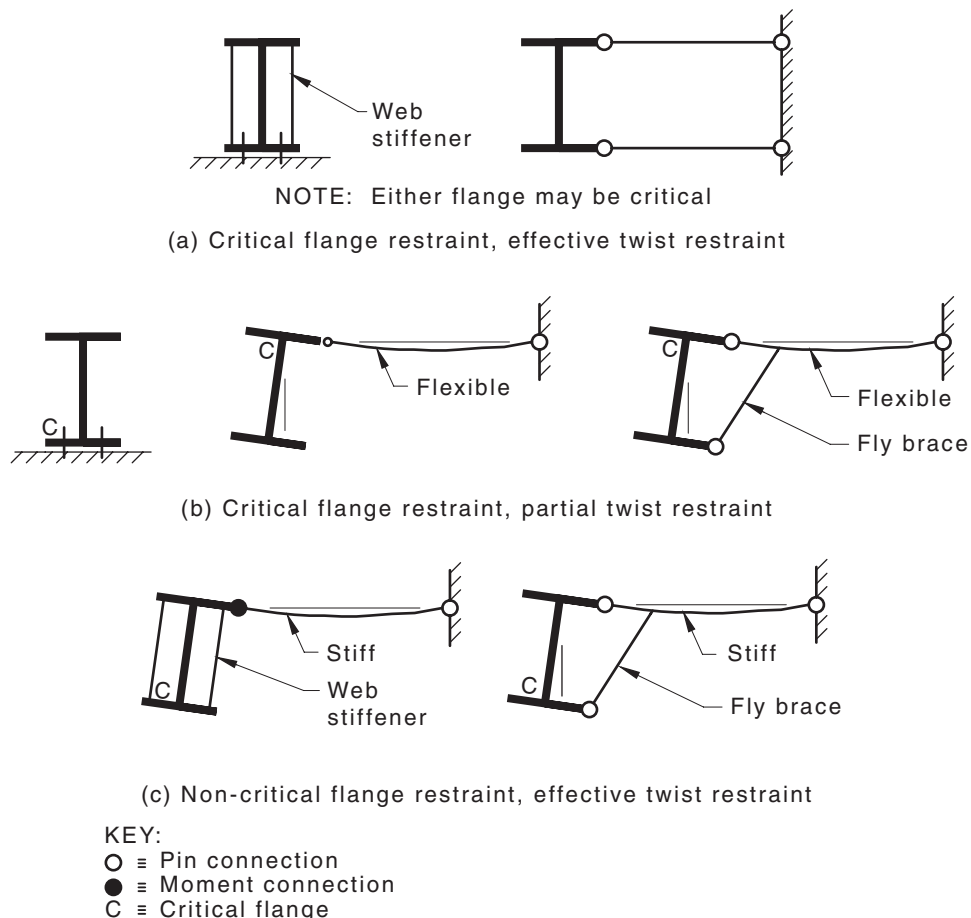


Figure 5.4.2.1 — Fully restrained cross-sections

### 5.4.2.2 Partially restrained

A cross-section of a member may be assumed to be partially restrained if the restraint or support effectively prevents lateral deflection of some point in the cross-section other than the critical flange, and partially prevents twist rotation of the section, as for example in [Figure 5.4.2.2](#).

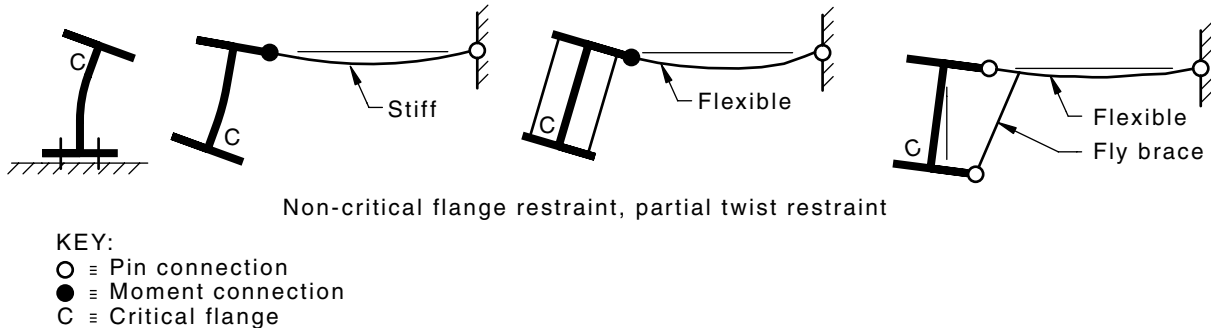


Figure 5.4.2.2 — Partially restrained cross-sections

### 5.4.2.3 Rotationally restrained

A cross-section of a member which may be assumed to be fully or partially restrained may be assumed to be rotationally restrained when the restraint or support provides significant restraint against lateral rotation of the critical flange (see [Clause 5.5](#)) out of the plane of bending, as for example in [Figure 5.4.2.3](#).

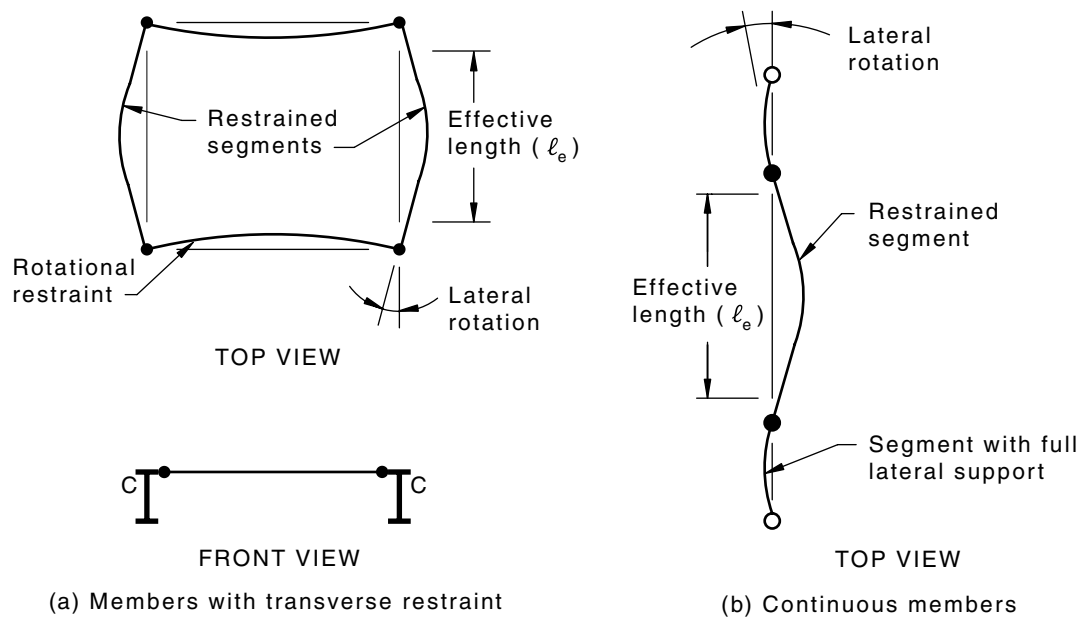
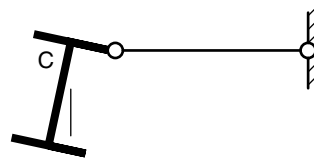


Figure 5.4.2.3 — Rotationally restrained cross-sections

### 5.4.2.4 Laterally restrained

A cross-section of a segment whose ends are fully or partially restrained may be assumed to be laterally restrained when the restraint effectively prevents lateral deflection of the critical flange (see [Clause 5.5](#))

but is ineffective in preventing twist rotation of the section, as for example in [Figure 5.4.2.4](#). Cross-sections in member segments with one end unrestrained shall not be assumed to be laterally restrained.



Critical flange restraint, no twist restraint

KEY:  
 ○ ≡ Pin connection  
 C ≡ Critical flange

**Figure 5.4.2.4 — Laterally restrained cross-section**

### 5.4.3 Restraining elements

#### 5.4.3.1 Restraint against lateral deflection

The lateral restraint at any cross-section considered to be fully, partially or laterally restrained in terms of [Clause 5.4.2](#) shall be designed to transfer a transverse force acting at the critical flange (see [Clause 5.5](#)) equal to 0.025 times the maximum force in the critical flanges of the adjacent segments or sub-segments, except where the restraints are more closely spaced than is required to ensure that  $M^*$  equals  $\phi M_b$ .

When the restraints are more closely spaced, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints which shall ensure that  $M^*$  equals  $\phi M_b$ . Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer the transverse force of 0.025 times the maximum force in the critical flanges of the equivalent adjacent segments or sub-segments.

#### 5.4.3.2 Restraint against twist rotation

A torsional restraint at a cross-section may be assumed to provide effective restraint against twist rotation if it is designed to transfer a transverse force equal to 0.025 times the maximum force in the critical flange from any unrestrained flange to the lateral restraint.

A torsional restraint at a cross-section may be assumed to provide partial restraint against twist rotation if it is able to provide an elastic restraint against twist rotation without rotational slip.

Flexible elements such as unstiffened webs may form part of such a restraint provided that they are connected in such a way as to prevent rotational slip.

Any restraint at a cross-section which permits rotational slip shall be assumed to be ineffective in restraining twist rotation.

NOTE Guidance on the effects of the stiffness of a torsional restraint on the resistance to lateral buckling is given in [Clause H.5.1](#) of [Appendix H](#).

#### 5.4.3.3 Parallel restrained members

When a series of parallel members is restrained by a line of restraints, each restraining element shall be designed to transfer a transverse force equal to the sum of 0.025 times the flange force from the connected member and 0.0125 times the sum of the flange forces in the connected members beyond, except that no more than seven members need be considered.

#### 5.4.3.4 Restraint against lateral rotation

A rotational restraint at a cross-section which is considered to be fully or partially restrained (see [Clauses 5.4.2.1, 5.4.2.2 and 5.4.2.3](#)) may be deemed to provide restraint against lateral rotation out of the plane of bending, providing its flexural stiffness in the plane of rotation is comparable with the corresponding stiffness of the restrained member.

NOTE Guidance on the effects of the stiffness of a rotational restraint on the resistance to lateral buckling is given in [Clause H.5.2 of Appendix H](#).

A segment which has full lateral restraint (see [Clause 5.3.2](#)) may be deemed to provide rotational restraint to an adjacent segment which is laterally continuous with it.

A segment that does not have full lateral restraint shall be assumed to be unable to provide rotational restraint to an adjacent segment, unless the member resistance is determined by the method of design by buckling analysis in accordance with [Clause 5.6.4](#).

### 5.5 Critical flange

#### 5.5.1 General

The critical flange at any cross-section is the flange which, in the absence of any restraint at that section, would deflect the farthest during buckling.

The critical flange may be determined by an elastic buckling analysis (see [Clause 5.6.4](#)) or as specified in [Clauses 5.5.2 and 5.5.3](#).

#### 5.5.2 Segments with both ends restrained

The critical flange at any section of a segment restrained at both ends shall be the compression flange.

#### 5.5.3 Segments with one end unrestrained

When gravity loads are dominant, the critical flange of a segment with one end unrestrained shall be the top flange.

When wind loads are dominant, the critical flange shall be the exterior flange in the case of external pressure or internal suction, and shall be the interior flange in the case of internal pressure or external suction.

### 5.6 Member capacity of segments without full lateral restraint

#### 5.6.1 Segments fully or partially restrained at both ends

##### 5.6.1.1 Open sections with equal flanges

For open sections with equal flanges, the following shall apply:

- (a) *Segments of constant cross-section* — The nominal member moment capacity ( $M_b$ ) shall be calculated as follows:

$$M_b = \alpha_m \alpha_s M_s \leq M_s \quad 5.6.1.1(1)$$

where

$\alpha_m$  = moment modification factor

$\alpha_s$  = slenderness reduction factor

$M_s$  = nominal section moment capacity determined in accordance with [Clause 5.2](#) for the gross section

The moment modification factor ( $\alpha_m$ ) shall be taken as one of the following:

- (i) 1.0;
- (ii) a value obtained from [Table 5.6.1](#);

$$(iii) \quad \alpha_m \frac{1.7M_m^*}{\sqrt{\left[ (M_2^*)^2 + (M_3^*)^2 + (M_4^*)^2 \right]}} \leq 2.5$$

where

$M_m^*$  = maximum design bending moment in the segment

$M_2^*, M_4^*$  = design bending moments at the quarter points of the segment

$M_3^*$  = design bending moment at the midpoint of the segment; or

- (iv) a value obtained from an elastic buckling analysis in accordance with [Clause 5.6.4](#), except that for sub-segments formed by intermediate lateral restraints in segments fully or partially restrained at both ends, the sub-segment moment distribution shall be used instead of the segment moment distribution when using (ii) or (iii).

The slenderness reduction factor ( $\alpha_s$ ) shall be determined as follows:

$$\alpha = 0.6 \left[ \sqrt{\left[ \left( \frac{M_s}{M_{oa}} \right)^2 + 3 \right]} - \left( \frac{M_s}{M_{oa}} \right) \right] \quad 5.6.1.1(2)$$

in which  $M_{oa}$  shall be taken as either —

- (A)  $M_{oa} = M_o$ , where  $M_o$  is the reference buckling moment; or
- (B) the value determined from an elastic buckling analysis in accordance with [Clause 5.6.4](#).

The reference buckling moment ( $M_o$ ) shall be determined as follows:

$$M_o = \sqrt{\left[ \left[ \left( \frac{\pi^2 E I_y}{l_e^2} \right) \right] \left[ GJ + \left( \frac{\pi^2 E I_w}{l_e^2} \right) \right] \right]} \quad 5.6.1.1(3)$$

where

$E, G$  = elastic moduli (see [Clause 1.4](#))

$I_y, J$ , and  $I_w$  = section constants (see [Clause 1.4](#))

$l_e$  = effective length determined in accordance with [Clause 5.6.3](#)

NOTE Values of E and G and expressions for J and  $I_w$  are given in [Clause H.4](#) of [Appendix H](#).

(b) *Segments of varying cross-section* — The nominal member moment capacity ( $M_b$ ) shall be determined in accordance with [Clause 5.6.1.1\(a\)](#) and using one of the following:

- (i) The properties of the minimum cross-section.
- (ii) The properties of the critical cross-section as specified in [Clause 5.3.3](#), provided that the value of  $M_{oa}$  determined in accordance with [Clause 5.6.1.1\(a\)](#) is reduced, before it is used in [Equation 5.6.1.1\(2\)](#), by multiplying it by the reduction factor ( $\alpha_{st}$ ) as follows:

$$\alpha_{st} = 1.0 - \left[ 1.2r_r (1 - r_s) \right]$$

where

$$r_r = l_r/l \text{ for stepped members}$$

$$= 0.5 \text{ for tapered members}$$

$$r_s = \frac{A_{fm}}{A_{fc}} \left[ 0.6 + \left( \frac{0.4d_m}{d_c} \right) \right]$$

$A_{fm}, A_{fc}$  = the flange areas at the minimum and critical cross-sections, respectively

$d_m, d_c$  = the section depths at the minimum and critical cross-sections, respectively

$l_r$  = the length of the segment over which the cross-section is reduced

$l$  = the length of the segment.

- (iii) The method of design by buckling analysis (see [Clause 5.6.4](#)).

### 5.6.1.2 I-sections with unequal flanges

The nominal member moment capacity ( $M_b$ ) shall be determined in accordance with [Clause 5.6.1.1\(a\)](#), except that the reference buckling moment ( $M_o$ ) shall be determined by using either —

$$(a) \quad M_o = \sqrt{\left( \frac{\pi^2 E I_y}{l_e^2} \right)} \left[ \sqrt{\left( GJ \right) + \left( \frac{\pi^2 E I_w}{l_e^2} \right) + \left( \frac{\beta_x^2 \pi^2 E I_y}{4 l_e^2} \right)} + \frac{\beta_x}{2} \sqrt{\left( \frac{\pi^2 E I_y}{l_e^2} \right)} \right]; \text{ or}$$

- (b) the method of design by buckling analysis (see [Clause 5.6.4](#)).

The monosymmetry section constant ( $\beta_x$ ) shall be determined using either —

$$(i) \quad \beta_x = 0.8d_f \left[ \left( \frac{2I_{cy}}{I_y} \right) - 1 \right]$$

where

$d_f$  = the distance between flange centroids

$I_{cy}$  = the second moment of area of the compression flange about the section minor principal y-axis; or



$$(ii) \quad \beta_x = \frac{1}{I_x} \int (x^2 y + y^3) dA - 2y_0$$

where  $y_0$  is the coordinate of the shear centre (see Reference 11 in [Appendix H](#)). The values of  $\beta_x$  are positive when the larger flange is in compression, and negative when the smaller flange is in compression.

### 5.6.1.3 Angle sections

The nominal member moment capacity ( $M_b$ ) of an angle section shall be determined in accordance with [Clause 5.6.1.1\(a\)](#) using  $I_w = 0$ .

### 5.6.1.4 Hollow sections

The nominal member moment capacity ( $M_b$ ) of a rectangular hollow section shall be determined in accordance with [Clause 5.6.1.1\(a\)](#) using  $I_w = 0$ .

## 5.6.2 Segments unrestrained at one end

The nominal member moment capacity ( $M_b$ ) of a segment unrestrained at one end and at the other end both —

- (a) fully or partially restrained; and
- (b) laterally continuous or restrained against lateral rotation

shall be determined using either —

- (i) [Equations 5.6.1.1\(1\)](#) and [5.6.1.1\(2\)](#) with  $M_{0a}$  equal to the value of  $M_0$  obtained from [Equation 5.6.1.1\(3\)](#), and the appropriate value of  $\alpha_m$  given in [Table 5.6.2](#); or
- (ii)  $M_b = \alpha_s M_s \leq M_s$

where the slenderness reduction factor ( $\alpha_s$ ) shall be determined as follows:

$$\alpha_s = 0.6 \left[ \sqrt{\left[ \left( \frac{M_s}{M_{ob}} \right)^2 + 3 \right]} - \left( \frac{M_s}{M_{ob}} \right) \right]$$

where  $M_s$  is the nominal section moment capacity determined in accordance with [Clause 5.2](#) for the gross section, and  $M_{ob}$  is determined by an elastic buckling analysis in accordance with [Clause 5.6.4](#).

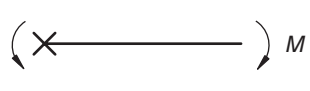


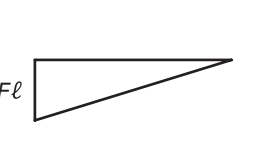
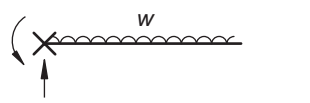

**Table 5.6.1 — Moment modification factors ( $\alpha_m$ ) for segments fully or partially restrained at both ends**

Beam segment	Moment distribution	Moment modification factor, $\alpha_m$	Range
		$1.75 + 1.05\beta_m + 0.3\beta_m^2$ 2.5	$-1 \leq \beta_m \leq 0.6$ $0.6 < \beta_m \leq 1$
		$1.0 + 0.35 \left(1 - \frac{2a}{l}\right)^2$	$0 \leq \frac{2a}{l} \leq 1$
		$1.35 + 0.4 \left(\frac{2a}{l}\right)^2$	$0 \leq \frac{2a}{l} \leq 1$
		$1.35 + 0.15\beta_m$ $-1.2 + 3.0\beta_m$	$0 \leq \beta_m < 0.9$ $0.9 \leq \beta_m \leq 1$
		$1.35 + 0.36\beta_m$	$0 \leq \beta_m \leq 1$
		$1.13 + 0.10\beta_m$ $-1.25 + 3.5\beta_m$	$0 \leq \beta_m \leq 0.7$ $0.7 \leq \beta_m \leq 1$
		$1.13 + 0.12\beta_m$ $-2.38 + 4.8\beta_m$	$0 \leq \beta_m \leq 0.75$ $0.75 \leq \beta_m \leq 1$
		1.00	
		1.75	
		2.50	

KEY:  
 $\equiv$  full or partial restraint

NOTE Diagrams indicate positive direction of forces.

**Table 5.6.2 — Moment modification factors ( $\alpha_m$ ) for segments unrestrained at one end**

Member segment	Moment distribution	Moment modification factor, $\alpha_m$
		0.25
		1.25
		2.25

KEY

X = Full or partial restraint

NOTE Diagrams indicate positive direction of forces.

### 5.6.3 Effective length

The effective length ( $l_e$ ) of a segment or sub-segment shall be determined as follows:

$$l_e = k_t k_l k_r l$$

where

- $k_t$  = twist restraint factor given in [Table 5.6.3\(A\)](#)  
 $k_l$  = load height factor given in [Table 5.6.3\(B\)](#)  
 $k_r$  = lateral rotation restraint factor given in [Table 5.6.3\(C\)](#)

and the length ( $l$ ) shall be taken as either —

- the segment length, for segments without intermediate restraints, or for segments unrestrained at one end, with or without intermediate lateral restraints; or
- the sub-segment length, for sub-segments formed by intermediate lateral restraints (see [Clauses 5.4.2.4](#) and [5.4.3.1](#)) in a segment which is fully or partially restrained at both ends.

The lateral rotation restraint factor ( $k_r$ ) shall only be taken as less than unity when effective rotational restraints, complying with [Clause 5.4.3.4](#), act at one or both ends of a segment which is fully, or partially restrained at both ends. The lateral rotation restraint factor shall be taken as unity for all segments which are unrestrained at one end.

**Table 5.6.3(A) — Twist restraint factors ( $k_t$ )**

Restraint arrangement	Factor, $k_t$
FF, FL, LL, FU	1.0
FP, PL, PU	$1 + \frac{\left[ \left( \frac{d_1}{l} \right) \left( \frac{t_f}{2t_w} \right)^3 \right]}{n_w}$
PP	$1 + \frac{\left[ 2 \left( \frac{d_1}{l} \right) \left( \frac{t_f}{2t_w} \right)^3 \right]}{n_w}$

**Table 5.6.3(B) — Load height factors ( $k_l$ ) for gravity loads**

Longitudinal position of the load	Restraint arrangement	Load height position	
		Shear centre	Top flange
Within segment	FF, FP, FL, PP, PL, LL	1.0	1.4
	FU, PU	1.0	2.0
At segment end	FF, FP, FL, PP, PL, LL	1.0	1.0
	FU, PU	1.0	2.0

**Table 5.6.3(C) — Lateral rotation restraint factors ( $k_r$ )**

Restraint arrangement	Ends with lateral rotation restraints (see <a href="#">Clause 5.4.3.4</a> )	Factor, $k_r$
FU, PU	Any	1.0
FF, FP, FL, PP, PL, LL	None	1.0
FF, FP, PP	One	0.85
FF, FP, PP	Both	0.70

In [Tables 5.6.3\(A\)](#), [5.6.3\(B\)](#) and [5.6.3\(C\)](#),

- $d_1$  = clear depth between flanges ignoring fillets or welds
- $n_w$  = number of webs
- $t_f$  = thickness of critical flange
- $t_w$  = thickness of web
- F = fully restrained
- L = laterally restrained
- P = partially restrained
- U = unrestrained

and two of the symbols F, L, P, U are used to indicate the conditions at the two ends. (For F, L, P and U restraint requirements, see [Clause 5.4.3](#).)

### 5.6.4 Design by buckling analysis

When a member is designed by this method, the elastic buckling bending moment ( $M_{ob}$ ) at the most critical section of the member shall be determined by using the results of an elastic flexural-torsional buckling analysis. This analysis shall take proper account of the member support, restraint, and loading conditions.

The value of  $M_{oa}$  to be used in [Clause 5.6.1.1\(a\)](#) shall be taken as follows:

$$M_{oa} = \frac{M_{ob}}{\alpha_m}$$

The moment modification factor ( $\alpha_m$ ) shall be determined by using either (a) or (b) given below —

(a) [Clause 5.6.1.1\(a\)](#); or

(b) the value given by  $\alpha_m = \frac{M_{os}}{M_{oo}}$

where

$M_{os}$  = elastic buckling moment for a segment, fully restrained at both ends, which is unrestrained against lateral rotation and loaded at the shear centre

$M_{oo}$  = reference elastic buckling moment given by [Equation 5.6.1.1\(3\)](#) with  $l_e = l$

NOTE Summaries and approximations of the results of elastic buckling analyses are given in [Appendix H](#) and in the references given in [Clause H.6](#).

## 5.7 Bending in a non-principal plane

### 5.7.1 Deflections constrained to a non-principal plane

When the deflection of a member is constrained to a non-principal plane by continuous lateral restraints which prevent lateral deflection, then the forces exerted by the restraints shall be determined, and the principal axis bending moments acting on the member shall be calculated from these forces and the applied forces by a rational analysis.

The calculated principal axis bending moments shall satisfy the requirements of [Clause 8.3.4](#).

### 5.7.2 Deflections unconstrained

When the deflections of a member loaded in a non-principal plane are unconstrained, the principal axis bending moments shall be calculated by a rational analysis.

The calculated principal axis bending moments shall satisfy [Clauses 8.3.4](#) and [8.4.5](#).

## 5.8 Separators and diaphragms

If separators or diaphragms are used to permit two or more I-section members or channels placed side by side to act together as a unit in the distribution of external loads between them, the separators and diaphragms shall meet the following requirements:

- (a) Separators, made up of spacers and through bolts, shall not be used to transmit forces between the members, other than those due to transverse forces (if any) and a design transverse force ( $Q^*$ ), taken as not less than 0.025 times the maximum design force occurring in the most heavily loaded compression flange of any member forming the unit. The design transverse force ( $Q^*$ ) shall be taken as shared equally between the separators.

- (b) Diaphragms shall be used where external vertical as well as transverse forces are to be transmitted from one member to another. The diaphragms and their fastenings shall be proportioned to distribute the forces applied to them and in addition, to resist the design transverse force ( $Q^*$ ) specified above, and resulting shear forces. The design transverse force ( $Q^*$ ) shall be taken as shared equally between the diaphragms.

## 5.9 Design of webs

### 5.9.1 General

The geometry and arrangement of beam webs, including any transverse or longitudinal stiffeners, shall satisfy [Clause 5.10](#).

A web subject to shear force shall satisfy [Clause 5.11](#).

A web subject to shear force and bending moment shall satisfy [Clause 5.12](#).

A web subject to bearing load shall satisfy [Clause 5.13](#).

Load-bearing stiffeners and end posts shall satisfy [Clause 5.14](#).

Intermediate transverse stiffeners shall satisfy [Clause 5.15](#).

Longitudinal stiffeners shall satisfy [Clause 5.16](#).

### 5.9.2 Definition of web panel

A web panel of thickness ( $t_w$ ) shall be assumed to extend over an unstiffened area of a web plate with longitudinal dimension ( $s$ ) and clear transverse dimension ( $d_p$ ).

The web panel may be bounded by flanges, transverse or longitudinal stiffeners, or free edges.

### 5.9.3 Minimum thickness of web panel

Unless a rational analysis would warrant a lesser value, the thickness of a web panel shall satisfy [Clauses 5.10.1](#), [5.10.4](#), [5.10.5](#) and [5.10.6](#).

## 5.10 Arrangement of webs

### 5.10.1 Unstiffened webs

The thickness of an unstiffened web bounded on both longitudinal sides by flanges shall not be less than —

$$\left( \frac{d_1}{180} \right) \sqrt{\left( \frac{f_y}{250} \right)}$$

where  $d_1$  is the clear depth of the web between flanges, ignoring fillets or welds.

The thickness of an unstiffened web bounded on one longitudinal side by a free edge shall not be less than —

$$\left( \frac{d_1}{90} \right) \sqrt{\left( \frac{f_y}{250} \right)}$$

where  $d_1$  is the clear depth of the web, ignoring fillets or welds.

### 5.10.2 Load bearing stiffeners

Load bearing stiffeners shall be provided where the design compressive bearing forces applied through a flange by loads or reactions exceed the design bearing capacity ( $\phi R_b$ ) of the web alone specified in [Clause 5.13.2](#), or when required to form an end post ([Clause 5.15.2.2](#)).

### 5.10.3 Side reinforcing plates

Additional side reinforcing plates may be provided to augment the strength of the web. Proper account shall be taken of any lack of symmetry. The proportion of shear force assumed to be resisted by such plates shall be limited by the amount of horizontal shear which can be transmitted through the fasteners to the web and to the flanges.

### 5.10.4 Transversely stiffened webs

The thickness of a web transversely stiffened but without longitudinal stiffeners shall not be less than —

- (a)  $\left(\frac{d_1}{200}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $1.0 \leq s/d_1 \leq 3.0$ ;
- (b)  $\left(\frac{s}{200}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $0.74 < s/d_1 \leq 1.0$ ; or
- (c)  $\left(\frac{d_1}{270}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $s/d_1 \leq 0.74$ .

All web lengths for which  $s/d_p$  is greater than 3.0 shall be assumed to be unstiffened, where  $d_p$  is the greatest panel depth in the length.

### 5.10.5 Webs with longitudinal and transverse stiffeners

The thickness of a web with a set of longitudinal stiffeners placed on one or both sides of the web at a distance  $0.2d_2$  from the compression flange shall not be less than —

- (a)  $\left(\frac{d_1}{250}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $1.0 \leq s/d_1 \leq 2.4$ ;
- (b)  $\left(\frac{s}{250}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $0.74 \leq s/d_1 \leq 1.0$ ; or
- (c)  $\left(\frac{d_1}{340}\right) \sqrt{\left(\frac{f_y}{250}\right)}$  when  $s/d_1 < 0.74$

where  $d_2$  is twice the clear distance from the neutral axis to the compression flange.

The thickness of a web with an additional set of longitudinal stiffeners placed on one or both sides of the web at the neutral axis shall be not less than —

$$\left(\frac{d_1}{400}\right) \sqrt{\left(\frac{f_y}{250}\right)} \text{ when } s/d_1 \leq 1.5$$

### 5.10.6 Webs of members designed plastically

The web thickness of a member assumed to contain a plastic hinge shall not be less than  $(d_1/82)\sqrt{(f_y/250)}$ .

Load bearing stiffeners shall be provided when a bearing load or shear force acts within  $d_1/2$  of a plastic hinge location and the design bearing load or design shear force exceeds 0.1 times the design shear yield capacity ( $\phi V_w$ ) of the member specified in [Clause 5.11.4](#).

These stiffeners shall be located within a distance  $d_1/2$  on either side of the hinge location and shall be designed in accordance with [Clause 5.14](#) to carry the greater of the design bearing load or the design shear force considered as a bearing load.

If the stiffeners are flat plates, their slenderness ( $\lambda_s$ ) as defined in [Clause 5.2.2](#) using the stiffener yield stress ( $f_{ys}$ ) shall be less than the plasticity limit ( $\lambda_{sp}$ ) specified in [Clause 5.2.2](#).

### 5.10.7 Openings in webs

Except for a castellated member, an opening in a web may be unstiffened provided that the greatest internal dimension of the opening ( $l_w$ ) satisfies either —

- (a)  $l_w/d_1 \leq 0.10$  for webs without longitudinal stiffeners; or
- (b)  $l_w/d_1 \leq 0.33$  for longitudinally stiffened webs,

provided that the longitudinal distance between boundaries of adjacent openings is at least three times the greatest internal dimension of the opening.

In addition, not more than one unstiffened opening shall be provided at any cross-section unless a rational analysis shows that stiffeners are not necessary.

The design of a castellated member or a member with stiffened openings shall be based on a rational analysis.

## 5.11 Shear capacity of webs

### 5.11.1 Shear capacity

A web subject to a design shear force ( $V^*$ ) shall satisfy —

$$V^* \leq \phi V_v$$

where

$\phi$  = capacity factor (see [Table 3.4](#))

$V_v$  = nominal shear capacity of the web determined from either [Clause 5.11.2](#) or [Clause 5.11.3](#)

NOTE The shear capacity of the section may be limited by the shear capacity of the flange-to-web connection, which should be checked.

### 5.11.2 Approximately uniform shear stress distribution

The nominal shear capacity ( $V_v$ ) of a web where the shear stress distribution is approximately uniform shall be taken as —

$$V_v = V_u$$

where  $V_u$  is the nominal shear capacity of a web with a uniform shear stress distribution given as follows:

- (a) When the maximum web panel depth to thickness ratio  $d_p/t_w$  satisfies —



$$\frac{d_p}{t_w} \leq \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity of the web ( $V_u$ ) shall be taken as —

$$V_u = V_w$$

where the nominal shear yield capacity of the web ( $V_w$ ) is specified in [Clause 5.11.4](#).

(b) When the maximum web panel depth to thickness ratio  $d_p/t_w$  satisfies —

$$\frac{d_p}{t_w} > \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity ( $V_u$ ) of the web shall be taken as —

$$V_u = V_b$$

where the nominal shear buckling capacity of the web ( $V_b$ ) is specified in [Clause 5.11.5](#).

### 5.11.3 Non-uniform shear stress distribution

The nominal shear capacity ( $V_v$ ) of a web with a non-uniform shear stress distribution, such as in a member with unequal flanges, varying web thickness or holes not used for fasteners, shall be calculated as follows:

$$V_v = \frac{2V_u}{0.9 + \left(\frac{f_{vm}^*}{f_{va}^*}\right)} \leq V_u$$

where

$V_u$  = nominal shear capacity of a web with a uniform shear stress distribution determined in accordance with [Clause 5.11.2](#)

$f_{vm}^*, f_{va}^*$  = maximum and average design shear stresses in the web determined by a rational elastic analysis

For a circular hollow section,  $V_v$  shall be taken as the nominal shear yield capacity ( $V_w$ ) specified in [Clause 5.11.4](#).

### 5.11.4 Shear yield capacity

The nominal shear yield capacity ( $V_w$ ) of a web shall be calculated as follows:

$$V_w = 0.6f_y A_w$$

where  $A_w$  is the gross sectional area of the web.

The nominal shear yield capacity ( $V_w$ ) of a circular hollow section shall be calculated as follows:

$$V_w = 0.36f_y A_e$$

where the effective sectional area ( $A_e$ ) shall be taken as the gross area of the circular hollow section provided either that there are no holes larger than those required for fasteners, or that the net area is greater than 0.9 times the gross area, or otherwise as the net area.

### 5.11.5 Shear buckling capacity

#### 5.11.5.1 Unstiffened web

The nominal shear buckling capacity ( $V_b$ ) for an unstiffened web or a web considered to be unstiffened (see [Clause 5.10.4](#)) shall be calculated as follows:

$$V_b = \alpha_v V_w \leq V_w$$

where

$$\alpha_v = \left[ \frac{82}{\left(\frac{d_p}{t_w}\right) \sqrt{\left(\frac{f_y}{250}\right)}} \right]^2$$

#### 5.11.5.2 Stiffened web

The nominal shear buckling capacity ( $V_b$ ) for a stiffened web with  $s/d_p \leq 3.0$  shall be calculated as follows:

$$V_b = \alpha_v \alpha_d \alpha_f V_w \leq V_w$$

where

$$\alpha_v = \left[ \frac{82}{\left(\frac{d_p}{t_w}\right) \sqrt{\left(\frac{f_y}{250}\right)}} \right]^2 \left[ \frac{0.75}{\left(\frac{s}{d_p}\right)^2} + 1.0 \right] \leq 1.0 \text{ when } 1.0 \leq s/d_p \leq 3.0$$

$$\alpha_v = \left[ \frac{82}{\left(\frac{d_p}{t_w}\right) \sqrt{\left(\frac{f_y}{250}\right)}} \right]^2 \left[ \frac{1}{\left(\frac{s}{d_p}\right)^2} + 0.75 \right] \leq 1.0 \text{ when } s/d_p \leq 1.0, \text{ and}$$

$$\alpha_d = 1 + \frac{1 - \alpha_v}{1.15 \alpha_v \sqrt{\left[ 1 + \left(\frac{s}{d_p}\right)^2 \right]}} ; \text{ or}$$

$\alpha_d = 1.0$  when required by [Clause 5.15.2.2](#), and

$d_p =$  depth of the deepest web panel.

Values of the product  $\alpha_v\alpha_d$  are given in [Table 5.11.5.2](#).

The flange restraint factor ( $\alpha_f$ ) shall be taken as either —

(a)  $\alpha_f = 1.0$ ; or

(b) 
$$\alpha_f = 1.6 - \frac{0.6}{\sqrt{\left[1 + \left(\frac{40b_{fo}t_f^2}{d_1^2t_w}\right)\right]}}$$
 for webs without longitudinal stiffeners, in which  $b_{fo}$  is the least of all of the following:

(i)  $\frac{12t_f}{\sqrt{(f_y / 250)}}$  ;

(ii) the distance from the mid-plane of the web to the nearer edge of the flange (taken as zero if there is no flange outstand);

(iii) half the clear distance between the webs if there are two or more webs; or

(c) shall be determined from a rational buckling analysis

NOTE Guidance on the shear buckling capacity of a web which contains an axial load is given in [Appendix I](#).

**Table 5.11.5.2 — Values of  $\alpha_v\alpha_d$**

$\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}$	$\frac{s}{d_p}$										
	0.3	0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.5	3.0	> 3.0
90	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.991	0.952	0.927	0.830
100	1.000	1.000	1.000	1.000	0.989	0.946	0.907	0.877	0.833	0.803	0.672
110	1.000	1.000	1.000	0.998	0.919	0.866	0.825	0.792	0.744	0.711	0.556
120	1.000	1.000	1.000	0.930	0.859	0.805	0.762	0.728	0.677	0.642	0.467
130	1.000	1.000	1.000	0.883	0.812	0.757	0.713	0.678	0.625	0.587	0.398
140	1.000	1.000	0.960	0.846	0.775	0.719	0.674	0.638	0.583	0.544	0.343
150	1.000	1.000	0.926	0.816	0.745	0.689	0.643	0.606	0.550	0.510	0.299
160	1.000	1.000	0.898	0.792	0.721	0.664	0.617	0.579	0.522	0.481	0.263
170	1.000	1.000	0.875	0.772	0.701	0.643	0.596	0.558	0.499	0.458	0.233
180	1.000	0.997	0.855	0.755	0.684	0.626	0.578	0.539	0.480	0.438	0.208
190	1.000	0.974	0.839	0.740	0.669	0.611	0.563	0.524	0.464	0.421	
200	1.000	0.955	0.825	0.728	0.657	0.598	0.550	0.511	0.450	0.407	
210	1.000	0.939	0.813	0.718							
220	1.000	0.924	0.803	0.709							
230	1.000	0.912	0.793	0.701							
240	1.000	0.901	0.785	0.694							
250	1.000	0.891	0.778	0.687							
260	1.000	0.883	0.772	0.682							
270	1.000	0.875	0.767	0.677							

## 5.12 Interaction of shear and bending

### 5.12.1 General

The nominal web shear capacity ( $V_{vm}$ ) in the presence of bending moment shall be calculated using the provisions of either [Clause 5.12.2](#) or [5.12.3](#).

### 5.12.2 Proportioning method

When the bending moment is assumed to be resisted only by the flanges and the design bending moment ( $M^*$ ) satisfies —

$$M^* \leq \phi M_f$$

where  $M_f$  is the nominal moment capacity calculated for the flanges alone and determined as follows:

$$M_f = A_{fm} d_f f_y$$

where

- $A_{fm}$  = lesser of the flange effective areas, determined using [Clause 6.2.2](#) for the compression flange and the lesser of  $A_{fg}$  and  $0.85A_{fn} f_u/f_y$  for the tension flange
- $A_{fg}$  = gross area of the flange
- $A_{fn}$  = net area of the flange
- $d_f$  = distance between flange centroids,

the member shall satisfy —

$$V^* \leq \phi V_{vm}$$

where

$$V_{vm} = V_v$$

and  $V_v$  is the nominal web shear capacity determined either from [Clause 5.11.2](#) or [Clause 5.11.3](#).

### 5.12.3 Shear and bending interaction method

When the bending moment is assumed to be resisted by the whole of the cross-section, the member shall be designed for combined bending and shear, and shall satisfy —

$$V^* \leq \phi V_{vm}$$

where

$$V_{vm} = \begin{cases} V_v & \text{for } M^* \leq 0.75\phi M_s; \text{ or} \\ V_v \left[ 2.2 - \left( \frac{1.6M^*}{\phi M_s} \right) \right] & \text{for } 0.75\phi M_s < M^* \leq \phi M_s; \end{cases}$$

where

$V_v$  = nominal shear capacity of a web in shear alone (see [Clause 5.11.1](#))

$M_s$  = nominal section moment capacity determined in accordance with [Clause 5.2](#)

NOTE Guidance on stiffened web panels required to resist bending moment, shear, axial and transverse loading is given in [Appendix I](#).

## 5.13 Compressive bearing action on the edge of a web

### 5.13.1 Dispersion of force to web

Where a force is applied to a flange either as a point load or through a stiff bearing of length ( $b_s$ ), it shall be assumed to be dispersed uniformly through the flange at a slope of 1:2.5 to the surface of the flange, as shown in [Figure 5.13.1.1](#), or to the top of the flat portion of the web for rectangular and square hollow sections to AS/NZS 1163, as shown in [Figure 5.13.1.3](#). The stiff bearing length is that length which cannot deform appreciably in bending. The dispersion of load to the flange shall be taken at a slope of 1:1 through solid material, as shown in [Figure 5.13.1.2](#).

### 5.13.2 Bearing capacity

The design bearing force ( $R^*$ ) on a web shall satisfy —

$$R^* \leq \phi R_b$$

where

$\phi$  = the capacity factor (see [Table 3.4](#))

$R_b$  = the nominal bearing capacity of the web under concentrated or patch loading, which shall be taken as the lesser of its nominal bearing yield capacity ( $R_{by}$ ) defined in [Clause 5.13.3](#), and its nominal bearing buckling capacity ( $R_{bb}$ ) defined in [Clause 5.13.4](#)

### 5.13.3 Bearing yield capacity

The nominal bearing yield capacity ( $R_{by}$ ) of a web shall be calculated as follows:

$$R_{by} = 1.25 b_{bf} t_w f_y$$

where  $b_{bf}$  is the bearing width shown in [Figure 5.13.1.1](#), except that for square and rectangular hollow sections to AS/NZS 1163, the nominal bearing yield capacity ( $R_{by}$ ) of both webs shall be calculated as follows:

$$R_{by} = 2 b_b t f_y \alpha_p$$

where

$b_b$  = bearing width (see [Figures 5.13.1.3\(b\) and \(c\)](#))

$\alpha_p$  = coefficient used to calculate the nominal bearing yield capacity ( $R_{by}$ ) for square and rectangular hollow sections to AS/NZS 1163

The coefficient ( $\alpha_p$ ) shall be determined as follows:

- (a) For interior bearing, where  $b_d$  is greater than or equal to  $1.5d_5$  —

$$\alpha_p = \frac{0.5}{k_s} \left[ 1 + \left( 1 - \alpha_{pm}^2 \right) \left( 1 + \frac{k_s}{k_v} - \left( 1 - \alpha_{pm}^2 \right) \frac{0.25}{k_v^2} \right) \right]$$

where

$b_d$  = distance from the stiff bearing to the end of the member (see [Figure 5.13.1.3\(b\)](#))

$d_5$  = flat width of web (see [Figure 5.13.1.3\(a\)](#))

$\alpha_{pm}$  = coefficient used to calculate  $\alpha_p$

$$= \frac{1}{k_s} + \frac{0.5}{k_v}$$

$k_s$  = ratio used to calculate  $\alpha_p$  and  $\alpha_{pm}$

$$= \frac{2r_{ext}}{t} - 1$$

$k_v$  = ratio of flat width of web ( $d_5$ ) to thickness ( $t$ ) of section

$$= \frac{d_5}{t}$$

$r_{ext}$  = outside radius of section (see [Figure 5.13.1.3\(a\)](#)).

The bearing width ( $b_b$ ) shall be calculated as follows:

$$b_b = b_s + 5r_{ext} + d_5$$

(b) For end bearing, where  $b_d$  is less than  $1.5d_5$  —

$$\alpha_p = \sqrt{(2 + k_s^2)} - k_s$$

The bearing width ( $b_b$ ) shall be calculated as follows:

$$b_b = b_s + 2.5r_{ext} + \frac{d_5}{2}$$

NOTE Guidance on the nominal yield capacity of a stiffened web in bearing in the presence of bending moment and axial load is given in [Appendix I](#).

#### 5.13.4 Bearing buckling capacity

The nominal bearing buckling capacity ( $R_{bb}$ ) of an I-section or C-section web without transverse stiffeners shall be taken as the axial compression capacity determined in accordance with [Section 6](#) using the following parameters:

- $\alpha_b = 0.5$ .
- $k_f = 1.0$ .
- area of web =  $t_w b_b$ .
- geometrical slenderness ratio taken as  $2.5d_1/t_w$  when the top and bottom flanges are effectively restrained against lateral movement out of the plane of the web or  $5.0d_1/t_w$  when only one flange is effectively restrained against lateral movement.
- $b_b$  is the total bearing width obtained by dispersions at a slope of 1:1 from  $b_{bf}$  to the neutral axis (if available), as shown in [Figure 5.13.1.1](#).

The nominal bearing buckling capacity ( $R_{bb}$ ) of a square or rectangular hollow section web to AS/NZS 1163 without transverse stiffeners shall be taken as the axial compression capacity determined in accordance with [Section 6](#) using the following parameters:

- (i)  $\alpha_b = 0.5$ .
- (ii)  $k_f = 1.0$ .
- (iii) area of web =  $t_w b_b$ .
- (iv) geometrical slenderness ratio taken as  $3.5d_5/t_w$  for interior bearing ( $b_d \geq 1.5d_5$ ) or  $3.8d_5/t_w$  for end bearing ( $b_d < 1.5d_5$ ).
- (v)  $b_b$  is the total bearing width as shown in [Figure 5.13.1.3](#).

NOTE Guidance on the nominal bearing buckling capacity ( $R_{bb}$ ) of a stiffened web with a bearing load between the stiffeners in the presence of bending moment and axial load is given in [Appendix I](#).

### 5.13.5 Combined bending and bearing of rectangular and square hollow sections

Rectangular and square hollow sections to AS/NZS 1163 subjected to combined bending and bearing force shall satisfy [Clauses 5.2](#), [5.13.2](#), and either —

$$1.2 \left( \frac{R^*}{\phi R_b} \right) + \left( \frac{M^*}{\phi M_s} \right) \leq 1.5 \quad \text{for } \frac{b_s}{b} \geq 1.0 \text{ and } \frac{d_1}{t_w} \leq 30$$

or

$$0.8 \left( \frac{R^*}{\phi R_b} \right) + \left( \frac{M^*}{\phi M_s} \right) \leq 1.0 \quad \text{otherwise}$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $R_b$  = nominal bearing capacity of a web specified in [Clause 5.13.2](#)
- $M_s$  = nominal section moment capacity determined in accordance with [Clause 5.2](#)
- $b_s$  = stiff bearing length
- $b$  = total width of section

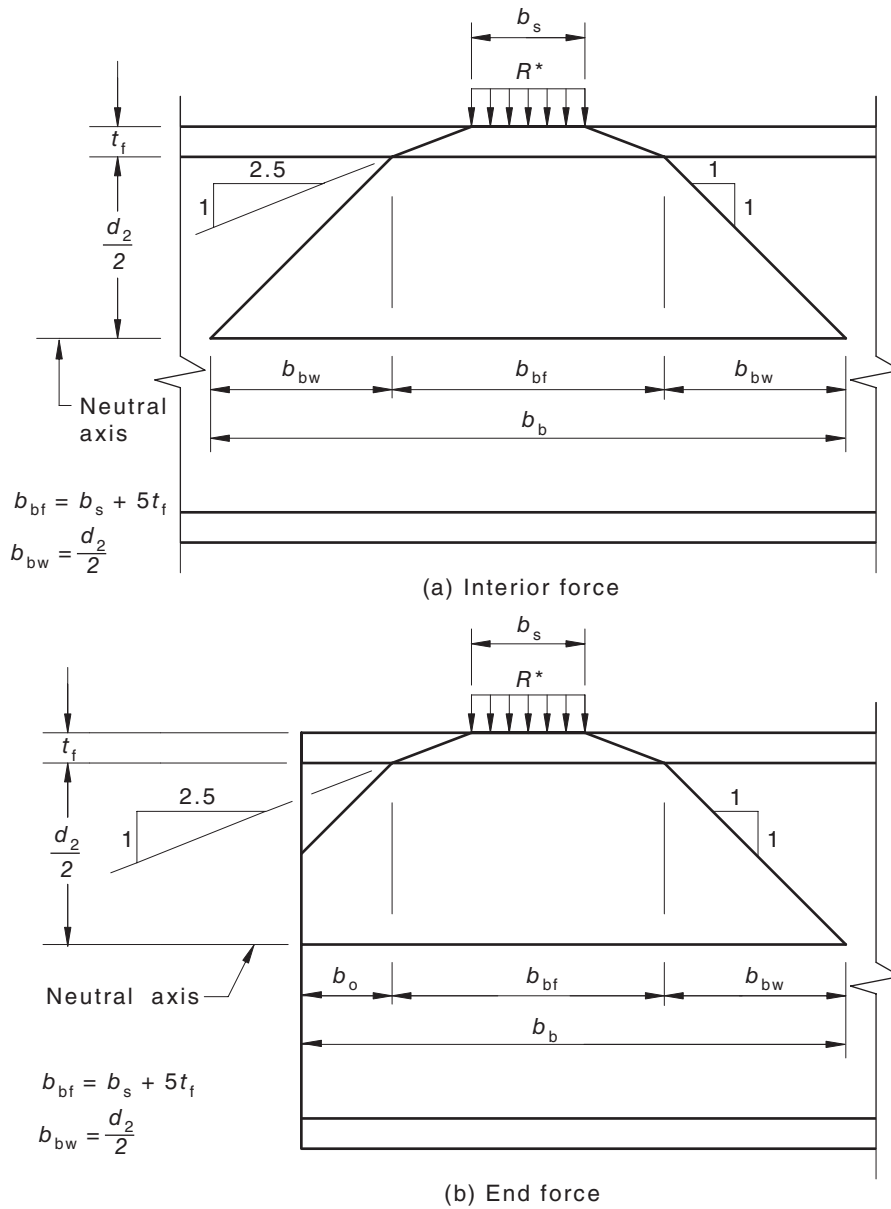


Figure 5.13.1.1 — Dispersions of force through flange and web

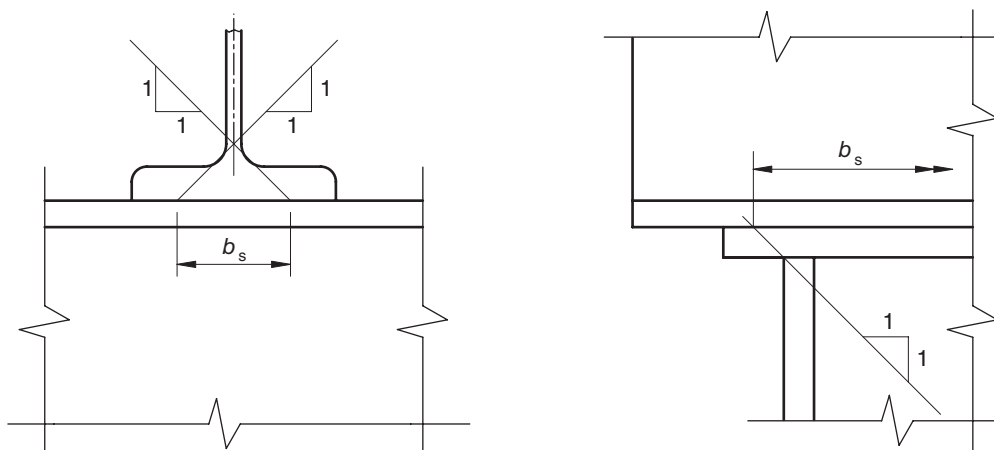
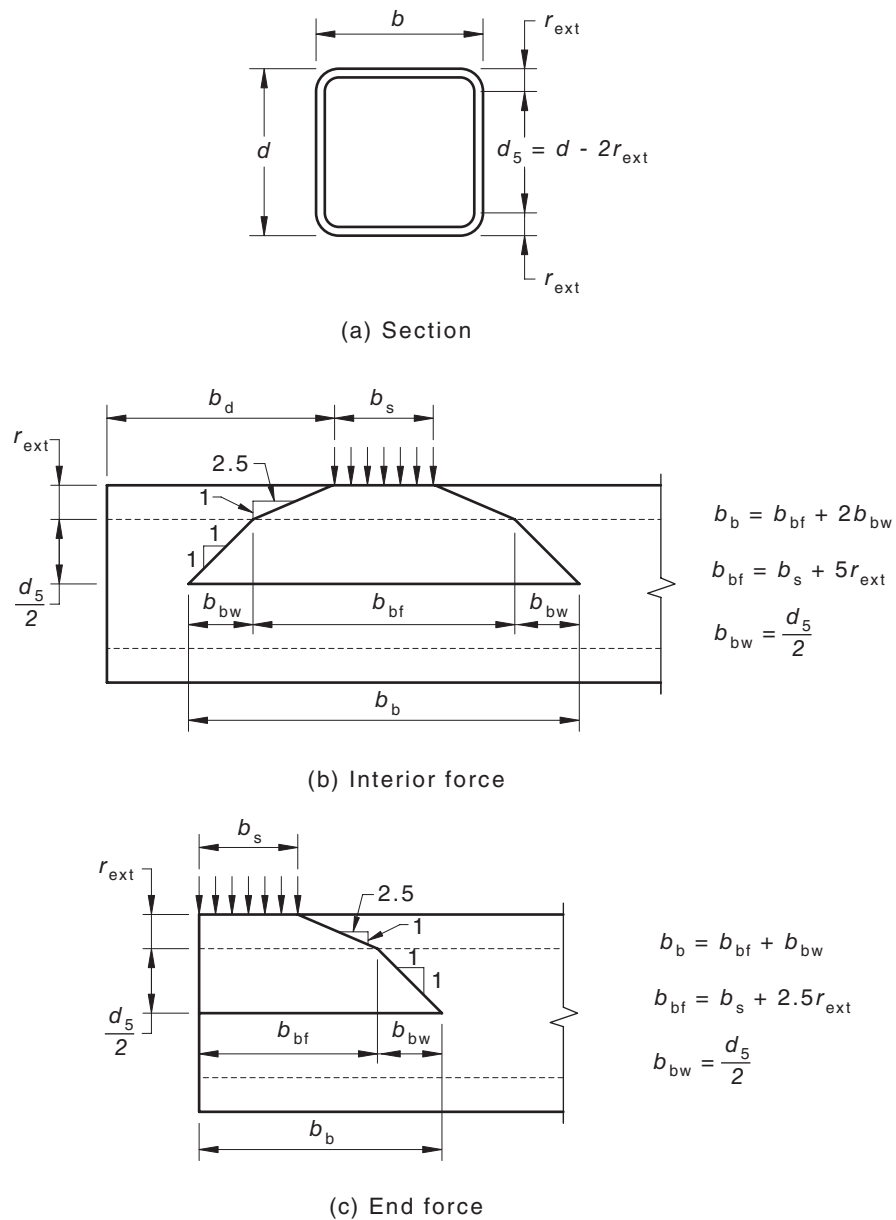


Figure 5.13.1.2 — Stiff bearing length on flange





**Figure 5.13.1.3 — Rectangular and square hollow sections — Dispersion of force through flange, radius and web**

## 5.14 Design of load bearing stiffeners

### 5.14.1 Yield capacity

When a load bearing stiffener is required, it shall satisfy —

$$R^* \leq \phi R_{sy}$$

where

- $R^*$  = design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener  
 $\phi$  = capacity factor (see [Table 3.4](#))  
 $R_{Sy}$  = nominal yield capacity of the stiffened web  
 $\quad = R_{by} + A_s f_{ys}$   
 $R_{by}$  = nominal bearing yield capacity (see [Clause 5.13.3](#))  
 $A_s$  = area of the stiffener in contact with the flange  
 $f_{ys}$  = yield stress of the stiffener

### 5.14.2 Buckling capacity

When a load bearing stiffener is required, it shall satisfy —

$$R^* \leq \phi R_{sb}$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))  
 $R_{sb}$  = nominal buckling capacity of the stiffened web, determined in accordance with [Section 6](#) using  $\alpha_b$  equals 0.5 and  $k_f$  equals 1.0 for a compression member whose radius of gyration is taken about the axis parallel to the web

The effective section of the compression member shall be taken as the area of the stiffener, together with a length of web on each side of the centreline not greater than the lesser of —

$$\frac{17.5t_w}{\sqrt{\left(\frac{f_y}{250}\right)}} \text{ and } \frac{s}{2}, \text{ if available.}$$

The effective length ( $l_e$ ) of the compression member used in calculating the buckling capacity ( $R_{sb}$ ) shall be determined as either —

$$l_e = 0.7d_1$$

where the flanges are restrained by other structural elements against rotation in the plane of the stiffener, or —

$$l_e = d_1$$

if either of the flanges is not so restrained.

### 5.14.3 Outstand of stiffeners

Unless the outer edge of a flat stiffener is continuously stiffened, the stiffener outstand from the face of a web ( $b_{es}$ ) shall satisfy —

$$b_{es} \leq \frac{15t_s}{\sqrt{\left(\frac{f_{ys}}{250}\right)}}$$

where

- $t_s$  = thickness of the stiffener  
 $f_{ys}$  = yield stress of the stiffener used in design

### 5.14.4 Fitting of load bearing stiffeners

A load bearing stiffener shall be fitted to provide a tight and uniform bearing against the loaded flange, unless welds are provided between the flange and stiffener for the purpose of transmitting the concentrated force or reaction. Where a point of concentrated force is directly over a support, this provision shall apply to both flanges.

Load bearing stiffeners shall be provided with sufficient welds or bolts to transmit their share of the design bearing force or design reaction ( $R^*$ ) to the web.

### 5.14.5 Design for torsional end restraint

When load bearing stiffeners are the sole means of providing torsional end restraint at the supports of a member, the second moment of area of a pair of stiffeners ( $I_s$ ) about the centreline of the web shall be such that —

$$I_s \geq \left(\frac{\alpha_t}{1000}\right) \left(\frac{d^3 t_f R^*}{F^*}\right)$$

where

- $\alpha_t$  =  $\frac{230}{(l_e / r_y)} - 0.60$  and  $0 \leq \alpha_t \leq 4$   
 $R^*$  = design reaction at the bearing  
 $F^*$  = total design load on the member between supports  
 $t_f$  = thickness of the critical flange (see [Clause 5.5](#))  
 $(l_e / r_y)$  = load bearing stiffener slenderness ratio used in [Clause 5.14.2](#)

## 5.15 Design of intermediate transverse web stiffeners

### 5.15.1 General

Intermediate transverse web stiffeners shall extend between each flange and shall terminate no further from a flange than four times the web thickness.

NOTE Intermediate stiffeners may be provided on one or both sides of a web.

## 5.15.2 Spacing

### 5.15.2.1 Interior panels

The spacing of intermediate web stiffeners ( $s$ ) that define internal panels shall satisfy [Clause 5.10.4](#) or [Clause 5.10.5](#).

### 5.15.2.2 End panels

An end panel shall be provided with an end post which satisfies [Clause 5.15.9](#), unless the width of the end panel ( $s$ ) is reduced so that its shear buckling capacity ( $V_b$ ) calculated by using  $\alpha_d$  equals 1.0 in [Clause 5.11.5.2](#) satisfies [Clauses 5.11.1](#) and [5.12](#).

### 5.15.3 Minimum area

An intermediate web stiffener not subject to external loads or moments shall have an area  $A_s$  which satisfies —

$$A_s \geq 0.5\gamma A_w (1 - \alpha_v) \left( \frac{V^*}{\phi V_u} \right) \left[ \left( \frac{s}{d_p} \right) - \frac{\left( \frac{s}{d_p} \right)^2}{\sqrt{1 + \left( \frac{s}{d_p} \right)^2}} \right]$$

where

- $\alpha_v$  = shear buckling coefficient for the web, determined in accordance with [Clause 5.11.5.2](#)
- $\gamma$  = 1.0 for a pair of stiffeners
- = 1.8 for a single angle stiffener
- = 2.4 for a single plate stiffener

### 5.15.4 Buckling capacity

An intermediate web stiffener shall satisfy —

$$V^* \leq \phi (R_{sb} + V_b)$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $V_b$  = nominal shear buckling capacity specified in [Clause 5.11.5.2](#) for a stiffened web using  $\alpha_d$  equals 1.0 and  $\alpha_f$  equals 1.0
- $R_{sb}$  = nominal buckling capacity of the intermediate stiffener determined in accordance with [Clause 5.14.2](#)

The effective length ( $l_e$ ) of the compression member used in calculating  $R_{sb}$  shall be taken as —

$$l_e = d_1$$

### 5.15.5 Minimum stiffness

An intermediate web stiffener not subject to external loads or moments shall have a minimum second moment of area ( $I_s$ ) about the centreline of the web such that —

$$I_s \geq 0.75d_1t_w^3 \text{ for } \frac{s}{d_1} \leq \sqrt{2} \text{ ; and}$$

$$I_s \geq \frac{1.5d_1t_w^3}{s^2} \text{ for } \frac{s}{d_1} > \sqrt{2}$$

### 5.15.6 Outstand of stiffeners

The outstand ( $b_{es}$ ) of an intermediate web stiffener shall satisfy [Clause 5.14.3](#).

### 5.15.7 External forces

#### 5.15.7.1 Increase in stiffness

Where an intermediate stiffener is used to transfer design forces ( $F_n^*$ ) normal to the web or design moments ( $M^* + F_p^*e$ ) acting normal to the web [including moments ( $F_p^*e$ ) caused by any eccentric force  $F_p^*$  parallel to the web], the minimum value of  $I_s$  in [Clause 5.15.5](#) shall be increased by —

$$\frac{d_1^4 \left\{ 2F_n^* + [(M^* + F_p^*e) / d_1] \right\}}{\phi E d_1 t_w}$$

#### 5.15.7.2 Increase in strength

When an intermediate stiffener is required to carry a transverse load parallel to the web, it shall be designed as a load bearing stiffener in accordance with [Clause 5.14](#).

### 5.15.8 Connection of intermediate stiffeners to web

The web connections of intermediate transverse stiffeners not subject to external loading shall be designed to resist a design shear force per unit length, in kilonewtons per millimetre (kN/mm), of not less than —

$$\frac{0.0008(t_w)^2 f_y}{b_{es}}$$

where

$b_{es}$  = outstand width of the stiffener from the face of the web, in millimetres

$t_w$  = web thickness, in millimetres

### 5.15.9 End posts

When an end post is required by [Clause 5.15.2.2](#), it shall be formed by a load bearing stiffener and a parallel end plate. The load bearing stiffener shall be designed in accordance with [Clause 5.14](#), and shall be no smaller than the end plate. The area of the end plate ( $A_{ep}$ ) shall satisfy —

$$A_{ep} \geq \frac{d_1 \left[ (V^* / \phi) - \alpha_v V_w \right]}{8ef_y}$$

where

$\alpha_v$  = as given in [Clause 5.11.5.2](#)

$V_w$  = as given in [Clause 5.11.4](#)

$e$  = distance between the end plate and load bearing stiffener

## 5.16 Design of longitudinal web stiffeners

### 5.16.1 General

Longitudinal web stiffeners shall be continuous or shall extend between and be attached to transverse web stiffeners.

### 5.16.2 Minimum stiffness

When a longitudinal stiffener is required at a distance  $0.2d_2$  from the compression flange, it shall have a second moment of area ( $I_s$ ) about the face of the web such that —

$$I_s \geq 4d_2 t_w^3 \left[ 1 + \frac{4A_s}{d_2 t_w} \left( 1 + \frac{A_s}{d_2 t_w} \right) \right]$$

where  $A_s$  is the area of the stiffener.

When a second longitudinal stiffener is required at the neutral axis of the section, it shall have a second moment of area ( $I_s$ ) about the face of the web such that —

$$I_s \geq d_2 t_w^3$$

## Section 6 Members subject to axial compression

### 6.1 Design for axial compression

A concentrically loaded member subject to a design axial compression force ( $N^*$ ) shall satisfy both —

$$N^* \leq \phi N_s, \text{ and}$$

$$N^* \leq \phi N_c$$

where

$\phi$  = capacity factor (see [Table 3.4](#))

$N_s$  = nominal section capacity determined in accordance with [Clause 6.2](#)

$N_c$  = nominal member capacity determined in accordance with [Clause 6.3](#)

### 6.2 Nominal section capacity

#### 6.2.1 General

The nominal section capacity ( $N_s$ ) of a concentrically loaded compression member shall be calculated as follows:

$$N_s = k_f A_n f_y$$

where

$k_f$  = form factor given in [Clause 6.2.2](#)

$A_n$  = net area of the cross-section, except that for sections with penetrations or unfilled holes that reduce the section area by less than  $100\{1 - [f_y/(0.85f_u)]\}\%$ , the gross area may be used. Deductions for fastener holes shall be made in accordance with [Clause 9.1.10](#)

#### 6.2.2 Form factor

The form factor ( $k_f$ ) shall be calculated as follows:

$$k_f = \frac{A_e}{A_g}$$

where

$A_e$  = effective area

$A_g$  = gross area of the section

The effective area ( $A_e$ ) shall be calculated from the gross area by summing the effective areas of the individual elements, whose effective widths are specified in [Clause 6.2.4](#).

### 6.2.3 Plate element slenderness

The slenderness ( $\lambda_e$ ) of a flat plate element shall be calculated as follows:

$$\lambda_e = \frac{b}{t} \sqrt{\left(\frac{f_y}{250}\right)}$$

where

$b$  = clear width of the element outstand from the face of the supporting plate element, or  
clear width of the element between the faces of the supporting plate elements

$t$  = thickness of the plate

For circular hollow sections, the element slenderness ( $\lambda_e$ ) shall be calculated as follows:

$$\lambda_e = \left(\frac{d_o}{t}\right) \sqrt{\left(\frac{f_y}{250}\right)}$$

where

$d_o$  = outside diameter of the section

$t$  = wall thickness of the section

### 6.2.4 Effective width

The effective width ( $b_e$ ) of a flat plate element of clear width ( $b$ ), or the effective outside diameter ( $d_e$ ) of a circular hollow section of outside diameter ( $d_o$ ), shall be calculated from the value of the element slenderness ( $\lambda_e$ ) given in [Clause 6.2.3](#) and the element yield slenderness limit ( $\lambda_{ey}$ ) given in [Table 6.2.4](#).

The effective width ( $b_e$ ) for a flat plate element shall be calculated as follows:

$$b_e = b \left(\frac{\lambda_{ey}}{\lambda_e}\right) \leq b$$

The effective outside diameter ( $d_e$ ) for a circular hollow section shall be the lesser of the following:

$$d_e = d_o \sqrt{\left(\frac{\lambda_{ey}}{\lambda_e}\right)} \leq d_o$$

$$d_e = d_o \left(\frac{3\lambda_{ey}}{\lambda_e}\right)^2$$

Alternatively, the effective width ( $b_e$ ) for a flat plate element may be obtained from the following:

$$b_e = b \left(\frac{\lambda_{ey}}{\lambda_e}\right) \sqrt{\left(\frac{k_b}{k_{bo}}\right)} \leq b$$

where  $k_b$  is the elastic buckling coefficient for the element.



For a flat plate element supported along both longitudinal edges —

$$k_{bo} = 4.0$$

and for a flat plate element supported along one longitudinal edge (outstand) —

$$k_{bo} = 0.425$$

The elastic buckling coefficient ( $k_b$ ) for the flat plate element shall be determined from a rational elastic buckling analysis of the whole member as a flat plate assemblage.

**Table 6.2.4 — Values of plate element yield slenderness limit**

Plate element type	Longitudinal edges supported	Residual stresses (see Note)	Yield slenderness limit, ( $\lambda_{ey}$ )
Flat	One (Outstand)	SR	16
		HR	16
		LW, CF	15
		HW	14
	Both	SR	45
		HR	45
		LW, CF	40
		HW	35
Circular hollow sections	SR	82	
	HR, CF	82	
	LW	82	
	HW	82	
<b>Key</b> SR = stress relieved HR = hot-rolled or hot-finished CF = cold-formed LW = lightly welded longitudinally HW = heavily welded longitudinally NOTE Welded members whose compressive residual stresses are less than 40 MPa may be considered to be lightly welded.			

## 6.3 Nominal member capacity

### 6.3.1 Definitions

For the purpose of this Clause, the definitions below apply.

*Geometrical slenderness ratio* — geometrical slenderness ratio ( $l_e/r$ ), taken as the effective length ( $l_e$ ), specified in [Clause 6.3.2](#), divided by the radius of gyration ( $r$ ) computed for the gross section about the relevant axis.

*Length* — actual length ( $l$ ) of an axially loaded member, taken as the length centre-to-centre of intersections with supporting members, or the cantilevered length in the case of free-standing members.

### 6.3.2 Effective length

The effective length ( $l_e$ ) of a compression member shall be determined as follows:

$$l_e = k_e l$$

where  $k_e$  is the member effective length factor determined in accordance with [Clause 4.6.3](#).

### 6.3.3 Nominal capacity of a member of constant cross-section subject to flexural buckling

The nominal member capacity ( $N_C$ ) of a member of constant cross-section subject to flexural buckling shall be determined as follows:

$$N_C = \alpha_c N_S \leq N_S$$

where

$N_S$  = nominal section capacity, determined in accordance with [Clause 6.2](#)

$\alpha_c$  = member slenderness reduction factor

$$= \xi \left[ 1 - \sqrt{1 - \left( \frac{90}{\xi \lambda} \right)^2} \right]$$

$$\xi = \frac{\left( \frac{\lambda}{90} \right)^2 + 1 + \eta}{2 \left( \frac{\lambda}{90} \right)^2}$$

$$\lambda = \lambda_n + \alpha_a \alpha_b$$

$$\eta = 0.00326(\lambda - 13.5) \geq 0$$

$$\lambda_n = \left( \frac{l_e}{r} \right) \sqrt{(k_f)} \sqrt{\left( \frac{f_y}{250} \right)}$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050}$$

$\alpha_b$  = appropriate member section constant given in [Table 6.3.3\(A\)](#) or [6.3.3\(B\)](#)

$k_f$  = form factor determined in accordance with [Clause 6.2.2](#)

Alternatively, values of the member slenderness reduction factor ( $\alpha_c$ ) may be obtained directly from [Table 6.3.3\(C\)](#) using the value of the modified member slenderness ( $\lambda_n$ ) and the appropriate member section constant ( $\alpha_b$ ) given in [Table 6.3.3\(A\)](#) or [6.3.3\(B\)](#).

Fabricated monosymmetric and non-symmetric sections other than unflipped angles, tees and cruciform sections, and hot-rolled channels braced about the minor principal axis, shall be designed for flexural torsional buckling in accordance with AS/NZS 4600 with a reduction factor of 0.85 applied to the nominal member capacity ( $N_C$ ). A capacity factor of 0.90 shall also be used.

**Table 6.3.3(A) — Values of member section constant ( $\alpha_b$ ) for  $k_f = 1.0$** 

Compression member section constant ( $\alpha_b$ )	Section description
-1.0	<ul style="list-style-type: none"> <li>— Hot-formed RHS and CHS</li> <li>— Cold-formed (stress relieved) RHS and CHS</li> </ul>
-0.5	<ul style="list-style-type: none"> <li>— Cold-formed (non-stress relieved) RHS and CHS</li> <li>— Welded H, I and box section fabricated from Grade 700 high strength quenched and tempered plate</li> </ul>
0	<ul style="list-style-type: none"> <li>— Hot-rolled UB and UC sections (flange thickness up to 40 mm)</li> <li>— Welded H and I sections fabricated from flame-cut plates</li> <li>— Welded box sections</li> </ul>
0.5	<ul style="list-style-type: none"> <li>— Tees flame-cut from universal sections, and angles</li> <li>— Hot-rolled channels</li> <li>— Welded H and I sections fabricated from as-rolled plates (flange thickness up to 40 mm)</li> <li>— Other sections not listed in this Table</li> </ul>
1.0	<ul style="list-style-type: none"> <li>— Hot-rolled UB and UC sections (flange thickness over 40 mm)</li> <li>— Welded H and I sections fabricated from as-rolled plates (flange thickness over 40 mm)</li> </ul>

**Table 6.3.3(B) — Values of member section constant ( $\alpha_b$ ) for  $k_f < 1.0$** 

Compression member section constant ( $\alpha_b$ )	Section description
-0.5	<ul style="list-style-type: none"> <li>— Hot-formed RHS and CHS</li> <li>— Cold-formed RHS and CHS (stress relieved)</li> <li>— Cold-formed RHS and CHS (non-stress relieved)</li> </ul>
0	<ul style="list-style-type: none"> <li>— Hot-rolled UB and UC sections (flange thickness up to 40 mm)</li> <li>— Welded box sections</li> </ul>
0.5	<ul style="list-style-type: none"> <li>— Welded H and I sections (flange thickness up to 40 mm)</li> </ul>
1.0	<ul style="list-style-type: none"> <li>— Other sections not listed in this Table</li> </ul>

**Table 6.3.3(C) — Values of member slenderness reduction factor ( $\alpha_c$ )**

Modified member slenderness ( $\lambda_n$ )	Compression member section constant ( $\alpha_b$ )				
	-1.0	-0.5	0	0.5	1.0
0	1.000	1.000	1.000	1.000	1.000
5	1.000	1.000	1.000	1.000	1.000
10	1.000	1.000	1.000	1.000	1.000
15	1.000	0.998	0.995	0.992	0.990
20	1.000	0.98	0.978	0.967	0.956
25	0.997	0.979	0.961	0.942	0.923
30	0.991	0.968	0.943	0.917	0.888
35	0.983	0.955	0.925	0.891	0.853

Table 6.3.3(C) (continued)

Modified member slenderness ( $\lambda_n$ )	Compression member section constant ( $\alpha_b$ )				
	-1.0	-0.5	0	0.5	1.0
40	0.973	0.940	0.905	0.865	0.818
45	0.959	0.924	0.884	0.837	0.782
50	0.944	0.905	0.861	0.808	0.747
55	0.927	0.885	0.836	0.778	0.711
60	0.907	0.862	0.809	0.746	0.676
65	0.886	0.837	0.779	0.714	0.642
70	0.861	0.809	0.748	0.680	0.609
75	0.835	0.779	0.715	0.646	0.576
80	0.805	0.746	0.681	0.612	0.545
85	0.772	0.711	0.645	0.579	0.516
90	0.737	0.675	0.610	0.547	0.487
95	0.700	0.638	0.575	0.515	0.461
100	0.661	0.600	0.541	0.485	0.435
105	0.622	0.564	0.508	0.457	0.412
110	0.584	0.528	0.477	0.431	0.389
115	0.546	0.495	0.448	0.406	0.368
120	0.510	0.463	0.421	0.383	0.348
125	0.476	0.434	0.395	0.361	0.330
130	0.445	0.406	0.372	0.341	0.313
135	0.416	0.381	0.350	0.322	0.297
140	0.389	0.357	0.330	0.304	0.282
145	0.364	0.336	0.311	0.288	0.268
150	0.341	0.316	0.293	0.273	0.255
155	0.320	0.298	0.277	0.259	0.242
160	0.301	0.281	0.263	0.246	0.231
165	0.283	0.265	0.249	0.234	0.220
170	0.267	0.251	0.236	0.222	0.210
175	0.252	0.238	0.224	0.212	0.200
180	0.239	0.225	0.213	0.202	0.192
185	0.226	0.214	0.203	0.193	0.183
190	0.214	0.203	0.193	0.184	0.175
195	0.204	0.194	0.185	0.176	0.168
200	0.194	0.185	0.176	0.168	0.161
205	0.184	0.176	0.168	0.161	0.154
210	0.176	0.168	0.161	0.154	0.148
215	0.167	0.161	0.154	0.148	0.142
220	0.160	0.154	0.148	0.142	0.137
225	0.153	0.147	0.142	0.137	0.132
230	0.146	0.141	0.136	0.131	0.127
235	0.140	0.135	0.131	0.126	0.122
240	0.134	0.130	0.126	0.122	0.118
245	0.129	0.125	0.121	0.117	0.114
250	0.124	0.120	0.116	0.113	0.110

Table 6.3.3(C) (continued)

Modified member slenderness ( $\lambda_n$ )	Compression member section constant ( $\alpha_b$ )				
	-1.0	-0.5	0	0.5	1.0
255	0.119	0.116	0.112	0.109	0.106
260	0.115	0.111	0.108	0.105	0.102
265	0.110	0.107	0.104	0.102	0.099
270	0.106	0.103	0.101	0.098	0.096
275	0.102	0.100	0.097	0.095	0.092
280	0.099	0.096	0.094	0.092	0.089
285	0.095	0.093	0.091	0.089	0.087
290	0.092	0.090	0.088	0.086	0.084
295	0.089	0.087	0.085	0.083	0.081
300	0.086	0.084	0.082	0.081	0.079
305	0.083	0.082	0.080	0.078	0.077
310	0.081	0.079	0.077	0.076	0.074
315	0.078	0.077	0.075	0.074	0.072
320	0.076	0.074	0.073	0.071	0.070
340	0.067	0.066	0.065	0.064	0.063
370	0.057	0.056	0.055	0.054	0.054
400	0.049	0.048	0.047	0.047	0.046
450	0.039	0.038	0.038	0.037	0.037
500	0.031	0.031	0.031	0.031	0.030
550	0.026	0.026	0.026	0.025	0.025
600	0.022	0.022	0.022	0.021	0.021

### 6.3.4 Nominal capacity of a member of varying cross-section

The nominal member capacity ( $N_c$ ) of a member of varying cross-section shall be determined using the provisions of [Clause 6.3.3](#) provided that the following are satisfied —

- the nominal section capacity ( $N_s$ ) is the minimum value for all cross-sections along the length of the member; and
- the modified member slenderness ( $\lambda_n$ ) given in [Clause 6.3.3](#) is replaced by the following:

$$\lambda_n = 90 \sqrt{\left( \frac{N_s}{N_{om}} \right)}$$

where  $N_{om}$  is the elastic flexural buckling load of the member in axial compression determined using a rational elastic buckling analysis.

## 6.4 Laced and battened compression members

### 6.4.1 Design forces

If a compression member composed of two or more main components which are parallel is intended to act as a single member, the main components and their connections shall be proportioned to resist

a design transverse shear force ( $V^*$ ) applied at any point along the length of the member in the most unfavourable direction. The design transverse shear force ( $V^*$ ) shall be calculated as follows:

$$V^* \frac{\pi \left( \frac{N_s}{N_c} - 1 \right) N^*}{\lambda_n} \geq 0.01 N^*$$

where

- $N_s$  = nominal section capacity of the compression member given by [Clause 6.2.1](#)
- $N_c$  = nominal member capacity of the compression member given by [Clause 6.3.3](#)
- $N^*$  = design axial force applied to the compression member
- $\lambda_n$  = modified member slenderness

The modified member slenderness ( $\lambda_n$ ) of a battened compression member shall be determined using [Clauses 6.4.3.2](#) and [6.3.3](#).

## 6.4.2 Laced compression members

### 6.4.2.1 Slenderness ratio of a main component

The maximum slenderness ratio  $(l_e/r)_c$  of a main component, based on its minimum radius of gyration and the length between consecutive points where lacing is attached, shall not exceed the lesser of 50 or 0.6 times the slenderness ratio of the member as a whole.

### 6.4.2.2 Slenderness ratio of a laced compression member

The slenderness ratio of a laced compression member shall be calculated by assuming that the main components act as an integral member but shall not be taken as less than  $1.4(l_e/r)_c$ .

### 6.4.2.3 Lacing angle

The angle of inclination of the lacing to the longitudinal axis of the member shall be within the following limits:

- (a)  $50^\circ$  to  $70^\circ$  for single lacing.
- (b)  $40^\circ$  to  $50^\circ$  for double lacing.

### 6.4.2.4 Effective length of a lacing element

The effective length of a lacing element shall be taken as the distance between the inner welds or fasteners for single lacing, and 0.7 times this distance for double lacing which is connected by welds or fasteners.

### 6.4.2.5 Slenderness ratio limit of a lacing element

The slenderness ratio of a lacing element shall not exceed 140.

### 6.4.2.6 Mutually opposed lacing

Single lacing systems mutually opposed in direction on opposite sides of two main components shall not be used unless allowance is made for the resulting torsional effects.

Double lacing systems and single lacing systems mutually opposed in direction on opposite sides of two main components shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the compression member, except for tie plates as specified in [Clause 6.4.2.7](#), unless all actions resulting from the deformation of the compression member are calculated and allowed for in design.

#### 6.4.2.7 Tie plates

Tie plates shall be provided at the ends of the lacing system, at points where the lacing system is interrupted, and at connections with other members. End tie plates shall have a width measured along the axis of the member of not less than the perpendicular distance between the centroids of their connections to the main components. Intermediate tie plates shall have a width of not less than three-quarters of this distance.

A tie plate and its connections shall be treated as battens for design purposes (see [Clause 6.4.3](#)). The thickness of a tie plate shall not be less than 0.02 times the distance between the innermost lines of welds or fastenings, except where the tie plate is effectively stiffened at the free edges. In the latter case, the edge stiffeners shall have a slenderness ratio less than 170.

### 6.4.3 Battened compression member

#### 6.4.3.1 Slenderness ratio of a main component

The maximum slenderness ratio  $(l_e/r)_c$  of a main component, based on its minimum radius of gyration and the length between consecutive points where battens are attached, shall not exceed the lesser of 50, or 0.6 times the slenderness ratio of the member as a whole determined using [Clause 6.4.3.2](#).

#### 6.4.3.2 Slenderness ratios of battened compression member

The slenderness ratio  $(l_e/r)_{bn}$  of a battened compression member about the axis normal to the plane of the battens shall be calculated as follows:

$$\left(\frac{l_e}{r}\right)_{bn} = \sqrt{\left[\left(\frac{l_e}{r}\right)_m^2 + \left(\frac{l_e}{r}\right)_c^2\right]}$$

where

$$\left(\frac{l_e}{r}\right)_m = \text{slenderness ratio of the whole member about the above axis calculated by assuming that the main components act as an integral member}$$

$$\left(\frac{l_e}{r}\right)_c = \text{maximum slenderness ratio of the main component, determined in accordance with [Clause 6.4.3.1](#)}$$

The slenderness ratio  $(l_e/r)_{bp}$  of a battened compression member about the axis parallel to the plane of the battens shall be taken as not less than  $1.4(l_e/r)_c$ .

#### 6.4.3.3 Effective length of a batten

The effective length of an end batten shall be taken as the perpendicular distance between the centroids of the main components. The effective length of an intermediate batten shall be taken as 0.7 times the perpendicular distance between the centroids of the main components.

#### 6.4.3.4 Maximum slenderness ratio of a batten

The slenderness ratio of a batten shall not exceed 180.

#### 6.4.3.5 Width of a batten

The width of an end batten shall be not less than the greater of the distance between the centroids of the main components and twice the width of the narrower main component.

The width of an intermediate batten shall be not less than the greater of half the distance between the main components and twice the width of the narrower main component.

#### 6.4.3.6 Thickness of a batten

The thickness of a batten shall be not less than 0.02 times the minimum distance between the innermost lines of welds or fasteners, except where the batten is effectively stiffened at the free edges. The batten shall be deemed to be effectively stiffened at the free edges when the edge stiffeners have a slenderness ratio of not greater than 170, where the radius of gyration is taken about the axis parallel to the member axis.

#### 6.4.3.7 Loads on battens

The batten and its connections shall be designed to transmit simultaneously to the main components a design longitudinal shear force ( $V_l^*$ ) calculated as follows:

$$V_l^* = \frac{V^* s_b}{n_b d_b}$$

and a design bending moment ( $M^*$ ) calculated as follows:

$$M^* = \frac{V^* s_b}{2n_b}$$

where

- $V^*$  = design transverse shear force specified in [Clause 6.4.1](#)
- $s_b$  = longitudinal centre-to-centre distance between the battens
- $n_b$  = number of parallel planes of battens
- $d_b$  = lateral distance between the centroids of the welds or fasteners

### 6.5 Compression members back to back

#### 6.5.1 Components separated

##### 6.5.1.1 Application

This Clause applies to compression members composed of two angle, channel or tee-section components discontinuously separated back to back by a distance not exceeding that required for the end gusset connection. If such a member is designed as a single integral member, then it shall conform to [Clauses 6.5.1.2](#) to [6.5.1.5](#).

##### 6.5.1.2 Configuration

The configuration of the main components shall be of similar sections arranged symmetrically with their corresponding rectangular axes aligned.



### 6.5.1.3 Slenderness

The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with [Clause 6.4.3.2](#).

### 6.5.1.4 Connection

The main components shall be interconnected by fasteners. Where the components are connected together, the member shall be designed as a battened compression member in accordance with [Clause 6.4.3](#). The main components shall be connected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member, the main components shall be connected by not less than two fasteners in each line along the length of the member, or by equivalent welds.

### 6.5.1.5 Design forces

The interconnecting fasteners shall be designed to transmit a design longitudinal shear force between the components induced by the transverse shear force ( $V^*$ ) given in [Clause 6.4.1](#). The design longitudinal shear force ( $V_l^*$ ) per connection shall be taken as follows:

$$V_l^* = 0.25V^* \left( \frac{l_e}{r} \right)_c$$

where  $(l_e/r)_c$  is the slenderness ratio of the main component between the interconnections.

## 6.5.2 Components in contact

### 6.5.2.1 Application

This Clause applies to compression members composed of two angle, channel or tee-section components back-to-back or separated by continuous steel packing. If such a member is designed as a single integral member, then it shall conform to [Clauses 6.5.2.2](#) to [6.5.2.5](#).

### 6.5.2.2 Configuration

The main components shall be of similar sections arranged symmetrically with their corresponding rectangular axes aligned.

### 6.5.2.3 Slenderness

The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with [Clause 6.4.3.2](#).

### 6.5.2.4 Connection

The main components shall be connected at intervals so that the member is divided into at least three bays of approximately equal length. At the ends of the member, the main components shall be interconnected by not less than two fasteners in each line along the length of the member, or by equivalent welds.

### 6.5.2.5 Design forces

The interconnecting fasteners or welds shall be designed to transmit a longitudinal shear force between the components induced by the transverse shear force ( $V^*$ ) in accordance with [Clause 6.4.1](#). The design longitudinal shear force ( $V_l^*$ ) per connection shall be as specified in [Clause 6.5.1.5](#).

## 6.6 Restraints

### 6.6.1 Restraint systems

The members and the connections of restraining systems required to brace compression members and reduce their effective lengths shall be determined by analysing the structure for its design loads, including any notional horizontal forces (see [Clause 3.2.4](#)), from the points where the forces arise to anchorage or reaction points, and by designing the members and connections as specified in [Clauses 6.6.2](#) and [6.6.3](#).

### 6.6.2 Restraining members and connections

At each restrained cross-section of a compression member, the restraining members and their connections which are required to brace the compression member shall be designed for the greater of —

- (a) the restraining member forces specified in [Clause 6.6.1](#); and
- (b) 0.025 times the maximum axial compression force in the member at the position of the restraint,

except where the restraints are more closely spaced than is required to ensure that —

$$N^* = \phi N_c$$

When the restraint spacing is less, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints that will ensure that  $N^*$  equals  $\phi N_c$ . Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer the transverse force determined for the position of the equivalent restraint.

### 6.6.3 Parallel braced compression members

When a series of parallel compression members is restrained by a line of restraints, each restraining element shall be designed to transfer the transverse force specified in [Clause 6.6.2](#), except that 0.025 times the axial compression force shall be replaced by the sum of 0.025 times the axial force in the connected compression member and 0.0125 times the sum of the axial forces in the connected compression members beyond, with no more than seven members considered in the summation.

## Section 7 Members subject to axial tension

### 7.1 Design for axial tension

A member subject to a design axial tension force ( $N^*$ ) shall satisfy —

$$N^* \leq \phi N_t$$

where

$\phi$  = capacity factor, see [Table 3.4](#)

$N_t$  = nominal section capacity in tension determined in accordance with [Clause 7.2](#)

### 7.2 Nominal section capacity

The nominal section capacity of a tension member shall be taken as the lesser of —

$$N_t = A_g f_y; \text{ and}$$

$$N_t = 0.85 k_t A_n f_u$$

where

$A_g$  = gross area of the cross-section

$f_y$  = yield stress used in design

$k_t$  = correction factor for distribution of forces determined in accordance with [Clause 7.3](#)

$A_n$  = net area of the cross-section, obtained by deducting from the gross area the sectional area of all penetrations and holes, including fastener holes. The deduction for all fastener holes shall be made in accordance with [Clause 9.1.10](#). For threaded rods, the net area shall be taken as the tensile stress area of the threaded portion, as defined in AS 1275

$f_u$  = tensile strength used in design

### 7.3 Distribution of forces

#### 7.3.1 End connections providing uniform force distribution

Where for design purposes it is assumed that the tensile force is distributed uniformly to a tension member, the end connections shall satisfy both the following:

- (a) The connections shall be made to each part of the member and shall be symmetrically placed about the centroidal axis of the member.
- (b) Each part of the connection shall be proportioned to transmit at least the maximum design force carried by the connected part of the member.


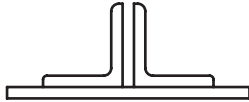
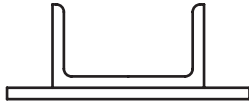
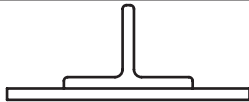


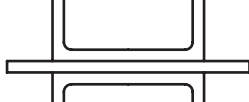
For connections satisfying these requirements, the value of  $k_t$  shall be taken as 1.0.

**7.3.2 End connections providing non-uniform force distribution**

If the end connections of a tension member do not satisfy the requirements of [Clause 7.3.1](#), then the member shall be designed to conform to [Section 8](#) using  $k_t$  equals 1.0, except that [Clause 7.2](#) may be used for the following members:

- (a) *Eccentrically-connected angles, channels and tees* — Eccentrically-connected angles, channels and tees may be designed in accordance with [Clause 7.2](#), using the appropriate value of  $k_t$  given in [Table 7.3.2](#).
- (b) *I-sections or channels connected by both flanges only* — A symmetrical rolled or built-up member of solid I-section or channel section connected by both flanges only may be designed in accordance with [Clause 7.2](#) using a value of  $k_t$  equal to 0.85, provided that —
  - (i) the length between the first and last rows of fasteners in the connection or, when the member is welded, the length of longitudinal weld provided to each side of the connected flanges shall be not less than the depth of the member; and
  - (ii) each flange connection shall be proportioned to transmit at least half of the maximum design force carried by the connected member.

**Table 7.3.2 — Correction factor ( $k_t$ )**

Configuration case	Correction factor, ( $k_t$ )
(a) 	0.75 for unequal angles connected by the short leg 0.85 otherwise
(b) 	As for Case (a)
(c) 	0.85
(d) 	0.90
(e) 	1.0
(f) 	1.0
(g) 	1.0

## 7.4 Tension members with two or more main components

### 7.4.1 General

A tension member composed of two or more main components intended to act as a single member shall conform to [Clauses 7.4.2 to 7.4.5](#).

### 7.4.2 Design forces for connections

If a tension member is composed of two or more main components, the connections between the components shall be proportioned to resist the internal actions arising from the external design forces and bending moments (if any). The design forces for lacing bars, and the design forces and bending moments (if any) for battens, shall be assumed to be divided equally among the connection planes parallel to the direction of force.

### 7.4.3 Tension member composed of two components back-to-back

A tension member composed of two flats, angles, channels or tees, discontinuously connected back-to-back either in contact or separated by a distance not exceeding that required for the end gusset connection, shall conform to the following:

- (a) *Where the components are separated* — They shall be connected either —
  - (i) together at regular intervals along their length by welding, or bolting, so that the slenderness ratio of the individual components between connections shall not exceed 300; or
  - (ii) by connections which conform to [Clauses 6.5.1.4](#) and [6.5.1.5](#).
- (b) *Where component members are in contact back-to-back* — They shall be connected together as required by [Clauses 6.5.2.4](#) and [6.5.2.5](#).

### 7.4.4 Laced tension member

A tension member composed of two components connected by lacing shall conform to [Clause 6.4.2](#) except as follows:

- (a) The slenderness ratio of the lacing elements shall not exceed 210.
- (b) The slenderness ratio of a main component based on its minimum radius of gyration and the length between consecutive points where lacing is attached shall not exceed 300.

For tie plates, the requirements of [Clause 6.4.2.7](#) shall be satisfied except that the thickness of tie plates shall be not less than 0.017 times the distance between the innermost lines of connections.

### 7.4.5 Battened tension member

A tension member composed of two components connected by battens shall conform to [Clause 6.4.3](#) except as follows:

- (a) The spacing of battens shall be such that the maximum slenderness ratio of each main component, based on its minimum radius of gyration and the length between consecutive battens, does not exceed 300.
- (b) Battens attached by bolts shall be connected by not less than two bolts and [Clause 6.4.3.7](#) shall not apply.
- (c) Batten plates shall have a thickness of not less than 0.017 times the distance between the innermost lines of connections.

- (d) Intermediate battens shall have a width of not less than half the effective width of end batten plates.

## 7.5 Members with pin connections

The nominal capacity of a pin connection shall be determined in accordance with [Clause 9.4](#). A pin connection in a tension member shall conform to the following additional requirements:

- (a) The thickness of an unstiffened element containing a hole for a pin connection shall be greater than or equal to 0.25 times the distance from the edge of the hole to the edge of the element measured at right angles to the axis of the member. This limit does not apply to the internal plies where the connected elements are clamped together by external nuts.
- (b) The net area beyond a hole for a pin, parallel to or within  $45^\circ$  of the axis of the member, shall be greater than or equal to the net area required for the member.
- (c) The sum of the areas at a hole for a pin, perpendicular to the axis of the member, shall be greater than or equal to 1.33 times the net area required for the member.
- (d) Pin plates provided to increase the net area of a member or to increase the bearing capacity of a pin shall be arranged to avoid eccentricity and shall be proportioned to distribute the load from the pin into the member.

## Section 8 Members subject to combined actions

### 8.1 General

A member subject to combined axial and bending actions shall be proportioned so that its design actions specified in [Clause 8.2](#), in combination with the nominal section and member capacities (see [Sections 5, 6 and 7](#)), satisfy [Clauses 8.3](#) and [8.4](#). For plastic design (see [Clause 4.5](#)), only the requirements of [Clause 8.4.3](#) need to be satisfied.

Eccentrically loaded double-bolted or welded angles in trusses shall be proportioned to satisfy [Clause 8.3](#), and either [Clause 8.4.5](#) or [Clause 8.4.6](#).

### 8.2 Design actions

For checking the section capacity at a section, the design axial force ( $N^*$ ), which may be tension or compression, shall be the force at the section, and the design bending moments ( $M_x^*$ ,  $M_y^*$ ) shall be the bending moments at the section about the major x- and minor y-principal axes, respectively.

For checking the member capacity, the design axial force ( $N^*$ ) shall be the maximum axial force in the member, and the design bending moments ( $M_x^*$ ,  $M_y^*$ ) shall be the maximum bending moments in the member.

$M_x^*$ ,  $M_y^*$  are the design bending moments resulting from frame action and transverse loading on the member, and include the second order design bending moments resulting from the design loads acting on the structure and its members in their displaced and deformed configuration.

The design bending moments ( $M_x^*$ ,  $M_y^*$ ) shall be determined from one of the following methods of analysis:

- (a) *First-order linear elastic analysis* — By modifying the first-order design bending moments, by using the appropriate moment amplification factors determined in accordance with [Clause 4.4.2](#).
- (b) *Second-order elastic analysis* — In which the design bending moments ( $M^*$ ) are obtained either directly, or by modifying the second-order end moments by using the moment amplification factors determined in accordance with [Appendix E](#).
- (c) *First-order plastic analysis* — In which the design bending moments ( $M^*$ ) are obtained directly for frames where the elastic buckling load factor ( $\lambda_c$ ) satisfies  $\lambda_c \geq 5$  and the requirements of [Clause 4.5.4](#) are satisfied.
- (d) *Second-order plastic analysis* — In which the design bending moments ( $M^*$ ) are obtained directly for frames where the elastic buckling load factor ( $\lambda_c$ ) satisfies  $\lambda_c < 5$ .
- (e) *Advanced structural analysis* — In which the design bending moments ( $M_x^*$  or  $M_y^*$ ) are obtained directly in accordance with [Appendix D](#), in which case only the section capacity requirements of [Clause 8.3](#) and the connection requirements of [Section 9](#) need to be satisfied.

### 8.3 Section capacity

#### 8.3.1 General

The member shall satisfy [Clauses 8.3.2](#), [8.3.3](#) and [8.3.4](#), as appropriate:

- (a) For bending about the major principal x-axis only, sections at all points along the member shall have sufficient capacity to satisfy [Clause 8.3.2](#).

- (b) For bending about the minor principal  $y$ -axis only, sections at all points along the member shall have sufficient capacity to satisfy [Clause 8.3.3](#).
- (c) For bending about a non-principal axis, or bending about both principal axes, sections at all points along the member shall have sufficient capacity to satisfy [Clause 8.3.4](#).

In this Section —

- $M_{sx}, M_{sy}$  = nominal section moment capacities about the  $x$ - and  $y$ -axes respectively, determined in accordance with [Clause 5.2](#)
- $N_s$  = nominal section axial load capacity determined in accordance with [Clause 6.2](#) for axial compression, or [Clause 7.2](#) for axial tension (for which  $N_s$  equals  $N_t$ ).

### 8.3.2 Uniaxial bending about the major principal $x$ -axis

Where uniaxial bending occurs about the major principal  $x$ -axis, the following shall be satisfied:

$$M_x^* \leq \phi M_{rx}$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $M_{rx}$  = nominal section moment capacity, reduced by axial force (tension or compression)
- $$= M_{sx} \left( 1 - \frac{N^*}{\phi N_s} \right)$$

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS/NZS 1163, which are compact as defined in [Clause 5.2.3](#),  $M_{rx}$  may be calculated by one of the following:

- (a) For compression members where  $k_f$  is equal to 1.0 and for tension members —

$$M_{rx} = 1.18 M_{sx} \left( 1 - \frac{N^*}{\phi N_s} \right) \leq M_{sx}$$

- (b) For compression members where  $k_f$  is less than 1.0 —

$$M_{rx} = M_{sx} \left( 1 - \frac{N^*}{\phi N_s} \right) \left[ 1 + 0.18 \left( \frac{82 - \lambda_w}{82 - \lambda_{wy}} \right) \right] \leq M_{sx}$$

$\lambda_w$  and  $\lambda_{wy}$  are the values of  $\lambda_e$  and  $\lambda_{ey}$  for the web (see [Clause 6.2.3](#) and [Table 6.2.4](#)).

### 8.3.3 Uniaxial bending about the minor principal $y$ -axis

Where uniaxial bending occurs about the minor principal  $y$ -axis, the design bending moment ( $M_y^*$ ) about the minor principal  $y$ -axis shall satisfy —

$$M_y^* \leq \phi M_{ry}$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))



$$\begin{aligned}
 M_{ry} &= \text{nominal section moment capacity, reduced by the axial tensile or compressive force} \\
 &= M_{sy} \left[ 1 - \frac{N^*}{\phi N_s} \right]
 \end{aligned}$$

Alternatively,  $M_{ry}$  may be calculated by one of the following:

- (a) For doubly symmetric I-sections which are compact, as defined in [Clause 5.2.3](#) —

$$M_{ry} = 1.19M_{sy} \left[ 1 - \left( \frac{N^*}{\phi N_s} \right)^2 \right] \leq M_{sy}$$

- (b) For rectangular or square hollow sections to AS/NZS 1163 which are compact, as defined in [Clause 5.2.3](#) —

$$M_{ry} = 1.18M_{sy} \left[ 1 - \left( \frac{N^*}{\phi N_s} \right) \right] \leq M_{sy}$$

### 8.3.4 Biaxial bending

Where biaxial bending occurs, the design tensile or compressive force ( $N^*$ ) and the design bending moments ( $M_x^*$ ) and ( $M_y^*$ ) about the major principal x-axis and minor principal y-axis shall satisfy —

$$\frac{N^*}{\phi N_s} + \frac{M_x^*}{\phi M_{sx}} + \frac{M_y^*}{\phi M_{sy}} \leq 1$$

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS/NZS 1163, which are compact as defined in [Clause 5.2.3](#), sections at all points along the member shall satisfy —

$$\left( \frac{M_x^*}{\phi M_{rx}} \right)^\gamma + \left( \frac{M_y^*}{\phi M_{ry}} \right)^\gamma \leq 1$$

where  $M_{rx}$  and  $M_{ry}$  shall be calculated in accordance with [Clauses 8.3.2](#) and [8.3.3](#) respectively, and

$$\gamma = 1.4 + \left( \frac{N^*}{\phi N_s} \right) \leq 2.0$$

## 8.4 Member capacity

### 8.4.1 General

The member shall satisfy [Clauses 8.4.2](#), [8.4.3](#) and [8.4.4](#), as appropriate:

- (a) For a member bent about the major principal x-axis only and where there is sufficient restraint to prevent lateral buckling, or for a member bent about the minor principal y-axis only, the member shall satisfy the in-plane requirements of [Clause 8.4.2](#) for a frame analysed elastically, or [Clause 8.4.3](#) for a frame analysed plastically.
- (b) For a member bent about the major principal x-axis only and with insufficient restraint to prevent lateral buckling, the member shall satisfy both the in-plane requirements of [Clause 8.4.2](#) and out-of-plane requirements of [Clause 8.4.4](#).

- (c) For a member bent about a non-principal axis, or bent about both principal axes, the member shall satisfy the biaxial bending requirements of [Clause 8.4.5](#).

## 8.4.2 In-plane capacity — Elastic analysis

### 8.4.2.1 Application

This Clause applies to a member analysed using an elastic method in accordance with [Clause 4.4](#), or to a member in a statically determinate structure.

### 8.4.2.2 Compression members

A member bent about a principal axis shall have sufficient in-plane capacity to satisfy the following:

$$M^* \leq \phi M_i$$

where

$M^*$  = design bending moment about the principal axis

$\phi$  = capacity factor (see [Table 3.4](#))

$M_i$  = nominal in-plane member moment capacity

$$= M_s \left( 1 - \frac{N^*}{\phi N_c} \right)$$

$M_s$  = nominal section moment capacity determined in accordance with [Clause 5.2](#) for bending about the same principal axis as the design bending moment

$N^*$  = design axial compressive force

$N_c$  = nominal member capacity in axial compression determined in accordance with [Clause 6.3](#) for buckling about the same principal axis, with the effective length factor ( $k_e$ ) taken as 1.0 for both braced and sway members, unless a lower value is calculated for braced members from [Clause 4.6.3.2](#), [4.6.3.3](#) or [Clause 4.6.3.5](#), provided [Clause 6.1](#) is satisfied for  $N_c$  calculated using  $l_e$  determined in accordance with [Clause 4.6.3](#)

Alternatively, for doubly symmetric I-sections and rectangular and square hollow sections to AS/NZS 1163, which are compact as defined in [Clause 5.2.3](#), and where the form factor ( $k_f$ ) determined in accordance with [Clause 6.2.2](#) is unity,  $M_i$  may be calculated as follows:

$$M_i = M_s \left\{ \left[ 1 - \left( \frac{1 + \beta_m}{2} \right)^3 \right] \left( 1 - \frac{N^*}{\phi N_c} \right) + 1.18 \left( \frac{1 + \beta_m}{2} \right)^3 \sqrt{1 - \frac{N^*}{\phi N_c}} \right\}$$

$$\leq M_{rx} \text{ or } M_{ry} \text{ as appropriate}$$

where

$\beta_m$  = ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature for members without transverse load, or  
 = value determined in accordance with [Clause 4.4.2.2](#) for members with transverse load

$M_{rx}$  or  $M_{ry}$  = nominal section moment capacity about the appropriate principal axis determined in accordance with [Clause 8.3](#)

### 8.4.2.3 Tension members

A member subject to a design axial tensile force ( $N^*$ ) and a design bending moment ( $M^*$ ) shall satisfy [Clause 8.3](#).

## 8.4.3 In-plane capacity — Plastic analysis

### 8.4.3.1 Application

This Clause applies only to compact doubly symmetric I-section members. When the distribution of moments in a frame is determined using a plastic method of analysis in accordance with [Clause 4.5](#), then the design axial compressive force ( $N^*$ ) in any member of the frame which is assumed to contain a plastic hinge shall satisfy the member slenderness requirements of [Clause 8.4.3.2](#), and the web slenderness requirements of [Clause 8.4.3.3](#).

The design plastic moment capacity reduced by axial force (tension or compression) for compact doubly symmetric I-sections shall be as specified in [Clause 8.4.3.4](#).

### 8.4.3.2 Member slenderness

The design axial compressive force ( $N^*$ ) in every member assumed to contain a plastic hinge shall satisfy the following:

$$\frac{N^*}{\phi N_s} \leq \left[ \frac{0.60 + 0.40\beta_m}{\sqrt{(N_s / N_{ol})}} \right]^2 \quad \text{when } \frac{N^*}{\phi N_s} \leq 0.15,$$

and

$$\frac{N^*}{\phi N_s} \leq \frac{1 + \beta_m - \sqrt{(N_s / N_{ol})}}{1 + \beta_m + \sqrt{(N_s / N_{ol})}} \quad \text{when } \frac{N^*}{\phi N_s} > 0.15,$$

where

$\beta_m$  = ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

$N_s$  = nominal section capacity in axial compression determined in accordance with [Clause 6.2](#)

$$N_{ol} = \frac{\pi^2 EI}{l^2}$$

$I$  = second moment of area for a the axis about which the design moment acts

$l$  = actual length of the member

A member for which —

$$\frac{N^*}{\phi N_s} > 0.15, \text{ and}$$

$$\frac{N^*}{\phi N_s} > \frac{1 + \beta_m - \sqrt{(N_s / N_{ol})}}{1 + \beta_m + \sqrt{(N_s / N_{ol})}}$$

shall not contain plastic hinges, although it shall be permissible to design the member as an elastic member in a plastically analysed structure to satisfy the requirements of [Clause 8.4.2](#).

#### 8.4.3.3 Web slenderness

The design axial compressive force ( $N^*$ ) in every member assumed to contain a plastic hinge shall satisfy the following:

(a) For webs where  $45 \leq \frac{d_1}{t} \sqrt{\left(\frac{f_y}{250}\right)} \leq 82$  —

$$\frac{N^*}{\phi N_s} \leq 0.60 - \left[ \frac{d_1}{t} \frac{\sqrt{(f_y / 250)}}{137} \right]$$

(b) For webs where  $25 < \frac{d_1}{t} \sqrt{\left(\frac{f_y}{250}\right)} < 45$  —

$$\frac{N^*}{\phi N_s} \leq 1.91 - \left[ \frac{d_1}{t} \frac{\sqrt{(f_y / 250)}}{27.4} \right] \leq 1.0$$

(c) For webs where  $0 \leq \frac{d_1}{t} \sqrt{\left(\frac{f_y}{250}\right)} \leq 25$  —

$$\frac{N^*}{\phi N_s} \leq 1.0$$

Members which have webs for which  $(d_1/t)\sqrt{(f_y/250)}$  exceeds 82 shall not contain plastic hinges, although it shall be permissible to design such a member as an elastic member in a plastically analysed structure to satisfy the requirements of [Clause 8.4.2](#).

#### 8.4.3.4 Plastic moment capacity

The design plastic moment capacity ( $\phi M_{pr}$ ) reduced for axial force (tension or compression) shall be calculated as follows:

(a) For members bent about the major principal x-axis —

$$\phi M_{prx} = 1.18\phi M_{sx} \left( 1 - \frac{N^*}{\phi N_s} \right) \leq \phi M_{sx}$$

(b) For members bent about the minor principal y-axis —

$$\phi M_{pry} = 1.19\phi M_{sy} \left[ 1 - \left( \frac{N^*}{\phi N_s} \right)^2 \right] \leq \phi M_{sy}$$

where  $M_{sx}$  and  $M_{sy}$  are the nominal section moment capacities determined in accordance with [Clauses 5.2.1](#) and [5.2.3](#).

## 8.4.4 Out-of-plane capacity

### 8.4.4.1 Compression members

A member subject to a design axial compressive force ( $N^*$ ) and a design bending moment ( $M_x^*$ ) about its major principal  $x$ -axis, and which may buckle laterally, shall satisfy [Clause 8.4.2](#) and also the following:

$$M_x^* \leq \phi M_{ox}$$

where

$\phi$  = capacity factor (see [Table 3.4](#))

$M_{ox}$  = nominal out-of-plane member moment capacity

$$= M_{bx} \left( 1 - \frac{N^*}{\phi N_{cy}} \right)$$

$M_{bx}$  = the nominal member moment capacity of the member without full lateral restraint and bent about the major principal  $x$ -axis, determined in accordance with [Clause 5.6](#) using a moment modification factor ( $\alpha_m$ ) appropriate to the distribution of design bending moment along the member

$N_{cy}$  = the nominal member capacity in axial compression, determined in accordance with [Clause 6.3](#) for buckling about the minor principal  $y$ -axis

Alternatively, for members without transverse loads which are of compact doubly symmetric I-section (see [Clause 5.2.3](#)), are fully or partially restrained at both ends, and have a form factor ( $k_f$ ) of unity determined in accordance with [Clause 6.2.2](#),  $M_{ox}$  may be calculated as follows:

$$M_{ox} = \alpha_{bc} M_{bxo} \sqrt{\left[ \left( 1 - \frac{N^*}{\phi N_{cy}} \right) \left( 1 - \frac{N^*}{\phi N_{oz}} \right) \right]} \leq M_{rx}$$

where

$$\frac{1}{\alpha_{bc}} = \frac{1 - \beta_m}{2} + \left( \frac{1 + \beta_m}{2} \right)^3 \left( 0.4 - 0.23 \frac{N^*}{\phi N_{cy}} \right)$$

$M_{bxo}$  = nominal member moment capacity without full lateral restraint and with a uniform distribution of design bending moment so that  $\alpha_m$  is unity, determined in accordance with [Clause 5.6](#)

$N_{cy}$  = nominal member capacity in axial compression, determined in accordance with [Clause 6.3](#) for buckling about the minor principal  $y$ -axis

$\beta_m$  = ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

$N_{oz}$  = nominal elastic torsional buckling capacity of the member, calculated as follows:

$$N_{oz} = \frac{GJ + \left( \pi^2 EI_w / l_z^2 \right)}{\left( I_x + I_y \right) / A}$$

$E, G$	=	elastic moduli
$A, I_w, I_x, I_y$ and $J$	=	section constants
$l_z$	=	distance between partial or full torsional restraints

NOTE Values of  $E$  and  $G$ , and expressions for  $I_w$  and  $J$  are given in [Appendix H](#).

#### 8.4.4.2 Tension members

A member subject to a design axial tensile force ( $N^*$ ) and a design bending moment ( $M_x^*$ ) about its major principal  $x$ -axis, and which may buckle laterally, shall satisfy the following:

$$M_x^* \leq \phi M_{ox}$$

where

$\phi$	=	capacity factor (see <a href="#">Table 3.4</a> )
$M_{ox}$	=	nominal out-of-plane member moment capacity
	=	$M_{bx} \left( 1 + \frac{N^*}{\phi N_t} \right) \leq M_{rx}$
$M_{bx}$	=	nominal member moment capacity defined in <a href="#">Clause 8.4.4.1</a>
$N_t$	=	nominal section capacity in axial tension determined in accordance with <a href="#">Clause 7.2</a>
$M_{rx}$	=	nominal section moment capacity reduced by axial force determined in accordance with <a href="#">Clause 8.3.2</a>

#### 8.4.5 Biaxial bending capacity

##### 8.4.5.1 Compression members

A member subject to a design axial compressive force ( $N^*$ ) and design bending moments ( $M_x^*$ ) and ( $M_y^*$ ) about the major  $x$ - and minor  $y$ - principal axes respectively shall satisfy the following:

where

$\phi$	=	capacity factor (see <a href="#">Table 3.4</a> )
$M_{cx}$	=	lesser of the nominal in-plane member moment capacity ( $M_{ix}$ ) and the nominal out-of-plane member moment capacity ( $M_{ox}$ ) for bending about the major principal $x$ -axis, determined in accordance with <a href="#">Clauses 8.4.2</a> and <a href="#">8.4.4</a> respectively
$M_{iy}$	=	nominal in-plane member moment capacity, determined in accordance with <a href="#">Clause 8.4.2</a> , for bending about the minor principal $y$ -axis

### 8.4.5.2 Tension members

A member subject to a design axial tensile force ( $N^*$ ) and design bending moments ( $M_x^*$ ) and ( $M_y^*$ ) about the major  $x$ - and minor  $y$ - principal axes respectively shall satisfy the following:

$$\left( \frac{M_x^*}{\phi M_{cx}} \right)^{1.4} + \left( \frac{M_y^*}{\phi M_{iy}} \right)^{1.4} \leq 1$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $M_{tx}$  = lesser of the nominal section moment capacity ( $M_{rx}$ ) reduced by axial tension and the nominal out-of-plane member moment capacity ( $M_{ox}$ ) determined in accordance with [Clauses 8.3.2](#) and [8.4.4.2](#) respectively
- $M_{ry}$  = nominal section moment capacity reduced by axial tension, determined in accordance with [Clause 8.3.3](#)

### 8.4.6 Eccentrically loaded double bolted or welded single angles in trusses

A member subject to a design axial tensile force ( $N^*$ ) and design bending moments ( $M_x^*$ ) and ( $M_y^*$ ) about the major  $x$ - and minor  $y$ -principal axes respectively shall satisfy the following:

$$\left( \frac{M_x^*}{\phi M_{tx}} \right)^{1.4} + \left( \frac{M_y^*}{\phi M_{ry}} \right)^{1.4} \leq 1$$

where

- $N^*$  = design axial compression force in the member
- $M_h^*$  = design bending moment acting about the rectangular  $h$ -axis parallel to the loaded leg
- $\phi$  = capacity factor (see [Table 3.4](#))
- $N_{ch}$  = nominal member capacity in axial compression, determined in accordance with [Clause 6.3](#), of a single angle compression member buckling with  $l_e$  equals  $l$  about the rectangular  $h$ -axis parallel to the loaded leg
- $M_{bx}$  = nominal member capacity, determined in accordance with [Clause 5.6](#), for an angle without full lateral support, bent about the major principal  $x$ -axis using a factor  $\alpha_m$  appropriate to the distribution of design bending moment along the member
- $\alpha$  = angle between  $x$ - and  $h$ -axes

For equal leg angles, where  $l/t \leq (210 + 175\beta_m)(250/f_y)$ ,  $M_{bx}$  may be taken as  $M_{sx}$ ,

where

- $M_{sx}$  = nominal section moment capacity about the  $x$ -principal axis, determined in accordance with [Clause 5.2](#)
- $l$  = member length
- $t$  = thickness of the angle

For other equal leg angles,  $M_{bx}$  may be determined by using [Clause 5.6.1.1](#) with —

$$M_o = \left( \frac{525t}{l} \right) \left( \frac{250}{f_y} \right) M_s$$

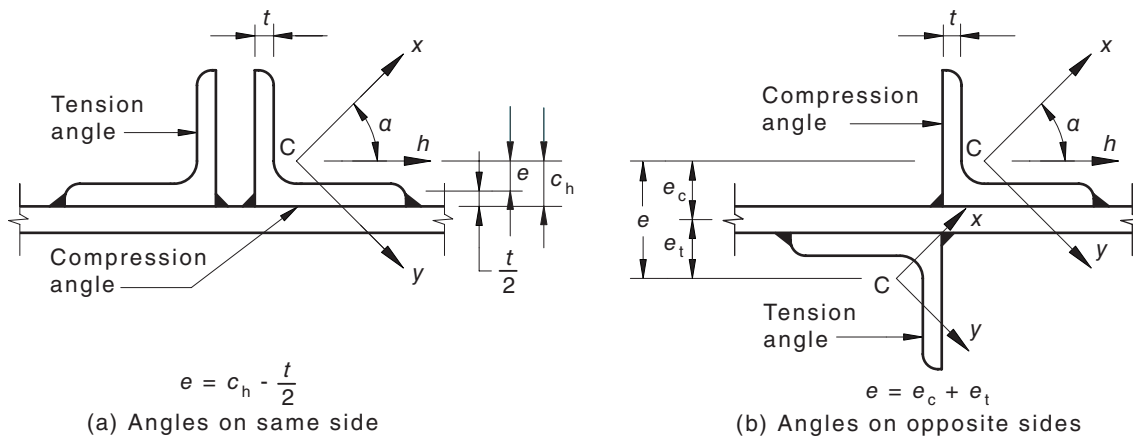
The design end bending moment ( $M_h^*$ ) shall be calculated from a rational elastic analysis of the truss, or shall be taken as not less than  $N^*e$ , resulting from the out-of-plane eccentricity ( $e$ ) of the design axial force ( $N^*$ ) in the member,

where

$$e = \left( c_h - \frac{t}{2} \right), \text{ for angles on the same side of the truss chord}$$

$$= \left( e_c + e_t \right), \text{ for angles on opposite sides of the truss chord}$$

(see [Figure 8.4.6](#)).



**Figure 8.4.6 — Single angles loaded through one leg**



## Section 9 Connections

### 9.1 General

#### 9.1.1 Requirements for connections

Connection elements consist of connection components (cleats, gusset plates, brackets, connecting plates) and connectors (bolts, pins and welds). The connections in a structure shall be proportioned so as to be consistent with the assumptions made in the analysis of the structure and to conform to this Section. Connections shall be capable of transmitting the calculated design action effects.

#### 9.1.2 Classification of connections

##### 9.1.2.1 Connections in rigid construction

The connections shall conform to [Clause 4.2.2](#). The joint deformations shall be such that they have no significant influence on the distribution of action effects nor on the overall deformation of the frame.

##### 9.1.2.2 Connections in semi-rigid construction

The connections shall conform to [Clause 4.2.3](#). Connections between members in semi-rigid construction shall provide a predictable degree of interaction between members, based on the actual action-deformation characteristics of the connection as determined experimentally.

##### 9.1.2.3 Connections in simple construction

The connections shall conform to [Clause 4.2.4](#). Connections between members in simple construction shall be capable of deforming to provide the required rotation at the connection. The connections shall not develop a level of restraining bending moment which adversely affects any part of the structure. The rotation capacity of the connection shall be provided by the detailing of the connection and shall have been demonstrated experimentally. The connection shall be treated as being subjected to reaction shear forces acting at an eccentricity appropriate to the connection detailing.

##### 9.1.2.4 Connections in structures analysed by the plastic method

Connections in structures analysed by the plastic method shall conform to [Clause 4.5.3](#), in addition to the requirements of this Section.

#### 9.1.3 Design of connections

Each element in a connection shall be designed so that the structure is capable of resisting all design actions. The design capacities of each element shall be not less than the calculated design action effects. For earthquake load combinations, the connection shall be designed for the calculated design action effects, exhibit the required ductility and shall conform to [Section 13](#).

Connections and the adjacent areas of members shall be designed by distributing the design action effects so that they conform to the following requirements:

- (a) The distributed design action effects are in equilibrium with the design action effects acting on the connection.
- (b) The deformations in the connection are within the deformation capacities of the connection elements.
- (c) All of the connection elements and the adjacent areas of members are capable of resisting the design action effects acting on them.
- (d) The connection elements shall remain stable under the design action effects and deformations.

Design shall be on the basis of a recognized method supported by experimental evidence.

Residual actions due to the installation of bolts need not be considered.

#### 9.1.4 Minimum design actions on connections

Connections carrying design action effects, except for lacing connections and connections to sag rods, purlins and girts, shall be designed to transmit the greater of —

- (a) the design action in the member; and
- (b) the minimum design action effects expressed either as the value or the factor times the member design capacity required by the strength limit state, specified as follows:
  - (i) Connections in rigid construction — A bending moment of 0.5 times the member design moment capacity.
  - (ii) Connections to beam in simple construction — A shear force of 0.15 times the member design shear capacity or 40 kN, whichever is the lesser.
  - (iii) Connections at the ends of tensile or compression members — A force of 0.3 times the member design capacity, except that for threaded rod acting as a bracing member with turnbuckles, the minimum tensile force shall be equal to the member design capacity.
  - (iv) Splices in members subject to axial tension — A force of 0.3 times the member design capacity in tension.
  - (v) Splices in members subject to axial compression — For ends prepared for full contact (refer to AS/NZS 5131), it shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there shall be sufficient fasteners to transmit a force of 0.15 times the member design capacity in axial compression.

In addition, splices located between points of effective lateral support shall be designed for the design axial force ( $N^*$ ) plus a design bending moment not less than the design bending moment ( $M^*$ )

where

$$M^* = \frac{\delta N^* l_s}{1000}$$

$\delta$  = appropriate amplification factor  $\delta_b$  or  $\delta_s$  determined in accordance with [Clause 4.4](#)

$l_s$  = distance between points of effective lateral support

When members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit a force of 0.3 times the member design capacity in axial compression.

- (vi) Splices in flexural members — A bending moment of 0.3 times the member design capacity in bending. This provision shall not apply to splices designed to transmit shear force only.

A splice subjected to a shear force only shall be designed to transmit the design shear force together with any bending moment resulting from the eccentricity of the force with respect to the centroid of the connector group.

- (vii) Splices in members subject to combined actions — A splice in a member subject to a combination of design axial tension or design axial compression and design bending moment shall satisfy Items (iv), (v) and (vi) simultaneously.

For earthquake load combinations, the design action effects specified in this Clause may need to be increased to meet the required behaviour of the steel frame and shall conform to [Section 13](#).

### 9.1.5 Intersections

Members or components meeting at a joint shall be arranged to transfer the design actions between the parts and wherever practicable, with their centroidal axes meeting at a point. Where there is eccentricity at joints, the members and components shall be designed for the design bending moments which result.

The disposition of fillet welds to balance the design actions about the centroidal axis or axes for end connections of single angle, double angle and similar type members is not required for statically loaded members but is required for members and connection components subject to fatigue loading.

Eccentricity between the centroidal axes of angle members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but shall be taken account of in members and connection components subject to fatigue loading.

### 9.1.6 Choice of fasteners

Where slip in the serviceability limit state shall be avoided in a connection, high-strength bolts in a friction-type joint (bolting category 8.8/TF or 10.9/TF), fitted bolts or welds shall be used.

Where a joint is subject to impact or vibration, high-strength bolts in a friction-type joint (bolting category 8.8/TF or 10.9/TF), locking devices or welds shall be used.

### 9.1.7 Combined connections

When non-slip fasteners (such as high-strength bolts in a friction-type connection or welds) are used in a connection in conjunction with slip-type fasteners (such as snug-tight bolts, or tensioned high-strength bolts in bearing-type connections), all of the design actions shall be assumed to be resisted by the non-slip fasteners.

Where a mixture of non-slip fasteners is used, sharing of the design actions may be assumed. However, when welding is used in a connection in conjunction with other non-slip fasteners —

- (a) any design actions initially applied directly to the welds shall not be assumed to be distributed to fasteners added after the application of the design actions; and
- (b) any design actions applied after welding shall be assumed to be resisted by the welds.

### 9.1.8 Prying forces

Where bolts are required to carry a design tensile force, the bolts shall be proportioned to resist any additional tensile force due to prying action.

### 9.1.9 Connection components

Connection components (cleats, gusset plates, brackets and the like) other than connectors shall have their design capacities assessed as follows:

- (a) Connection components subject to shear — using [Clause 5.11](#).
- (b) Connection components subject to tension — using [Clause 7.2](#).
- (c) Connection components subject to compression — using [Section 6](#).

- (d) Connection components subject to bending — using [Section 5](#).
- (e) A connection component, including a member framing onto the connection component, subject to a design shear force or design tension force ( $R_{bs}^*$ ) shall satisfy the following equation:

$$R_{bs}^* \leq \phi R_{bs}$$

where

$\phi$  = capacity factor

= 0.75

$R_{bs}$  = nominal design capacity in block shear

=  $0.6 f_{uc} A_{nv} + k_{bs} f_{uc} A_{nt}$

$\leq 0.6 f_{yc} A_{gv} + k_{bs} f_{uc} A_{nt}$

$f_{uc}$  = minimum tensile strength of connection element

$f_{yc}$  = yield stress of connection element

$A_{nv}$  = net area subject to shear at rupture

$A_{nt}$  = net area subject to tension at rupture

$A_{gv}$  = gross area subject to shear at rupture

$k_{bs}$  = a factor to account for the effect of eccentricity on the block shear capacity

= 1.0 when tension stress is uniform

= 0.5 when tension is non-uniform

### 9.1.10 Deductions for fastener holes

#### 9.1.10.1 Hole area

In calculating the deductions to be made for holes for fasteners (including countersunk holes), the gross areas of the holes in the plane of their axes shall be used.

#### 9.1.10.2 Holes not staggered

For holes that are not staggered, the area to be deducted shall be the maximum sum of the areas of the holes in any cross-sections at right angles to the direction of the design action in the member.

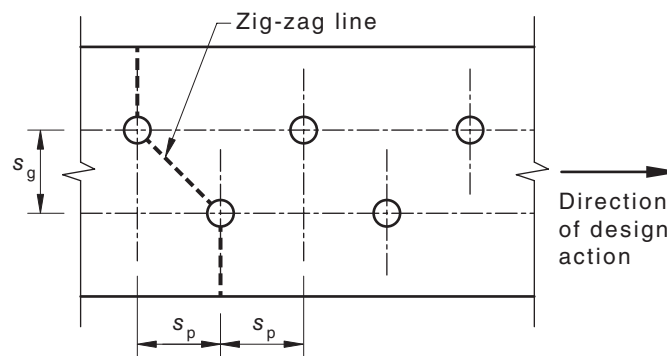
#### 9.1.10.3 Staggered holes

When holes are staggered, the area to be deducted shall be the greater of —

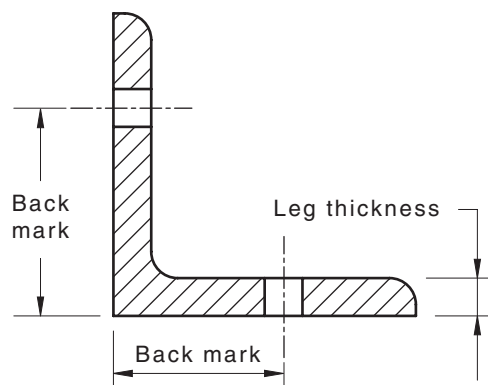
- (a) the deduction for non-staggered holes; or
- (b) the sum of the areas of all holes in any zig-zag line extending progressively across the member or part of the member, less ( $s_p^2 t / 4s_g$ ) for each gauge space in the chain of holes

where

- $s_p$  = staggered pitch, the distance measured parallel to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see [Figure 9.1.10.3\(A\)](#))
- $t$  = thickness of the holed material
- $s_g$  = gauge, the distance, measured at right angles to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see [Figure 9.1.10.3\(A\)](#)). For sections such as angles with holes in both legs, the gauge shall be taken as the sum of the back marks to each hole, less the leg thickness (see [Figure 9.1.10.3\(B\)](#))



**Figure 9.1.10.3(A) — Staggered holes**



**Figure 9.1.10.3(B) — Angles with holes in both legs**

### 9.1.11 Hollow section connections

When design actions from one member are applied to a hollow section at a connection, the design shall include local effects on the hollow section.

## 9.2 Design of bolts

### 9.2.1 Bolts and bolting category

The bolts and bolting categories listed in [Table 9.2.1](#) shall be designed in accordance with this Clause and [Clause 9.3](#).

Other property classes of bolts conforming to AS 1110 series, AS 1111 series and AS/NZS 1559 may be designed in accordance with the provisions of this Clause and [Clause 9.3](#).

**Table 9.2.1 — Bolts and bolting category**

Bolting category	Bolt Standard	Bolt grade	Method of tensioning	Minimum tensile strength ( $f_{uf}$ ) (see Note 2), MPa
4.6/S	AS 1111 (series), AS 1110 (series)	4.6	Snug tight	400
8.8/S	AS/NZS 1252.1, AS 1110 (series)	8.8	Snug tight	830
8.8/TB	AS/NZS 1252.1	8.8	Full tensioning	830
8.8/TF (see Note 1)	AS/NZS 1252.1	8.8	Full tensioning	830
10.9/S (see Note 4)	AS/NZS 1252.1	10.9	Snug tight	1040
10.9/TB (see Note 4)	AS/NZS 1252.1	10.9	Full tensioning	1040
10.9/TF (see Notes 1 and 4)	AS/NZS 1252.1	10.9	Full tensioning	1040

NOTE 1 Special category used in connections where slip at the serviceability limit state is to be restricted (see [Clauses 3.5.5](#) and [9.1.6](#)).

NOTE 2  $f_{uf}$  is the minimum tensile strength of the bolt as specified in AS 4291.1:2015 except for grade 8.8 bolts less than 16 mm diameter where the minimum tensile strength is 800 MPa.

NOTE 3 Bolts to AS 1110 (series) and AS 1111 (series) are not suitable for full tensioning.

NOTE 4 Post-manufacture treatment of high strength bolts may adversely affect material properties.

## 9.2.2 Bolt strength limit states

### 9.2.2.1 Bolt in shear

A bolt subject to a design shear force ( $V_f^*$ ) shall satisfy —

$$V_f^* \leq \phi V_f$$

where

$\phi$  = capacity factor (see [Table 3.4](#))

$V_f$  = nominal shear capacity of a bolt

The nominal shear capacity of a bolt ( $V_f$ ) shall be calculated as follows:

$$V_f = 0.62 f_{uf} k_{rd} k_r (n_n A_c + n_x A_o)$$

where

$f_{uf}$  = minimum tensile strength of the bolt as specified in the relevant Standard (see [Table 9.2.1](#))

$k_{rd}$  = Reduction factor to account for the reduced ductility of grade 10.9 bolts when subjected to shear where threads intercept the shear plane

= 1.0 for grade 4.6 and grade 8.8 bolts

= 1.0 for grade 10.9 bolts where threads do not intercept the shear plane

= 0.83 for grade 10.9 bolts where threads intercept the shear plane

- $k_r$  = reduction factor given in [Table 9.2.2.1](#) to account for the length of a bolted lap connection ( $l_j$ ). For all other connections,  $k_r$  equals 1.0  
 $n_n$  = number of shear planes with threads intercepting the shear plane  
 $A_c$  = minor diameter area of the bolt as defined in AS 1275  
 $n_x$  = number of shear planes without threads intercepting the shear plane  
 $A_o$  = nominal plain shank area of the bolt

**Table 9.2.2.1 — Reduction factor for a bolted lap connection ( $k_r$ )**

Length, mm	$l_j < 300$	$300 \leq l_j \leq 1300$	$l_j > 1300$
$k_r$	1.0	$1.075 - (l_j/4000)$	0.75

### 9.2.2.2 Bolt in tension

A bolt subject to a design tension force ( $N_{tf}^*$ ) shall satisfy —

$$N_{tf}^* \leq \phi N_{tf}$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))  
 $N_{tf}$  = nominal tensile capacity of a bolt

The nominal tension capacity of a bolt ( $N_{tf}$ ) shall be calculated as follows:

$$N_{tf} = A_s f_{uf}$$

where  $A_s$  is the tensile stress area of a bolt as specified in AS 1275.

### 9.2.2.3 Bolt subject to combined shear and tension

A bolt required to resist both design shear ( $V_f^*$ ) and design tensile forces ( $N_{tf}^*$ ) at the same time shall satisfy —

$$\left( \frac{V_f^*}{\phi V_f} \right)^2 + \left( \frac{N_{tf}^*}{\phi N_{tf}} \right)^2 \leq 1.0$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))  
 $V_f$  = nominal shear capacity calculated in accordance with [Clause 9.2.2.1](#)  
 $N_{tf}$  = nominal tensile capacity calculated in accordance with [Clause 9.2.2.2](#)

#### 9.2.2.4 Ply in bearing

A ply subject to a design bearing force ( $V_b^*$ ) due to a bolt in shear shall satisfy —

$$V_b^* \leq \phi V_b$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $V_b$  = nominal bearing capacity of a ply

The nominal bearing capacity of a ply ( $V_b$ ) shall be calculated as follows:

$$V_b = 3.2d_f t_p f_{up} \tag{9.2.2.4(1)}$$

provided that, for a ply subject to a component of force acting towards an edge, the nominal bearing capacity of a ply ( $V_b$ ) shall be the lesser of that given by [Equation 9.2.2.4\(1\)](#) and that given by [Equation 9.2.2.4\(2\)](#) —

$$V_b = a_e t_p f_{up} \tag{9.2.2.4(2)}$$

where

- $d_f$  = diameter of the bolt
- $t_p$  = thickness of the ply
- $f_{up}$  = tensile strength of the ply
- $a_e$  = minimum distance from the edge of a hole to the edge of a ply, measured in the direction of the component of a force, plus half the bolt diameter. The edge of a ply shall be deemed to include the edge of an adjacent bolt hole

#### 9.2.2.5 Filler plates

For connections in which filler plates exceed 6 mm in thickness but are less than 20 mm in thickness, the nominal shear capacity of a bolt ( $V_f$ ) specified in [Clause 9.2.2.1](#) shall be reduced by multiplying by  $[1 - 0.0154(t - 6)]$ , where  $t$  is the total thickness of the filler, including any paint film, up to 20 mm. Any filler plate shall extend beyond the connection and the extension of the filler plate shall be secured with enough bolts to distribute the calculated design force in the connected element over the combined cross-section of the connected element and filler plate. For multi-shear plane connections with more than one filler plate through which a bolt passes, the reduction shall be determined using the maximum thickness of filler plate on any shear plane through which the bolt passes.



## 9.2.3 Bolt serviceability limit state

### 9.2.3.1 Design

For friction-type connections (bolting categories 8.8/TF or 10.9/TF) in which slip in the serviceability limit state is required to be limited, a bolt subjected only to a design shear force ( $V_{sf}^*$ ) in the plane of the interfaces shall satisfy —

$$V_{sf}^* \leq \phi V_{sf}$$

where

- $\phi$  = capacity factor (see [Clause 3.5.5](#))  
 $V_{sf}$  = nominal shear capacity of a bolt, for a friction-type connection

The nominal shear capacity of a bolt ( $V_{sf}$ ) shall be calculated as follows:

$$V_{sf} = \mu n_{ei} N_{ti} k_h \tag{9.2.3.1}$$

where

- $\mu$  = slip factor as specified in [Clause 9.2.3.2](#)  
 $n_{ei}$  = number of effective interfaces  
 $N_{ti}$  = minimum bolt tension at installation as specified in [Clause 15.2.2.2](#)  
 $k_h$  = factor for different hole types, as specified in [Clause 14.3.2](#)  
 = 1.0 for standard holes  
 = 0.85 for short slotted and oversize holes  
 = 0.70 for long slotted holes

The strength limit state shall be separately assessed in accordance with [Clause 9.2.2](#).

### 9.2.3.2 Contact surfaces

Where the surfaces in contact are clean “as-rolled” surfaces, the slip factor ( $\mu$ ) shall be taken as 0.35. If any applied finish, or other surface condition including a machined surface, is used, the slip factor shall be based upon test evidence. Tests performed in accordance with the procedure specified in [Appendix J](#) shall be deemed to provide satisfactory test evidence.

A connection involving 8.8/TF or 10.9/TF bolting category shall be identified as such, and the drawings shall clearly indicate the surface treatment required at such a connection and whether masking of the connection surfaces is required during painting operations.

NOTE Slip factors for common surface conditions are included in EN 1090-2:2018. Further information on slip factors is available from Galvanizers Association of Australia.

### 9.2.3.3 Combined shear and tension

Bolts in a connection for which slip in the serviceability limit state shall be limited, which are subject to a design tension force ( $N_{tf}^*$ ), shall satisfy —

$$\left( \frac{V_{sf}^*}{\phi V_{sf}} \right) + \left( \frac{N_{tf}^*}{\phi N_{tf}} \right) \leq 1.0$$

where

$V_{sf}^*$  = design shear force on the bolt in the plane of the interfaces

$N_{tf}^*$  = design tensile force on the bolt

$\phi$  = capacity factor (see [Clause 3.5.5](#))

$V_{sf}$  = nominal shear capacity of the bolt as specified in [Clause 9.2.3.1](#)

$N_{tf}$  = nominal tensile capacity of the bolt

The nominal tensile capacity of the bolt ( $N_{tf}$ ) shall be taken as —

$$N_{tf} = N_{ti}$$

where  $N_{ti}$  is the minimum bolt tension at installation as specified in [Table 15.2.2.2](#).

The strength limit state shall also be separately assessed in accordance with [Clause 9.2.2.3](#).

## 9.3 Assessment of the strength of a bolt group

### 9.3.1 Bolt group subject to in-plane loading

The design actions in a bolt group shall be determined by an analysis based on the following assumptions:

- (a) The connection plates shall be assumed to be rigid and to rotate relative to each other about a point known as the instantaneous centre of the bolt group.
- (b) In the case of a bolt group subject to a pure couple only, the instantaneous centre of rotation coincides with the bolt group centroid.

In the case of a bolt group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of rotation is at infinity and the design shear force is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the bolt group centroid shall be superposed, or a recognized method of analysis shall be used.

- (c) The design shear force in each bolt shall be assumed to act at right angles to the radius from the bolt to the instantaneous centre, and shall be taken as proportional to that radius.

Each bolt shall satisfy the requirements of [Clause 9.2.2.1](#) using the capacity factor ( $\phi$ ) for a bolt group (see [Table 3.4](#)) and the ply in bearing shall satisfy [Clause 9.2.2.4](#).

### 9.3.2 Bolt group subject to out-of-plane loading

The design actions in any bolt in a bolt group subject to out-of-plane loading shall be determined in accordance with [Clause 9.1.3](#).

Each bolt shall conform to [Clauses 9.2.2.1](#), [9.2.2.2](#) and [9.2.2.3](#) using the capacity factor ( $\phi$ ) for a bolt group (see [Table 3.4](#)), and the ply in bearing shall conform to [Clause 9.2.2.4](#).

### 9.3.3 Bolt group subject to combinations of in-plane and out-of-plane loadings

The design actions in any bolt in a bolt group shall be determined in accordance with [Clauses 9.3.1](#) and [9.3.2](#).

Each bolt shall conform to [Clauses 9.2.2.1](#), [9.2.2.2](#) and [9.2.2.3](#) using the capacity factor ( $\phi$ ) for a bolt group (see [Table 3.4](#)), and the ply in bearing shall conform to [Clause 9.2.2.4](#).

## 9.4 Design of a pin connection

### 9.4.1 Pin in shear

A pin subject to a design shear force ( $V_f^*$ ) shall satisfy —

$$V_f^* \leq \phi V_f$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $V_f$  = nominal shear capacity of the pin

The nominal shear capacity of a pin ( $V_f$ ) shall be calculated as follows:

$$V_f = 0.62 f_{yp} n_s A_p$$

where

- $f_{yp}$  = yield stress of the pin
- $n_s$  = number of shear planes
- $A_p$  = cross-sectional area of the pin

### 9.4.2 Pin in bearing

A pin subject to a design bearing force ( $V_b^*$ ) shall satisfy —

$$V_b^* \leq \phi V_b$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $V_b$  = nominal bearing capacity of the pin

The nominal bearing capacity of a pin ( $V_b$ ) shall be calculated as follows:

$$V_b = 1.4 f_{yp} d_f t_p k_p$$

where

- $f_{yp}$  = yield stress of the pin
- $d_f$  = pin diameter
- $t_p$  = connecting plate thickness(es)
- $k_p$  = 1.0 for pins without rotation, or  
= 0.5 for pins with rotation

### 9.4.3 Pin in bending

A pin subject to a design bending moment ( $M^*$ ) shall satisfy —

$$M^* \leq \phi M_p$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $M_p$  = nominal moment capacity of the pin

The nominal moment capacity of a pin ( $M_p$ ) shall be calculated as follows:

$$M_p = f_{yp} S$$

where

- $f_{yp}$  = yield stress of the pin
- $S$  = plastic section modulus of the pin

### 9.4.4 Ply in bearing

A ply subject to a design bearing force ( $V_b^*$ ) due to a pin in shear shall satisfy [Clause 9.2.2.4](#).

## 9.5 Design details for bolts and pins

### 9.5.1 Minimum pitch

The distance between centres of fastener holes shall be not less than 2.5 times the nominal diameter of the fastener ( $d_f$ ).

NOTE The minimum pitch may also be affected by [Clause 9.2.2.4](#).

### 9.5.2 Minimum edge distance

The minimum edge distance shall be as follows:

- (a) *Standard holes* — The minimum edge distance for a standard size bolt hole (see [Clause 14.3.2](#)) shall be as given in [Table 9.5.2](#), where the edge distance is measured from the centre of a hole to the edge of a plate or rolled section.
- (b) *Non-standard holes* — The minimum edge distance for a non-standard size bolt hole shall be as given in [Table 9.5.2](#), where the edge distance is measured from the nearer edge of a hole to the physical edge of a plate or rolled section, plus half the fastener diameter ( $d_f$ ).

**Table 9.5.2 — Minimum edge distance**

Sheared or hand flame cut edge	Rolled plate, flat bar or section: machine cut, sawn or planed edge	Rolled edge of a rolled flat bar or section
$1.75d_f$	$1.50d_f$	$1.25d_f$
NOTE The edge distance may also be affected by <a href="#">Clause 9.2.2.4</a> .		

### 9.5.3 Maximum pitch

The maximum distance between centres of fasteners shall be the lesser of  $15t_p$  (where  $t_p$  = thickness of thinner ply connected) or 200 mm. However, in the following cases, the maximum distances shall be as follows:

- (a) For fasteners which are not required to carry design actions in regions not liable to corrosion — The lesser of  $32t_p$  or 300 mm.
- (b) For an outside line of fasteners in the direction of the design action — The lesser of  $(4t_p + 100)$  mm, or 200 mm.

### 9.5.4 Maximum edge distance

The maximum distance from the centre of any fastener to the nearest edge of parts in contact with one another shall be 12 times the thickness of the thinnest outer connected ply under consideration, but shall not exceed 150 mm.

### 9.5.5 Holes

Holes for bolts and pins shall be in accordance with AS/NZS 5131.

## 9.6 Design of welds

### 9.6.1 Scope

#### 9.6.1.1 General

Welding shall be in accordance with AS/NZS 1554.1, AS/NZS 1554.2, AS/NZS 1554.4 or AS/NZS 1554.5.

#### 9.6.1.2 Weld types

For the purpose of this Standard, welds shall be butt, fillet, slot or plug welds, or compound welds.

### 9.6.1.3 Weld quality

Weld quality shall be either SP or GP as specified in AS/NZS 1554.1 or AS/NZS 1554.4, as appropriate, except that where a higher quality weld is required by [Clause 11.1.4](#), weld quality conforming with AS/NZS 1554.5 shall be used. Weld quality shall be specified on the design drawings.

NOTE The requirements of [Sections 10](#) and [11](#) may influence required weld quality.

## 9.6.2 Complete and incomplete penetration butt welds

### 9.6.2.1 Definitions

For the purpose of this Clause, the definitions below apply.

*Complete penetration butt weld* — butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

*Incomplete penetration butt weld* — butt weld in which fusion exists over less than the complete depth of the joint.

*Prequalified weld preparation* — joint preparation prequalified in terms of AS/NZS 1554.1 or AS/NZS 1554.4.

### 9.6.2.2 Size of weld

The size of a complete penetration butt weld, other than a complete penetration butt weld in a T-joint or a corner joint, and the size of an incomplete penetration butt weld shall be the minimum depth to which the weld extends from its face into a joint, exclusive of reinforcement.

The size of a complete penetration butt weld for a T-joint or a corner joint shall be the thickness of the part whose end or edge butts against the face of the other part.

### 9.6.2.3 Design throat thickness

Design throat thickness shall be as follows:

- (a) *Complete penetration butt weld* — The design throat thickness for a complete penetration butt weld shall be the size of the weld.
- (b) *Incomplete penetration butt weld* — The design throat thickness for an incomplete penetration butt weld shall be as follows:
  - (i) Prequalified preparation for incomplete penetration butt weld except as otherwise provided in (iii), as specified in AS/NZS 1554.1 or AS/NZS 1554.4.
  - (ii) Non-prequalified preparation for incomplete penetration butt weld except as provided in (iii) —
    - (A) where  $\theta \leq 60^\circ$ 
      - $(d - 3)$  mm, for single V weld;
      - $[(d_3 + d_4) - 6]$  mm, for double V weld
    - (B) where  $\theta > 60^\circ$ 
      - $d$  mm, for single V weld;
      - $(d_3 + d_4)$  mm, for double V weld

where

$d$	=	depth of preparation ( $d_3$ and $d_4$ are the values of $d$ for each side of the weld)
$\theta$	=	angle of preparation

- (iii) For an incomplete penetration butt weld made by an automatic arc welding process for which it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved, an increase in design throat thickness up to the depth of preparation may be allowed. If the macro test shows penetration beyond the depth of preparation, an increase in design throat thickness up to that shown in [Figure 9.6.3.4](#) may be allowed.

NOTE It is only necessary to specify the design throat thickness required, leaving the fabricator to determine the welding procedure necessary to achieve the specified design throat thickness.

#### 9.6.2.4 Effective length

The effective length of a butt weld shall be the length of the continuous full size weld.

#### 9.6.2.5 Effective area

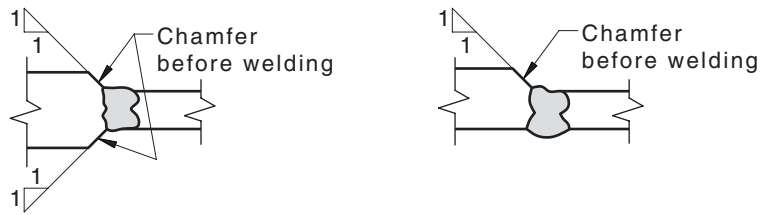
The effective area of a butt weld shall be the product of the effective length and the design throat thickness.

#### 9.6.2.6 Transition of thickness or width

Butt welded joints between parts of different thickness or unequal width that are subject to tension shall have a smooth transition between surfaces or edges.

The transition shall be made by chamfering the thicker part or by sloping the weld surfaces or by any combination of those, as shown in [Figure 9.6.2.6](#).

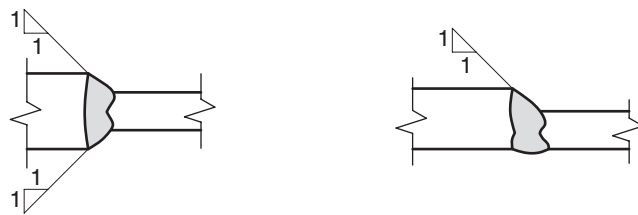
The transition slope between the parts shall not exceed 1:1. However, the provisions of [Section 11](#) require a lesser slope than this or a curved transition between the parts for some fatigue detail categories.



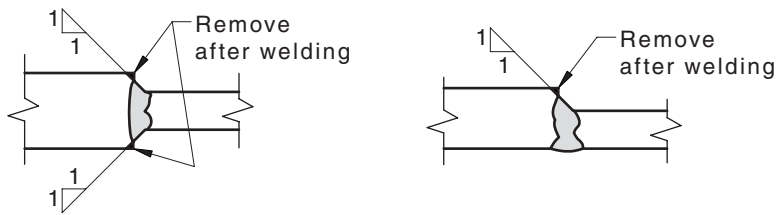
Centre-line alignment  
(Particularly applicable to web plates)

Offset alignment  
(Particularly applicable to flange plates)

(i) Transition by chamfering thicker part

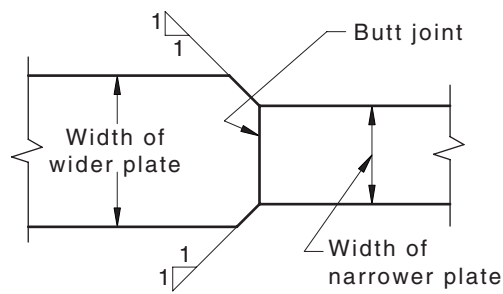


(ii) Transition by sloping weld surface



(iii) Transition by sloping weld surface and chamfering

(a) Transition of butt joints in parts of unequal thickness



(b) Transition of butt joints in parts of unequal width — Transition by chamfering wider part

NOTE Transition slopes shown in (a) and (b) are the maximum permitted.

**Figure 9.6.2.6 — Transitions of thickness or width for butt welds subject to tension**



### 9.6.2.7 Strength assessment of a butt weld

The assessment of a butt weld for the strength limit state shall be as follows:

- (a) *Complete penetration butt weld* — The design capacity of a complete penetration butt weld shall be taken as equal to the nominal capacity of the weaker part of the parts joined, multiplied by the appropriate capacity factor ( $\phi$ ) for butt welds given in [Table 3.4](#), provided that the welding procedures are qualified in accordance with AS/NZS 1554.1, AS/NZS 1554.4 or AS/NZS 1554.5. The butt weld shall be made using a welding consumable which will produce butt tensile test specimens in accordance with AS 2205.2.1 for which the minimum strength is not less than that given in [Table 2.1](#) for the parent material.
- (c) *Incomplete-penetration butt weld* — The design capacity of an incomplete-penetration butt weld shall be calculated as for a fillet weld (see [Clause 9.6.3.10](#)) using the design throat thickness determined in accordance with [Clause 9.6.2.3\(b\)](#).

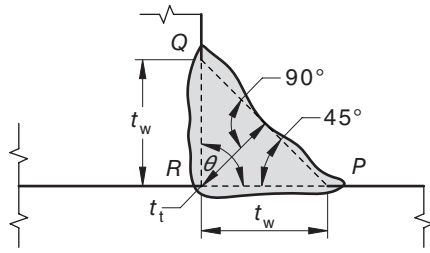
## 9.6.3 Fillet welds

### 9.6.3.1 Size of a fillet weld

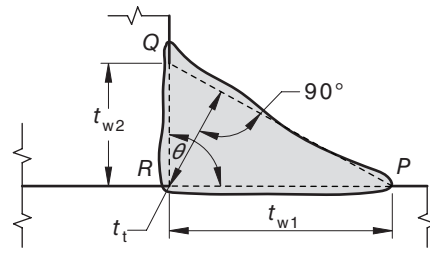
The size of a fillet weld shall be specified by the leg lengths. The leg lengths shall be defined as the lengths ( $t_{w1}$ ,  $t_{w2}$ ) of the sides lying along the legs of a triangle inscribed within the cross-section of the weld (see [Figures 9.6.3.1\(a\) and \(b\)](#)). When the legs are of equal length, the size shall be specified by a single dimension ( $t_w$ ).

Where there is a root gap, the size ( $t_w$ ) shall be given by the lengths of the legs of the inscribed triangle reduced by the root gap as shown in [Figure 9.6.3.1\(c\)](#).

NOTE The preferred sizes of a fillet weld less than 15 mm are — 3, 4, 5, 6, 8, 10 and 12 mm.

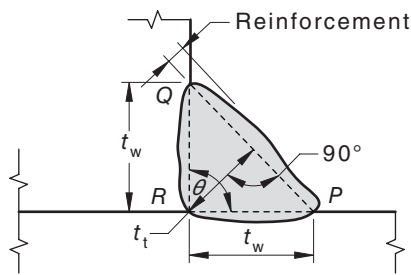


Equal leg fillet weld

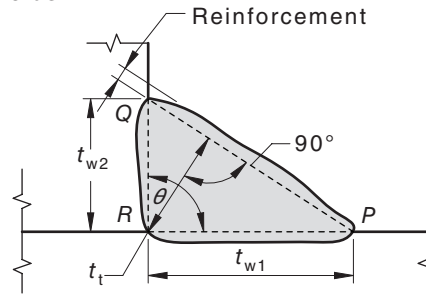


Unequal leg fillet weld

(a) Concave fillet welds

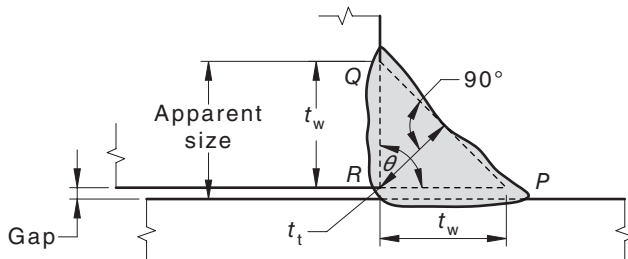


Equal leg fillet weld

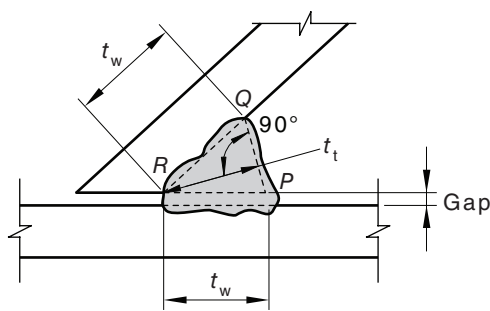


Unequal leg fillet weld

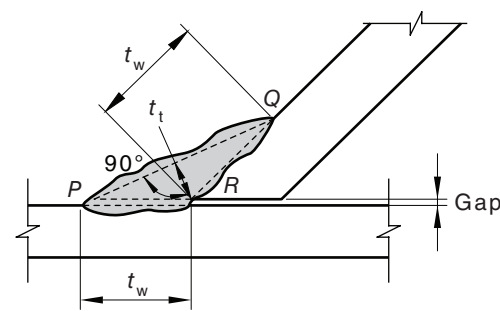
(b) Convex fillet welds



(c) Fillet weld with root gap



(d) Fillet weld at angled connection - Acute angle side



(e) Fillet weld at angled connection - Obtuse angle side

KEY:

*PQR* = triangle inscribed within the cross-section of the weld

$t_w, t_{w1}, t_{w2}$  = sizes of fillet welds

$t_t$  = design throat thickness

Figure 9.6.3.1 — Fillet weld size

### 9.6.3.2 Minimum size of a fillet weld

The minimum size of a fillet weld, other than a fillet weld used to reinforce a butt weld, shall conform with [Table 9.6.3.2](#), except that the size of the weld need not exceed the thickness of the thinner part joined.

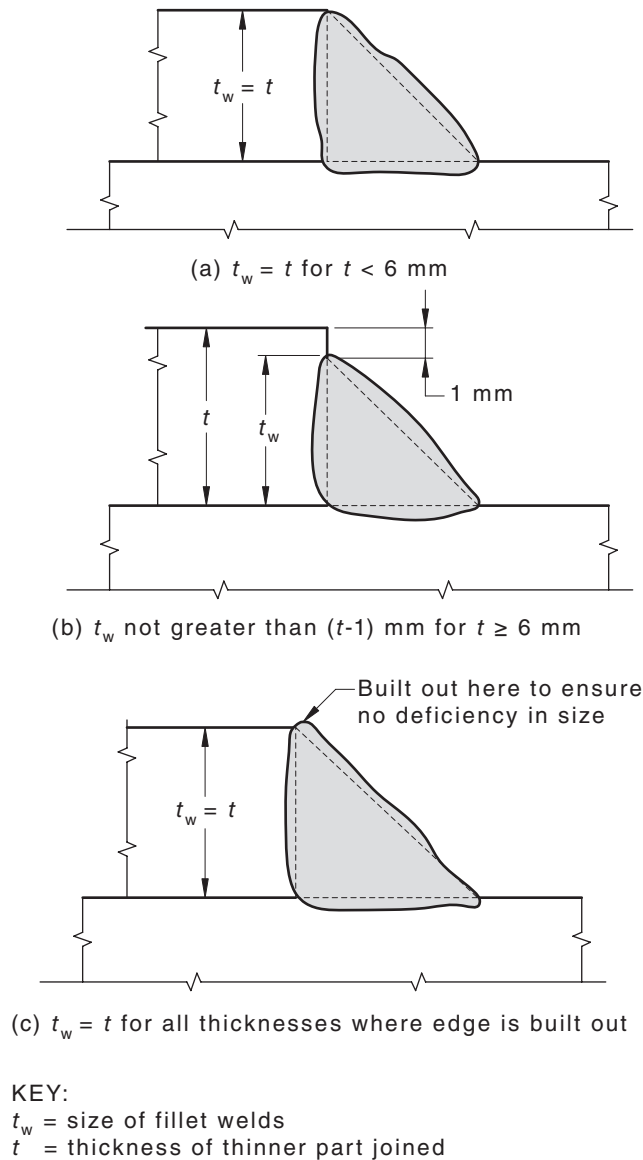
**Table 9.6.3.2 — Minimum size of a fillet weld**

Thickness of thickest part ( <i>t</i> ) mm	Minimum size of a fillet weld ( <i>t<sub>w</sub></i> ) mm
$t \leq 7$	3
$7 < t \leq 10$	4
$10 < t \leq 15$	5
$t > 15$	6

### 9.6.3.3 Maximum size of a fillet weld along an edge

The maximum size of a fillet weld along an edge of material shall be —

- (a) for material less than 6 mm in thickness, the thickness of the material (see [Figure 9.6.3.3\(a\)](#)); and
- (b) for material 6 mm or more in thickness (see [Figure 9.6.3.3\(b\)](#)), unless the weld is designated on the drawing to be built out to obtain the design throat thickness (see [Figure 9.6.3.3\(c\)](#)), 1 mm less than the thickness of the material.

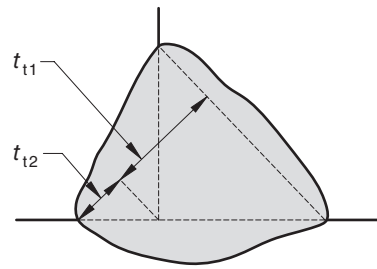


**Figure 9.6.3.3 — Maximum sizes of fillet welds along edges**

**9.6.3.4 Design throat thickness**

The design throat thickness ( $t_t$ ) of a fillet weld shall be as shown in [Figure 9.6.3.1](#).

For a weld made by an automatic arc welding process, an increase in design throat thickness may be allowed as shown in [Figure 9.6.3.4](#), provided that it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved. Where such penetration is achieved, the size of weld required may be correspondingly reduced to give the specified design throat thickness.



Design throat thickness for deep penetration welds made by automatic processes:

$$t_t = t_{t1} + 0.85t_{t2}$$

**Figure 9.6.3.4 — Deep penetration weld**

### 9.6.3.5 Effective length

The effective length of a fillet weld shall be the overall length of the full-size fillet, including end returns. The minimum effective length of a fillet weld shall be 4 times the size of the weld. However, if the ratio of the effective length of the weld to the size of the weld does not conform to this requirement, the size of the weld for design purposes shall be taken as 0.25 times the effective length. The minimum length requirement shall also apply to lap joints.

Any segment of intermittent fillet weld shall have an effective length of not less than 40 mm or 4 times the nominal size of the weld, whichever is the greater.

### 9.6.3.6 Effective area

The effective area of a fillet weld shall be the product of the effective length and the design throat thickness.

### 9.6.3.7 Transverse spacing of fillet welds

If two parallel fillet welds connect two components in the direction of the design action to form a built-up member, the transverse distance between the welds shall not exceed  $32t_p$ , except that in the case of intermittent fillet welds at the ends of a tension member, the transverse distance shall not exceed either  $16t_p$  or 200 mm, where  $t_p$  is the thickness of the thinner of the two components connected.

It shall be permissible to use fillet welds in slots and holes in the direction of the design action in order to satisfy this Clause.

### 9.6.3.8 Intermittent fillet welds

Except at the ends of a built-up member, the clear spacing between the lengths of consecutive collinear intermittent fillet welds shall not exceed the lesser of —

- (a) for elements in compression,  $16t_p$  and 300 mm; and
- (b) for elements in tension,  $24t_p$  and 300 mm

### 9.6.3.9 Built-up members — Intermittent fillet welds

If intermittent fillet welds connect components forming a built-up member, the welds shall conform to the following requirements:

- (a) At the ends of a tension or compression component of a beam, or at the ends of a tension member, when side fillets are used alone, they shall have a length along each joint line at least equal to the width of the connected component. If the connected component is tapered, the length of weld shall be the greater of —
  - (i) the width of the widest part; and

- (ii) the length of the taper.
- (b) At the cap plate or baseplate of a compression member, welds shall have a length along each joint line of at least the maximum width of the member at the contact face.
- (c) Where a beam is connected to the face of a compression member, the welds connecting the compression member components shall extend between the levels of the top and bottom of the beam and in addition —
- (i) for an unrestrained connection, a distance ( $d$ ) below the lower face of the beam; and
- (ii) for a restrained connection, a distance ( $d$ ) above and below the upper and lower faces of the beam,

where  $d$  is the maximum cross-sectional dimension of the compression member.

### 9.6.3.10 Strength limit state for fillet weld

A fillet weld subject to a design force per unit length of weld ( $v_w^*$ ) shall satisfy —

$$v_w^* \leq \phi v_w$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))
- $v_w$  = nominal capacity of a fillet weld per unit length

The design force per unit length ( $v_w^*$ ) shall be the vectorial sum of the design forces per unit length on the effective area of the weld.

The nominal capacity of a fillet weld per unit length ( $v_w$ ) shall be calculated as follows:

$$v_w = 0.6 f_{uw} t_t k_r$$

where

- $f_{uw}$  = nominal tensile strength of weld metal (see [Table 9.6.3.10\(A\)](#))
- $t_t$  = design throat thickness
- $k_r$  = reduction factor given in [Table 9.6.3.10\(B\)](#) to account for the length of a welded lap connection ( $l_w$ ). For all other connection types,  $k_r$  equals 1.0

**Table 9.6.3.10(A) — Nominal tensile strength of weld metal ( $f_{uw}$ ) (see Notes and [Table 10.4.4](#))**

Manual metal arc (AS/NZS 4855)	Submerged arc (AS 1858.1)	Flux cored arc (AS/NZS ISO 17632)	Gas metal arc (AS/NZS 2717.1) (ISO 14341)	Gas tungsten arc (ISO 636)	Nominal tensile strength of weld metal ( $f_{uw}$ )
<b>Structural steel welding to AS/NZS 1554.1 and AS/NZS 1554.5 — Steel Types 1–8C</b>					
A E35, A 38 B E43XX	W40X	A-T35, A-T38 B-T43	A-G35, A-G38 B-G43	A-W35, A-W38 B-W43	430
A E42, A E46 B E49XX	W50X	A-T42, A-T46 B-T49	A-G42, A-G46 B-G49, W500	A-W42, A-W46 B-W49	490

**Table 9.6.3.10(A) (continued)**

A E50 B E55XX	W55X	A-T50 B-T55, B-T57	A-G50 B-G55, B-G57 W55X, W62X	A-W50 B-W55, B-W57	550
<b>Structural steel welding to AS/NZS 1554.4 — Steel Types 8Q-10Q</b>					
<b>Manual metal arc (AS/NZS 4855, AS/NZS 4857)</b>	<b>Submerged arc (AS 1858.1 AS 1858.2)</b>	<b>Flux cored arc (AS/NZS ISO 17632 AS/NZS ISO 18276)</b>	<b>Gas metal arc (AS/NZS 2717.1) (ISO 14341, ISO 16834)</b>	<b>Gas tungsten arc (ISO 636, ISO 16834)</b>	<b>Nominal tensile strength of weld meta, (<math>f_{uw}</math>)</b>
A E35, A 38 B E43XX	W40X	A-T35, A-T38 B-T43	A-G35, A-G38 B-G43	A-W35, A-W38 B-W43	430
A E42, A E46 B E49XX	W50X	A-T42, A-T46 B-T49	A-G42, A-G46 B-G49 W50X	A-W42, A-W46 B-W49	490
A E50 B E55XX B E57XX B E59XX	W55X	A-T50 B-T55, B-T57, B-T59	A-G50 B-G55, B-G57, B-G59 W55X	A-W50 B-W55 B-W57, B-W59	550
A E55 B E62XX	W62X	A-T55 B-T62	A-G55 B-G62 W62X	A-W55 B-W62	620
A E62 B E69XX	W69X	A-T62 B-T69	A-G62 B-G69 W69X	A-W62 B-W69	690
A E69 B E76XX B E78XX	W76X	A-T69 B-T76, B-78	A-G69 B-G76, B-G78 W76X	A-W69 B-W76, B-W78	760
A E79 B E83XX	W83X	A-T79 B-T83	A-G79 B-G83, W83X	A-W79 B-W83	830
NOTE 1 The minimum tensile strength of the European type A classification series consumables is slightly higher than that shown in this Table.					
NOTE 2 The B-E57XX, B-E59XX, B-E78XX and equivalent strength consumables for other welding processes, may be difficult to source commercially.					
NOTE 3 The letter "X" represents any flux type (manual metal arc welding process) or impact energy value (submerged arc and gas metal arc welding processes).					

**Table 9.6.3.10(B) — Reduction factor for a welded lap connection ( $k_r$ )**

Length of weld, $l_w$ m	$l_w \leq 1.7$	$1.7 < l_w \leq 8.0$	$l_w > 8.0$
$k_r$	1.00	$1.10 - 0.06 l_w$	0.62

## 9.6.4 Plug and slot welds

### 9.6.4.1 Plug and slot welds in the form of fillet welds around the circumference of the hole or slot

These plug and slot welds shall be regarded as a fillet weld with an effective length as defined in [Clause 9.6.3.5](#), and a nominal capacity as defined in [Clause 9.6.3.10](#). The minimum size shall be as for a fillet weld (see [Clause 9.6.3.2](#)).

### 9.6.4.2 Plug and slot welds in hole filled with weld metal

The effective shear area ( $A_w$ ) of a plug or slot weld in a hole filled with weld metal shall be assumed to be the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

Such a plug or slot weld subject to a design shear force ( $V_w^*$ ) shall satisfy —

$$V_w^* \leq \phi V_w$$

where

- $\phi$  = capacity factor (see [Table 3.4](#))  
 $V_w$  = nominal shear capacity of the weld

The nominal shear capacity ( $V_w$ ) of the weld shall be calculated as follows:

$$V_w = 0.60 f_{uw} A_w$$

### 9.6.4.3 Limitations

Plug or slot welds may only be used to transmit shear in lap joints or to prevent buckling of lapped parts or to join component parts of built-up members.

## 9.6.5 Compound weld

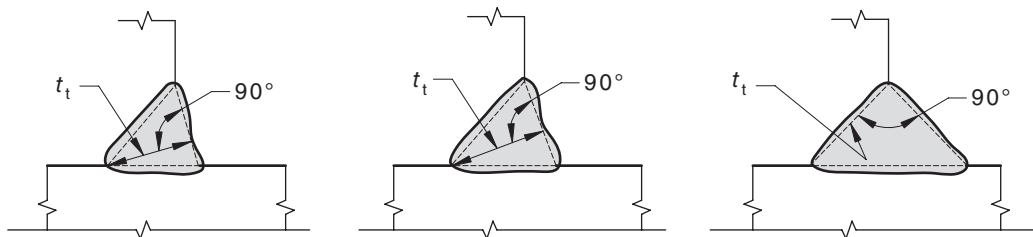
### 9.6.5.1 Description of a compound weld

A compound weld, defined as a fillet weld superimposed on a butt weld, shall be as specified in AS 1101.3.

### 9.6.5.2 Design throat thickness

The design throat thickness of a compound weld, for use in design calculations, shall be —

- for a complete penetration butt weld, the size of the butt weld without reinforcement; and
- for an incomplete penetration butt weld, the shortest distance from the root of the incomplete penetration butt weld to the face of the fillet weld as determined by the largest inscribed triangle in the total weld cross-section, with a maximum value equal to the thickness of the part whose end or edge butts against the face of the other part (see [Figure 9.6.5.2](#)).



NOTE The design throat thickness ( $t_t$ ) of a weld is the minimum distance from the root of a weld to its face, less any reinforcement. The three diagrams above illustrate this concept.

**Figure 9.6.5.2 — Design throat thicknesses of compound welds**

### 9.6.5.3 Strength limit state

The weld shall satisfy the requirements of [Clause 9.6.2.7](#).



## 9.7 Assessment of the strength of a weld group

### 9.7.1 Weld group subject to in-plane loading

#### 9.7.1.1 General method of analysis

The design force per unit length in a fillet weld group subject to in-plane loading shall be determined in accordance with the following:

- (a) The connection plates shall be deemed to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the weld group.
- (b) In the case of a weld group subject to a pure couple only, the instantaneous centre of rotation coincides with the weld group centroid.

In the case of a weld group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of the rotation is at infinity, and the design force per unit length ( $v_w^*$ ) is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the weld group centroid shall be superposed, or a recognized method of analysis shall be used.

- (c) The design force per unit length ( $v_w^*$ ) at any point in the fillet weld group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

A fillet weld shall satisfy the requirements of [Clause 9.6.3.10](#) at all points in the fillet weld group using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)). In the case of a fillet weld group of constant throat thickness, it will be sufficient to check only that point in the group defined by the maximum value of the radius to the instantaneous centre.

#### 9.7.1.2 Alternative analysis

The design force per unit length in the fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and proportioning the design force per unit length in the fillet weld group to satisfy equilibrium between the fillet weld group and the elements of the connected member.

A fillet weld shall satisfy the requirements of [Clause 9.6.3.10](#) at all points in the fillet weld group using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)).

### 9.7.2 Weld group subject to out-of-plane loading

#### 9.7.2.1 General method of analysis

The design force per unit length in a fillet weld group subject to out-of-plane loading shall be determined in accordance with the following:

- (a) The fillet weld group shall be analysed in isolation from the connected element.
- (b) The design force per unit length in the fillet weld resulting from a design bending moment shall be deemed to vary linearly with the distance from the relevant centroidal axes. The design force per unit length in the fillet weld group resulting from any shear force or axial force shall be assumed to be uniformly distributed over the length of the fillet weld group.

A fillet weld shall satisfy the requirements of [Clause 9.6.3.10](#) at all points in the fillet weld group, using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)).

### 9.7.2.2 Alternative analysis

The design force per unit length in a fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member.

A fillet weld shall satisfy the requirements of [Clause 9.6.3.10](#) at all points in the fillet weld group, using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)).

## 9.7.3 Weld group subject to in-plane and out-of-plane loading

### 9.7.3.1 General method of analysis

The design force per unit length as determined from analyses in accordance with [Clauses 9.7.1.1](#) and [9.7.2.1](#) shall satisfy [Clause 9.6.3.10](#) at all points in the fillet weld group, using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)).

### 9.7.3.2 Alternative analysis

The design force per unit length as determined from analyses in accordance with [Clauses 9.7.1.2](#) and [9.7.2.2](#) shall satisfy [Clause 9.6.3.10](#) at all points in the fillet weld group, using the appropriate capacity factor ( $\phi$ ) for a weld group (see [Table 3.4](#)).

## 9.7.4 Combination of weld types

If two or more types of weld are combined in a single connection, the design capacity of the connection shall be the sum of the design capacities of each type, determined in accordance with this Section.

## 9.8 Packing in construction

Where packing is welded between two members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent buckling, the packing shall be trimmed flush with the edges of the element subject to the design action and the size of the welds along the edges shall be increased over the required size by an amount equal to the thickness of the packing.

Otherwise the packing shall extend beyond the edges and shall be welded to the piece to which it is fitted.

## Section 10 Brittle fracture

### 10.1 Methods

The steel grade shall be selected either by the notch-ductile range method as specified in [Clause 10.2](#), or by using a fracture assessment carried out as specified in [Clause 10.5](#).

### 10.2 Notch-ductile range method

The steel grade shall be selected to operate in its notch-ductile temperature range.

The design service temperature for the steel shall be determined in accordance with [Clause 10.3](#). The appropriate steel type suitable for the design service temperature and material thickness shall be selected in accordance with [Clauses 10.4.1](#), [10.4.2](#) and [10.4.3](#).

### 10.3 Design service temperature

#### 10.3.1 General

The design service temperature shall be the estimated lowest metal temperature to be encountered in service or during erection or testing and taken as the basic design temperature as defined in [Clause 10.3.2](#), except as modified in [Clause 10.3.3](#).

#### 10.3.2 Basic design temperature

Lowest one-day mean ambient temperature (LODMAT) isotherms for Australia are given in [Figure 10.3.2](#). The basic design temperature shall be the LODMAT temperature, except that —

- (a) structures that may be subject to especially low local ambient temperatures shall have a basic service temperature of 5 °C cooler than the LODMAT temperature; and
- (b) critical structures, located where the Bureau of Meteorology records indicate the occurrence of abnormally low local ambient temperatures cause the temperature of the critical structure to be lowered below the LODMAT temperature, shall have a basic design service temperature equal to such a lowered temperature of the critical structure.

NOTE In special cases, metal temperatures lower than the LODMAT may occur where there is minimum insulation, minimum heat capacity and radiation shielding, and where abnormally low local temperatures may occur, such as frost conditions.



NOTE 1 Lowest one day mean ambient temperature.

NOTE 2 Based on records 1957 to 1971 supplied by Australian Bureau of Meteorology.

NOTE 3 Isotherms in degree Celsius.

**Figure 10.3.2 — LODMAT isotherms**

### 10.3.3 Modifications to the basic design temperature

The design service temperature shall be the basic design temperature, except that for parts that are subject to artificial cooling below the basic design service temperature (for example, in refrigerated buildings), the design service temperature shall be the minimum expected temperature for the part.

## 10.4 Material selection

### 10.4.1 Selection of steel type

The steel type for the material thickness shall be selected from [Table 10.4.1](#) so that the permissible service temperature listed in [Table 10.4.1](#) is less than the design service temperature determined in accordance with [Clause 10.3](#). The permissible service temperatures listed in [Table 10.4.1](#) shall be subject to the limitations and modifications specified in [Clauses 10.4.2](#) and [10.4.3](#) respectively.

**Table 10.4.1 — Permissible service temperatures according to steel type and thickness**

Steel type (see <a href="#">Table 10.4.4</a> )	Permissible service temperature, °C					
	Thickness, mm					
	≤ 6	> 6 ≤ 12	> 12 ≤ 20	> 20 ≤ 32	> 32 ≤ 70	> 70
1	-20	-10	0	0	0	5
2	-30	-20	-10	-10	0	0
2S	0	0	0	0	0	0
3	-40	-30	-20	-15	-15	10
4	-10	0	0	0	0	5
5	-30	-20	-10	0	0	0
5S	0	0	0	0	0	0
6	-40	-30	-20	-15	-15	-10
7A	-10	0	0	0	0	—
7B	-30	-20	-10	0	0	—
7C	-40	-30	-20	-15	-15	—
8C	-40	-30	—	—	—	—
8Q	-20	-20	-20	-20	-20	-20
9Q	-20	-20	-20	-20	-20	-20
10Q	-20	-20	-20	-20	-20	-20

**Key**  
— = material is not available in these thicknesses.

The permissible service temperature for steels with a L20, L40, L50, Y20 or Y40 designation shall be the colder of the temperature shown in [Table 10.4.1](#), and the specified impact test temperature.

This Table is based on available statistical data on notch toughness characteristics of steels currently made in Australia. Confirmation should be sought from other manufacturers that there is statistical data that confirms their steels meet these same requirements and that the facility where the steel is produced has factory production control. The statistical data shall meet the requirements of [Appendix K](#).

NOTE Steel types 8Q, 9Q and 10Q conforming to AS 3597 are impact tested at -20 °C and therefore meet the requirements of this Table.

## 10.4.2 Limitations

[Table 10.4.1](#) shall only be used without modification for members and components which conform to the fabrication and erection provisions of [Sections 14](#) and [15](#) and AS/NZS 5131 and with the provisions of AS/NZS 1554.1 or AS/NZS 1554.4, as appropriate.

[Table 10.4.1](#) may be used without modification for welded members and connection components which are not subject to more than 1.0 % outer bend fibre strain during fabrication. Members and components subject to greater outer bend fibre strains shall be assessed using the provisions of [Clause 10.4.3](#).

## 10.4.3 Modification for certain applications

### 10.4.3.1 Steel subject to strain between 1.0 % and 10.0 %

Where a member or component is subjected to an outer bend fibre strain during fabrication of between 1.0 % and 10.0 %, the permissible service temperature for each steel type shall be increased by at least 20 °C above the value given in [Table 10.4.1](#).

NOTE Local strain due to weld distortion should be disregarded.

### 10.4.3.2 Steel subject to a strain of not less than 10.0 %

Where a member or component is subjected to an outer bend fibre strain during fabrication of not less than 10.0 %, the permissible service temperature for each steel type shall be increased by at least 20 °C above the value given in [Table 10.4.1](#) plus 1 °C for every 1.0 % increase in outer bend fibre strain above 10.0 %.

NOTE Local strain due to weld distortion should be disregarded.

### 10.4.3.3 Post-weld heat-treated members

Where a member or component has been welded or strained and has been subjected to a post-weld heat-treatment temperature of more than 500 °C, but not more than 620 °C, the permissible service temperature given in [Table 10.4.1](#) shall not be modified.

NOTE Guidance on appropriate post-weld heat-treatment can be found in AS 4458.

### 10.4.3.4 Non-conforming conditions

Steels, for which the permissible service temperature (as modified where applicable) is not known or is warmer than the design service temperature, shall not be used, unless conformance to each of the following requirements is demonstrated:

- (a) A mock-up of the joint or member shall be fabricated from the desired grade of steel, having similar dimensions and strains of not less than that of the service component.
- (b) Three Charpy test specimens shall be taken from the area of maximum strain and tested at the design service temperature.
- (c) The impact properties as determined from the Charpy tests shall be not less than the minimum specified impact properties for the grade of steel under test.
- (d) Where the Standard to which the steel complies does not specify minimum impact properties, the average absorbed energy for three 10 mm × 10 mm test specimens shall be not less than 27 J, provided none of the test results is less than 20 J.
- (e) Where a plate thickness prevents a 10 mm × 10 mm test piece from being used, the standard test thickness closest to the plate thickness shall be used and the minimum value energy absorption requirements shall be reduced proportionally.

### 10.4.4 Selection of steel grade

The steel grade shall be selected to match the required steel type given in [Table 10.4.4](#).

**Table 10.4.4 — Steel type relationship to steel grade**

Steel type (see Note)	Specification and grade of parent steel					
	AS/NZS 1163	AS/NZS 1594	AS/NZS 3678	AS/NZS 3679.2	AS/NZS 3679.1	AS 3597
1	C250	HA200 HA250 HU250 HA300 HA300/1 HU300 HU300/1	200 250 300		300	—
2	C250L0	—	—		300L0	—
2S	—	—	250S0		300S0	

Table 10.4.4 (continued)

Steel type (see Note)	Specification and grade of parent steel				
	AS/NZS 1163	AS/NZS 1594	AS/NZS 3678 AS/NZS 3679.2	AS/NZS 3679.1	AS 3597
			300S0		—
3		XF300	250L15 250L20 250Y20 250L40 250Y40 300L15 300L20 300Y20 300L40 300Y40	300L15	—
4	C350	HA350 HA400 WR350	350 WR350 400	350	—
5	C350L0	—	WR350L0	350L0	—
5S	—	—	350S0	350S0	—
6		XF400	350L15 350L20 350Y20 350L40 350Y40 400L15 400L20 400Y20 400L40 400Y40		—
7A	C450	—	450	—	—
7B	C450L0	—	—	—	—
7C	—	—	450L15 450L20 450Y20 450L40 450Y40	—	—
8C	—	XF500	—	—	—
8Q		—	—	—	500
9Q		—	—	—	600
10Q	—	—	—	—	700

NOTE Steel types 8Q, 9Q and 10Q are quenched and tempered steels currently designated as steel types 8, 9 and 10 respectively in AS/NZS 1554.4.

## 10.5 Fracture assessment

A fracture assessment shall be made, using a fracture mechanics analysis coupled with fracture toughness measurements of the steel selected, weld metal and heat-affected zones and non-destructive examination of the welds and their heat-affected zones.

NOTE For methods of fracture assessment, see BS 7910 and WTIA Technical Note 10.



## Section 11 Fatigue

### 11.1 General

#### 11.1.1 Requirements

This Section applies to the design of structures and structural elements subject to loadings which could lead to fatigue.

The following effects are not covered by this Section:

- (a) Reduction of fatigue life due to corrosion or immersion.
- (b) High stress — low cycle fatigue.
- (c) Thermal fatigue.
- (d) Stress corrosion cracking.

The design shall verify that at each point of the structure the requirements of this Section are satisfied for the design life of the structure.

A structure or structural element which is designed in accordance with this Section shall also conform to the requirements of this Standard for the strength and serviceability limit states.

#### 11.1.2 Notation

For the purposes of this Section the following notation applies:

$d_x, d_y$	=	distances of the extreme fibres from the neutral axes
$f_c$	=	fatigue strength corrected for thickness of material
$f_f$	=	uncorrected fatigue strength
$f_{rn}$	=	detail category reference fatigue strength at $n_r$ — normal stress
$f_{rs}$	=	detail category reference fatigue strength at $n_r$ — shear stress
$f_y$	=	yield stress
$f_3$	=	detail category fatigue strength at constant amplitude fatigue limit ( $5 \times 10^6$ cycles)
$f_5$	=	detail category fatigue strength at cut off limit ( $10^8$ cycles)
$f^*$	=	design stress range
$f_i^*$	=	design stress range for loading event $i$
$l$	=	member length
$n_i$	=	number of cycles of nominal loading event $i$ , producing $f_i^*$
$n_r$	=	reference number of stress cycles ( $2 \times 10^6$ cycles)
$n_{sc}$	=	number of stress cycles
$t_f$	=	flange thickness
$t_p$	=	plate thickness

$\alpha_s$  = inverse of the slope of the S-N curve

$\phi$  = capacity factor

### 11.1.3 Limitation

In all stress cycles, the magnitude of the design stress shall not exceed  $f_y$  and the stress range shall not exceed  $1.5f_y$ .

### 11.1.4 Designation of weld category

The welds in the welded details given in [Tables 11.5.1\(B\)](#) and (D) for Detail Categories 112 and below shall conform with Category SP as defined in AS/NZS 1554.1 or AS/NZS 1554.4, as appropriate.

The welds in the welded details given in [Table 11.5.1\(B\)](#) for Detail Category 125 shall have a weld quality conforming to that defined in AS/NZS 1554.5.

### 11.1.5 Method

For the reference design condition, the capacity factor ( $\phi$ ) shall be taken as 1.0.

The reference design condition implies the following:

- (a) The detail is located on a redundant load path, in a position where failure at that point alone shall not lead to overall collapse of the structure.
- (b) The stress history is estimated by conventional methods.
- (c) The load cycles are not highly irregular.
- (d) The detail is accessible for, and subject to, regular inspection.

The capacity factor ( $\phi$ ) shall be reduced when any of the above conditions do not apply.

For non-redundant load paths, the capacity factor ( $\phi$ ) shall be less than or equal to 0.70.

### 11.1.6 Thickness effect

The thickness correction factor ( $\beta_{tf}$ ) shall be taken as —

$$\beta_{tf} = 1.0$$

except for a transverse fillet or butt welded connection involving a plate thickness ( $t_p$ ) greater than 25 mm, where  $\beta_{tf}$  shall be calculated as follows:

$$\beta_{tf} = \left( \frac{25}{t_p} \right)^{0.25}$$

The uncorrected fatigue strength ( $f_f$ ) shall be reduced to a corrected fatigue strength ( $f_c$ ) using —

$$f_c = \beta_{tf} f_f$$

The uncorrected detail category reference fatigue strength for normal stress ( $f_{rn}$ ) shall be reduced to a corrected detail category reference fatigue for normal stress ( $f_{rnc}$ ) using —

$$f_{rnc} = \beta_{tf} f_{rn}$$

The uncorrected detail category reference fatigue strength for shear stress ( $f_{rs}$ ) shall be reduced to a corrected detail category reference fatigue strength for shear stress ( $f_{rsc}$ ) using —

$$f_{rsc} = \beta_{tf} f_{rs}$$

The uncorrected detail category reference fatigue strength at constant amplitude fatigue limit ( $f_3$ ) shall be reduced to a corrected detail category reference fatigue strength at constant amplitude fatigue limit ( $f_{3c}$ ) using —

$$f_{3c} = \beta_{tf} f_3$$

The uncorrected detail category reference fatigue strength at cut-off limit ( $f_5$ ) shall be reduced to a corrected detail category reference fatigue strength at cut-off limit ( $f_{5c}$ ) using —

$$f_{5c} = \beta_{tf} f_5$$

## 11.2 Fatigue loading

The fatigue loading shall be obtained from the referring Standards where applicable. These include —

AS 1418.1, *Cranes, hoists and winches, Part 1: General requirements*

AS 1418.3, *Cranes, hoists and winches, Part 3: Bridge, gantry, portal (including container cranes) and jib cranes*

AS 1418.5, *Cranes, hoists and winches, Part 5: Mobile cranes*

AS 1418.18, *Cranes, hoists and winches, Part 18: Crane runways and monorails*

AS 5100.1, *Bridge design, Part 1: Scope and general principles*

AS 5100.2, *Bridge design, Part 2: Design loads*

In other cases, the fatigue loading used in design shall be the actual service loading, including dynamic effects.

## 11.3 Design spectrum

### 11.3.1 Stress determination

The design stresses shall be determined from an elastic analysis of the structure or from the stress history obtained from strain measurements.

The design stresses shall be determined as normal or shear stresses taking into account all design actions on the member but excluding stress concentrations due to the geometry of the detail as described in [Tables 11.5.1\(A\) to 11.5.1\(D\)](#). The effect of stress concentrations which are not characteristic of the detail shall be taken into account separately.

Unless noted otherwise, each arrow in [Tables 11.5.1\(A\) to 11.5.1\(D\)](#) indicates the location and direction of the stresses acting in the base material on a plane normal to the arrow for which the stress range is to be calculated.

For the fatigue assessment of trusses made of open sections in which the connections are not pinned, the effects of secondary bending moments shall be taken into account unless —

$$l/d_x > 40; \text{ or}$$

$$l/d_y > 40$$

For truss connections using hollow sections, the stress range in the members may be calculated without consideration of the effects of connection stiffness and eccentricities, subject to the following:

- For a truss connection involving circular hollow sections, the stress range shall be multiplied by the appropriate factor given in [Table 11.3.1\(A\)](#).
- For a truss connection involving rectangular hollow sections, the calculated stress range shall be multiplied by the appropriate factor given in [Table 11.3.1\(B\)](#).
- The design throat thickness of a fillet weld shall be greater than the wall thickness of the connected member.

**Table 11.3.1(A) — Multiplying factors for calculated stress range — Circular hollow sections**

Type of connection		Chords	Verticals	Diagonals
Gap connections	K type	1.5	1.0	1.3
	N type	1.5	1.8	1.4
Overlap connections	K type	1.5	1.0	1.2
	N type	1.5	1.65	1.25

**Table 11.3.1(B) — Multiplying factors for calculated stress range — Rectangular hollow sections**

Type of joint		Chords	Verticals	Diagonals
Gap connections	K type	1.5	1.0	1.5
	N type	1.5	2.2	1.6
Overlap connections	K type	1.5	1.0	1.3
	N type	1.5	2.0	1.4

### 11.3.2 Design spectrum calculation

The stress spectrum of a nominal loading event producing irregular stress cycles shall be obtained by a rational stress cycle counting method. Rainflow counting or an equivalent method may be used.

## 11.4 Exemption from assessment

Fatigue assessment is not required for a member, connection or detail, if the normal and shear design stress ranges ( $f^*$ ) satisfy —

$$f^* < \phi \times 27 \text{ MPa}$$

or if the number of stress cycles ( $n_{sc}$ ) satisfies

$$n_{sc} < 2 \times 10^6 \left( \frac{\phi \times 36}{f^*} \right)^3$$

## 11.5 Detail category

### 11.5.1 Detail categories for normal stress

A detail category for normal stress shall be assigned for each structural member, connection or detail in the structure. The detail categories are given in [Tables 11.5.1\(A\) to \(D\)](#).

The classifications in these tables are divided into four parts which correspond to four basic groups:

Group 1: Non-welded details — plain material and bolted plates. (See [Table 11.5.1\(A\)](#))

Group 2: Welded details — not in hollow sections. (See [Table 11.5.1\(B\)](#))

Group 3: Bolts. (See [Table 11.5.1\(C\)](#))

Group 4: Welded details — in hollow sections. (See [Table 11.5.1\(D\)](#))

Details not classified in [Tables 11.5.1\(A\) to \(D\)](#) shall be treated as the lowest detail category of a similar detail, unless a superior fatigue strength is proved by testing or by analysis and testing.

### 11.5.2 Detail categories for shear stress

A detail category for shear stress shall be assigned for each relevant detail in the structure. The detail categories for shear stress are given in [Table 11.5.1\(B\)](#) (Descriptions 39 and 40) and in [Table 11.5.1\(C\)](#) (Description 41).

Table 11.5.1(A) — Detail category classification group 1 non-welded details

Detail category	Constructional details	
	Illustration (see Note)	Description
160		<p><b>ROLLED AND EXTRUDED PRODUCTS</b></p> <p>(1) Plates and flats                  (2) Rolled sections                  (3) Seamless tubes</p> <p>Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress</p>
140		<p><b>BOLTED CONNECTIONS</b></p> <p>(4) and (5) Stress range calculated on the gross section for 8.8/TF bolting category and on the net section otherwise.</p> <p>Unsupported one-sided coverplate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses.</p> <p><b>MATERIAL WITH GAS-CUT OR SHEARED EDGES WITH NO DRAGLINES (6)</b></p> <p>All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.</p>
125		<p><b>MATERIAL WITH MACHINE GAS-CUT EDGES WITH DRAGLINES OR MANUAL GAS-CUT MATERIAL (7)</b></p> <p>Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.</p>

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**Table 11.5.1(A)** (continued)

Detail category	Constructional details	
	Illustration (see Note)	Description
NOTE The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.		

**Table 11.5.1(B) — Detail category classification group 2 welded details — Not in hollow sections**

Detail category	Constructional details	
	Illustration (see Note 2)	Description
125	<p>(8)</p> <p>(9)</p>	<p><b>WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS</b></p> <p>(8) and (9) Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having unrepaired stop-start positions.</p> <p>(See Note 1)</p>
112	<p>(10)</p> <p>(11)</p> <p>(12)</p>	<p><b>WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS</b></p> <p>(10) and (11) Zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having unrepaired stop-start positions.</p> <p>(12) Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continual manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 100.</p>

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Table 11.5.1(B) (continued)

Detail category	Constructional details	
	Illustration (see Note 2)	Description
90	<p>(13)</p>	<p><b>WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS</b></p> <p>(13) Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.</p>
80	<p>(14)</p>	<p><b>INTERMITTENT LONGITUDINAL WELDS</b></p> <p>(14) Zones of intermittent longitudinal welds.</p>
71	<p>(15)</p>	<p><b>INTERMITTENT LONGITUDINAL WELDS</b></p> <p>(15) Zones containing cope holes in longitudinally welded T joints. Cope hole not to be filled with weld.</p>
112	<p>(16) (17) (18)</p> <p>Taper <math>\leq 1:4</math></p>	<p><b>TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)</b></p> <p>Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides.</p> <p>(16) Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 % NDT inspection, and weld surface to be free of exposed porosity in the weld metal.</p> <p>(17) Plate girders welded as (16) before assembly.</p> <p>(18) Transverse splices as (16) with radiused or tapered transition with taper <math>\leq 1:4</math>.</p>



Table 11.5.1(B) (continued)

Detail category	Constructional details	
	Illustration (see Note 2)	Description
90	<p>(19) (20)</p> <p>Taper <math>\leq 1 : 4</math></p> <p>(21)</p>	<p><b>TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)</b></p> <p>Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides.</p> <p>(19) Transverse splices of plates, rolled sections, or plate girders.</p> <p>(20) Transverse splices of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 71, as for (15).</p> <p>(21) Transverse splices in plates or flats being tapered in width or in thickness where the taper is <math>\leq 1:4</math>.</p>
80	<p>(22)</p> <p><math>1 : 4 &lt; \text{taper} \leq 1 : 2.5</math></p>	<p><b>TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)</b></p> <p>Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides.</p> <p>(22) Transverse splices as for (21) with taper in width or thickness <math>&gt; 1:4</math> and <math>\leq 1:2.5</math>.</p>
71	<p>(23) (24)</p> <p><math>\geq 10 \text{ mm}</math></p> <p><math>\geq 10 \text{ mm}</math></p> <p><math>\geq 10 \text{ mm}</math></p>	<p><b>TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)</b></p> <p>(23) Transverse butt welded splices made on a backing bar. The end of the fillet weld of the backing strip shall be greater than 10 mm from the edges of the stressed plate.</p> <p>(24) Transverse butt welds as for (23) with taper on width or thickness <math>&lt; 1:2.5</math>.</p>

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Table 11.5.1(B) (continued)

Detail category	Constructional details		
	Illustration (see Note 2)	Description	
50	<p>(25)</p>	<p><b>TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)</b></p> <p>(25) Transverse butt welds as (23) where fillet welds end closer than 10 mm to plate edge</p>	
71	<p>(26)</p>	<p><b>CRUCIFORM JOINTS WITH LOAD-CARRYING WELDS</b></p> <p>(26) Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be <math>&lt; 0.15</math> times the thickness of intermediate plate</p>	
56	(27)		(27) Partial penetration or fillet welds with stress range calculated on plate area
36	(28)		(28) Partial penetration or fillet welds with stress range calculated on throat area of weld
63		<p>(29)</p>	<p><b>OVERLAPPED WELDED JOINTS</b></p> <p>(29) Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration</p>
56	(30)		(30) Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements
45	(31)		(31) Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld

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Table 11.5.1(B) (continued)

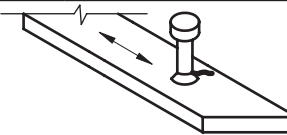
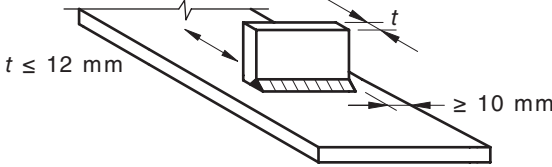
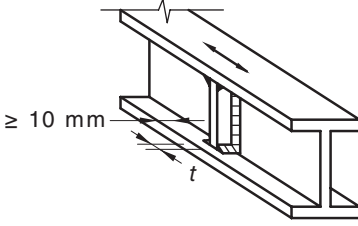
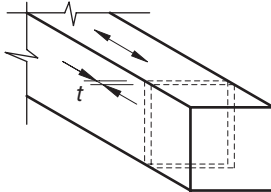
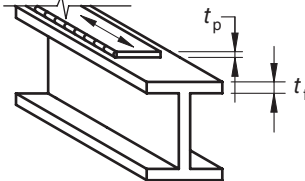
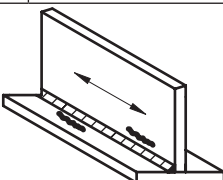
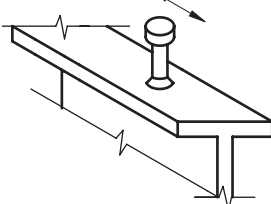
Detail category	Constructional details		Description
	Illustration (see Note 2)		
90	(32)	(33)	<p><b>WELDED ATTACHMENTS (NON-LOAD CARRYING WELDS) LONGITUDINAL WELDS</b></p> <p>(32) Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted.</p>
	—	$\frac{1}{3} \leq \frac{r}{b}$	
80	$l \leq 50 \text{ mm}$	—	<p>(33) Gusset welded to the edge of a plate or beam flange. Smooth transition radius (<math>r</math>) formed by machining or flame-cutting plus grinding. Class of detail varies according to <math>r/b</math> ratio as noted.</p>
71	$50 < l \leq 100 \text{ mm}$	$\frac{1}{6} \leq \frac{r}{b} < \frac{1}{3}$	
50	$100 \text{ mm} < l$	—	
45	—	$\frac{r}{b} < \frac{1}{6}$	
80	 <p>(34)</p>		<p><b>WELDED ATTACHMENTS</b></p> <p>(34) Shear connectors on base material (failure in base material)</p>

Table 11.5.1(B) (continued)

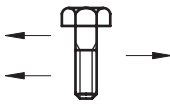
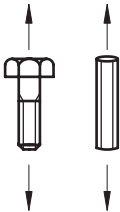
Detail category	Constructional details		
	Illustration (see Note 2)	Description	
80	$t \leq 12 \text{ mm}$	 <p>(35)</p>  <p>(36)</p>  <p>(37)</p>	
71	$t > 12 \text{ mm}$		
50	$t_f \text{ and } t_p \leq 25 \text{ mm}$		
36	$t_f \text{ and } t_p > 25 \text{ mm}$	 <p>(38)</p>	<p><b>COVER PLATES IN BEAMS AND PLATE GIRDERS</b></p> <p>(38) End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential. (See Description 31 for the fatigue check in the weld itself.)</p>
80		 <p>(39)</p>  <p>(40)</p>	<p><b>WELDS LOADED IN SHEAR</b></p> <p>(39) Fillet welds transmitting shear. Stress range to be calculated on weld throat area.</p> <p>(40) Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud).</p>

NOTE 1 See [Clause 11.1.4](#) regarding weld category

Table 11.5.1(B) (continued)

Detail category	Constructional details	
	Illustration (see Note 2)	Description
NOTE 2 The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.		

Table 11.5.1(C) — Detail category classification group 3 bolts

Detail category	Constructional details	
	Illustration (see Note 3)	Description
100	 <p>(41)</p>	<p><b>(41) BOLTS IN SHEAR (8.8/TB BOLTING CATEGORY ONLY)</b></p> <p>Shear stress range calculated on the minor diameter area of the bolt (<math>A_c</math>)</p> <p>(See Note 1)</p>
36	 <p>(42)</p>	<p><b>(42) BOLTS AND THREADED RODS IN TENSION</b> (tensile stress to be calculated on the tensile stress area <math>A_s</math>).</p> <p>Additional forces due to prying effects shall be taken into account. For tensioned bolts (8.8/TF and 8.8/TB bolting categories), the stress range depends on the connection geometry.</p> <p>(See Note 2)</p>

NOTE 1 If the shear on the joint is insufficient to cause slip of the joint (see [Clause 9.2.3](#)), the shear in the bolt need not be considered in fatigue.

NOTE 2 In connections with tensioned bolts (see [Clause 15.2](#)), the change in the force in the bolts is often less than the applied force, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections. However, Standards Australia is not prepared to recommend methods for calculation of the stress range in tensioned bolts.

NOTE 3 The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 11.5.1(D) — Detail category classification group 4 welded details — In hollow sections

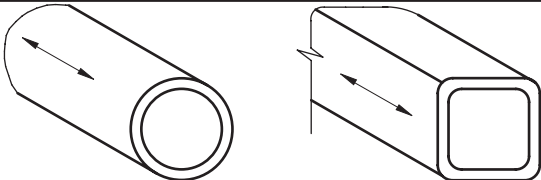




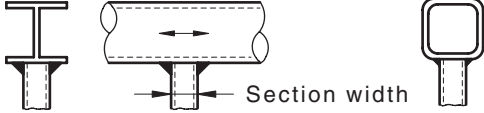


Detail category	Constructional details	
	Illustration (see Note)	Description
140	 <p>(43)</p>	<p><b>CONTINUOUS AUTOMATIC LONGITUDINAL WELDS</b></p> <p>(43) No stop-starts, or as manufactured</p>
90 ( $t \geq 8$ mm) 71 ( $t < 8$ mm)	 <p>(44)</p>	<p><b>TRANSVERSE BUTT WELDS</b></p> <p>(44) Butt-welded end-to-end connection of circular hollow sections</p>
71 ( $t \geq 8$ mm) 56 ( $t < 8$ mm)	 <p>(45)</p>	<p>(45) Butt-welded end-to-end connection of rectangular hollow sections</p>

Table 11.5.1(D) (continued)

Detail category	Constructional details	
	Illustration (see Note)	Description
56 ( $t \geq 8$ mm)	 <p>(46)</p>	<b>BUTT WELDS TO INTERMEDIATE PLATE</b> (46) Circular hollow sections, end-to-end butt welded with an intermediate plate
50 ( $t < 8$ mm)		
50 ( $t \geq 8$ mm)	 <p>(47)</p>	(47) Rectangular hollow sections, end-to-end butt welded with an intermediate plate
41 ( $t < 8$ mm)		
71	 <p>(48)</p>	<b>WELDED ATTACHMENTS</b> (Non-load-carrying) (48) Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction $\leq 100$ mm
45 ( $t \geq 8$ mm)	 <p>(49)</p>	<b>FILLET WELDS TO INTERMEDIATE PLATE</b> (49) Circular hollow sections, end-to-end fillet welded with an intermediate plate
40 ( $t < 8$ mm)		
40 ( $t \geq 8$ mm)	 <p>(50)</p>	(50) Rectangular hollow sections, end-to-end fillet welded with an intermediate plate
36 ( $t < 8$ mm)		

NOTE The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

## 11.6 Fatigue strength

### 11.6.1 Definition of fatigue strength for normal stress

The uncorrected fatigue strength ( $f_f$ ) for each detail category ( $f_{rn}$ ) subject to normal stress is defined by —

$$f_f^3 = \frac{f_{rn}^3 \times 2 \times 10^6}{n_{sc}} \text{ when } n_{sc} \leq 5 \times 10^6$$

$$f_f^5 = \frac{f_5^5 \times 10^8}{n_{sc}} \text{ when } 5 \times 10^6 < n_{sc} \leq 10^8$$

where  $n_{sc}$  is the number of stress cycles.

Values of  $f_f$ ,  $f_3$  and  $f_5$  are shown in [Figure 11.6.1](#) for each detail category ( $f_{rn}$ ).

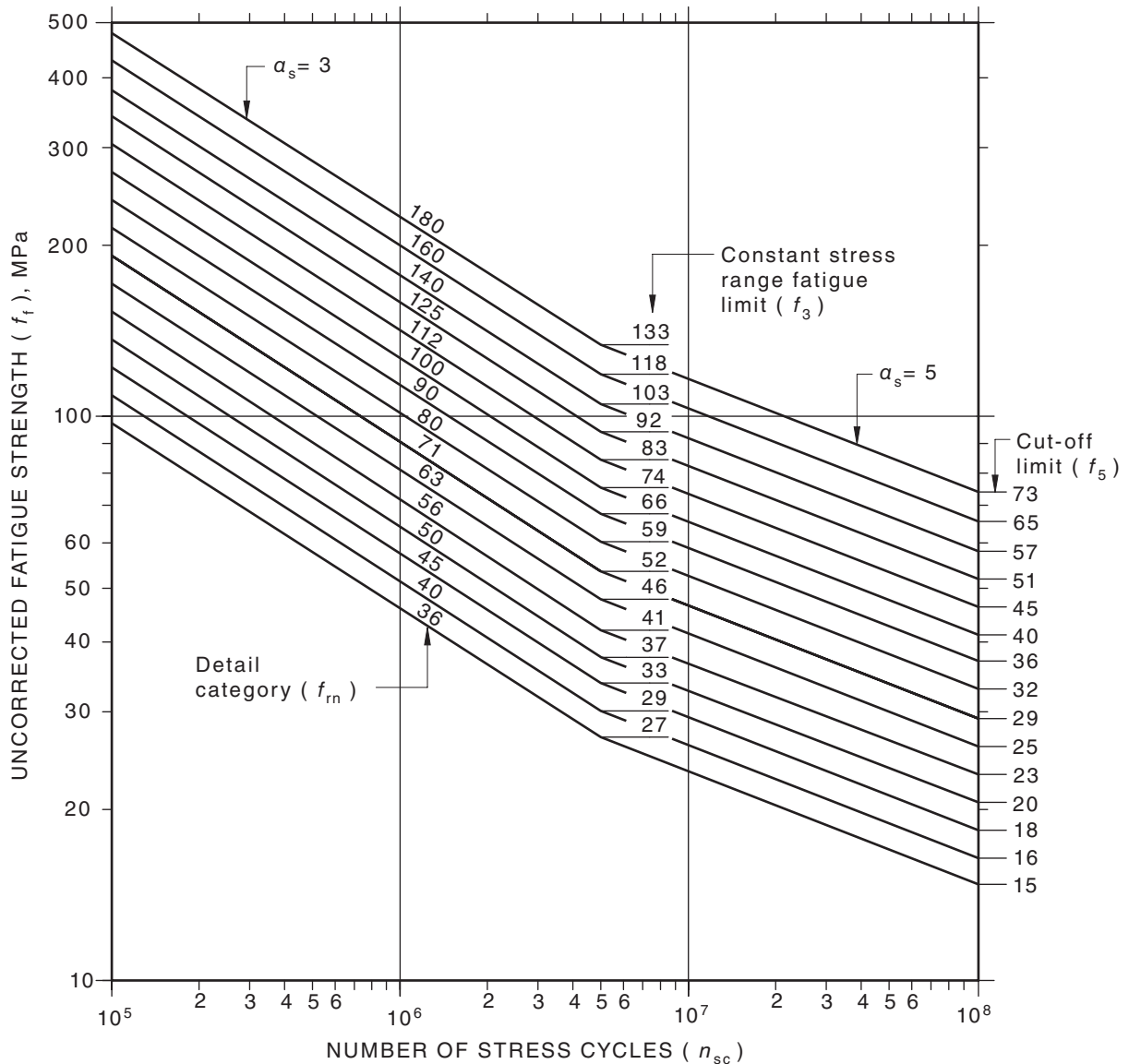


Figure 11.6.1 — S-N curve for normal stress

### 11.6.2 Definition of fatigue strength for shear stress

The uncorrected fatigue strength ( $f_f$ ) for each detail category ( $f_{rs}$ ) subject to shear stress is defined by —

$$f_f^5 = \frac{f_{rs}^5 \times 2 \times 10^6}{n_{sc}} \text{ when } n_{sc} \leq 10^8$$

where  $n_{sc}$  is the number of stress cycles.

Values of  $f_f$  and  $f_5$  are shown in [Figure 11.6.2](#) for each detail category ( $f_{rs}$ ).

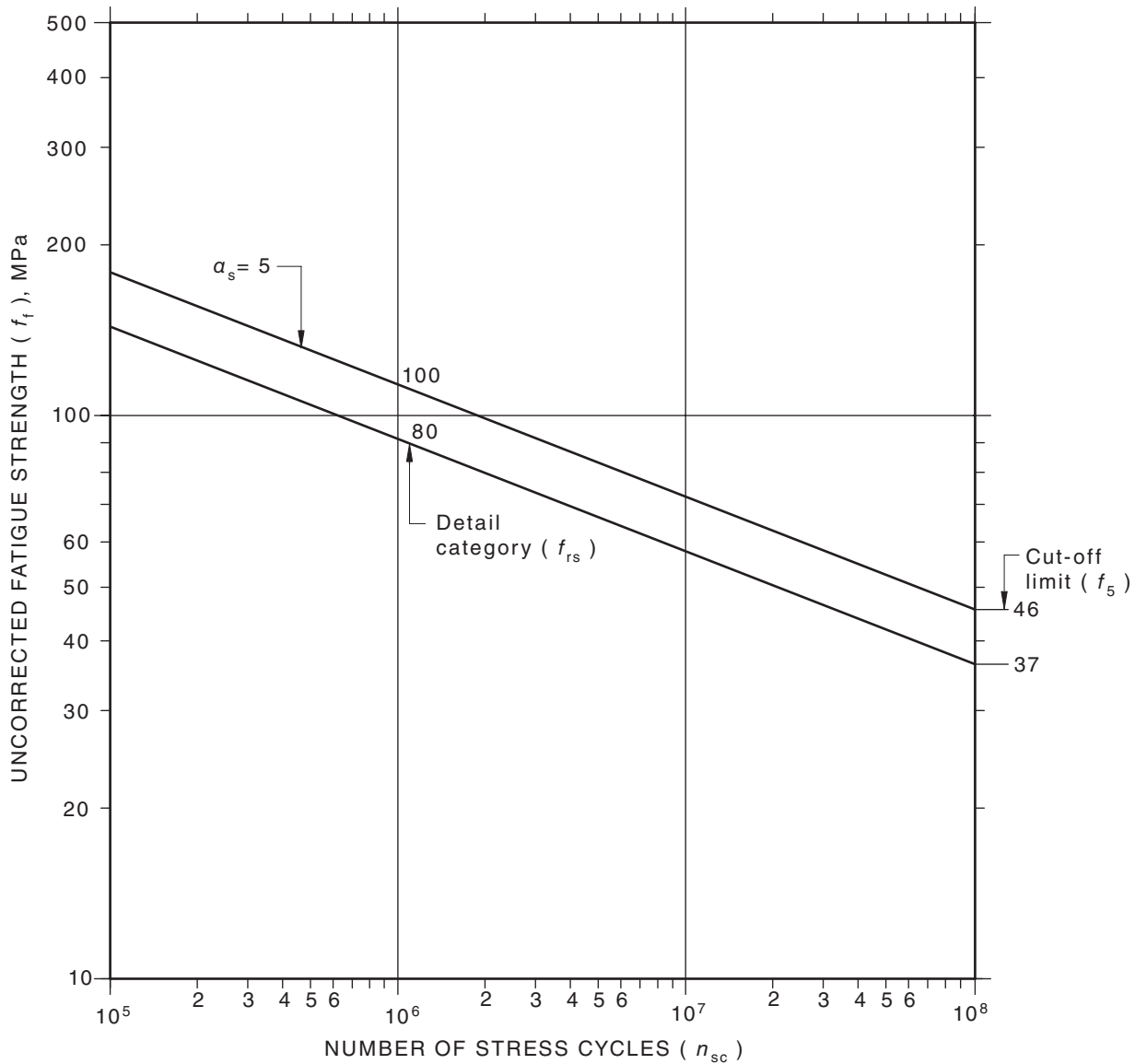


Figure 11.6.2 — S-N curve for shear stress

### 11.7 Exemption from further assessment

At any point in the structure at which the design stress range ( $f^*$ ) is less than  $(\phi f_{3c})$  for all normal stress ranges, no further assessment at that point is required.

### 11.8 Fatigue assessment

#### 11.8.1 Constant stress range

The design stress range ( $f^*$ ) at any point in the structure subject only to constant stress range cycles shall satisfy —

$$\frac{f^*}{\phi f_c} \leq 1.0$$



### 11.8.2 Variable stress range

The design stress ranges ( $f^*$ ) at any point in the structure at which the stress range varies shall satisfy —

(a) for normal stresses:

$$\frac{\sum_i n_i (f_i^*)^3}{5 \times 10^6 (\phi f_{3c})^3} + \frac{\sum_j n_j (f_j^*)^5}{5 \times 10^6 (\phi f_{3c})^5} \leq 1.0$$

(b) for shear stresses:

$$\frac{\sum_k n_k (f_k^*)^5}{2 \times 10^6 (\phi f_{rsc})^5} \leq 1.0$$

where

- (i) summation  $\sum_i$  is for  $i$  design stress ranges ( $f_i^*$ ) for which  $\phi f_{3c} \leq f_i^*$ ;
- (ii) summation  $\sum_j$  is for  $j$  design stress ranges ( $f_j^*$ ) for which  $\phi f_{5c} \leq f_j^* < \phi f_{3c}$ ; and
- (iii) summation  $\sum_k$  is for  $k$  design stress ranges ( $f_k^*$ ) for shear stresses  $\phi f_{5c} \leq f_k^*$ .

### 11.9 Punching limitation

For members and connections requiring assessment for fatigue in accordance with this Section, a punched hole shall only be permitted in material whose thickness does not exceed 12.0 mm.

## Section 12 Fire

### 12.1 Requirements

This Section applies to steel building elements required to have a fire-resistance level (FRL).

For protected steel members and connections, the thickness of protection material ( $h_i$ ) shall be greater than or equal to that required to give a period of structural adequacy (PSA) equal to the required FRL.

For unprotected steel members and connections, the exposed surface area to mass ratio ( $k_{sm}$ ) shall be less than or equal to that required to give a PSA equal to the required FRL.

The PSA shall be determined in accordance with [Clause 12.3](#), using the variations of the mechanical properties of steel with temperature as specified in [Clause 12.4](#).

Connections and web penetrations shall be in accordance with [Clause 12.10](#).

### 12.2 Definitions

For the purpose of this Section, the definitions below apply.

*Exposed surface area to mass ratio* — ratio of the surface area exposed to the fire to the mass of steel.

NOTE In the case of members with fire protection material applied, the exposed surface area is to be taken as the internal surface area of the fire protection material.

Fire exposure condition —

- (a) *three-sided fire exposure condition* — steel member incorporated in or in contact with a concrete or masonry floor or wall.

NOTE 1 Three-sided fire exposure condition is to be considered separately unless otherwise specified in [Clause 12.9](#).

NOTE 2 Members with more than one face in contact with a concrete or masonry floor or wall may be treated as three-sided fire exposure.

- (b) *four-sided fire exposure condition* — steel member exposed to fire on all sides.

*Fire protection system* — fire protection material and its method of attachment to the steel member.

*Fire-resistance level (FRL)* — fire-resistance grading period for structural adequacy only, in minutes, which is required to be attained in the standard fire test.

*Period of structural adequacy (PSA)* — time ( $t$ ), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test.

*Prototype* — test specimen representing a steel member and its fire protection system which is subjected to the standard fire test.

*Standard fire test* — fire-resistance test specified in AS 1530.4.

*Stickability* — ability of the fire protection system to remain in place as the member deflects under load during a fire test, as specified in AS 1530.4.

*Structural adequacy* — ability of the member exposed to the standard fire test to carry the test load specified in AS 1530.4.

### 12.3 Determination of period of structural adequacy

The PSA shall be determined using one of the following methods —

- (a) by calculation —
  - (i) by determining the limiting temperature of the steel ( $T_l$ ) in accordance with [Clause 12.5](#); and then
  - (ii) by determining the PSA as the time from the start of the test ( $t$ ) to the time at which the limiting steel temperature is attained in accordance with [Clause 12.6](#) for protected members and [Clause 12.7](#) for unprotected members;
- (b) by direct application of a single test in accordance with [Clause 12.8](#); or
- (c) by structural analysis in accordance with [Section 4](#), using mechanical properties which vary with temperature in accordance with [Clause 12.4](#). Calculation of the temperature of the steel member shall be by using a rational method of analysis confirmed by test data.

### 12.4 Variation of mechanical properties of steel with temperature

#### 12.4.1 Variation of yield stress with temperature

The influence of temperature on the yield stress of steel shall be taken as follows:

$$\frac{f_y(T)}{f_y(20)} = 1.0 \quad \text{when } 0^\circ\text{C} < T \leq 215^\circ\text{C}; \text{ and}$$

$$= \frac{905 - T}{690} \quad \text{when } 215^\circ\text{C} < T \leq 905^\circ\text{C}$$

where

$$f_y(T) = \text{yield stress of steel at } T^\circ\text{C}$$

$$f_y(20) = \text{yield stress of steel at } 20^\circ\text{C}$$

$$T = \text{temperature of the steel in } ^\circ\text{C}$$

This relationship is shown by Curve 1 in [Figure 12.4](#).

#### 12.4.2 Variation of modulus of elasticity and shear modulus with temperature

The influence of temperature on the modulus of elasticity and shear modulus of steel shall be taken as follows:

$$\frac{E(T)}{E(20)} = 1.0 + \left[ \frac{T}{2000 \left[ \ln \left( \frac{T}{1100} \right) \right]} \right] \quad \text{when } 0^\circ\text{C} < T \leq 600^\circ\text{C}; \text{ and}$$

$$= \frac{690 \left( 1 - \frac{T}{1000} \right)}{T - 53.5} \quad \text{when } 600^\circ\text{C} < T \leq 1000^\circ\text{C}$$

$$G(T) = \frac{E(T)}{2(1 + \nu)}$$

where

- $E(T)$  = modulus of elasticity of steel at  $T$  °C  
 $E(20)$  = modulus of elasticity of steel at 20 °C  
 $G(T)$  = shear modulus of steel at  $T$  °C

This relationship is shown by Curve 2 in [Figure 12.4](#).

### 12.4.3 Slenderness at elevated temperature

In the calculation of the terms  $\sqrt{(f_y/250)}$ ,  $\sqrt{(250/f_y)}$ ,  $(f_y/250)$ ,  $(250/f_y)$  in [Clauses 5.2.2](#), [5.3.2.4](#), [5.10](#), [5.11](#), [5.14](#), [6.2.3](#), [6.3.3](#), [8.4.3.3](#) and [8.4.6](#), the value of  $f_y$  shall be  $f_y(20)$ . All other values of  $f_y$  shall be  $f_y(T)$ .

NOTE 1 In the calculation of slenderness in the Sections listed above, the modulus of elasticity is assumed implicitly at  $E(20)$ . Adjustment of  $f_y$  with temperature without a corresponding adjustment of  $E$  may lead to unsafe designs.

NOTE 2 The simplified method introduced in [Clause 12.4.3](#) implicitly assumes that  $E(T)$  and  $f_y(T)$  reduce by the same fraction with temperature. Hence the slenderness ratios effectively remain unchanged and only the section capacities reduce with  $f_y(T)$ .

### 12.5 Determination of limiting steel temperature

The limiting steel temperature ( $T_l$ ) shall be calculated as follows:

$$T_l = 905 - 690r_f$$

where  $r_f$  is the ratio of the design action on the member under the design load for fire specified in Section 4 of AS/NZS 1170.0 to the design capacity of the member ( $\phi R_u$ ) at room temperature.

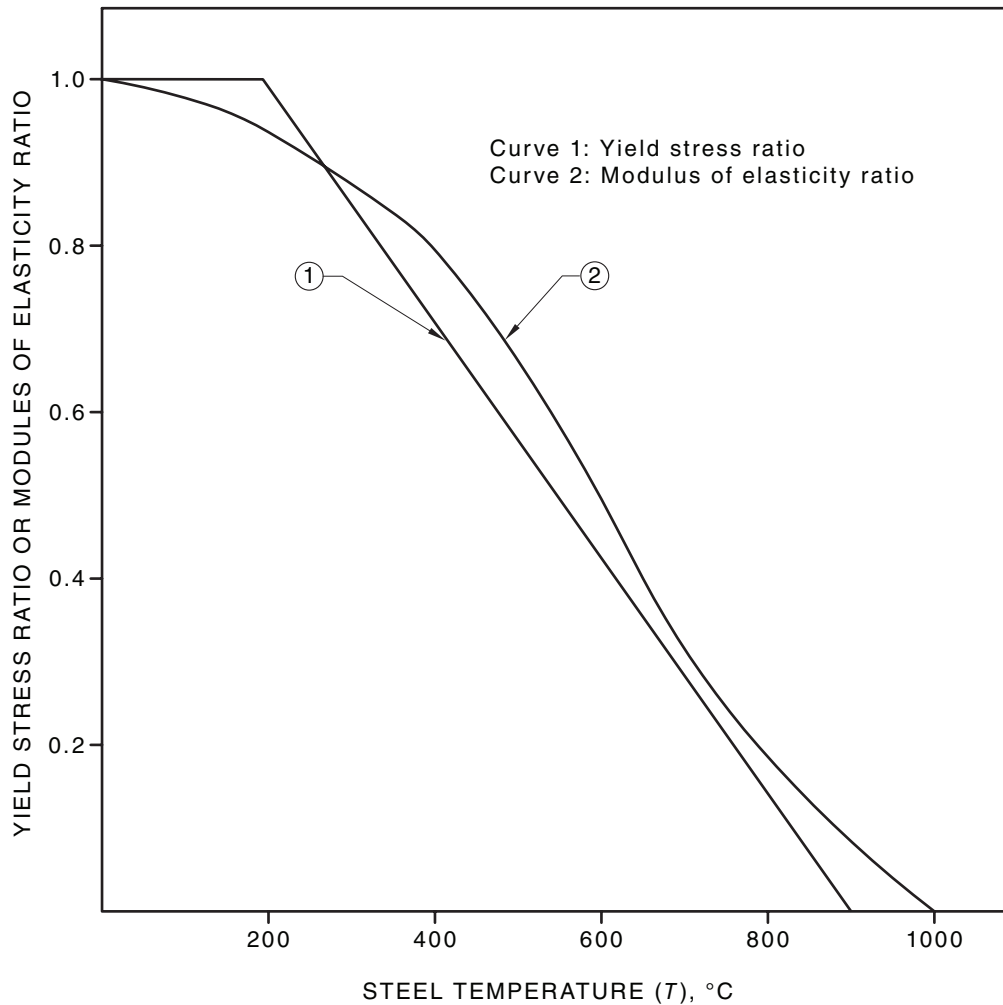


Figure 12.4 — Variation of mechanical properties of steel with temperature

## 12.6 Determination of time at which limiting temperature is attained for protected members

### 12.6.1 Methods

The time ( $t$ ) at which the limiting temperature ( $T_l$ ) is attained shall be determined by calculation on the basis of a suitable series of fire tests in accordance with [Clause 12.6.2](#) or from the results of a single test in accordance with [Clause 12.6.3](#).

For beams and for all members with a four-sided fire exposure condition, the limiting temperature ( $T_l$ ) shall be taken as the average of all of the temperatures measured at the thermocouple locations shown in AS 1530.4.

For columns with a three-sided fire exposure condition, the limiting temperature ( $T_l$ ) shall be taken as the average of the temperatures measured at the thermocouple locations on the face farthest from the wall. Alternatively, the temperatures from members with a four-sided fire exposure condition and the same surface area to mass ratio may be used.

Alternatively, methods of assessment in accordance with EN 13381-4 and EN 13381-8 may be used.

## 12.6.2 Temperature based on test series

### 12.6.2.1 General

Calculation of the variation of steel temperature with time shall be by interpolation of the results of a series of fire tests using the regression analysis equation specified in [Clause 12.6.2.2](#) subject to the limitations and conditions of [Clause 12.6.2.3](#).

### 12.6.2.2 Regression analysis

The relationship between temperature ( $T$ ) and time ( $t$ ) for a series of tests on a group shall be calculated by least-squares regression as follows:

$$t = k_0 + k_1 h_i + k_2 \left( \frac{h_i}{k_{sm}} \right) + k_3 T + k_4 h_i T + k_5 \left( \frac{h_i T}{k_{sm}} \right) + k_6 \left( \frac{T}{k_{sm}} \right)$$

where

$t$	=	time from the start of the test, in minutes
$k_0$ to $k_6$	=	regression coefficients
$h_i$	=	thickness of fire protection material, in millimetres
$T$	=	steel temperature, in degrees Celsius, $T > 250$ °C
$k_{sm}$	=	exposed surface area to mass ratio, in square metres/tonne

### 12.6.2.3 Limitations and conditions on use of regression analysis

Test data to be utilized in accordance with [Clause 12.6.2.2](#) shall satisfy the following:

- Steel members shall be protected with board, sprayed blanket or similar insulation materials having a dry density less than 1000 kg/m<sup>3</sup>.  
NOTE Experience has shown that the above regression method can also be used for materials such as intumescent and ablative coatings subject to the coefficient of correlation exceeding 0.9.
- All tests shall incorporate the same fire protection system.
- All members shall have the same fire exposure condition.
- The test series shall include at least nine tests.
- The test series may include prototypes which have not been loaded provided that stickability has been demonstrated.
- All members subject to a three-sided fire exposure condition shall be within a group in accordance with [Clause 12.9](#).

The regression equation shall only be used for interpolation. The window defining the limits of interpolation shall be determined as shown in [Figure 12.6.2.3](#).

The regression equation obtained for one fire protection system may be applied to another system using the same fire protection material and the same fire exposure condition provided that stickability has been demonstrated for the second system.

A regression equation obtained using prototypes with a four-sided fire exposure condition may be applied to a member with a three-sided fire exposure condition provided that stickability has been demonstrated for the three-sided case.

### 12.6.3 Temperature based on single test

The variation of steel temperature with time measured in a standard fire test may be used without modification provided —

- (a) the fire protection system is the same as the prototype;
- (b) the fire exposure condition is the same as the prototype;
- (c) the fire protection material thickness is equal to or greater than that of the prototype;
- (d) the surface area to mass ratio is equal to or less than that of the prototype; and
- (e) where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

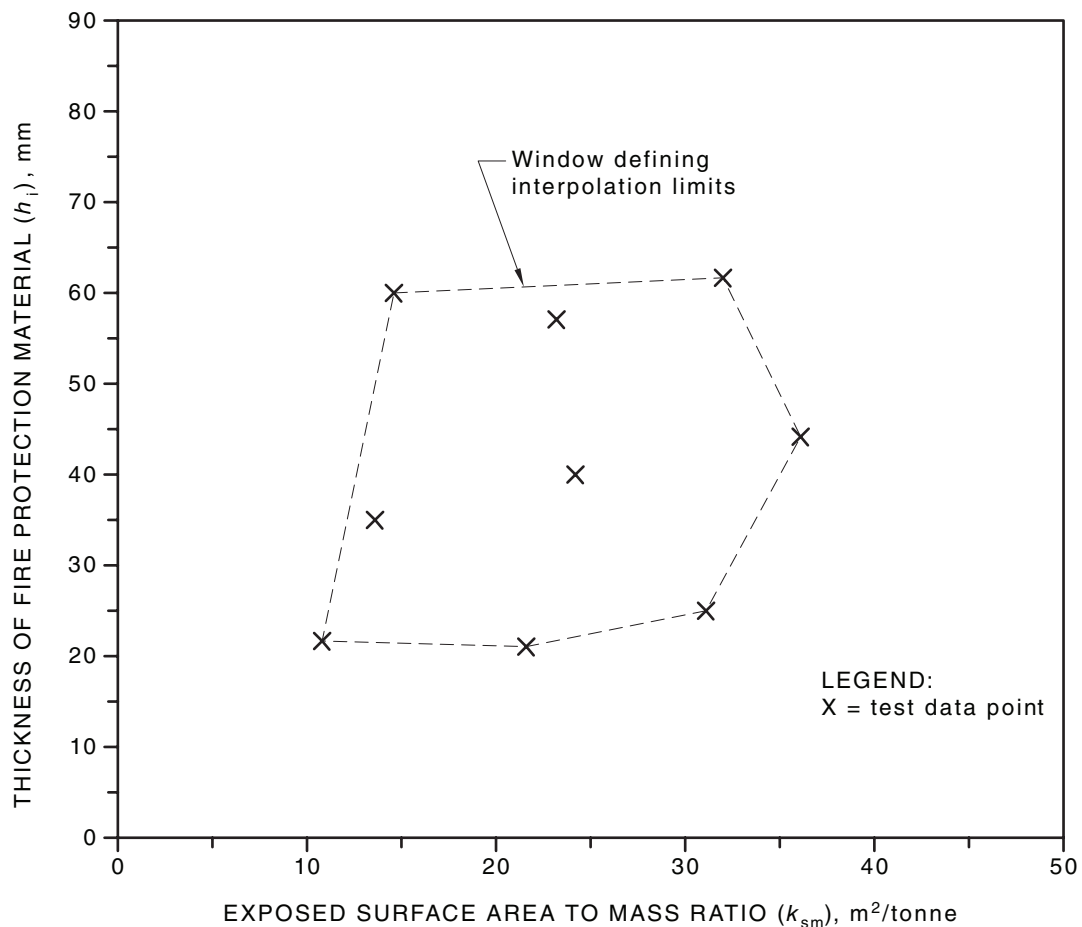


Figure 12.6.2.3 — Definition of window for interpolation limits

### 12.7 Determination of time at which limiting temperature is attained for unprotected members

The time ( $t$ ) at which the limiting temperature is attained shall be calculated for —

- (a) three-sided fire exposure condition as follows:

$$t = -5.2 + 0.0221T + \left( \frac{0.433T}{k_{sm}} \right)$$

- (b) four-sided fire exposure condition as follows:

$$t = -4.7 + 0.0263T + \left( \frac{0.213T}{k_{sm}} \right)$$

where

- $t$  = time from the start of the test, in minutes  
 $T$  = steel temperature, in degrees Celsius,  $500\text{ °C} \leq T \leq 750\text{ °C}$   
 $k_{sm}$  = exposed surface area to mass ratio,  $2\text{ m}^2/\text{tonne} \leq k_{sm} \leq 35\text{ m}^2/\text{tonne}$

For temperatures below 500 °C, linear interpolation shall be used based on the time at 500 °C and an initial temperature of 20 °C at  $t$  equals 0.

## 12.8 Determination of PSA from a single test

The PSA determined in accordance with AS 1530.4 from a single test may be applied without modification provided —

- (a) the fire protection system is the same as the prototype;
- (b) the fire exposure condition is the same as the prototype;
- (c) the fire protection material thickness is equal to or greater than that of the prototype;
- (d) the surface area to mass ratio is less than or equal to that of the prototype;
- (e) the conditions of support are the same as the prototype and the restraints are not less favourable than those of the prototype; and
- (f) the ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

## 12.9 Three-sided fire exposure condition

Members subject to a three-sided fire exposure condition shall be deemed to be in separate groups unless the following conditions are satisfied:

- (a) The characteristics of the members of a group shall not vary one from the other by more than —

- (i) concrete density:  $\left( \frac{\text{highest in group}}{\text{lowest in group}} \right) \leq 1.25$ ; and

- (ii) effective thickness ( $h_e$ ):  $\left( \frac{\text{largest in group}}{\text{smallest in group}} \right) \leq 1.25$

where the effective thickness ( $h_e$ ) is equal to the cross-sectional area excluding voids per unit width, as shown in [Figure 12.9\(a\)](#).

- (b) Rib voids shall be either —
  - (i) all open; or
  - (ii) all blocked as shown in [Figure 12.9\(b\)](#).

Concrete slabs may incorporate permanent steel deck formwork.



## 12.10 Special considerations

### 12.10.1 Connections

Connections shall be protected with the maximum thickness of fire protection material required for any of the members framing into the connection to achieve their respective fire-resistance levels. This thickness shall be maintained over all connection components, including bolt heads, welds and splice plates.

### 12.10.2 Web penetrations

The thickness of fire protection material at and adjacent to web penetrations shall be the greatest of that required for —

- the area above the penetration considered as a three-sided fire exposure condition ( $k_{sm1}$ ) (see [Figure 12.10.2](#));
- the area below the penetration considered as a four-sided fire exposure condition ( $k_{sm2}$ ) (see [Figure 12.10.2](#)); and
- the section as a whole considered as a three-sided fire exposure condition ( $k_{sm}$ ) (see [Figure 12.10.2](#)).

This thickness shall be applied over the full beam depth and shall extend each side of the penetration for a distance at least equal to the beam depth and not less than 300 mm.

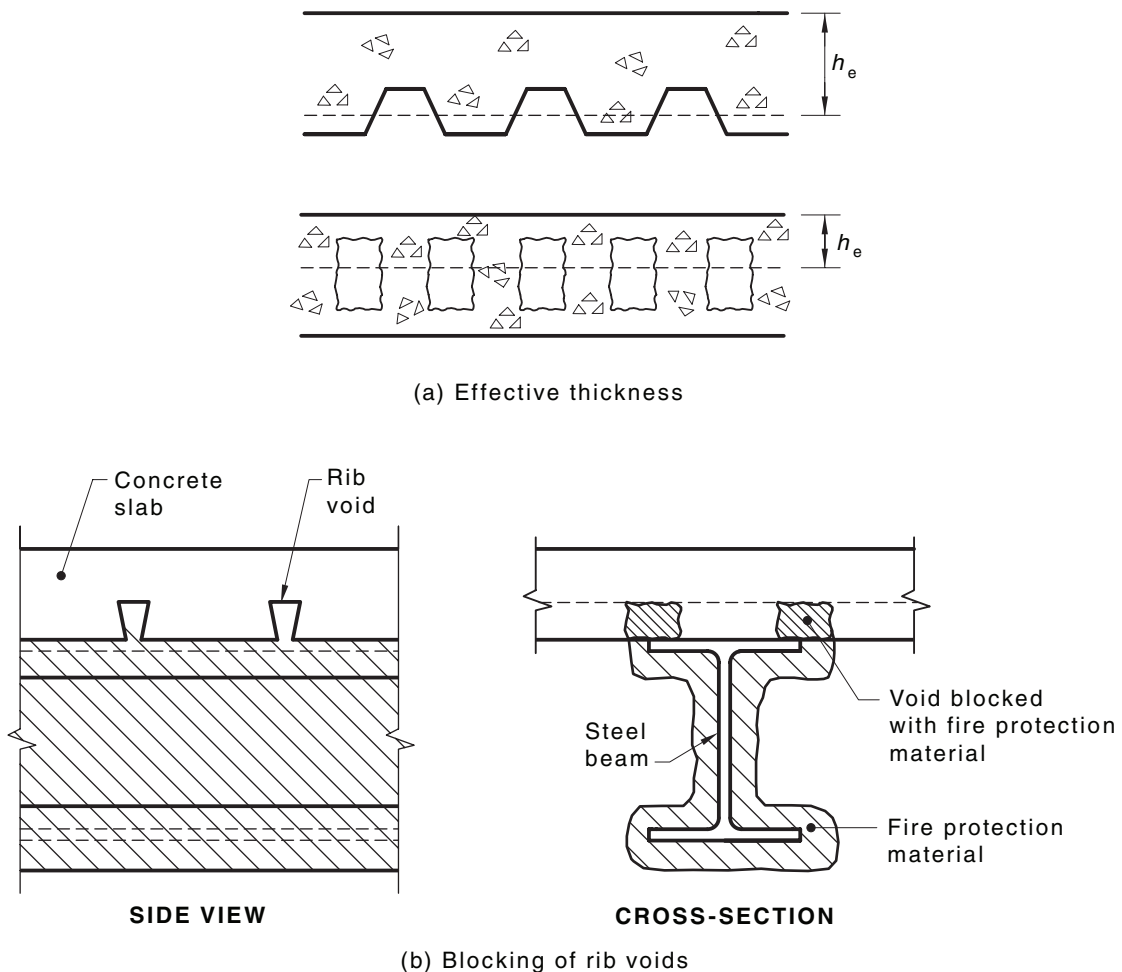
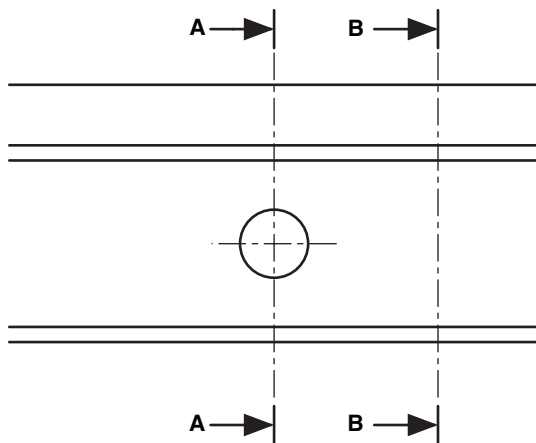


Figure 12.9 — Three-sided fire exposure condition requirements



Side view of beam with penetration

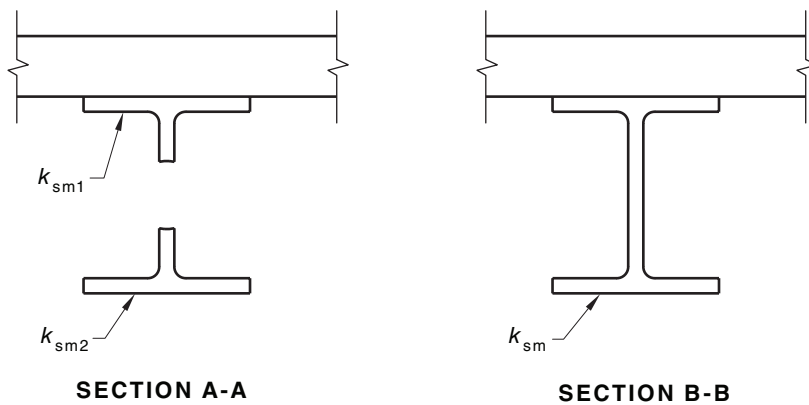


Figure 12.10.2 — Web penetrations

## Section 13 Earthquake

### 13.1 General

This Section sets out the additional minimum design and detailing requirements for steel structures, structural members, and connections which form the whole or parts of a building or structure subject to the earthquake forces specified in AS 1170.4.

### 13.2 Definitions

For the purposes of this Section, the definitions given in Clause 1.3 of AS 1170.4—2007 shall apply for the following terms:

*Bearing wall system*

*Braced frame*

*Braced frame, concentric*

*Braced frame, eccentric*

*Ductility (of a structure)*

*Moment-resisting frame*

*Moment-resisting frame, intermediate*

*Moment-resisting frame, ordinary*

*Moment-resisting frame, special*

*Seismic-force-resisting system*

*Space frame*

*Structural ductility factor*

*Structural performance factor*

### 13.3 Design and detailing requirements

#### 13.3.1 General

Design and detailing requirements for a structure shall be based on the earthquake design category and structural system assigned to the structure in accordance with AS 1170.4.

Limited ductile steel structures shall conform to [Clause 13.3.5](#).

Moderately ductile steel structures shall conform to [Clause 13.3.6](#).

Fully ductile steel structures shall conform to [Clause 13.3.7](#).

#### 13.3.2 Stiff elements

A stiff element that is deemed not to be part of the seismic-force-resisting system may be incorporated into a steel structure, provided its effects on the behaviour of the seismic-force-resisting system are considered and provided for in the analysis and design.

### 13.3.3 Non-structural elements

A non-structural element which is attached to or encloses the exterior of a steel structure shall be capable of accommodating the movements resulting from earthquake forces as follows:

- (a) All connections and panel joints shall permit relative movement between storeys equal to the design storey deflection calculated in accordance with AS 1170.4, or 6 mm, whichever is the greater.
- (b) Connections shall be ductile and shall have a rotation capacity to preclude brittle failure.
- (c) Connections which permit movements in the plane of a panel shall include sliding connections using slotted or oversize holes, or connection details which permit movement by bending, or other connection details which have been demonstrated by test to be adequate.

### 13.3.4 Structural ductility factor and structural performance factor

The structural ductility factor ( $\mu$ ) and structural performance factor ( $S_p$ ) for steel structures and members shall be as given in [Table 13.3.4](#).

**Table 13.3.4 — Structural ductility factor ( $\mu$ ) and structural performance factor ( $S_p$ ) — Steel structures**

Description of structural system	$\mu$	$S_p$
Special moment-resisting frames (fully ductile) — see Note	4	0.67
Intermediate moment-resisting frames (moderately ductile)	3	0.67
Ordinary moment-resisting frames (limited ductile)	2	0.77
Moderately ductile concentrically braced frames	3	0.67
Limited ductile concentrically braced frames	2	0.77
Fully ductile eccentrically braced frames — see Note	4	0.67
Other steel structures not defined above	2	0.77
NOTE The design of structures with $\mu > 3$ is outside the scope of this Standard (see <a href="#">Clause 13.3.7</a> ).		

### 13.3.5 Requirements for “limited ductile” steel structures ( $\mu = 2$ )

Limited ductile steel structures shall conform to the following requirements:

- (a) The minimum yield stress specified for the grade of steel shall not exceed 350 MPa.
- (b) *Concentrically braced frames* — Connections of diagonal brace members that are expected to yield shall be designed for the full member design capacity.
- (c) *Ordinary moment resisting frames* — No additional requirements.

### 13.3.6 Requirements for “moderately ductile” steel structures ( $\mu = 3$ )

#### 13.3.6.1 General

The minimum yield stress specified for the grade of steel shall not exceed 350 MPa.

### 13.3.6.2 Bearing wall and building frame systems

Concentrically braced frames in bearing wall and building frame systems shall conform to the following:

- (a) The design axial force for each diagonal tension brace member shall be limited to 0.85 times the design tensile capacity. Connections of each diagonal brace member shall be designed for the full member design capacity.
- (b) Any web stiffeners in beam-to-column connections shall extend over the full depth between flanges and shall be butt welded to both flanges.
- (c) All welds shall be weld category SP in accordance with AS/NZS 1554.1. Welds shall be subjected to non-destructive examination as given in [Table 13.3.6.2](#) and all such non-destructive examination shall be in accordance with AS/NZS 1554.1.

**Table 13.3.6.2 — Minimum requirements for non-destructive examination**

Weld type	Visual scanning %	Visual examination %	Magnetic particle or dye penetrant %	Ultrasonics or radiography %
Butt welds in members or connections in tension	100	100	100	10
Butt welds in members or connections others than those in tension	100	50	10	2
All other welds in members or connections	100	20	5	2

### 13.3.6.3 Moment-resisting frame, intermediate

Intermediate moment-resisting frames shall conform to the following additional requirements:

- (a) The minimum yield stress specified for the grade of steel shall not exceed 350 MPa.
- (b) Web stiffeners in beam to column connections shall extend over the full depth between flanges and shall be butt welded to both flanges.
- (c) Members in which plastic hinges will form during inelastic displacement of the frame shall conform to the requirements for plastic analysis specified in [Clause 4.5](#).

### 13.3.6.4 Fabrication in areas of plastic deformation

All areas of plastic deformation shall satisfy the following:

- (a) *Edge* — In parts of a member or connection subject to plastic deformation, a sheared edge shall not be permitted unless the edge is sheared oversize and machined to remove all signs of the sheared edge. A gas cut edge shall have a maximum surface roughness of 12  $\mu\text{m}$  (Centre Line Average Method).
- (b) *Punching* — In parts subject to plastic deformation, fastener holes shall not be punched full size. If punched, holes shall be punched undersize and reamed or drilled to remove the entire sheared surface.

### 13.3.7 Requirements for “fully ductile” structures ( $\mu > 3$ )

A steel structure that is fully ductile has a structural ductility factor  $> 3$  and is required by AS 1170.4 to be designed in accordance with NZS 1170.5. Steel members and connections for such structures shall be designed and detailed in accordance with NZS 3404.

## Section 14 Fabrication

### 14.1 General

A fabricated item shall be liable to rejection if —

- (a) the material does not satisfy the requirements of [Clause 14.2](#);
- (b) the fabrication does not satisfy the requirements of [Clause 14.3](#); or
- (c) it does not satisfy the tolerances specified in [Clause 14.4](#).

The fabricated item may be accepted nonetheless if —

- (i) it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby; or
- (ii) it passes testing in accordance with the appropriate Clauses of [Section 17](#).

Fabricated items which do not satisfy either (i) or (ii) above, and which do not satisfy [Clause 14.2](#), [Clause 14.3](#) or [Clause 14.4](#) shall be rejected.

### 14.2 Material

#### 14.2.1 General

All material shall satisfy the requirements of the appropriate material Standard specified in [Clauses 2.2](#), [2.3](#) and [2.4](#).

Surface defects in the steel shall be removed using the methods specified in the appropriate Standards listed in [Clause 2.2.1](#).

#### 14.2.2 Identification

The steel grade shall be identifiable at all stages of fabrication, or the steel shall be classed as unidentified steel and only used in accordance with [Clause 2.2.3](#). Any marking of steelwork shall be such as to not damage the material.

### 14.3 Fabrication procedures

#### 14.3.1 General

All material shall be fabricated to the requirements of AS/NZS 5131. Fabrication methods shall not reduce the properties of the material below the values used in design.

#### 14.3.2 Hole size

The nominal diameter of a completed hole, other than a hole in a base plate, shall be 2 mm larger than the nominal bolt diameter for a bolt not exceeding 24 mm in diameter, and not more than 3 mm larger for a bolt of greater diameter.

For a hole in a base plate, the hole diameter shall be not more than 6 mm greater than the anchor bolt diameter. A special plate washer of minimum thickness 4 mm shall be used under the nut if the hole diameter is 3 mm or more larger than the bolt diameter. The plate washer shall completely cover the hole such that the minimum distance from the edge of the hole to the edge of the plate washer shall be 0.5 times the hole diameter.

An oversize or slotted hole and the limitations on its use shall conform to the following:

- (a) *Oversize or slotted hole* — An oversize or slotted hole shall be permitted, provided that the following requirements are satisfied:
- (i) An oversize hole shall not exceed  $1.25d_f$  or  $(d_f + 8)$  mm in diameter, whichever is the greater, where  $d_f$  is the nominal bolt diameter, in millimetres.
  - (ii) A short slotted hole shall not exceed the appropriate hole size of this Clause in width and  $1.33 d_f$  or  $(d_f + 10)$  mm in length, whichever is the greater.
  - (iii) A long slotted hole shall not exceed the appropriate hole size of this Clause in width and  $2.5 d_f$  in length, where the length of the slotted hole is taken as the total length from one hole edge to another along the longest dimension.
- (b) *Limitations on use* — The use of an oversize or slotted hole shall be limited so that the following requirements are satisfied:
- (i) *Oversize hole* — An oversize hole may be used in any or all plies of bearing-type and friction-type connections, provided hardened or plate washers are installed over the oversize hole under both the bolt head and the nut. The plate washer shall completely cover the hole such that the minimum distance from the edge of the hole to the edge of the plate washer shall be 0.5 times the hole diameter.
  - (ii) *Short slotted hole* — A short slotted hole may be used in any or all plies of a friction-type or a bearing-type connection, provided hardened or plate washers are installed over the holes under both the bolt head and the nut. The plate washer shall completely cover the hole such that the minimum distance from the edge of the hole to the edge of the plate washer shall be 0.5 times the hole diameter.  
  
In a friction-type connection subject to a shear force, a short slotted hole may be used without regard to the direction of loading.  
  
In a bearing-type connection subject to a shear force, a short slotted hole may be used only where the connection is not eccentrically loaded and the bolt can bear uniformly, and where the slot is normal to the direction of the design action.
  - (iii) *Long slotted hole* — A long slotted hole may be used only in alternate plies of either a friction-type or bearing-type connection, provided a plate washer not less than 8 mm thick is used to completely cover any long slotted hole under both the bolt head and the nut. The plate washer shall completely cover the hole such that the minimum distance from the edge of the hole to the edge of the plate washer shall be 0.5 times the hole diameter.  
  
In a friction-type connection subject to a shear force, a long slotted hole may be used without regard to direction of loading.  
  
In a bearing-type connection subject to a shear force, a long slotted hole may be used only where the connection is not eccentrically loaded and where the bolt can bear uniformly, and where the slot is normal to the direction of the load.
- NOTE The plate washer is only required adjacent to the nut or bolt head that would otherwise be bearing against the ply (plate) containing the oversized hole.

Material for plate washers shall conform to AS/NZS 3678.

### 14.3.3 Bolting

#### 14.3.3.1 General

All bolts and associated nuts and washers shall conform to the appropriate bolt material Standard specified in [Clause 2.3.1](#). All material within the grip of the bolt shall be steel.

NOTE Conforming to this requirement ensures there is no compressible material within the grip of the bolt.

The length of a bolt shall be such that at least one clear thread shows above the nut and at least one thread plus the thread run out is clear beneath the nut after tightening.

One washer shall be provided under the rotated part.

Where the slope of the surfaces of parts in contact with the bolt head or nut exceeds 1:20 with respect to a plane normal to the bolt axis, a suitably tapered washer shall be provided against the tapered surface and the non-rotating part shall be placed against the tapered washer.

The nuts used in a connection subject to vibration shall be secured to prevent loosening. (See [Clause 9.1.6](#).)

#### 14.3.3.2 Friction-type connection

The contact surfaces of a joint using a tensioned bolt shall be prepared in accordance with AS/NZS 5131. Where the contact surfaces do not meet the requirements for a clean “as-rolled” surface or equivalent, refer to [Clause 9.2.3.2](#).

#### 14.3.3.3 Bearing-type connection

For a bearing-type connection, an applied finish on the contact surfaces shall be permitted.

#### 14.3.3.4 Bolt tensioning

When installed during fabrication a fully tensioned high strength bolt shall be installed in accordance with [Clause 15.2](#).

## 14.4 Geometrical tolerances

### 14.4.1 General

Tolerances on fabrication (i.e. permitted deviations) shall be in accordance with AS/NZS 5131. The tolerance limits shall be satisfied after fabrication is completed and any corrosion protection has been applied.

The following two types of permitted deviations are defined in AS/NZS 5131:

- (a) *Essential tolerances* — These comprise manufacturing, fabrication tolerances applicable to a range of criteria that are essential for the mechanical resistance and stability of the completed structure. In many cases these are compatible with assumptions made in formulating the expressions for design capacity in this Standard and AS/NZS 5100.6.
- (b) *Functional tolerances* — Required to fulfil other criteria, such as fit-up and appearance. Functional tolerances have two classes defined, Class 1 and Class 2, for which requirement strictness increases from Class 1 to Class 2. Where functional tolerances are not specified, Class 1 shall apply.

NOTE Tolerance measurements exclude coating thickness.



#### 14.4.2 Nonconformance of tolerances

For essential tolerances, the values specified are permissible deviations. If the actual deviation exceeds the permissible value, the actual deviation shall be corrected such that the actual deviation is within the permissible value.

The deviation of an essential tolerance above the specified value may be acceptable, when the excessive deviation is included explicitly in a revised calculation of design capacity using the relevant design provisions of this Standard.

For other than essential tolerances, the values specified are permissible deviations. If the actual deviation exceeds the permissible value, the actual deviation shall be corrected such that the actual deviation is within the permissible value

## Section 15 Erection

### 15.1 General

#### 15.1.1 Rejection of an erected item

An erected item shall be liable to rejection if —

- (a) the erection does not satisfy the requirements of [Clause 15.2](#); or
- (b) it does not satisfy the tolerances specified in [Clause 15.3](#).

The erected item may be accepted nonetheless if —

- (i) it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby; or
- (ii) it passes testing in accordance with the appropriate clauses of [Section 17](#).

Erected items which do not satisfy either (i) or (ii) above and which do not satisfy either [Clause 15.2](#) or [15.3](#) shall be rejected.

Bolts, nuts and washers shall be liable to rejection if, in the erected structure, they do not conform to [Clauses 14.3.3](#) and [15.2](#), unless it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby.

#### 15.1.2 Safety during erection

During the erection of a structure, steelwork shall be made safe against erection loading, including loading due to erection equipment or its operation, and wind.

### 15.2 Erection procedures

#### 15.2.1 General

The erection procedure and any modifications to the steelwork in the course of erection shall be in accordance with the requirements of AS/NZS 5131.

#### 15.2.2 Assembly of a connection involving bolts

##### 15.2.2.1 General

The assembly of connections involving bolts shall be in accordance with the requirements of AS/NZS 5131. Bolted connections shall be either —

- (a) snug tight; or
- (b) fully tensioned to the requirements of [Clause 15.2.2.2](#).

##### 15.2.2.2 Methods of tensioning

The method of tensioning shall be in accordance with either [Clause 15.2.2.3](#) or [Clause 15.2.2.4](#).

In the completed connection, all bolts shall have at least the minimum bolt tension specified in [Table 15.2.2.2](#) for the bolt grade specified when all bolts in the bolt group are tightened.

**Table 15.2.2.2 — Minimum bolt tension**

Nominal diameter of bolt	Minimum bolt tension, kN	
	Bolt grade	
	8.8	10.9
M16	95	130
M20	145	205
M24	210	295
M30	335	465
M36	490	680

NOTE The minimum bolt tensions given in this Table are approximately equivalent to the minimum proof loads derived from a proof load stress of 600 MPa for grade 8.8 bolts and 830 MPa for grade 10.9 bolts, as specified in AS 4291.1.

### 15.2.2.3 Part-turn method of tensioning

Tensioning of bolts by the part-turn method shall be in accordance with the requirements of AS/NZS 5131.

### 15.2.2.4 Tensioning by use of direct-tension indication device

Tensioning of bolts using a direct tension indication device shall be in accordance with the requirements of AS/NZS 5131.

## 15.3 Geometrical tolerances

### 15.3.1 General

Tolerances on erection (i.e. permitted deviations) shall be in accordance with AS/NZS 5131. The tolerance limits shall be satisfied after erection is completed.

The following two types of permitted deviations are defined in AS/NZS 5131:

- (a) *Essential tolerances* — Applicable to a range of criteria that are essential for the mechanical resistance and stability of the completed structure. In many cases these are compatible with assumptions made in formulating the expressions for design capacity in this Standard and AS/NZS 5100.6.
- (b) *Functional tolerances* — Required to fulfil other criteria, such as fit-up and appearance. Functional tolerances have two classes defined, Class 1 and Class 2, for which requirement strictness increases from Class 1 to Class 2. Where functional tolerances are not specified, Class 1 shall apply.

### 15.3.2 Nonconformance of tolerances

For essential tolerances, the values specified are permissible deviations. If the actual deviation exceeds the permissible value, the actual deviation shall be corrected such that the actual deviation is within the permissible value.

The deviation of an essential tolerance above the specified value may be acceptable, when the excessive deviation is included explicitly in a revised calculation of design capacity using the relevant design provisions of this Standard.

For other than essential tolerances, the values specified are permissible deviations. If the actual deviation exceeds the permissible value, the actual deviation shall be corrected such that the actual deviation is within the permissible value.

## Section 16 Modification of existing structures

### 16.1 General

All provisions of this Standard apply equally to the modification of existing structures or parts of a structure except as modified in this Section.

Site modifications during erection, and modification and repair of existing structures shall conform to the requirements of AS/NZS 5131.

### 16.2 Materials

The types of base metal involved shall be determined before preparing the drawings and specifications covering the strengthening of, the repair of, or the welding procedures for, an existing structure or parts of a structure.

## Section 17 Testing of structures or elements

### 17.1 General

#### 17.1.1 Scope of Section

The methods of test given in this Section are applicable to proof tests and prototype tests of complete structures, sub-structures, individual members or connections. The methods are not applicable to the testing of structural models, nor to the establishment of general design criteria or data.

#### 17.1.2 Circumstances requiring tests

Structures or parts of structures designed in accordance with this Standard are not required to be tested. Tests may be accepted as an alternative to calculation or may become necessary in special circumstances.

### 17.2 Definitions

For the purposes of this Section, the definitions below apply.

*Proof testing* — application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that one unit under test.

*Prototype testing* — application of test loads to one or more structures, sub-structures, members or connections to ascertain the structural characteristics of that class of structures, sub-structures, members or connections which are nominally identical to the units tested.

### 17.3 Test requirements

The test load shall be determined in accordance with [Clause 17.4.2](#) or [Clause 17.5.2](#), as appropriate.

Loading devices shall be calibrated, and care shall be exercised to ensure that no artificial restraints are applied by the loading systems. The test load shall be applied to the unit at a rate as uniform as practicable. The distribution and duration of forces applied in the test shall represent those forces to which the structure is deemed to be subjected under the requirements of [Section 3](#).

Deformations shall, as a minimum requirement, be recorded at the following times:

- (a) Prior to the application of the test load.
- (b) After the test load has been applied.
- (c) After the removal of the test load.

### 17.4 Proof testing

#### 17.4.1 Application

This Clause applies to the testing of a structure, sub-structure, member or connection to determine whether that particular structure, sub-structure, member or connection complies with the requirements for the strength or serviceability limit state, as appropriate.

#### 17.4.2 Test load

The test load shall be equal to the design load for the relevant limit state as determined from [Clause 3.2.3](#).

### 17.4.3 Criteria for acceptance

Criteria for acceptance shall be as follows:

- (a) *Acceptance for strength* — The test structure, sub-structure, member or connection shall be deemed to conform to the requirements for strength if it is able to sustain the strength limit state test load for at least 15 min. It shall then be inspected to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be reviewed and, if necessary, appropriate repairs to the damaged parts carried out.
- (b) *Acceptance for serviceability* — The maximum deformation of the structure or member under the serviceability limit state test load shall be within the serviceability limits appropriate to the structure.

## 17.5 Prototype testing

### 17.5.1 Test specimen

The materials and fabrication of the prototype shall conform to [Sections 2](#) and [14](#), respectively. Any additional requirements of a manufacturing specification shall be conformed to and the method of erection used shall simulate that which will be used in production.

### 17.5.2 Test load

The test load shall be equal to the design load for the relevant limit state determined in accordance with [Clause 3.2.3](#), multiplied by the appropriate factor given in [Table 17.5.2](#), unless a reliability analysis shows that a smaller value can be adopted.

### 17.5.3 Criteria for acceptance

Criteria for acceptance shall be as follows:

- (a) *Acceptance for strength* — The test unit shall be deemed to conform to the requirements for strength if it is able to sustain the strength limit state test load for at least 5 min.
- (b) *Acceptance for serviceability* — The maximum deformation of the unit under the serviceability limit state test load shall be within the serviceability limits appropriate to the structure.

### 17.5.4 Acceptance of production units

Production-run units shall be similar in all respects to the unit or units tested.

**Table 17.5.2 — Factors to allow for variability of structural units**

Number of similar units to be tested	Strength limit state	Serviceability limit state
1	1.5	1.2
2	1.4	1.2
3	1.3	1.2
4	1.3	1.1
5	1.3	1.1
10	1.2	1.1

## 17.6 Report of tests

The report of the test on each unit 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 report shall also contain a statement as to whether or not the structure or part tested satisfies the acceptance criteria.

## **Appendix A (normative)**

### **Not used**

This Appendix has been left blank intentionally.



## Appendix B (informative)

### Suggested deflection limits

#### B.1 Suggested vertical deflection limits for beams

The vertical deflection of beams may be controlled using the suggested limits given in [Table B.1](#). Alternatively, the guidance given in Appendix C of AS/NZS 1170.0:2002 may be used, where appropriate.

**Table B.1 — Suggested limits on calculated vertical deflections of beams**

Type of beam	Deflection to be considered	Deflection limit ( $\Delta$ ) for span ( $l$ ) (see Note 1)	Deflection limit ( $\Delta$ ) for cantilever ( $l$ ) (see Note 2)
Beam supporting masonry partitions	The deflection which occurs after the addition or attachment of partitions	$\frac{\Delta}{l} \leq \frac{1}{500}$ where provision is made to minimize the effect of movement, otherwise	$\frac{\Delta}{l} \leq \frac{1}{250}$ where provision is made to minimize the effect of movement, otherwise
		$\frac{\Delta}{l} \leq \frac{1}{1000}$	$\frac{\Delta}{l} \leq \frac{1}{500}$
All beams	The total deflection	$\frac{\Delta}{l} \leq \frac{1}{250}$	$\frac{\Delta}{l} \leq \frac{1}{125}$
NOTE 1 Suggested deflection limits in this Table may not safeguard against ponding.			
NOTE 2 For cantilevers, the values of $\Delta/l$ given in this Table apply, provided that the effect of the rotation at the support is included in the calculation of $\Delta$ .			

#### B.2 Suggested horizontal deflection limits

The relative horizontal deflection between adjacent frames at eaves level of industrial portal frame buildings under the serviceability wind load specified in AS/NZS 1170.0 and AS/NZS 1170.2 may be limited to the following:

- (a) Building clad with steel or aluminium sheeting, with no ceilings, with no internal partitions against external walls and no gantry cranes operating in the building — Frame spacing/200.
- (b) As in (a) but with gantry cranes operating — Frame spacing/250.
- (c) As in (a) but with external masonry walls supported by steelwork in lieu of steel or aluminium sheeting — Frame spacing/200.

The absolute horizontal deflection of a frame in an industrial portal frame building under the serviceability wind load specified in AS/NZS 1170.0 and AS/NZS 1170.2 may be limited to the following:

- (i) Building clad with steel or aluminium sheeting, with no ceilings, with no internal partitions against external walls and no gantry cranes operating in the building — Eaves height/150.
- (ii) As in (i) but with gantry cranes operating — Crane rail height/250.
- (iii) As in (i) but with external masonry walls supported by steelwork in lieu of steel or aluminium sheeting — Eaves height/250.

Alternatively, the guidance given in Appendix C of AS/NZS 1170.0:2002 may be used where appropriate.

## Appendix C (informative)

### Selection of corrosion protection requirements

#### C.1 Scope

This Appendix applies to the corrosion protection of steel members and connection components.

#### C.2 Selection of an appropriate corrosion protection system

The selection of an appropriate corrosion protection system should take into account:

- (a) The exposure environment.
- (b) The shape and detailing of the structure.
- (c) The required coating life to first maintenance or design life, as appropriate.
- (d) The cost-benefit of the available coating systems over the lifetime of the structure.

AS 2312.1 provides guidance on selection of an appropriate paint system to meet these requirements. AS/NZS 2312.2 provides guidance on protection utilizing hot dip galvanizing.

NOTE Guidance for applying AS 2312.1 and AS/NZS 2312.2 can be found in: Australian Steel Institute (ASI), ASI Number 870, *Australian steelwork corrosion and coatings guide*.

#### C.3 Classification of environments

The corrosion of steel and the life of a coating system are influenced by both the macroclimatic environment and also by micro-environments produced in localized areas or due to particular structure detailing.

Based on ISO 9223, AS 4312 provides a classification of the atmospheric corrosivity categories (macroclimatic environment) into categories C1 to CX in order of increasing corrosion rate.

AS 2312.1 and AS/NZS 2312.2 provide for durability of paint systems and galvanizing types respectively, based on these corrosivity categories.

Guidance on the influence of micro-environmental factors is provided in AS 4312, AS 2312.1 and AS/NZS 2312.2.

#### C.4 Protection systems

AS 2312.1 provides guidance on selection of an appropriate paint system. The paint systems are described by a standardized “system designation”, which includes the requirements for surface preparation. The durability, expressed as years to first maintenance, is defined for each system designation and corrosion category.

AS/NZS 2312.2 provides guidance on corrosion protection and design utilizing hot dip galvanizing. The durability class and expected life are defined for each corrosion category and galvanizing type.

## C.5 Application

The preparation of steel surfaces for the subsequent application of corrosion protection, and the application of corrosion protection systems, including painting and galvanizing, is defined in AS/NZS 5131.

## Appendix D (normative)

### Advanced structural analysis

#### D.1 General

For a frame comprising members of compact section (see [Clause 5.2.3](#)) with full lateral restraint (see [Clauses 5.3](#) and [5.4](#)), an advanced structural analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frame.

The analysis shall take into account the relevant material properties, residual stresses, geometrical imperfections, second-order effects, erection procedures and interaction with the foundations.

An advanced structural analysis for earthquake loads shall take the following into account, where appropriate:

- (a) Torsional response.
- (b) Pounding against adjacent structures.
- (c) Strain rate effects.

#### D.2 Design

For the strength limit state, it shall be sufficient to satisfy the section capacity requirements of [Clause 8.3](#) for the members and the requirements of [Section 9](#) for the connections.

An advanced structural analysis for earthquake loads shall take into account that the earthquake loads calculated in accordance with AS 1170.4 correspond to the load at which the first significant plastic hinge forms in the structure.

## Appendix E (normative)

### Second order elastic analysis

#### E.1 Analysis

In a second-order elastic analysis, the members shall be assumed to remain elastic, and changes in frame geometry under the design load and changes in the effective stiffnesses of the members due to axial forces shall be accounted for, except that for a frame where the elastic buckling load factor ( $\lambda_c$ ) of the frame as determined in accordance with [Clause 4.7](#) is greater than 5, the changes in the effective stiffnesses of the members due to axial forces may be neglected.

#### E.2 Design bending moment

The design bending moment ( $M^*$ ) shall be taken as the maximum bending moment in the length of the member. It shall be determined either —

- (a) directly from the second-order analysis;
- (b) approximately, if the member is divided into a sufficient number of elements, as the greatest element end bending moment; or
- (c) by amplifying the maximum calculated design bending moment ( $M_m^*$ ) taken as the maximum bending moment along the length of a member and obtained by superposition of the simple beam bending moments resulting from any transverse loading on the member with the second-order end bending moments ( $M_e^*$ ) determined by the analysis.

For a member with zero axial force or a member subject to axial tension, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = M_m^*$$

For a member with a design axial compressive force ( $N^*$ ) as determined from the analysis, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where  $\delta_b$  is the moment amplification factor for a braced member determined in accordance with [Clause 4.4.2.2](#).

## Appendix F (normative)

### Moment amplification for a sway member

For a sway member which forms part of a rectangular frame, the design end bending moments ( $M_f^*$ ), obtained from a first-order elastic analysis in which relative lateral displacements of the ends of members are not prevented shall be separated into two components  $M_{fb}^*$  and  $M_{fs}^*$ ,

where

$M_{fb}^*$  = design end bending moment obtained from a first-order elastic analysis of the frame with sway prevented (i.e. a braced frame)

$M_{fs}^*$  =  $M_f^* - M_{fb}^*$

For a frame where gravity design loads do not cause sway, it shall be permissible to calculate  $M_{fb}^*$  from the gravity design loads acting alone on the frame and  $M_{fs}^*$  from the transverse design loads acting alone.

The amplified end bending moments ( $M_e^*$ ) on a sway member shall be calculated as follows:

$$M_e^* = M_{fb}^* + \delta_s M_{fs}^*$$

where  $\delta_s$  is the moment amplification factor for a sway member (see [Clause 4.4.2.3](#)).

The maximum calculated design bending moment ( $M_m^*$ ) shall be taken as the maximum bending moment along the length of the sway member obtained by superposition of the simple beam bending moments resulting from any transverse loading on the member with the amplified end bending moments ( $M_e^*$ ).

For a sway member with zero axial force or a member subject to axial tension, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = M_m^*$$

For a sway member with a design axial compressive force ( $N^*$ ) as determined from the analysis, the design bending moment ( $M^*$ ) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where  $\delta_b$  is the moment amplification factor for a braced member (see [Clause 4.4.2.2](#)).

## Appendix G (normative)

### Braced member buckling in frames

The member elastic flexural buckling load ( $N_{om}$ ) of a braced compression member in a frame shall be determined as follows:

$$N_{om} = \frac{\pi^2 EI}{(k_e l)^2}$$

where  $k_e$  is the member effective length factor obtained from [Figure 4.6.3.3\(a\)](#) and the values of  $\gamma_1$  and  $\gamma_2$  for each restrained end of the compression member under consideration shall be calculated as the ratio of the stiffness of that member to the total stiffness of the braced members restraining that end as follows:

$$\gamma = \frac{\left(\frac{I}{l}\right)_m}{\sum \beta_e \alpha_{sr} \left(\frac{I}{l}\right)_r}$$

$(I/l)_m$  = stiffness in the plane of bending of the compression member under consideration

$\sum \beta_e \alpha_{sr} (I/l)_r$  = summation of the stiffnesses in the plane of bending of all the braced restraining members rigidly connected at that end to the member under consideration (except the member itself)

$\beta_e$  = modifying factor given in [Table 4.6.3.4](#) to account for the end conditions at the far end of the braced restraining member

$\alpha_{sr}$  = theoretical stability function multiplier, or the approximation shown in [Figure G.1](#) to account for the effect of the design axial force ( $N_r^*$ ) in the braced restraining member on its flexural stiffness

In [Figure G.1](#) the value of  $\rho$  is calculated as follows:

$$\rho = \frac{N_r^*}{N_{olr}}$$

where

$$N_{olr} = \frac{\pi^2 EI_r}{l_r^2}$$

For a braced restraining member in tension,  $\alpha_{sr}$  may conservatively be taken as 1.0. Where a braced restraining member is connected by a detail with negligible moment transmitting capacity, the contribution of that member to the total stiffness shall be taken as zero.



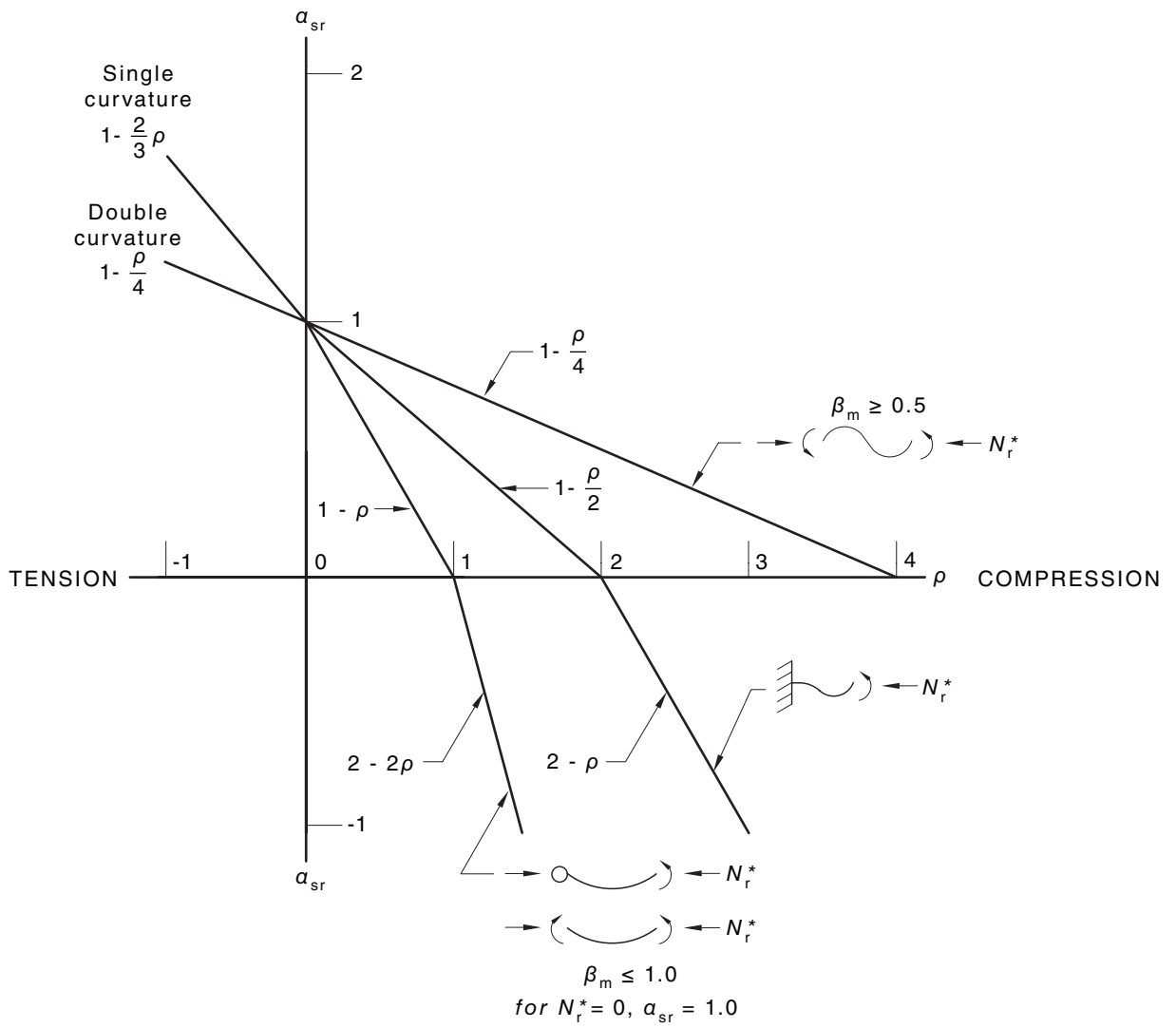


Figure G.1 — Stability function multipliers

## Appendix H (informative)

### Elastic resistance to lateral buckling

#### H.1 General

The elastic resistance of a beam to lateral buckling is influenced by many factors, including the beam geometry, the distribution of the loading on it, and the effects of end and intermediate restraints. Because of this, simple design rules can be formulated only for a limited number of situations. Such a set of simple rules is included in [Clauses 5.6.1](#), [5.6.2](#) and [5.6.3](#).

While these rules are generally on the safe side, there are many situations where they are overly conservative. When it is desirable to avoid undue conservatism, then [Clause 5.6.4](#) may be used, which requires the use of the results of an elastic flexural-torsional buckling analysis. This may be carried out by using computer programs such as those described in References 1 and 2 (see [Clause H.6](#) for details of references).

Alternatively, the published results of elastic flexural-torsional buckling analyses may be used. There are very many such publications, either in textbooks and surveys such as those listed in References 3 to 7, or in research publications such as References 8 to 10.

However, it is often the case that suitable computer programs are not available, and that the designer is daunted by the complexity and scope of the research publications. In this case, it is desirable that there should be a second level of approximation, more general and more accurate than the provisions of [Clauses 5.6.1](#), [5.6.2](#) and [5.6.3](#). Such a set of approximations is given in [Clauses H.2](#), [H.3](#) and [H.5](#). They may be used in conjunction with the method of design by buckling analysis of [Clause 5.6.4](#).

#### H.2 Segments restrained at both ends

The effects of geometry and loading distribution on the elastic flexural-torsional buckling of a uniform equal flanged segment restrained at both ends may be estimated by calculating approximately the maximum bending moment ( $M_{ob}$ ) in the segment at elastic buckling as follows:

$$M_{ob} = \alpha_m \alpha_l M_o \quad \text{H.2(1)}$$

where

$\alpha_m$  is given in [Clause 5.6.1\(a\)](#), or may be approximated in accordance with [Clause 5.6.4](#)

$$\alpha_l = \sqrt{\left[ 1 + \left[ \frac{0.4\alpha_m y_L}{M_o} \left( \frac{\pi^2 EI_y}{l^2} \right) \right]^2 \right]} + \left[ \frac{0.4\alpha_m y_L}{M_o} \left( \frac{\pi^2 EI_y}{l^2} \right) \right] \quad \text{H.2(2)}$$

$M_o$  is given by [Equation H.4\(2\)](#)

$y_L$  is the distance of the gravity loading below the centroid (and is positive when the load acts below the centroid)

Alternatively, for uniform equal flanged segments loaded so that  $-d_o/2 \leq y_L \leq d_o/2$ , where  $d_o$  is the overall section depth of the segment, the amended elastic buckling moment ( $M_{oa}$ ) used in [Equation 5.6.1.1\(2\)](#) may be taken as —

$$M_{oa} = M_o + \left[ 0.4\alpha_m y_L \left( \frac{\pi^2 E I_y}{l^2} \right) \right] \quad \text{H.2(3)}$$

### H.3 Segments unrestrained at one end

The effects of geometry and loading distribution on the elastic flexural-torsional buckling of a uniform equal flanged segment unrestrained at one end and both fully or partially restrained and laterally continuous or restrained against lateral rotation at the other end may be estimated by calculating the maximum bending moment ( $M_{ob}$ ) in the segment at elastic buckling as follows:

$$M_{ob} = \alpha_{mc} \alpha_{lc} M_o \quad \text{H.3(1)}$$

where

$$\alpha_{mc} = \frac{(C_3 + C_4 K)}{\pi \sqrt{(1 + K^2)}} \quad \text{H.3(2)}$$

$$\alpha_{lc} = 1 + \frac{\left( \frac{2y_L K}{d_f 2} \right)}{\sqrt{\left[ 1 + \left( \frac{2y_L K}{d_f 2} \right)^2 \right]}} \quad \text{H.3(3)}$$

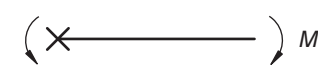
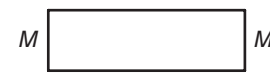
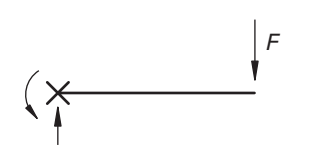
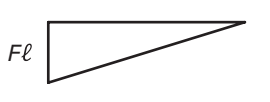
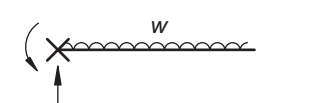
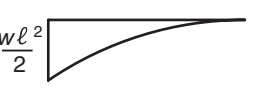
$M_o$  is given by [Equation H.4\(2\)](#)

$K$  is given by [Equation H.4\(3\)](#)

$C_3, C_4$  are given in [Table H.3](#)

The elastic flexural-torsional buckling of a uniform equal flanged segment, unrestrained at one end, and both fully or partially restrained and unrestrained against lateral rotation at the other end, may be estimated by calculating the maximum bending moment ( $M_{ob}$ ) in the segment at elastic buckling by using  $C_4 = 0$  in [Equation H.3\(2\)](#).

**Table H.3 — Factors ( $C_3$ ) and ( $C_4$ ) for beams unrestrained at one end**

Beam segment	Moment distribution	Factor, $C_3$	Factor, $C_4$
		1.6	0.8
		4.0	3.7
		7.0	8.0

KEY:  
 X = Full or partial restraint

### H.4 Reference elastic buckling moment

The elastic buckling moment ( $M_o$ ) of a simply supported segment in uniform bending may be used as a reference moment.

This moment is calculated as follows:

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{l^2}\right)} \left\{ \sqrt{\left[ GJ + \frac{\pi^2 EI_w}{l^2} + \left(\frac{\beta_x^2}{4} \frac{\pi^2 EI_y}{l^2}\right) \right]} + \frac{\beta_x}{2} \sqrt{\left(\frac{\pi^2 EI_y}{l^2}\right)} \right\} \tag{H.4(1)}$$

For sections bent about an axis of symmetry,  $\beta_x = 0$ , and [Equation H.4\(1\)](#) simplifies to —

$$M_o = \left(\frac{\pi \sqrt{(EI_y GJ)}}{l}\right) \sqrt{(1 + K^2)} \tag{H.4(2)}$$

where

$$K = \sqrt{\left(\frac{\pi^2 EI_w}{GJl^2}\right)} \tag{H.4(3)}$$

$$E = 200\,000 \text{ MPa}$$

$$G \approx 80\,000 \text{ MPa}$$

$$I_w = \frac{I_y (d_f)^2}{4} \tag{for doubly-symmetric I-section,}$$

$$= I_{cy} d_f^2 \left(1 - \frac{I_{cy}}{I_y}\right) \tag{for a monosymmetric I-section}$$

$$\begin{aligned}
 &= \frac{b_f^3 t_f b_w^2}{48} \left( 8 - \frac{3 b_f t_f b_w^2}{I_x} \right) && \text{for a thin-walled channel section,} \\
 &= 0 && \text{for an angle section, a tee-section, or a narrow} \\
 & && \text{rectangular section, and may be taken as 0 for a} \\
 & && \text{hollow section} \\
 J &\approx \sum \left( \frac{b t^3}{3} \right) && \text{for an open section} \\
 &\approx \frac{1}{I_x} \int (x^2 y + y^3) dA - 2y_o && \text{for a hollow section where } A_e \text{ is the area enclosed} \\
 & && \text{by the hollow section,} \\
 \beta_x &= \frac{1}{I_x} \int (x^2 y + y^3) dA - 2y_o \\
 &\approx 0.8 d_f \left( \frac{2 I_{cy}}{I_y} - 1 \right) && \text{for a monosymmetric I-section}
 \end{aligned}$$

Expressions for the properties of other thin-walled sections are given in Reference 11, while more accurate approximations for  $J$  are given in Reference 12.

## H.5 Effects of end restraints

### H.5.1 Torsional end restraints

The approximations given in [Clauses H.2](#) and [H.3](#) for the elastic buckling moments are for segments which are rigidly restrained torsionally at their supports against twist rotations. When the torsional end restraints are elastic, the buckling twists increase, and the resistance to buckling decreases. The decreased resistance ( $M_{obr}$ ) may be approximated as follows:

$$M_{obr} = M_{ob} \sqrt{\left[ \frac{2\beta_t}{(1 + \beta_t)} \right]} \leq M_{ob}$$

in which  $\beta_t$  depends on the elastic stiffness ( $\alpha_{rz}$ ) of the torsional end restraint (i.e. the ratio of the restraining torque supplied to the twist rotation).

For segments restrained at both ends —

$$\beta_t \approx \frac{\alpha_{rz} l / GJ}{5(1 + K^2)}$$

For segments unrestrained at one end and both fully or partially restrained and laterally continuous or restrained against lateral rotation at the other end —

$$\beta_t \approx \frac{\alpha_{rz} l / GJ}{25(1 + 2K^2) / (1 + K^2)}$$

and for segments unrestrained at one end and both fully or partially restrained and unrestrained against lateral rotation at the other end —

$$\beta_t \approx \frac{\alpha_{rz} l / Gj}{5(1 + 2K^2) / (1 + K^2)}$$

## H.5.2 End restraints against lateral rotation

### H.5.2.1 Segments restrained at both ends

Continuity of a segment with adjacent segments may introduce restraining moments which reduce the lateral rotations and increase the elastic buckling moment. The restraint effects depend on the relative minor axis flexural stiffnesses of the adjacent segments, and these depend in turn on the moment distributions in these segments. The restraining effects may be calculated approximately by using the method referred to in References 6 and 9 to calculate the effective length ( $l_e$ ), and by using  $l_e$  instead of the segment length ( $l$ ) in [Equations H.4\(2\)](#) and [H.4\(3\)](#).

### H.5.2.2 Segments unrestrained at one end

The approximate elastic buckling moments obtained by using the values of  $C_3$  and  $C_4$  of [Table H.3](#) in [Equations H.4\(1\)](#) to [H.4\(3\)](#) are for segments which are prevented from rotating laterally at their restrained ends. For segments which are unrestrained against lateral rotation at their restrained ends, the values of  $C_4$  used in [Equation H.3\(2\)](#) should be reduced to zero. For a segment with an elastic restraint against lateral rotation at its restrained end, a reduced value  $C_{4r}$  should be used which may be approximated as follows:

$$\frac{C_{4r}}{C_4} = \frac{1.5\alpha_{ry} l / EI_y}{5 + (\alpha_{ry} l / EI_y)} \leq 1.0$$

in which  $\alpha_{ry}$  is the elastic stiffness of the flexural end restraint (i.e. the ratio of the restraining minor axis moment supplied to the end lateral rotation).

## H.6 References

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- 9 NETHERCOT, D.A., and TRAHAIR, N.S. Lateral buckling approximations for elastic beams. *The Structural Engineer*, Vol. 54, No. 6, June 1976, pp. 197–204.
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## Appendix I (informative)

### Strength of stiffened web panels under combined actions

#### I.1 Yielding check

The design bending, shear, axial and bearing actions (or reactions) ( $M_w^*$ ), ( $V_w^*$ ), ( $N_w^*$ ) and ( $R_w^*$ ) on a web panel (see [Figure I.1](#)) should satisfy the yielding criterion —

$$\left( \frac{R_w^*}{\phi b_{bf} t_w} \right)^2 - \frac{f_w^*}{\phi} \left( \frac{R_w^*}{\phi b_{bf} t_w} \right) + \left( \frac{f_w^*}{\phi} \right)^2 + \left( \frac{V_w^*}{0.6 \phi A_w} \right)^2 \leq (f_y)^2$$

where

$$f_w^* = \frac{N_w^*}{A_w} + \left( 0.77 \frac{M_w^*}{Z_{we}} \right)$$

$b_{bf}$  = width of the bearing load on the edge of the web dispersed at 2.5:1 through the flange as shown in [Figure 5.13.1.1](#)

$M_w^*$  = design bending moment in the web, calculated by elastic theory for sections with non-compact or slender flanges (see [Clause 5.2](#)), or by plastic theory for sections with compact flanges (see [Clause 5.2](#))

$Z_{we}$  = elastic section modulus of the web panel

$$= \frac{t_w (d_p)^2}{6}$$



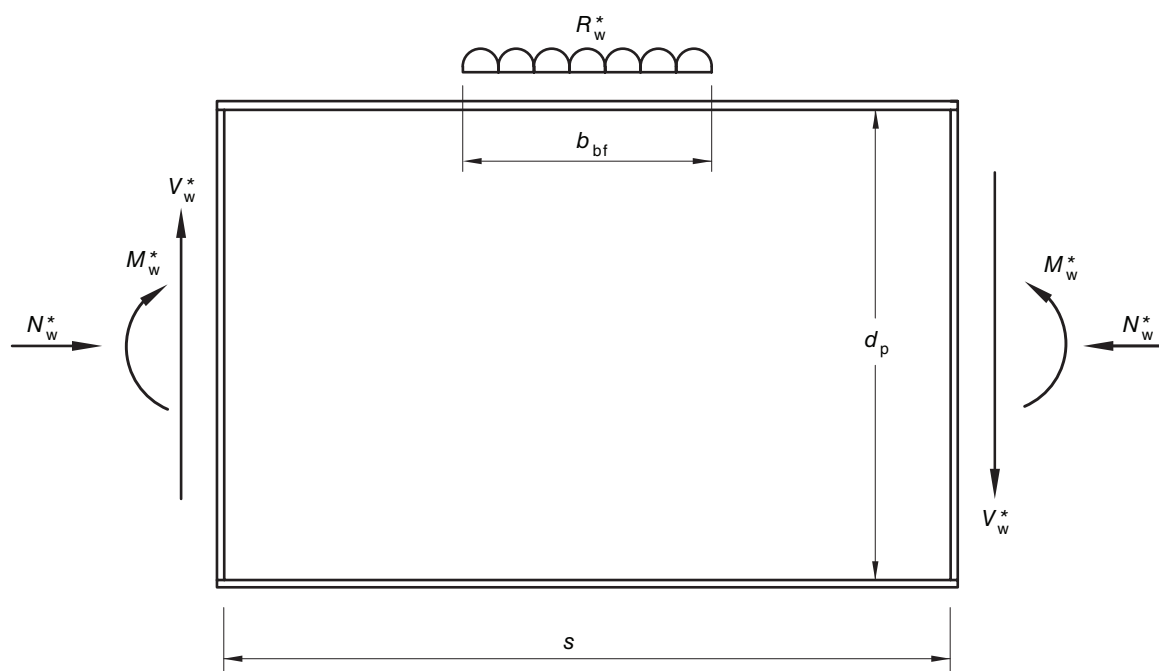


Figure I.1 — Actions on web panel

## I.2 Buckling check

The design bending, shear, axial and bearing actions (or reactions) ( $M_w^*$ ), ( $V_w^*$ ), ( $N_w^*$ ) ( $R_w^*$ ) on a web panel should satisfy the buckling criterion —

$$\left( \frac{R_w^*}{\phi R_{sb}} \right) + \left( \frac{N_w^*}{\phi N_{w0}} \right) + \left( \frac{V_w^*}{\phi V_v} \right)^2 + \left( \frac{M_w^*}{\phi M_w} \right)^2 \leq 1$$

where

$N_{w0}$  = nominal axial load capacity of the web panel if the web panel resisted axial load alone

$$= \frac{45A_w f_y}{\left( \frac{d_p}{t_w} \right) \sqrt{\left( \frac{f_y}{250} \right)}} \leq A_w f_y$$

$V_v$  = nominal shear capacity of the web panel if the web panel resisted shear alone, as specified in [Clause 5.11](#)

$M_w$  = nominal section moment capacity of the web if the web resisted bending alone, as specified in [Clause 5.2](#)

$R_{sb}$  = nominal buckling capacity of a transversely stiffened web in bearing alone

$$= \beta_w b_{bf} t_w f_y$$

$$\beta_w = \frac{0.10 + \frac{20}{\left(\frac{d_e}{t_w}\right) \sqrt{\left(\frac{f_y}{250}\right)}}}{\alpha_w}$$

$$d_e = \frac{1.9 \sqrt{(b_{bf} d_p)}}{\alpha_w}$$

$$\alpha_w = \left[ 3.4 + \left( \frac{2.2 d_p}{s} \right) \right] \left[ 0.4 + \left( \frac{0.5 b_{bf}}{s} \right) \right]$$

## Appendix J (normative)

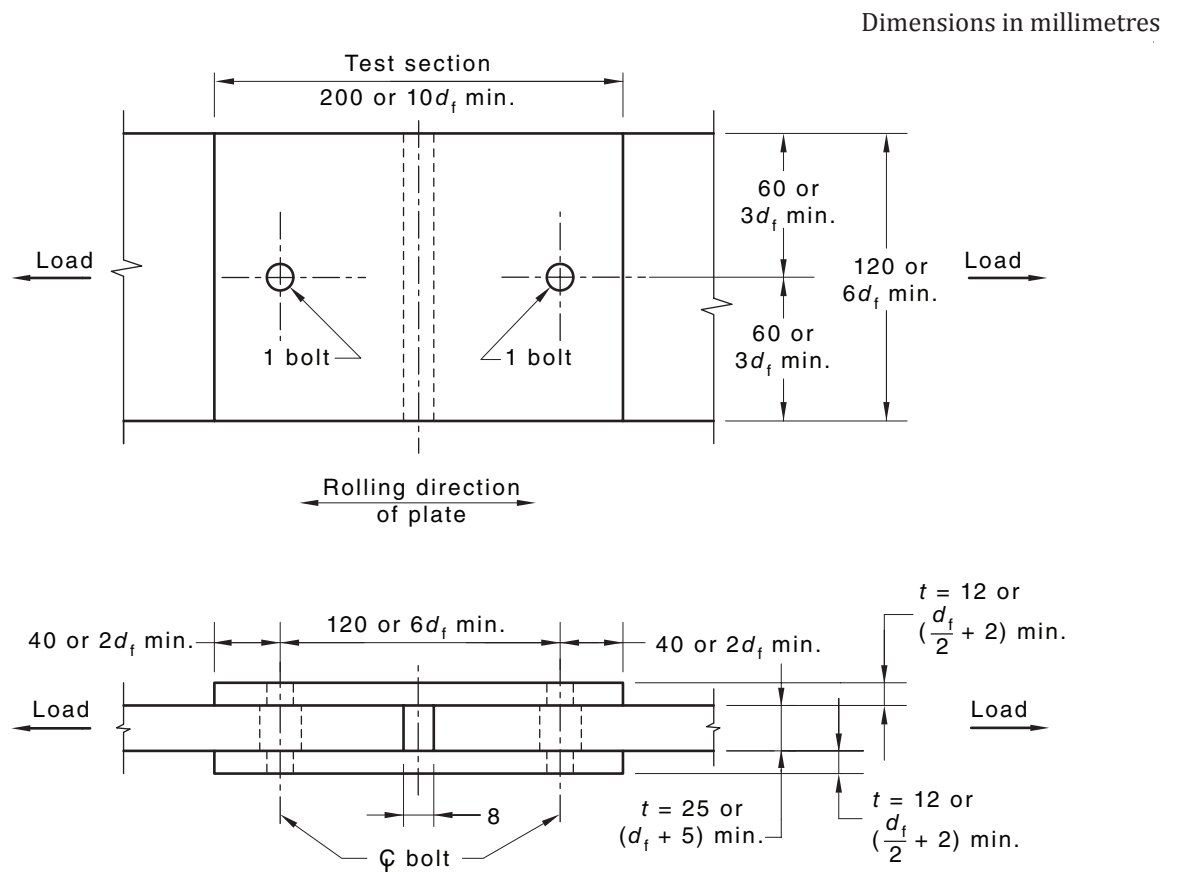
### Standard test for evaluation of slip factor

#### J.1 Test specimens

##### J.1.1 Form

The standard test specimen shall be a symmetrical double cover-plated butt connection as shown in [Figure J.1](#). The inner plates shall be equal in thickness.

NOTE It is suggested that the use of M20 bolts will prove to be most convenient, with 25 mm inner plates and 12 mm outer plates.



#### Key

$d_f$  = diameter of the bolt

$t$  = thickness of the plate

NOTE 1 Holes in plates: (a) Cover plates 22 mm or  $d_f + 2$  mm (b) Inner plates 23 mm or  $d_f + 3$  mm

NOTE 2 Length and width of inner plates outside of test section may be increased to suit the laboratory's testing facilities.

NOTE 3 Dimensions are shown for the use of M20 bolts. Dimensions in parentheses are for use of bolts with nominal diameter  $d_f$  mm, which should not be less than 16 mm.

## Figure J.1 — Standard test specimen

### J.1.2 Assembly and measurement

Care shall be taken in assembling the specimen to ensure that neither bolt is in bearing in the direction of loading, and that the surface condition of the friction faces is maintained in the same condition to be achieved in the field. If it is necessary to machine the ends of the inner plates to fit into the loading machine grip, machining oil shall not be allowed to contaminate the surfaces. Bolts shall be tensioned in the same manner as that to be used in the field and shall develop at least the minimum bolt tension given in [Table 15.2.2.2](#).

Between snug-tightening and final tensioning, the bolt extension shall be measured using a dial gauge micrometer or a displacement transducer with a resolution of 0.003 mm or finer. The final measurement shall be made immediately prior to testing. The cone-sphere anvil measuring technique described in AS/NZS 1252 for proof load measurements or other equivalent technique is suitable.

Bolt tension shall be ascertained from a calibration curve determined from load cell tests of at least three bolts of the test batch. In establishing the calibration curve, the bolt grip through the load cell shall be as close as practicable to that used in the specimens, the same method of extension measurement and tensioning shall be employed, and the calibration shall be based on the mean result. For the purposes of this test only, the initial snug-tight condition shall be finger tight.

Alternatively, when a bolt tension load cell is not available, the bolts shall be tensioned to at least 80 % and not more than 100 % of their specified proof loads, and the tension induced in the bolts calculated from the following equation:

$$\frac{E\Delta \times 10^{-3}}{\frac{a_o}{A_o} + \left[ \frac{a_t + \frac{t_n}{2}}{A_s} \right]} \quad \text{J.1}$$

where

- $N_{ti}$  = tension induced in the bolt, in kilonewtons
- $E$  = Young's modulus of elasticity, 200 000 MPa
- $\Delta$  = measured total extension of the bolt when tightened from a finger-tight condition to final tensioned condition, in millimetres
- $a_o$  = length of the unthreaded portion of the bolt shank contained within the grip before tensioning, in millimetres. In this context, the grip includes the washer thickness
- $A_o$  = plain shank area of the unthreaded portion of the bolt, in square millimetres
- $a_t$  = length of the threaded portion of the bolt contained within the grip before tensioning, in millimetres. In this context, the grip includes the washer thickness
- $t_n$  = thickness of the nut, in millimetres
- $A_s$  = tensile stress area of the bolt as defined in AS 1275, in square millimetres

It is not necessary for both bolts in the one specimen to have identical tension induced in them.

### J.1.3 Number of specimens

Tests on at least three specimens shall be undertaken, but five is preferred as a practical minimum number.

## J.2 Instrumentation

Two pairs of dial gauge micrometers or displacement transducers having an effective resolution achieving 0.003 mm or finer shall be symmetrically disposed over gauge lengths of  $3d_f$  on each edge of the specimen so as to measure the deformation between the inner plates from the bolt positions to the centre of the cover plates. The deformation of each half of the joint shall be taken as the mean of the deformation at each edge. The deformation so measured is therefore the sum of the elastic extension of the cover plates and any slip at the bolt positions.

[Figure J.2](#) shows a typically instrumented test specimen. It is essential that the micrometers or transducers be securely mounted since they may be shock loaded as slip occurs.

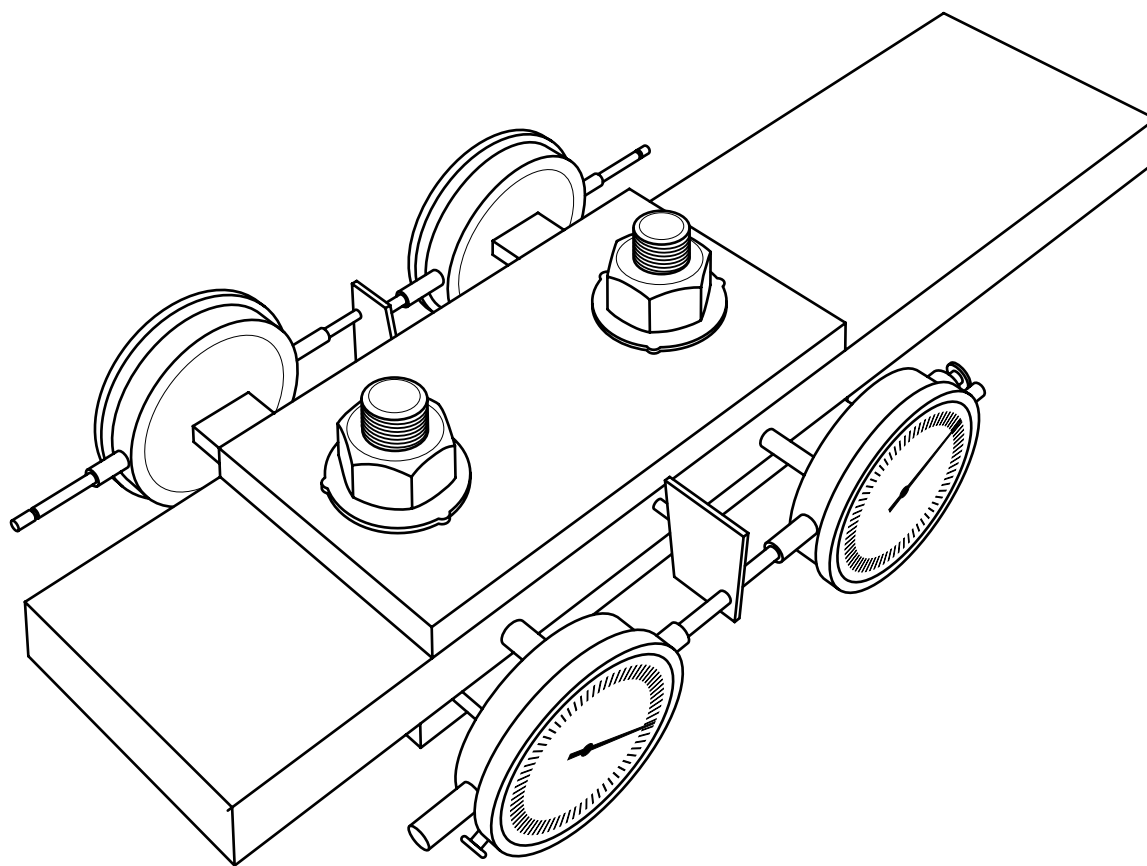


Figure J.2 — Typically instrumented test assembly

## J.3 Method of testing

The method of testing shall satisfy the following requirements:

- (a) *Type of loading* — Specimens shall be tested only by tensile loading.
- (b) *Loading rate* — Up to the slip load, force shall be applied in increments exceeding neither 25 kN nor 0.25 times of the slip load of the connection assuming a slip factor of 0.35 and the calculated bolt tension. The loading rate shall be approximately uniform at not more than 50 kN/min within each load increment. Slower loading rates are preferred. Each load

increment shall be applied after creep at constant load due to the preceding load increment has effectively ceased.

NOTE 1 Since slip will in all probability occur at one bolt position before the other, it is clear that the first bolt may slip into bearing before the slip load at the other bolt position is attained.

NOTE 2 After attainment of the slip load at one bolt position, the loading rate and increment size may be adjusted at the discretion of the operator.

## J.4 Slip load

Slip is usually well defined and easily detected when a sudden increase in deformation occurs. One or more sharp clearly audible reports may also be heard. However, with some types of surface, and occasionally with normal surfaces, the incidence of slip is not so well defined. In these cases, the load corresponding to a slip of 0.13 mm shall be used to define the slip load.

## J.5 Slip factor

The slip factor ( $\mu$ ) to be used in design shall be calculated from —

$$\mu = k(\mu_m - 1.64\delta)$$

where

$k$  = 0.85 when 3 specimens are tested,  
= 0.90 when 5 or more specimens are tested

$\mu_m$  = mean value of slip factor for all tests

$\Delta$  = standard deviation of slip factor for all tests

$$\mu_m = \frac{1}{2n} \left( \sum_{i=1}^{2n} \mu_i \right)$$

$$\mu_i = \frac{1}{2} \left( \frac{V_{si}}{N_{ti}} \right)$$

$$\delta = \sqrt{\left[ \frac{1}{2n-1} \sum_{i=1}^{2n} (\mu_i - \mu_m)^2 \right]}$$

$n$  = number of specimens tested, each providing two estimates of  $\mu$

$V_{si}$  = measured slip load at the position of the  $i^{\text{th}}$  bolt

$N_{ti}$  = tension induced in the  $i^{\text{th}}$  bolt by the tensioning as calculated from [Equation J.1](#)

However, if the calculated value of  $\mu$  is less than the lowest of all values of  $\mu_i$ , then  $\mu$  may be taken as equal to the lowest value of  $\mu_i$ .

## Appendix K (normative)

### Statistical data

#### K.1 Scope

This Appendix sets out the test requirements to produce the statistical data required for products not covered by [Table 10.4.1](#).

#### K.2 Statistical data

Statistical data collected from impact tests conducted in accordance with the appropriate Standard shall be assessed to ensure products meet the requirements of [Table 10.4.1](#).

Cold-formed structural hollow section tests shall be in accordance with the requirements of AS/NZS 1163.

Hot-rolled plate and hot-rolled steel flat product tests shall be in accordance with the requirements of AS/NZS 3678.

Hot-rolled bars and section tests shall be in accordance with the requirements of AS/NZS 3679.1.

#### K.3 Testing

##### K.3.1 General

This Appendix sets out the test requirements to produce the statistical data required for products not covered by [Table 10.4.1](#).

Testing shall be in accordance with [Section 17](#) and the following Standards, as applicable:

- (a) AS/NZS 1163.
- (b) AS/NZS 3678.
- (c) AS/NZS 3679.1.
- (d) AS/NZS 3679.2.

##### K.3.2 Minimum number of tests

The minimum number of tests shall be in accordance with the initial type testing requirements for impact grades in [Appendix B](#) of the relevant Standard noted in AS/NZS 1163, AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 3679.2. All tests shall be from the same facility with factory production control in accordance with the requirements in AS/NZS 1163, AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 3679.2.

#### K.4 Mechanical properties

The absorbed energy values, in joules, for all tests conducted for the temperatures listed in [Table 10.4.1](#) shall conform to the limits given in [Table K.4](#). Retests shall only be permitted in accordance with the retest requirements in AS/NZS 1163, AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 3679.2.

**Table K.4 — Charpy v-notch impact test requirements — Minimum absorbed energy values (in joules)**

Size of test piece, mm					
10 × 10		10 × 7.5		10 × 5	
Average of 3 tests	Individual test	Average of 3 tests	Individual test	Average of 3 tests	Individual test
27 J	20 J	22 J	16 J	18 J	13 J



## Appendix L (informative)

### Guidance on determination of the construction category

#### L.1 Introduction

This Appendix provides guidance on the choice of the construction category relevant to the building or structure as a whole or to components of the structure where it is appropriate to assign different construction categories.

**NOTE** The process outlined for defining a construction category is consistent with the philosophy and principles on which AS/NZS 1170.0 is based and intended to provide a level of consistency between the basis for the design assumptions and those for the ensuing fabrication and erection of the building or structure.

The determination of the construction category is undertaken in the design phase, based on the known loading for the building, the intended function, what maintenance and inspection measures will be in place, the elements that comprise the structure and the expected complexity of fabrication or erection for the structure. The construction specification should embody the specifics of these decisions and the additional and optional information necessary to fully define the necessary requirements for the chosen construction category or categories.

#### L.2 Background

The selection of a “construction category”, as applicable to a steel structure or components thereof, is a risk based approach intended to provide consistency with the reliability based philosophy and principles on which the fundamental load assessment (AS/NZS 1170 series) and structural design (this Standard and AS/NZS 5100.6) is based. The approach translates into a fit-for-purpose assessment that ensures the fabrication and erection of steel structures is based on a rational risk assessment, recognizing the importance of the structure, what maintenance and inspection measures will be in place, the consequences of failure and the complexity of the fabrication and erection.

AS/NZS 1170 series is based on the philosophy and principles set out in ISO 2394 (adopted as AS 5104), which provides a common basis for defining design rules relevant to the construction and use of a wide variety of buildings, bridges and civil engineering works. It includes methods for establishing and calibrating limit states design Standards. The required reliability is related to the expected social and economic consequences from a design failure. Significantly, the required reliability may be achieved through suitable combinations of the following measures:

- (a) Measures related to design, such as choice of values of action variables, reliability of design calculations, accuracy of mechanical models used and the like.
- (b) Measures relating to quality assurance, to reduce the risk of hazards from gross human errors, design and execution (fabrication and erection).

The construction category classification provides a fit-for-purpose level of quality assurance to reduce risks associated with fabrication and erection. It achieves this through reliability differentiation from inspection and supervision levels.

AS/NZS 1170.0 references the “importance level” for the building or structure as the primary indicator of the relative risk to life in extreme events (consequences of failure), and is based on the philosophy and principles set out in AS 5104 (ISO 2394). For Australia, the importance level is defined in the *National Construction Code* (NCC). For New Zealand, or for structures in Australia not covered by the NCC, AS/NZS 1170.0:2002, Section 3 is utilized.

The importance level is one component of the risk assessment that provides the basis for the calculation of the construction category. Other components reflect the type of loading the structure is subjected to and the complexity of the fabrication. Taken together, these components formalize the reliability differentiation that is included in AS 5104 and is implicit within AS/NZS 1170.0.

### L.3 Input factors determining the choice of construction category

#### L.3.1 General

The selection of the construction category defined in [Clause L.4](#) is based on three input variables:

- (a) The “importance factor”, which reflects the risk to life and consequences of failure (see [Clause L.3.2](#)).
- (b) The “service category”, which reflects the actions to which the structure and its parts are likely to be exposed, such as earthquake or fatigue (see [Clause L.3.3](#)).
- (c) The “fabrication category”, which reflects the complexity of the fabrication of the structure and its components (see [Clause L.3.4](#)).

#### L.3.2 Importance factor

The *National Construction Code* (for Australia) or AS/NZS 1170.0:2002, Section 3 defines the importance levels for different structure types. Importance levels are designated from 1 (representing the lowest risk to life) up to 4 (representing the highest risk to life and/or post disaster recover functions).

Where the structure type falls outside the scope of the NCC, AS/NZS 1170.0:2002, Section 3 should be referred to.

NOTE There is no provision in the NCC or in AS/NZS 1170.0 for designating parts of a structure with different importance factors.

#### L.3.3 Service category

The selected service category reflects the uncertainty in the exposure of the structure to actions that may expose flaws in the structure during use. The service category may be selected based on [Table L.1](#).

**Table L.1 — Suggested criteria for service categories**

Service category	Criteria
SC1	(a) Structures or components designed for predominantly quasi-static actions only. Examples include typical multi-level buildings, warehouses and storage facilities. <i>or</i> (b) Structures and components subject to low seismic demand (Earthquake design Categories I and II). <i>or</i> (c) Structures and components designed for low level fatigue actions where fatigue assessment is not required (e.g. for applications that satisfy <a href="#">Clause 11.4</a> or cranes classified S1 to S3 according to AS 1418.1—2002)
SC2	(d) Structures and components with members and connections subject to fatigue assessment in accordance with this Standard or AS/NZS 5100.6. Examples include road and railway bridges, cranes and immediate supporting structure (where supported off the building or structure) and structures susceptible to vibrations produced by wind, crowds or vibrating machinery. <i>or</i> (e) Structures and connections subject to medium to high seismic demand (Earthquake design Category III).

The structure or part of the structure can contain components or structural details that are categorized under different service categories.

NOTE The service category is also used to assess the recommended extent of non-destructive examination (NDE) of welds in AS/NZS 5131 and therefore should be nominated in the construction specification.

### L.3.4 Fabrication category

The selected fabrication category reflects the complexity of the fabrication inherent in the structure or parts of the structure. The fabrication category is selected based on [Table L.2](#).

**Table L.2 — Suggested criteria for fabrication categories**

Fabrication category	Criteria
FC1	(a) Non-welded components manufactured from any steel grade products. (b) Welded components manufactured from steel grade components less than or equal to Grade 450.
FC2	(c) Welded components manufactured from steel above Grade 450. (e) Site welded components essential for structural integrity. (f) Components receiving thermic treatment during manufacturing. (g) Components of CHS trusses requiring end profile cuts.

The structure or part of the structure can contain components or structural details that are categorized under different fabrication categories.

## L.4 Determination of the construction category

The construction category may be determined by the following process:

- Selection of the building or structure importance level, from the *National Construction Code* (for projects in Australia covered by the NCC) or AS/NZS 1170.0:2002, Section 3 (for projects in Australia not covered by the NCC).
- Selection of the service category (see [Table L.1](#)).
- Selection of the fabrication category (see [Table L.2](#)).
- Determination of the construction category from [Table L.3](#).

**Table L.3 — Risk matrix for determination of the construction category**

Importance level		1		2		3		4	
Service categories		SC1	SC2	SC1	SC2	SC1	SC2	SC1	SC2
Fabrication categories	FC1	CC1	CC3	CC2	CC3	CC2/CC3 <sup>a</sup>	CC3	CC3	CC3
	FC2	CC2	CC3	CC2	CC3	CC3	CC3	CC3	CC4

<sup>a</sup> The assessment of CC2 or CC3 for this designation marked should be based on engineering judgement and the relative simplicity of fabrication and erection of the structure.

NOTE 1 The determination of the construction category is the responsibility of the designer, taking national provisions, published guidance from industry associations and the relevant Work, Health and Safety regulations and Codes of Practice into account.

NOTE 2 The requirements for CC4 are additional to CC3. The additional requirements are not fully defined in this Standard. CC4 applies to unusual or special structures for which it is expected that requirements additional to those for CC3 will be defined at a project specific level or by a particular organization, agency or business.

## Appendix M (informative)

### Selection of materials for the avoidance of lamellar tearing

#### M.1 General

Lamellar tearing is a weld induced flaw in the material which generally becomes evident during ultrasonic examination. The main risk of tearing is with cruciform, T- and corner joints and with complete penetration welds where high levels of strain are known to occur in the through-thickness direction in the element to which the connection is made. This strain arises from the shrinkage of the weld metal as it cools. It is greatly increased where free movement is restrained by other portions of the structure.

The susceptibility of the material can be determined by measuring the through-thickness ductility as specified in AS/NZS 3678, which is expressed in terms of quality classes identified by Z-values.

NOTE Further guidance on the avoidance of lamellar tearing is given in Weld Australia Technical Note 6.

#### M.2 Procedure

The risk of lamellar tearing occurring is minimal if the following condition is satisfied:

$$Z_{Ed} \leq Z_{Rd} \quad \text{M.2.1}$$

where

$Z_{Ed}$  = the required design Z-value resulting from the magnitude of strains from restrained metal shrinkage under the weld beads

$Z_{Rd}$  = the available design Z-value for the material according to AS/NZS 3678, i.e. Z15, Z25 or Z35

The required design value  $Z_{Ed}$  may be determined using —

$$Z_{Ed} = Z_a + Z_b + Z_c + Z_d + Z_e \quad \text{M.2.2}$$

in which  $Z_a$ ,  $Z_b$ ,  $Z_c$ ,  $Z_d$  and  $Z_e$  are as given in [Table M.2](#).

NOTE The welding details should be adjusted to minimize  $Z_{Ed}$  when practical.

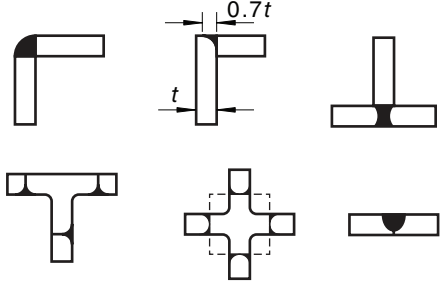
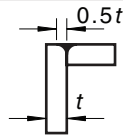
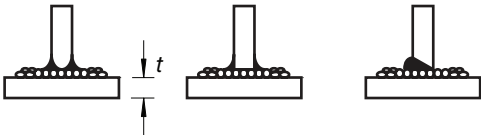
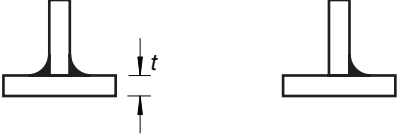
#### M.3 Non destructive examination

For high risk fabrications, fabricators may find it useful to examine plates to be welded using ultrasonic techniques to establish the location of internal discontinuities such as non metallic inclusions. The ultrasonic sensitivity to be used for parent metal scanning in areas where welds are to be made in accordance with AS 2207 i.e. set the gain so that the second back wall echo is displayed at full screen height. Where reflectors produce echoes approximately 25 % in plate thickness or higher, it may be beneficial to place welds away from such areas or to turn the plate over and place critical welds on the side of the plate furthest from the indications.

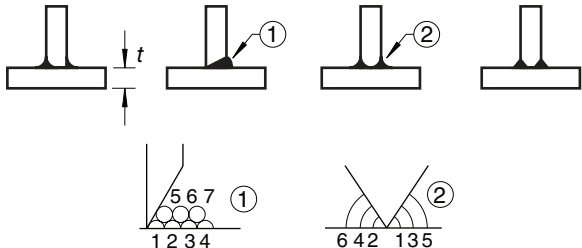
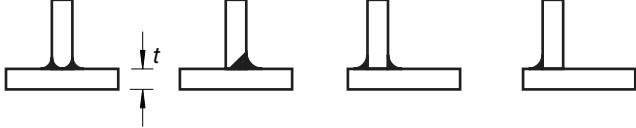

Should discontinuities be later detected in the weld heat affected zone it should be remembered that imperfections of parent metal origin are not considered a cause for rejection of the weld (see AS/NZS 1554.1:2014 Clause 6.2.2).

Fabricators and inspectors should be aware that ultrasonic examinations will not typically detect inclusions normally associated with lamellar tearing (e.g. manganese sulphides), and similarly are not an indicator of through – thickness ductility.

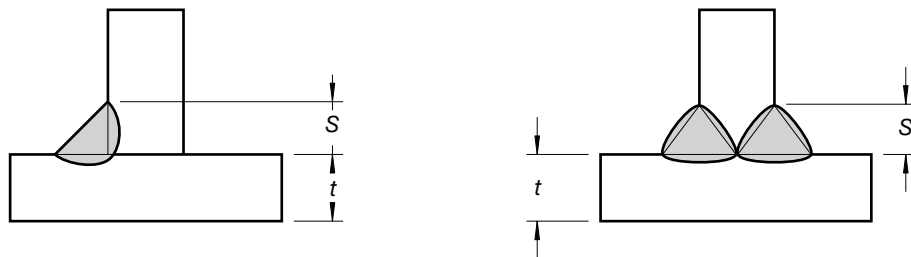
**Table M.2 — Criteria affecting the target value of  $Z_{Ed}$**   
(Adapted from Table 3.2 of EN 1993-1-10. Copyright © 2005. CEN, Belgium. [www.cen.eu](http://www.cen.eu))

Item	Effective weld depth ( $a_{eff}$ ) (see <a href="#">Figure M.2</a> ), mm	$Z_i$	
(a) Weld depth relevant for straining from metal shrinkage	$S \leq 7$	$Z_a = 0$	
	$7 < S \leq 10$	$Z_a = 3$	
	$10 < S \leq 20$	$Z_a = 6$	
	$20 < S \leq 30$	$Z_a = 9$	
	$30 < S \leq 40$	$Z_a = 12$	
	$40 < S \leq 50$	$Z_a = 15$	
	$50 < S$	$Z_a = 15$	
(b) Shape and position of welds in T- and cruciform- and corner-connections		$Z_b = -25$	
	Corner joints		$Z_b = -10$
	Single run fillet weld $Z_a = 0$ or fillet welds with $Z_a > 1$ with buttering with low strength weld material		$Z_b = -5$
	Multi run fillet welds		$Z_b = 0$

**Table M.2 (continued)**

Item	Effective weld depth ( $a_{eff}$ ) (see <a href="#">Figure M.2</a> ), mm		$Z_i$
	Partial and full penetration welds	with appropriate welding sequence to reduce shrinkage effects 	$Z_b = 3$
	Partial and full penetration welds		$Z_b = 5$
	Corner joints		$Z_b = 8$
(c)	Effect of material thickness $t$ on restraint to shrinkage	$t \leq 10$	$Z_b = 2^a$
		$10 < t \leq 20$	$Z_c = 4^a$
		$20 < t \leq 30$	$Z_c = 6^a$
		$30 < t \leq 40$	$Z_c = 8^a$
		$40 < t \leq 50$	$Z_c = 10^a$
		$50 < t \leq 60$	$Z_c = 12^a$
		$60 < t \leq 70$	$Z_c = 15^a$
		$70 < t$	$Z_c = 15^a$
(d)	Remote restraint of shrinkage after welding by other portions of the structure	Low restraint: Free shrinkage possible (e.g. T-joints)	$Z_d = 0$
		Medium restraint: Free shrinkage restricted (e.g. diaphragms in box girders)	$Z_d = 3$
		High restraint: Free shrinkage not possible (e.g. stringers in orthotropic deck plates)	$Z_d = 5$
(e)	Influence of preheating	Without preheating	$Z_e = 0$
		Preheating $\geq 100^\circ\text{C}$	$Z_e = -8$

<sup>a</sup> May be reduced by 50 % for material stressed, in the through-thickness direction, by compression due to predominantly static loads



**Figure M.2 — Effective weld depth  $S$  for shrinkage**

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## NOTES



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GPO Box 476 Sydney NSW 2001  
Phone (02) 9237 6000  
mail@standards.org.au  
www.standards.org.au