AS 4100—1998
(Incorporating Amendment No. 1)

AS 4100—1998

Australian Standard®

Steel structures

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This Australian Standard® was prepared by Committee BD-001, Steel Structures. It was approved on behalf of the Council of Standards Australia on 17 April 1998.
This Standard was published on 5 June 1998.

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This Standard was issued in draft form for comment as DR 97347.

Standards Australia wishes to acknowledge the participation of the expert individuals that contributed to the development of this Standard through their representation on the Committee and through the public comment period.

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(Incorporating Amendment No. 1)

Australian Standard®

Steel structures

Originated in part as SAA INT 351—1956.
Previous edition AS 4100—1990.
Second edition 1998.
Reissued incorporating Amendment No. 1 (February 2012).

Published by SAI Global Limited under licence from Standards Australia Limited, GPO Box 476, Sydney, NSW 2001, Australia

ISBN 0 7337 1981 3

PREFACE

This Standard was prepared by the Standards Australia Committee BD-001, Steel Structures, to supersede AS 4100—1990.

This Standard incorporates Amendment No. 1 (February 2012). The changes required by the Amendment are indicated in the text by a marginal bar and amendment number against the clause, note, table, figure or part thereof affected.

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.

This new edition of the Standard incorporates Amendments No. 1—1992, No. 2—1993, No. 3—1995 and draft Amendment No. 4 issued for public comment as DR 97347. Draft Amendment No. 4 was not published separately as a green slip.

Amendment No. 1—1992 includes the following major changes:

(a) Strength of steels complying with AS 1163 and AS/NZS 1594. (Table 2.1.)

(b) Shear buckling capacity for stiffened web. (Clause 5.11.5.2.)

(c) Bearing buckling capacity. (Clause 5.13.4.)

Amendment No. 2—1993 includes the following major changes:

(a) Shear and bending interaction method. (Clause 5.12.3.)

(b) Minimum area for the design of intermediate transverse web stiffeners. (Clause 5.15.3.)

(c) Section capacity of members subject to combined actions. (Clause 8.3.)

(d) Strength assessment of a butt weld. (Clause 9.7.2.7.)

(e) Fatigue. (Section 11.)

Amendment No. 3—1993 includes the following major changes:

(a) Compressive bearing action on the edge of a web. (Clause 5.13.)

(b) Section capacity of members subject to combined actions. (Clause 8.3.)

(c) In-plane and out-of-plane capacity of compression members. (Clauses 8.4.2.2 and 8.4.41.)

(d) Strength assessment of a butt weld. (Clause 9.7.2.7.)

(e) Earthquake. (Section 13.)

Amendment No. 4 includes the following major changes:

(a) Strengths of steels complying with AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 3679.2. (Table 2.1.)

(b) Minimum edge distance of fasteners. (Clause 9.6.2.)

(c) Permissible service temperatures according to steel type and thickness. (Table 10.4.1.)

(d) Steel type relationship to steel grade. (Table 10.4.4.)

(e) Welding of concentrically braced frames for structures of earthquake Design Category D and E. (Clause 13.3.4.2.)

A1 *Amendment No. 1—2012 to the 1998 edition* includes the following major changes:

(a) Revisions to AS/NZS 1163, AS/NZS 3678, AS/NZS 3679.1 and AS/NZS 3679.2 reflected by amendments to Sections 2 and 10.

(b) Revisions to AS/NZS 1554.1, AS/NZS 1554.4 and AS/NZS 1554.5 reflected by amendments to Sections 9 and 10.

(c) Section 13 brought into line with revisions to AS 1170.4.

(d) Quenched and tempered steels included by adding 'AS 3597' to listed material Standards in Section 2.

(e) Typographical errors corrected.

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

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APPENDICES

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SECTION 1 SCOPE AND GENERAL

1.1 SCOPE AND APPLICATION

1.1.1 Scope

This Standard sets out minimum requirements for the design, fabrication, 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.

A1 **'Text deleted'**

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

A1 (a) Steel elements less than 3 mm thick, with the exception of sections complying with AS/NZS 1163 and packers.

A1 (b) Steel members for which the value of the yield stress used in design (f_y) exceeds 690 MPa.

A1 (c) Cold-formed members, other than those complying with AS/NZS 1163, which shall be designed in accordance with AS/NZS 4600.

(d) Composite steel-concrete members, which shall be designed in accordance with AS 2327.

A1 (e) Road, railway and pedestrian bridges, which shall be designed in accordance with AS 5100.1, AS 5100.2 and AS 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.

A1 ### 1.1.2 'Text deleted'

1.2 REFERENCED DOCUMENTS

The documents referred to in this Standard are listed in Appendix A.

1.3 DEFINITIONS

For the purpose of this Standard, the definitions below apply. Definitions peculiar to a particular Clause or Section are also given in that Clause or Section.

Action—the cause of stress or deformations in a structure.

Action effect or load effect—the internal force or bending moment due to actions or loads.

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

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.

Bearing-wall system—see AS 1170.4.

Braced frame—see AS 1170.4.

A1 *Braced frame, concentric*—see AS 1170.4.

Braced frame, eccentric—see AS 1170.4.

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

A1 **'Text deleted'**

Capacity factor—a factor used to multiply the nominal capacity to obtain the design capacity.

Complete penetration butt weld—a butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

A1 **'Text deleted'**

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).

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).

Design action effect or design load effect—the action or load effect computed from the design actions or design loads.

A1 **'Text deleted'**

A1 *Design action or design load*—the 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.

Design capacity—the product of the nominal capacity and the capacity factor.

Design life—period over which a structure or structural element is required to perform its function without repair.

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

Design spectrum—sum of the stress spectra from all of the nominal loading events expected during the design life.

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

Discontinuity—an absence of material, causing a stress concentration.

A1 **'Text deleted'**

'Text deleted'

Ductility (of structure)—see AS 1170.4.

'Text deleted'

A1 **'Text deleted'**

'Text deleted'

Exposed surface area to mass ratio—the ratio of the surface area exposed to the fire to the mass of steel.

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

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.

Fatigue strength—the 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.

Fire exposure condition—

(a) *three-sided fire exposure condition*—steel member incorporated in or in contact with a concrete or masonry floor or wall.

(b) *four-sided fire exposure condition*—a steel member exposed to fire on all sides.

Fire protection system—the fire protection material and its method of attachment to the steel member.

Fire-resistance level (FRL)—the fire-resistance grading period for structural adequacy only, in minutes, which is required to be attained in the standard fire test.

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.

Full tensioning—a method of installing and tensioning a bolt in accordance with Clauses 15.2.4 and 15.2.5.

Geometrical slenderness ratio—the 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.

Incomplete penetration butt weld—a butt weld in which the depth of penetration is less than the complete depth of the joint.

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.

A1 **'Text deleted'**

Length (of a compression member)—the 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.

Limit state—any limiting condition beyond which the structure ceases to fulfil its intended function.

Load—an externally applied force.

Miner's summation—cumulative damage calculation based on the Palmgren-Miner summation or equivalent.

A1 **'Text deleted'**

A1 *Moment-resisting frame*—see AS 1170.4.

Moment-resisting frame, intermediate—see AS 1170.4.

Moment-resisting frame, ordinary—see AS 1170.4.

Moment-resisting frame, special—see AS 1170.4.

Nominal action or load—an action or load, as specified in Clause 3.2.1 or 3.2.2.

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

Nominal loading event—the loading sequence for the structure or structural element.

Non-slip fasteners—fasteners which 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.

A1 **'Text deleted'**

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.

Period of structural adequacy (PSA) (fire)—the time (t), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test.

Pin—an unthreaded fastener manufactured out of round bar.

Plastic hinge—a yielding zone with significant inelastic rotation which forms in a member when the plastic moment is reached.

Prequalified weld preparation—a joint preparation prequalified in terms of AS/NZS 1554.1.

Proof testing—the 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 (fire)—a test specimen representing a steel member and its fire protection system which is subjected to the standard fire test.

Prototype testing—the 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.

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.

A1 *Quenched and tempered steel*—high strength steel manufactured by heating, quenching, tempering and levelling steel plate.

Segment (in a member subjected to bending)—the 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.

Serviceability limit state—a limit state of acceptable in-service condition.

Shear wall—a wall designed to resist lateral forces parallel to the plane of the wall.

S-N curve—curve defining the limiting relationship between the number of stress cycles and stress range for a detail category.

Snug tight—the tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

Space frame—see AS 1170.4.

A1 **'Text deleted'**

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

Standard fire test—the fire-resistance test specified in AS 1530.4.

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

Strength limit state—a limit state of collapse or loss of structural integrity.

Stress cycle—one cycle of stress defined by stress cycle counting.

Stress cycle counting method—any rational method used to identify individual stress cycles from the stress history.

Stress range—algebraic difference between two extremes of stress.

Stress spectrum—histogram of the stress cycles produced by a nominal loading event.

Structural adequacy (fire)—the ability of the member exposed to the standard fire test to carry the test load specified in AS 1530.4.

A1 *Structural ductility factor*—see AS 1170.4.

Structural performance factor—see AS 1170.4.

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

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

Yield stress—the 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.

A superscripted '*' 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
A1 A_e	=	effective sectional area of a hollow section in shear; *or*
	=	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; or
	=	lesser of the flange effective areas

A_{fn}	=	net area of a flange
A_g	=	gross area of a cross-section
A1 A_{gv}	=	gross area subject to shear at rupture
A1 A_n	=	net area of a cross-section
A1 A_{nt}	=	net area subject to tension at rupture
A1 A_{nv}	=	net area subject to shear at rupture
A1 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; *or*
	=	area of a stiffener or stiffeners in contact with a flange; *or*
	=	area of an intermediate web stiffener
A_w	=	gross sectional area of a web; *or*
	=	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; *or*
	=	lesser dimension of a web panel; *or*
	=	clear width of an element outstand from the face of a supporting plate element; *or*
	=	clear width of a supported element between faces of supporting plate elements
b_b, b_{bf}, b_{bw}, b_o	=	bearing widths defined in Clause 5.13
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
A1 b_{fo}	=	half the clear distance between the webs; *or*
	=	least of 3 dimensions defined in Clause 5.11.5.2
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 Table H3 and Paragraph H5

	c_h	= perpendicular distance to centroid of an angle section from the face of the loaded leg of the angle
A1	c_m	= factor for unequal end moments
A1	d	= depth of a section; *or*
		= depth of preparation for incomplete penetration butt weld; *or*
		= maximum cross-sectional dimension of a built-up compression member
A1	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; *or*
		= factor defined in Appendix I
	d_f	= diameter of a fastener (bolt or pin); *or*
		= distance between flange centroids
	d_m	= depth of a section at minimum cross-section
	d_o	= overall section depth including out-of-square dimensions; *or*
		= overall section depth of a segment; *or*
		= outside diameter of a circular hollow section
	d_p	= clear transverse dimension of a web panel; *or*
		= 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
A1	d_5	= flat width of web of hollow sections
	E	= Young's modulus of elasticity, 200×10^3 MPa
	$E(T)$, $E(20)$	= E at T, 20 degrees Celsius respectively
	e	= eccentricity; *or*
		= web off-centre dimension; *or*
		= distance between an end plate and a load-bearing stiffener
	e_c, e_t	= eccentricities of compression and tension angles (Clause 8.4.6)
	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
A1 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
A1 f_{yc}	=	yield stress of connection element
$f_y(T)$, $f_y(20)$	=	yield stresses of steel at T, 20 degrees Celsius 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 (Appendix I)
G	=	shear modulus of elasticity, 80×10^3 MPa; *or*
	=	nominal dead load
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
A1 i	=	number of loading event (Section 11)
J	=	torsion constant for a cross-section
K	=	$\sqrt{\left[\pi^2 EI_w / \left(GJL^2\right)\right]}$
K_d	=	deflection amplification factor
k	=	coefficient used in Appendix J
k_b	=	elastic buckling coefficient for a plate element
k_{bo}	=	basic value of k_b
A1 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
A1 k_l	=	effective length factor for load height
k_p	=	factor for pin rotation
k_r	=	effective length factor for restraint against lateral rotation; *or*
	=	effective length factor for a restraining member; *or*
	=	reduction factor to account for the length of a bolted or welded lap splice connection
k_s	=	ratio used to calculate α_p and α_{pm}
k_{sm}	=	exposed surface area to mass ratio
A1 k_t	=	effective length factor for twist restraints; *or*
	=	correction factor for distribution of forces in a tension member
A1 k_v	=	ratio of flat width of web (d_5) to thickness (t) of hollow section
k_0-k_6	=	regression coefficients (Section 12)
A1 l	=	span; or
	=	member length; *or*
	=	member length from centre to centre of its intersections with supporting members; *or*
	=	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; *or*
	=	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 parrallel 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; *or*

= 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; or

= 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; *or*

= 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

A1 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_{ol} = $\dfrac{\pi^2 EI}{l^2}$

N_{olr} = $\dfrac{\pi^2 EI_r}{l_r^2}$

A1 N_{om} = elastic buckling load

N_{omb} = N_{om} for a braced member

N_{oms} = N_{om} for a sway member

N_{oz} = nominal elastic torsional buckling capacity of a member

N_s = nominal section capacity of a compression member; *or*

= 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; *or*
		= 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; *or*
		= design live load
	R_b	= nominal bearing capacity of a web
A1	R_{bb}	= nominal bearing buckling capacity of a web
A1	R_{bs}	= nominal design capacity in block shear
A1	R_{by}	= nominal bearing yield capacity of a web
A1	**'Text deleted'**	
	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; *or*
		= design reaction
A1	R_{bs}^*	= design reaction
	R_w^*	= design bearing force or reaction on a web panel
A1	r	= radius of gyration

A1 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_r	=	ratio defined in Clause 5.6.1.1
r_s	=	ratio defined in Clause 5.6.1.1
r_y	=	radius of gyration about minor principal y-axis
S	=	plastic section modulus
A1 S_p	=	structural performance factor
S^*	=	design action effect
s	=	spacing of stiffeners; *or*
	=	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
A1 t	=	thickness; *or*
	=	element thickness; *or*
	=	thickness of thinner part joined; *or*
	=	wall thickness of a circular hollow section; *or*
	=	thickness of an angle section; *or*
	=	time
t_f	=	thickness of a flange; *or*
	=	thickness of the critical flange
t_n	=	thickness of a nut
A1 t_p	=	thickness of a ply; *or*
	=	thickness of thinner ply connected; *or*
	=	thickness of a plate
	=	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
A1 t_w	=	thickness of a web or web panel
A1 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; *or*
	=	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 ith 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; *or*
	=	nominal shear capacity of a plug or slot weld
A1 V^*	=	design shear force; *or*
	=	design horizontal storey shear force at column ends; *or*
	=	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; *or*
	=	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
α	=	angle between x- and h-axes for an angle section
α_a	=	compression member factor, as defined in Clause 6.3.3
α_b	=	compression member section constant, as defined in Clause 6.3.3
α_{bc}	=	moment modification factor for bending and compression
α_c	=	compression member slenderness reduction factor
α_d	=	tension field coefficient for web shear buckling
α_f	=	flange restraint factor for web shear buckling
α_l α_{lc} α_{mc}	=	factors for bending defined in Paragraphs H2 and H3
α_m	=	moment modification factor for bending
A1 α_p	=	coefficient used to calculate the nominal bearing yield capacity (R_{by}) for square and rectangular hollow sections to AS/NZS 1163

α_{pm}	=	coefficient used to calculate α_p
α_{ry}	=	elastic stiffness of a flexural end restraint
α_{rz}	=	elastic stiffness of a torsional end restraint
α_s	=	slenderness reduction factor; *or*
	=	inverse of the slope of the S-N curve for fatigue
α_{sr}	=	stability function multiplier
α_{st}	=	reduction factor for members of varying cross-section
α_T	=	coefficient of thermal expansion for steel, 11.7×10^{-6} per degree Celsius
α_t	=	factor for torsional end restraint defined in Clause 5.14.5
α_v	=	shear buckling coefficient for a web
α_w	=	factor defined in Appendix I
β_e	=	modifying factor to account for conditions at the far ends of beam members
β_m	=	ratio of smaller to larger bending moment at the ends of a member; *or*
	=	ratio of end moment to fixed end moment
β_t	=	measure of elastic stiffness of torsional end restraint used in Appendix H
A1 β_{tf}	=	thickness correction factor for fatigue
β_x	=	monosymmetry section constant
β_w	=	factor defined in Appendix I
A1 γ	=	index used in Clause 8.3.4; *or*
	=	factor for transverse stiffener arrangement in stiffened web (Clause 5.15.3)
γ, γ_1, γ_2,	=	ratios of compression member stiffness to end restraint stiffness used in Clause 4.6.3.3
Δ	=	deflection; *or*
	=	deviation from nominated dimension; *or*
	=	measured total extension of a bolt when tightened
Δ_{ct}	=	mid-span deflection of a member resulting from transverse loading together with both end bending moments
Δ_{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
Δ_f	=	out-of-flatness of a flange plate
Δh_b	=	deviation from h_b
Δl_c	=	deviation from l_c
Δ_s	=	translational displacement of the top relative to the bottom for a storey height
Δ_v	=	deviation from verticality of a web at a support

Δ_w	=	out-of-flatness of a web
δ	=	standard deviation
δ_b	=	moment amplification factor for a braced member
δ_m	=	moment amplification factor, taken as the greater of δ_b and δ_s
δ_p	=	moment amplification factor for plastic design
δ_s	=	moment amplification factor for a sway member
ξ	=	compression member factor defined in Clause 6.3.3
η	=	compression member imperfection factor defined in Clause 6.3.3
θ	=	angle of preparation of an incomplete penetration butt weld
π	=	pi ($\approx$ 3.14159)
λ	=	slenderness ratio; *or*
	=	elastic buckling load factor
λ_c	=	elastic buckling load factor
λ_e	=	plate element slenderness
λ_{ed}	=	plate element deformation slenderness limit
λ_{ep}	=	plate element plasticity slenderness limit
λ_{ey}	=	plate element yield slenderness limit
λ_m	=	elastic buckling load factor for a member
λ_{ms}	=	elastic buckling load factor for the storey under consideration
λ_n	=	modified compression member slenderness
λ_s	=	section slenderness
λ_{sp}	=	section plasticity slenderness limit
λ_{sy}	=	section yield slenderness limit
λ_w, λ_{ew}	=	values of λ_e and λ_{ey} for the web
A1 μ	=	slip factor
	=	structural ductility factor
μ_m	=	mean value of the slip factor
A1 μ_i	=	individual test result from test for slip factor
v	=	Poisson's ratio, 0.25
ρ	=	ratio of design axial force in a restraining member to the elastic buckling load for a member of length l (Appendix G); *or*
	=	I_{cy}/I_y
ϕ	=	capacity factor
A1 ϕR_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 complied with.

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.

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 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 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) Any other requirements for fabrication, erection and operation.

1.7 CONSTRUCTION

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 specification, are satisfied.

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.

2.2 STRUCTURAL STEEL

2.2.1 Australian Standards

Except as otherwise permitted in Clause 2.2.3, all structural steel coming within the scope of this Standard shall, before fabrication, comply with the requirements of the following Standards, as appropriate:

A1 **'Text deleted'**

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	Structural steel
AS/NZS 3679.1	Part 1: Hot-rolled bars and sections
AS/NZS 3679.2	Part 2: Welded I sections

2.2.2 Acceptance of steels

A1 Test reports or test certificates that comply with the minimum requirements of the appropriate Standard listed in Clause 2.2.1 shall constitute sufficient evidence of compliance of the steel with the Standards listed in Clause 2.2.1. The test reports or test certificates shall be provided by the manufacturer or an independent laboratory accredited by signatories to the International Laboratory Accreditation Corporation (Mutual Recognition Arrangement) ILAC MRA or the Asia Pacific Laboratory Accreditation Cooperation (APLAC) on behalf of the manufacturer. In the event of a dispute as to the compliance of the steel with any of the Standards listed in Clause 2.2.1, the reference testing shall be carried out by independent laboratories accredited by signatories to ILAC MRA or APLAC.

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 will 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.

2.3 FASTENERS

2.3.1 Steel bolts, nuts and washers

Steel bolts, nuts and washers shall comply with the following Standards, as appropriate:

A1

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/NZS 1252	High strength steel bolts with associated nuts and washers for structural engineering
AS/NZS 1559	Hot-dip galvanized steel bolts with associated nuts and washers for tower construction

Test certificates that state that the bolts, nuts and washers comply with all the provisions of the appropriate Standard listed in Clause 2.3.1 shall constitute sufficient evidence of compliance with the appropriate Standard. Such test reports shall be provided by the bolt manufacturer or bolt importer and shall be carried out by an independent laboratory accredited by signatories to the International Laboratory Accreditation Corporation (Mutual Recognition Arrangement) ILAC MRA or the Asia Pacific Laboratory Accreditation Cooperation (APLAC) on behalf of the manufacturer, importer or customer. In the event of a dispute as to the compliance of the bolt, nut or washer with any of the Standards listed in Clause 2.3.1, the reference testing shall be carried out by independent laboratories accredited by signatories to ILAC MRA or APLAC.

NOTE: Acceptable bolts and associated bolting categories are specified in Table 9.3.1.

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 shall be permitted provided that evidence of their equivalence to high strength bolts complying with AS/NZS 1252 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 comply with AS/NZS 1252 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 complying with AS/NZS 1252 of the same nominal dimensions. Equivalent fasteners may differ in other dimensions from those specified in AS/NZS 1252.

(c) The method of tensioning and the inspection procedure for equivalent fasteners may differ in detail from those specified in Clauses 15.2.5 and 15.4 respectively, provided that the minimum fastener tension is not less than the minimum bolt tension given in Table 15.2.5.1 and that the tensioning procedure is able to be checked.

2.3.3 Welds

A1 All welding consumables and deposited weld metal for steel parent material with a specified yield strength ≤ 500 MPa shall comply with AS/NZS 1554.1 except when welding to quenched and tempered steel according to AS 3597, where the welding consumables and deposited weld metal for steel parent material with a specified yield strength ≤ 690 MPa shall comply with AS/NZS 1554.4. Where required by Clause 11.1.5, the welds shall comply with AS/NZS 1554.5.

2.3.4 Welded studs

All welded studs shall comply with, and shall be installed in accordance with
A1 AS/NZS 1554.2.

2.3.5 Explosive fasteners

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

2.3.6 Anchor bolts

Anchor bolts shall comply with either the bolt Standards of Clause 2.3.1 or shall be manufactured from rods complying with the steel Standards of Clause 2.2.1 provided that the threads comply with AS 1275.

2.4 STEEL CASTINGS

All steel castings shall comply with AS 2074.

A1

TABLE 2.1

STRENGTHS OF STEELS COMPLYING WITH AS/NZS 1163, AS/NZS 1594, AS/NZS 3678, AS/NZS 3679.1, AS/NZS 3679.2 (Note 2) 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 (Note 3)	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 HA250/1 HU250	All	250	350
		HA200	All	200	300
	Plate and strip	HA4N	All	170	280
		HA3	All	200	300
		HA1	All	(See Note 1)	(See Note 1)
		XF500	$t \leq 8$	480	570
		XF400	$t \leq 8$	380	460
		XF300	All	300	440

(*continued*)

TABLE 2.1 (*continued*)

A1

Steel Standard	Form	Steel grade	Thickness of material, t mm	Yield stress (f_y) MPa	Tensile strength (f_u) MPa
AS/NZS 3678 (Note 2 and 3)	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
		350	$20 < t \leq 80$	340	450
AS/NZS 3678 (Note 3)		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
		300	$12 < t \leq 20$	300	430
		300	$20 < t \leq 50$	280	430
		300	$50 < t \leq 80$	270	430
		300	$80 < t \leq 150$	260	430
		250	$t \leq 8$	280	410
		250	$8 < t \leq 12$	260	410
		250	$12 < t \leq 50$	250	410
		250	$50 < t \leq 80$	240	410
		250	$80 < t \leq 150$	230	410
		200	$t \leq 12$	200	300
AS/NZS 3679.1 (Note 3)	Flats and sections	350	$t \leq 11$	360	480
		350	$11 < t < 40$	340	480
		350	$t \geq 40$	330	480
		300	$t < 11$	320	440
		300	$11 \leq t \leq 17$	300	440
		300	$t > 17$	280	440
	Hexagons, rounds and squares	350	$t \leq 50$	340	480
		350	$50 < t < 100$	330	480
		350	$t \geq 100$	320	480
		300	$t \leq 50$	300	440
		300	$50 < t < 100$	290	440
		300	$t \geq 100$	280	440

(*continued*)

TABLE 2.1 (*continued*)

A1

Steel Standard	Form	Steel grade	Thickness of material, t mm	Yield stress (f_y) MPa	Tensile strength (f_u) MPa
AS 3597	Plate	500	$5 \leq t \leq 110$	500	590
		600	$5 \leq t \leq 110$	600	690
		700	$t \leq 5$	650	750
		700	$5 < t \leq 65$	690	790
		700	$65 < t \leq 110$	620	720

NOTES:

1 For design purposes, yield and tensile strengths approximate those of structural Grade HA200. For specific information, contact the supplier.

2 Welded I-sections complying with AS/NZS 3679.2 are manufactured from hot-rolled structural steel plates complying 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.

3 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.

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, fatigue, fire and earthquake in accordance with the procedures given in this Standard, as appropriate.

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:

A1 (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.

(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.

(e) Other specific loads, as required.

NOTES:

A1 1 For the design of bridges, loads specified in AS 5100.2 should be used.

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
A1 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 be considered to act in conjunction with only
A1 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.

A1 ### 3.2.5 Structural robustness

All steel structures, including members and connection components, shall comply with 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
A1 the strength limit state specified in AS/NZS 1170.0.

(c) The design resistance effect shall be calculated as 0.8 times the part of the dead load tending to resist the instability plus the design capacity (ϕR_u) of any elements contributing towards resisting the instability, where ϕ 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 (ϕR_u) shall be determined from the nominal capacity (R_u) determined from Sections 5 to 9, as appropriate, where the capacity factor (ϕ) shall not exceed the appropriate value given in Table 3.4.

(d) All members and connections shall be proportioned so that the design capacity (ϕR_u) is not less than the design action effect (S^*), i.e.—

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

TABLE 3.4

CAPACITY FACTORS (ϕ) FOR STRENGTH LIMIT STATES

Design capacity for	Clauses	Capacity factor (ϕ)	
Member subject to bending			
—full lateral support	5.1, 5.2 & 5.3	0.90	
—segment without full lateral support	5.1 & 5.6	0.90	
—web in shear	5.11 & 5.12	0.90	
—web in bearing	5.13	0.90	
—stiffener	5.14, 5.15 & 5.16	0.90	
Member subject to axial compression			
—section capacity	6.1 & 6.2	0.90	
—member capacity	6.1 & 6.3	0.90	
Member subject to axial tension	7.1 & 7.2	0.90	
Member subject to combined actions			
—section capacity	8.3	0.90	
—member capacity	8.4	0.90	
A1 Connection component other than a bolt, pin or weld	9.1.9(a), (b), (c), and (d)	0.90	
	9.1.9(e)	0.75	
Bolted connection			
—bolt in shear	9.3.2.1	0.80	
—bolt in tension	9.3.2.2	0.80	
—bolt subject to combined shear and tension	9.3.2.3	0.80	
—ply in bearing	9.3.2.4	0.90	
—bolt group	9.4	0.80	
Pin connection			
—pin in shear	9.5.1	0.80	
—pin in bearing	9.5.2	0.80	
—pin in bending	9.5.3	0.80	
—ply in bearing	9.5.4	0.90	
Welded connection		SP Category	GP Category
—complete penetration butt weld	9.7.2.7	0.90	0.60
—longitudinal fillet weld in RHS ($t < 3$ mm)	9.7.3.10	0.70	—
—other fillet weld and incomplete penetration butt weld	9.7.3.10	0.80	0.60
—plug or slot weld	9.7.4	0.80	0.60
—weld group	9.8	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, as appropriate, 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 comply 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 (ϕ) shall be taken as 0.7 and the bolts shall be designed in accordance with Clause 9.3.3.

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 after consideration has been given to the use of the structure, its maintenance, and the climatic or other local conditions.

NOTE: Recommendations on corrosion protection may be found in Appendix C.

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, as appropriate, 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 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.

3.9 FIRE

The structure, its component members and connections shall be designed in accordance with Section 12.

3.10 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.

3.11 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 considered where relevant and, if significant, shall be taken into account in the design of the structure in accordance with the principles of this Standard and appropriate engineering principles.

A1 The design of bridges for loads resulting from floods or collision shall be carried out in accordance with AS 5100.2, as appropriate.

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.

A1 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:

(a) *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.

(b) *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 dependable and 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 analyzed in its entirety except as follows:

(a) Regular building structures may be analyzed 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 considered 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 considered as substructures, the bending moment at a support may be determined from the assumption 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 I/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 considered and, where significant, 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 (δ_b) or (δ_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 (δ_b) or (δ_s), calculated in accordance with Clause 4.4.2.2 or Clause 4.4.2.3 as appropriate, is greater than 1.4, a second-order elastic analysis in accordance with Appendix E shall be carried out.

The maximum calculated bending moment $\left(M_m^*\right)$ 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 δ_b is a moment amplification factor for a braced member calculated as follows:

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

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 β_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 β_m is determined by one of the following methods:

(a) $\beta_m = -1.0$;

(b) β_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 Figure 4.4.2.2; or

(c) $\beta_m = 1 - \left(\dfrac{2\Delta_{ct}}{\Delta_{cw}}\right)$ with $-1.0 \leq \beta_m \leq 1.0$

where

Δ_{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

Δ_{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

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 (δ_m) shall be taken as the greater of—

δ_b = the moment amplification factor for a braced member determined in accordance with Clause 4.4.2.2, and

δ_s = the 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 (δ_s) shall be calculated from—

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

where Δ_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) $$\delta_s = \frac{1}{1-\left(\dfrac{1}{\lambda_{ms}}\right)}$$

where the elastic buckling load factor (λ_{ms}) for the storey under consideration is determined in accordance with Clause 4.7.2.2.; or

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

where the elastic buckling load factor (λ_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 (δ_s) for each sway member shall be taken as the value for the frame calculated as follows:

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

where the elastic buckling load factor (λ_c) is determined from a rational buckling analysis of the whole frame (see Clause 4.7.2)

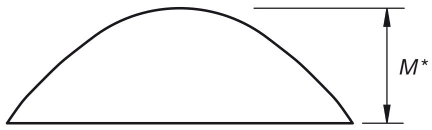
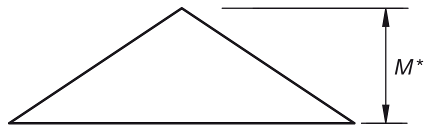
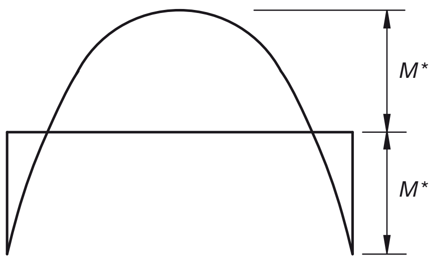
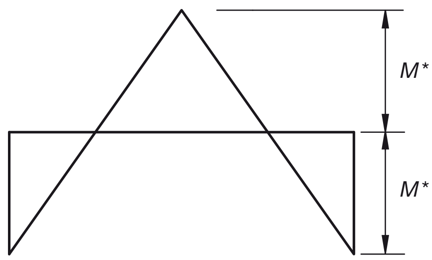
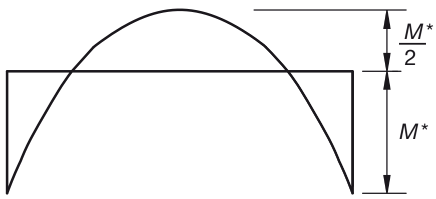
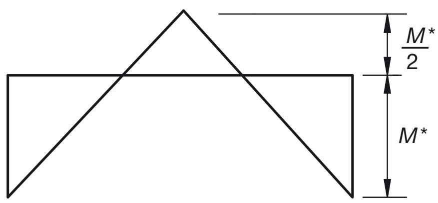
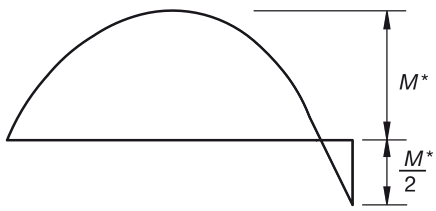
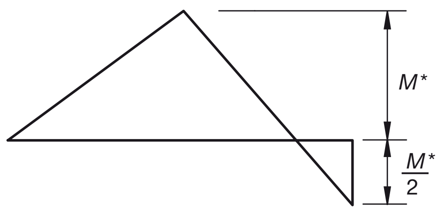
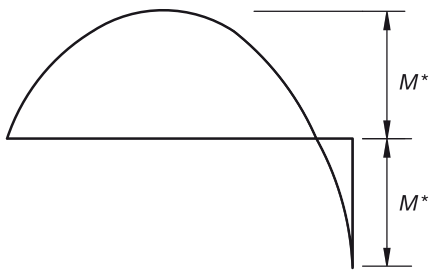
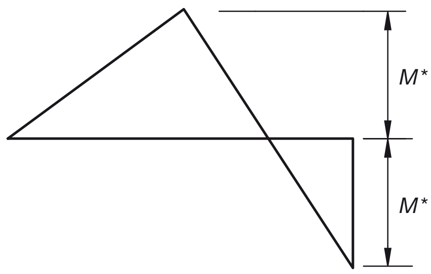
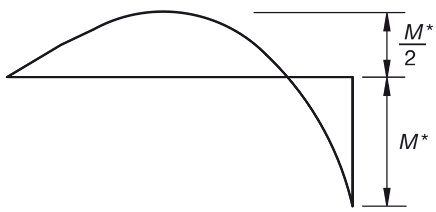
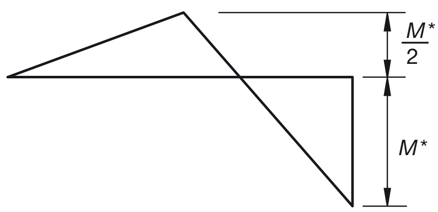
Moment distribution	β_m	Moment distribution	β_m
	-1.0		-1.0
	+0.2		+0.5
	+0.6		+1.0
	-0.5		+0.4
	+0.2		+0
	+0.2		+0.5

FIGURE 4.4.2.2 (in part) VALUES OF β_m FOR VARIOUS DISTRIBUTIONS OF BENDING MOMENT

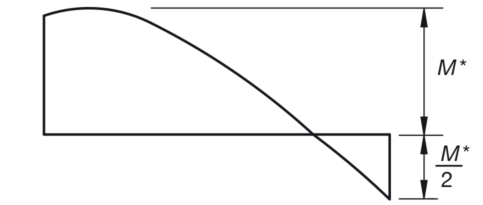
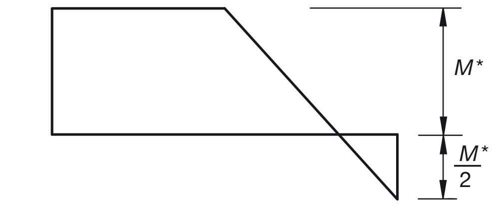
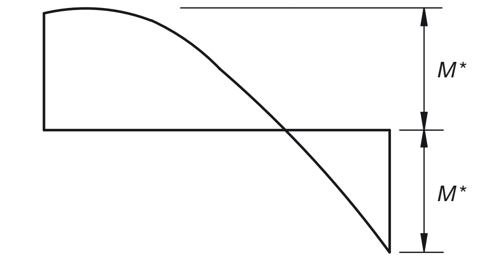
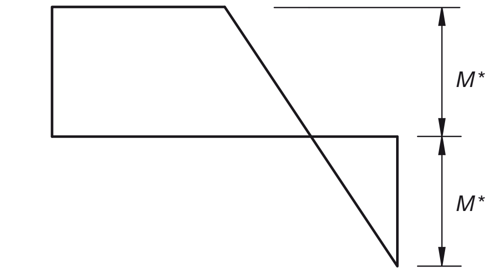
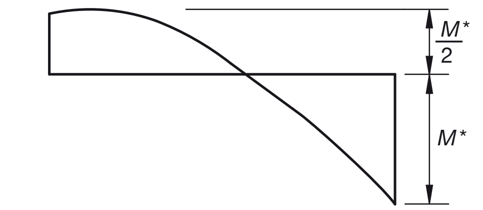
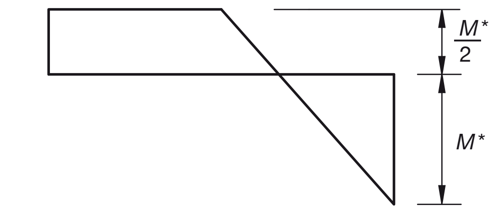
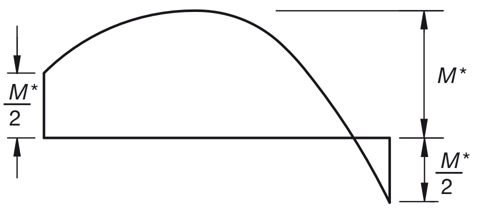
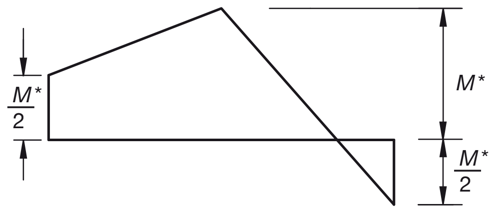
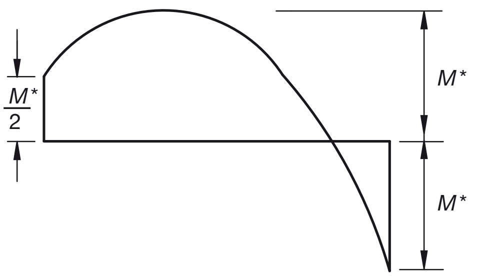
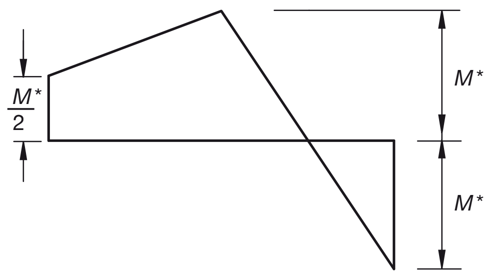
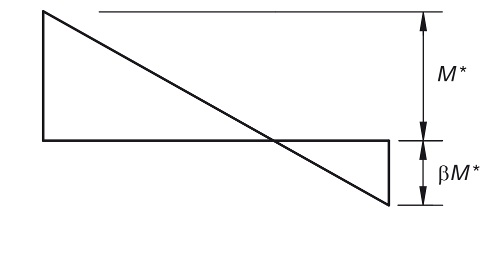
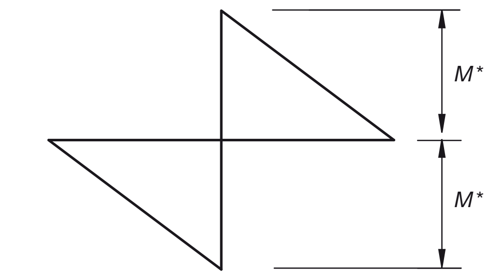
Moment distribution	β_m	Moment distribution	β_m
	-0.4		-0.5
	+0.1		-0.1
	+0.7		+0.3
	-0.5		-0.4
	-0.2		-0.1
	β		+1.0

FIGURE 4.4.2.2 (in part) VALUES OF β_m FOR VARIOUS DISTRIBUTIONS OF BENDING MOMENT

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 not be significantly different from those obtained for steels complying 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 complying 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 comply with 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 (λ_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 δ_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 (δ_b) in Clause 4.4.2.2, and the member buckling load (N_{oms}) in the determination of the elastic buckling load factor (λ_{ms}) in Clause 4.7.2.2 which is used in the determination of the moment amplification factor for a sway member (δ_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 is the member effective length factor, determined in accordance with Clause 4.6.3 and l is the 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.

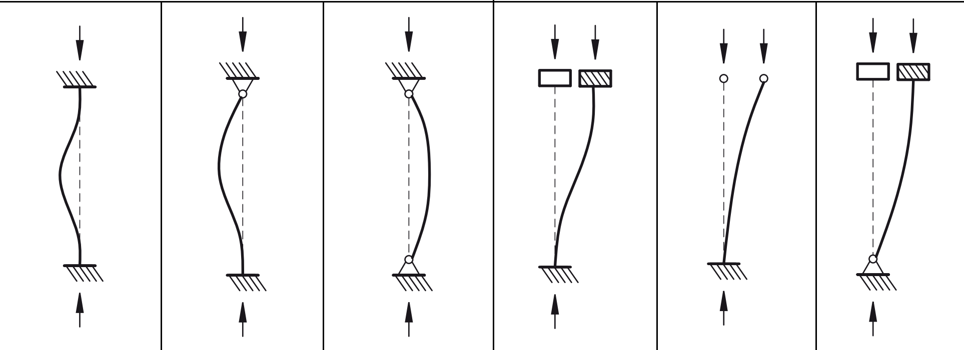
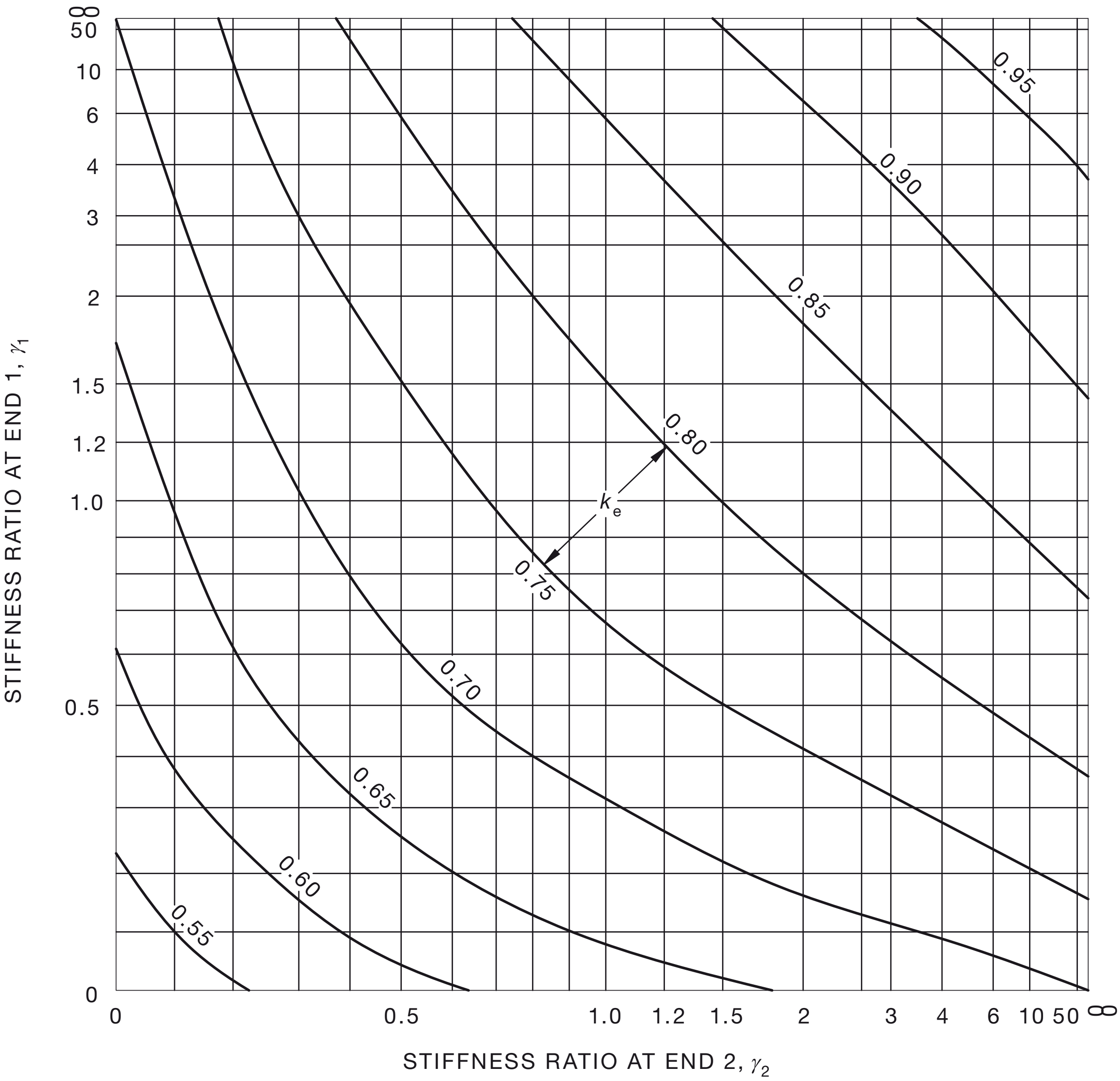
	Braced member			Sway member		
Buckled shape						
Effective length factor (k_e)	0.7	0.85	1.0	1.2	2.2	2.2
A1 Symbols for end restraint conditions	= Rotation fixed, translation fixed			= Rotation fixed, translation free		
	= Rotation free, translation fixed			= Rotation free, translation free		

FIGURE 4.6.3.2 EFFECTIVE LENGTH FACTORS FOR MEMBERS FOR IDEALIZED CONDITIONS OF END RESTRAINT

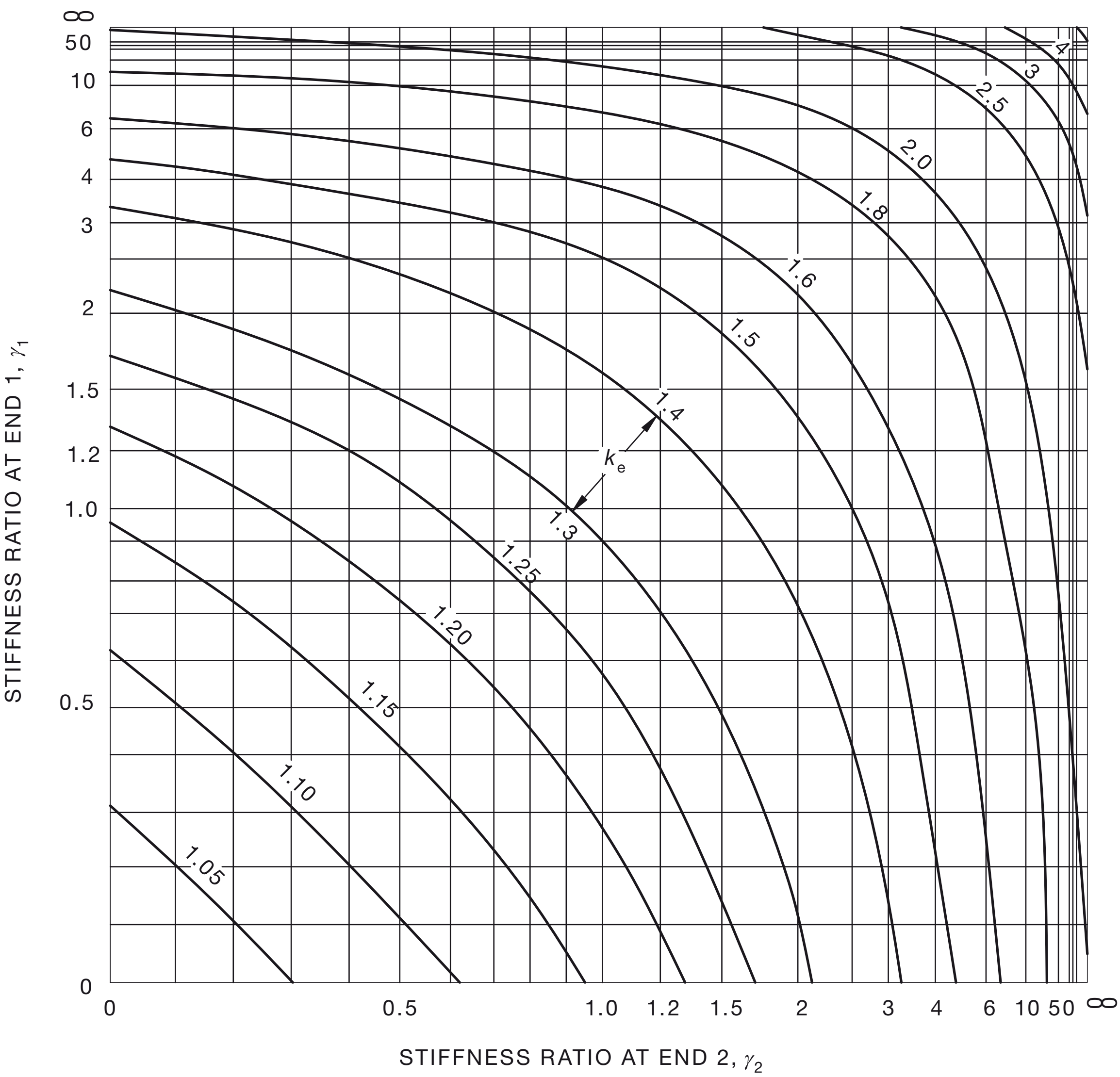
4.6.3.3 *Members in frames*

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, γ_1 and γ_2 are the ratios of the compression member stiffness to the end restraint stiffnesses. The γ- values shall be determined in accordance with Clause 4.6.3.4 or Appendix G, as appropriate.



(a) For braced members

FIGURE 4.6.3.3 (in part) EFFECTIVE LENGTH FACTORS



(b) For sway members

FIGURE 4.6.3.3 (in part) EFFECTIVE LENGTH FACTORS

4.6.3.4 *Stiffness ratios in rectangular frames*

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

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

except that—

(a) for a compression member whose base is not rigidly connected to a footing, the γ-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 γ-value shall not be taken as less than 0.6, unless a rational analysis would justify a different value.

The quantity $\Sigma(I/l)_c$ 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 $\Sigma\beta_e(I/l)_b$ 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 (β_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 (β_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 (λ_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 (λ_c) is used in the determination of the moment amplification factor for a sway member (δ_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 λ_c depends on the load set.

4.7.2 In-plane frame buckling

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

(a) one of the approximate methods of Clauses 4.7.2.1 and 4.7.2.2; or

(b) a rational elastic buckling analysis of the whole frame.

NOTE: The value of λ_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 (λ_m) for each column shall be determined as follows:

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

The elastic buckling load factor (λ_c) for the whole frame shall be taken as the lowest of all the λ_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 (λ_{ms}) for each storey shall be determined as follows:

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

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 (λ_c) for the whole frame shall be taken as the lowest of all the λ_{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 analyzed by the elastic method (see Clause 4.4) shall satisfy—

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

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

where

M_x^* = the design bending moment about the *x*-axis determined in accordance with Clause 4.4

ϕ = the capacity factor (see Table 3.4)

M_{sx} = the nominal section moment capacity, as specified in Clause 5.2, for bending about the *x*-axis

M_{bx} = the 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 analyzed by the elastic method (see Clause 4.4) shall satisfy—

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

where

M_y^* = the design bending moment about the *y*-axis determined in accordance with Clause 4.4.

M_{sy} = the nominal section moment capacity, as specified in Clause 5.2, for bending about the *y*-axis.

A member which is analyzed 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^* = the design bending moment determined in accordance with Clause 4.5, and

M_s = the nominal section moment capacity as specified in Clause 5.2.1

A member whose deflections are constrained to a non-principal plane shall be analyzed 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 analyzed 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 satisfy 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 as appropriate.

5.2.2 Section slenderness

For a section with flat compression plate elements, the section slenderness (λ_s) shall be taken as the value of the plate element slenderness (λ_e) for the element of the cross-section which has the greatest value of λ_e/λ_{ey}—

where

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

λ_{ey} = the plate element yield slenderness limit (see Table 5.2)

b = the 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 = the element thickness

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

For circular hollow sections, the section slenderness (λ_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 (λ_{sp}) and (λ_{sy}) respectively shall be taken as the values of the element slenderness limits (λ_{ep}) and (λ_{ey}) respectively given in Table 5.2.

TABLE 5.2

VALUES OF PLATE ELEMENT SLENDERNESS LIMITS

<table>
<tr><th>Plate element type</th><th>Longitudinal edges supported</th><th>Residual stresses (see Notes)</th><th>Plasticity limit (λ_{ep})</th><th>Yield limit (λ_{ey})</th><th>Deformation limit (λ_{ed})</th></tr>
<tr><td>Flat</td><td>One</td><td rowspan="2">SR
HR
LW,CF
HW</td><td rowspan="2">10
9
8
8</td><td rowspan="2">16
16
15
14</td><td rowspan="2">35
35
35
35</td></tr>
<tr><td colspan="2">(Uniform compression)</td></tr>
<tr><td>Flat</td><td>One</td><td rowspan="2">SR
HR
LW,CF
HW</td><td rowspan="2">10
9
8
8</td><td rowspan="2">25
25
22
22</td><td rowspan="2">—
—
—
—</td></tr>
<tr><td colspan="2">(Maximum compression at unsupported edge, zero stress or tension at supported edge)</td></tr>
<tr><td>Flat</td><td>Both</td><td rowspan="2">SR
HR
LW, CF
HW</td><td rowspan="2">30
30
30
30</td><td rowspan="2">45
45
40
35</td><td rowspan="2">90
90
90
90</td></tr>
<tr><td colspan="2">(Uniform compression)</td></tr>
<tr><td>Flat</td><td>Both</td><td rowspan="2">Any</td><td rowspan="2">82</td><td rowspan="2">115</td><td rowspan="2">—</td></tr>
<tr><td colspan="2">(Compression at one edge, tension at the other)</td></tr>
<tr><td colspan="2">Circular hollow sections</td><td>SR
HR, CF
LW
HW</td><td>50
50
42
42</td><td>120
120
120
120</td><td>—
—
—
—</td></tr>
</table>

NOTES:

1 SR—stress relieved
HR—hot-rolled or hot-finished
CF—cold formed
LW—lightly welded longitudinally
HW—heavily welded longitudinally

2 Welded members whose compressive residual stresses are less than 40 MPa may be considered 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 λ_{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 λ_{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 considered to have 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 considered to have 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) which divide the segment into a series of sub-segments may be considered to have 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 considered to have full lateral restraint, provided its length (l) satisfies—

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

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

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

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

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

where

A = area of cross-section

b_f, b_w = the flange width and web depth, respectively

b_1, b_2 = the greater and lesser leg lengths, respectively

d_f = the distance between flange centroids

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

I_y = the second moment of area of the section about the section minor principal y-axis

r_y = the radius of gyration about the minor principal y-axis

t = the thickness of an angle section

Z_e = the effective section modulus (see Clause 5.2)

ρ = I_{cy}/I_y

The ratio β_m shall be taken as one of the following as appropriate:

(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 considered 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 considered 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 considered 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).

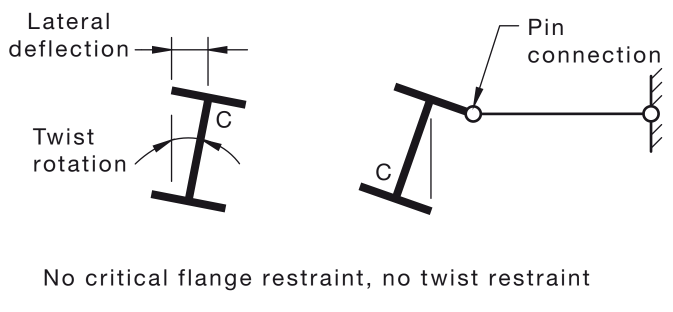


FIGURE 5.4.1 UNRESTRAINED CROSS-SECTIONS

5.4.2 Restraints at a cross-section

5.4.2.1 *Fully restrained*

A cross-section of a member may be considered 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).

(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).

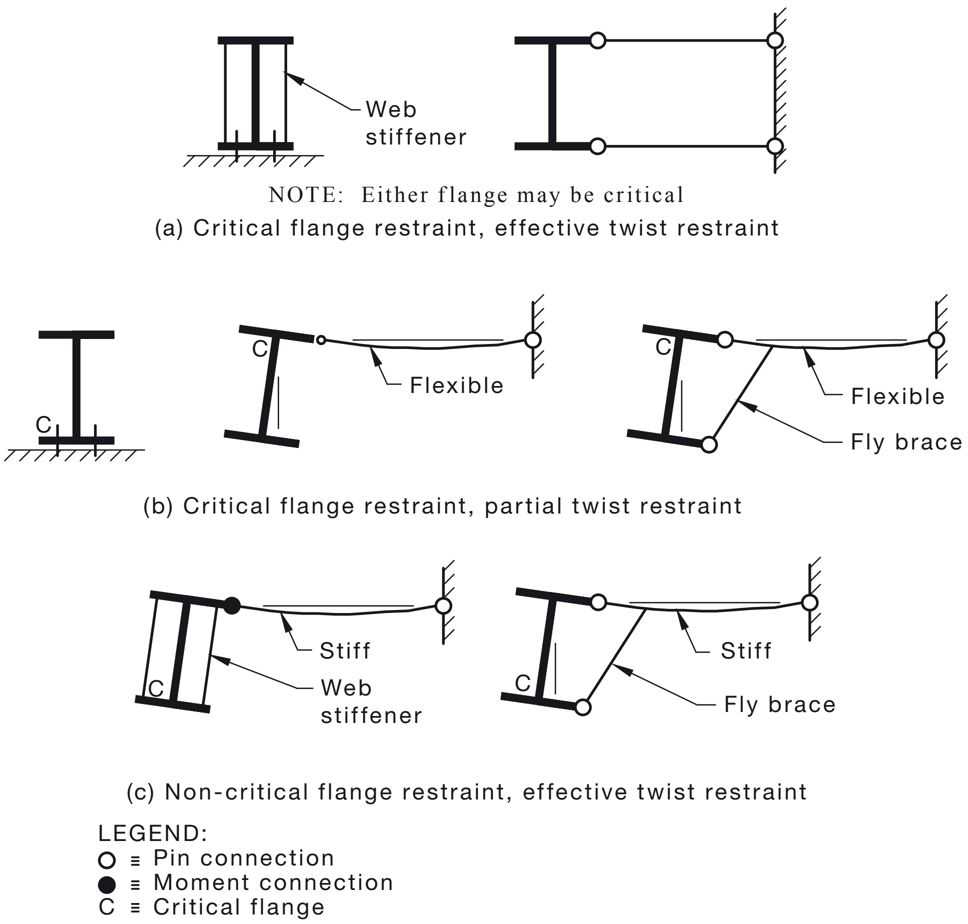
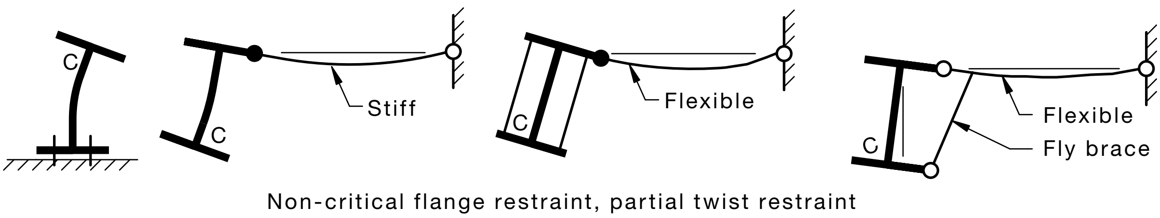


FIGURE 5.4.2.1 FULLY RESTRAINED CROSS-SECTIONS

5.4.2.2 *Partially restrained*

A cross-section of a member may be considered 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.

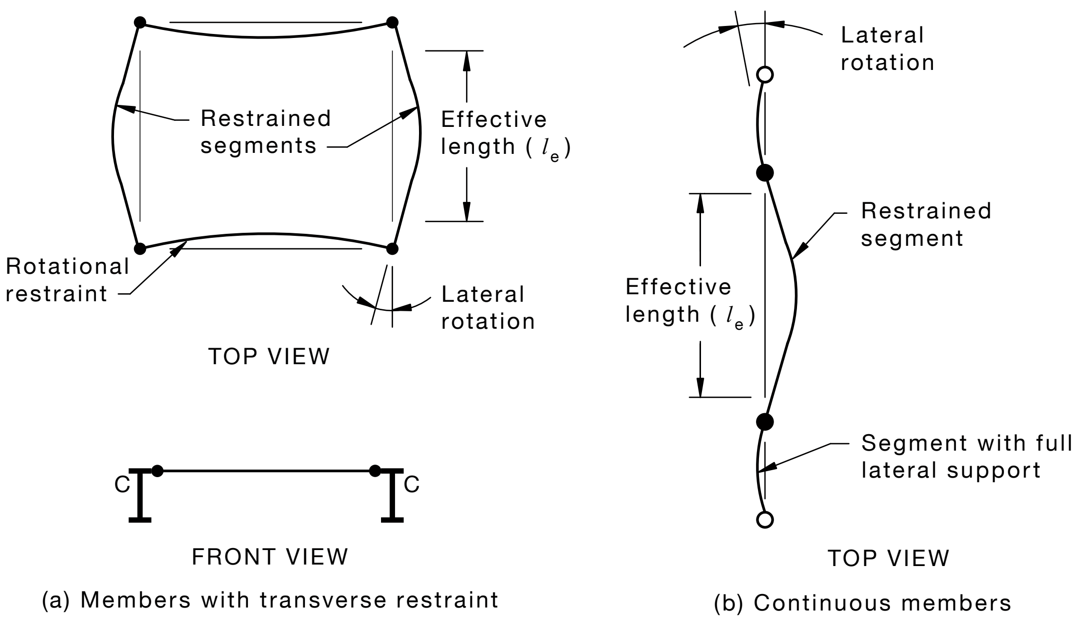


LEGEND:
O ≡ Pin connection
● ≡ Moment connection
C ≡ Critical flange

FIGURE 5.4.2.2 PARTIALLY RESTRAINED CROSS-SECTIONS

5.4.2.3 *Rotationally restrained*

A cross-section of a member which may be considered to be fully or partially restrained may be considered 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.

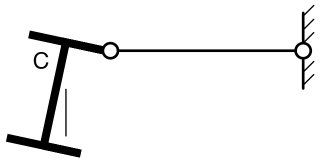


LEGEND:
O ≡ Pin connection
● ≡ Moment connection
C ≡ Critical flange

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 considered 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 considered to be laterally restrained.



Critical flange restraint, no twist restraint

LEGEND:
O ≡ 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 ϕM_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 will ensure that M^* equals ϕM_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 deemed 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 deemed 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 deemed 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 Paragraph H5.1 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 Paragraph H5.2 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 which 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 farther 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 be considered:

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

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

where

α_m = a moment modification factor

α_s = a slenderness reduction factor

M_s = the nominal section moment capacity determined in accordance with Clause 5.2 for the gross section

The moment modification factor (α_m) shall be taken as one of the following:

(i) 1.0;

(ii) a value obtained from Table 5.6.1;

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

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 (α_s) shall be determined as follows:

$$\alpha_s = 0.6\left[\sqrt{\left[\left(\frac{M_s}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_s}{M_{oa}}\right)\right] \qquad \text{. . . 5.6.1.1(2)}$$

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:

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

where

E, G = the elastic moduli (see Clause 1.4)

I_y, J, and I_w = section constants (see Clause 1.4)

l_e = the 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 Paragraph H4 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 either—

(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 (α_{st}) as follows:

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

where

r_r = l_r/l for stepped members

= 0.5 for tapered members

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

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; or

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

(b) the method of design by buckling analysis (see Clause 5.6.4).

The monosymmetry section constant (β_x) shall be determined using either—

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

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) $$\beta_x = \frac{1}{I_x}\int\left(x^2 y + y^3\right) dA - 2 y_o$$

where y_o is the coordinate of the shear centre (see Reference H6.11 of Appendix H). The values of β_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 equals 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 equals 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_{oa} equal to the value of M_o obtained from Equation 5.6.1.1(3), and the appropriate value of α_m given in Table 5.6.2; or

(ii) $M_b = \alpha_s M_s \leq M_s$

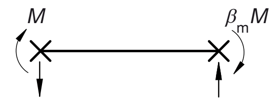
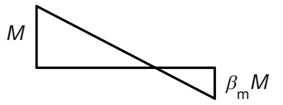
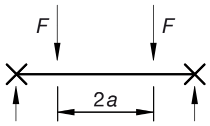
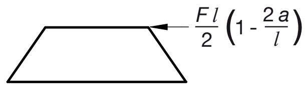
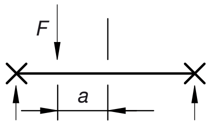
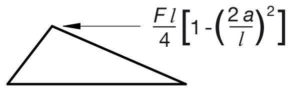
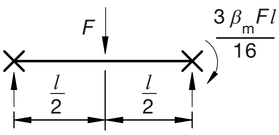
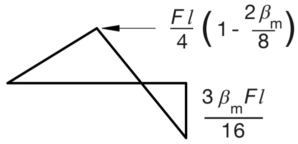
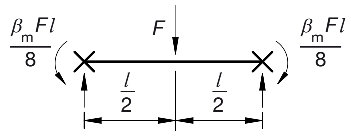
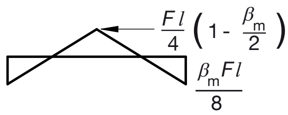
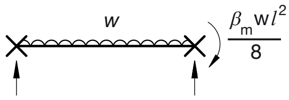
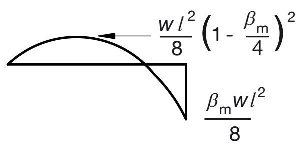
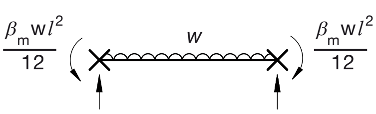
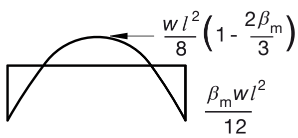
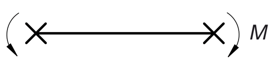
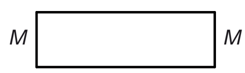
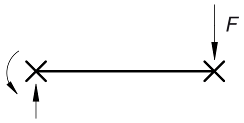
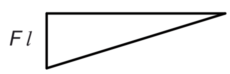
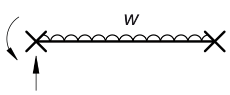
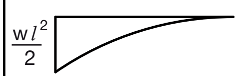
where the slenderness reduction factor (α_s) shall be determined as follows:

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

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 (α_m) FOR SEGMENTS FULLY OR PARTIALLY RESTRAINED AT BOTH ENDS

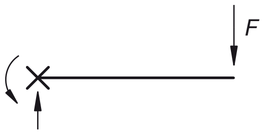
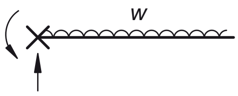
Beam segment	Moment distribution	Moment modification factor, α_m	Range
		$1.75+1.05\beta_m+0.3\,\beta_m^2$ 2.5	$-1 \leqslant \beta_m \leqslant 0.6$ $0.6 < \beta_m \leqslant 1$
		$1.0+0.35\left(1-\frac{2a}{l}\right)^2$	$0 \leqslant \frac{2a}{l} \leqslant 1$
		$1.35+0.4\left(\frac{2a}{l}\right)^2$	$0 \leqslant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

A1

NOTE: ✕ ≡ full or partial restraint

TABLE 5.6.2

MOMENT MODIFICATION FACTORS (α_m) FOR SEGMENTS UNRESTRAINED AT ONE END

Member segment	Moment distribution	Moment modification factor, α_m
		0.25
		1.25
		2.25

NOTE: × ≡ Full or partial restraint

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 = a twist restraint factor given in Table 5.6.3(1)

k_l = a load height factor given in Table 5.6.3(2)

k_r = a lateral rotation restraint factor given in Table 5.6.3(3)

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

(a) the segment length, for segments without intermediate restraints, or for segments unrestrained at one end, with or without intermediate lateral restraints; or

(b) 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(1)

TWIST RESTRAINT FACTORS (k_t)

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

TABLE 5.6.3(2)

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 FU, PU	1.0 1.0	1.4 2.0
At segment end	FF, FP, FL, PP, PL, LL FU, PU	1.0 1.0	1.0 2.0

TABLE 5.6.3(3)

LATERAL ROTATION RESTRAINT FACTORS (k_r)

Restraint arrangement	Ends with lateral rotation restraints (see Clause 5.4.3.4)	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(1), 5.6.3(2) and 5.6.3(3),

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 (α_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 $\alpha_m = \dfrac{M_{os}}{M_{oo}}$

where

M_{os} = the 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} = the 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 Paragraph H6.

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 considered 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 in pairs where the design compressive bearing forces applied through a flange by loads or reactions exceed the design bearing capacity (ϕR_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) $\left(\dfrac{d_1}{200}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $1.0 \leq s/d_1 \leq 3.0$;

(b) $\left(\dfrac{s}{200}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $0.74 < s/d_1 \leq 1.0$; or

(c) $\left(\dfrac{d_1}{270}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $s/d_1 \leq 0.74$.

All web lengths for which s/d_p is greater than 3.0 shall be considered 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) $\left(\dfrac{d_1}{250}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $1.0 \leq s/d_1 \leq 2.4$;

(b) $\left(\dfrac{s}{250}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $0.74 \leq s/d_1 \leq 1.0$; or

(c) $\left(\dfrac{d_1}{340}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ when $s/d_1 < 0.74$.

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—

$\left(\dfrac{d_1}{400}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}$ 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 (ϕV_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 (λ_s) as defined in Clause 5.2.2 using the stiffener yield stress (f_{ys}) shall be less than the plasticity limit (λ_{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

ϕ = the capacity factor (see Table 3.4)

V_v = the nominal shear capacity of the web determined from either Clause 5.11.2 or Clause 5.11.3

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—

$$\frac{d_p}{t_w} \leq \frac{82}{\sqrt{\frac{f_y}{250}}}$$

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—

$$\frac{d_p}{t_w} > \frac{82}{\sqrt{\frac{f_y}{250}}}$$

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 = the nominal shear capacity of a web with a uniform shear stress distribution determined in accordance with Clause 5.11.2

f^*_{vm}, f^*_{va} = the 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:

$$V_w = 0.6 f_y A_w$$

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:

$$V_w = 0.36 f_y A_e$$

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

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

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

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

α_d = 1.0 when required by Clause 5.15.2.2, and

d_p = the 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 (α_f) shall be taken as either—

(a) $\alpha_f = 1.0$; or

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

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

(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	0.998	0.946	0.907	0.877	0.833	0.803	0.672
110	1.000	1.000	1.000	0.989	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} = the lesser of the flange effective areas, determined using Clause 6.2.2 for the compression flange and the lesser of A_{fg} and $0.85 A_{fn} f_u / f_y$ for the tension flange

A_{fg} = the gross area of the flange

A_{fn} = the net area of the flange

d_f = the 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

$$V_{vm} = V_v \quad \text{for } M^* \leq 0.75\phi M_s\text{; or}$$

$$= V_v\left[2.2 - \left(\frac{1.6M^*}{\phi M_s}\right)\right] \text{for } 0.75\phi M_s \leq M^* \leq \phi M_s;$$

where

V_v = the nominal shear capacity of a web in shear alone (see Clause 5.11.1)

M_s = the 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 considered as 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
A1 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

ϕ = 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
A1 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))

α_p = coefficient used to calculate the nominal bearing yield capacity (R_{by}) for square
A1 and rectangular hollow sections to AS/NZS 1163

The coefficient (α_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))

α_{pm} = coefficient used to calculate α_p

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

k_s = ratio used to calculate α_p and α_{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 $1.5d_5$—

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

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

A1 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:

(a) $\alpha_b = 0.5$.

(b) $k_f = 1.0$.

(c) area of web = $t_w b_b$.

(d) 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.

(e) 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

A1 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

ϕ = 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

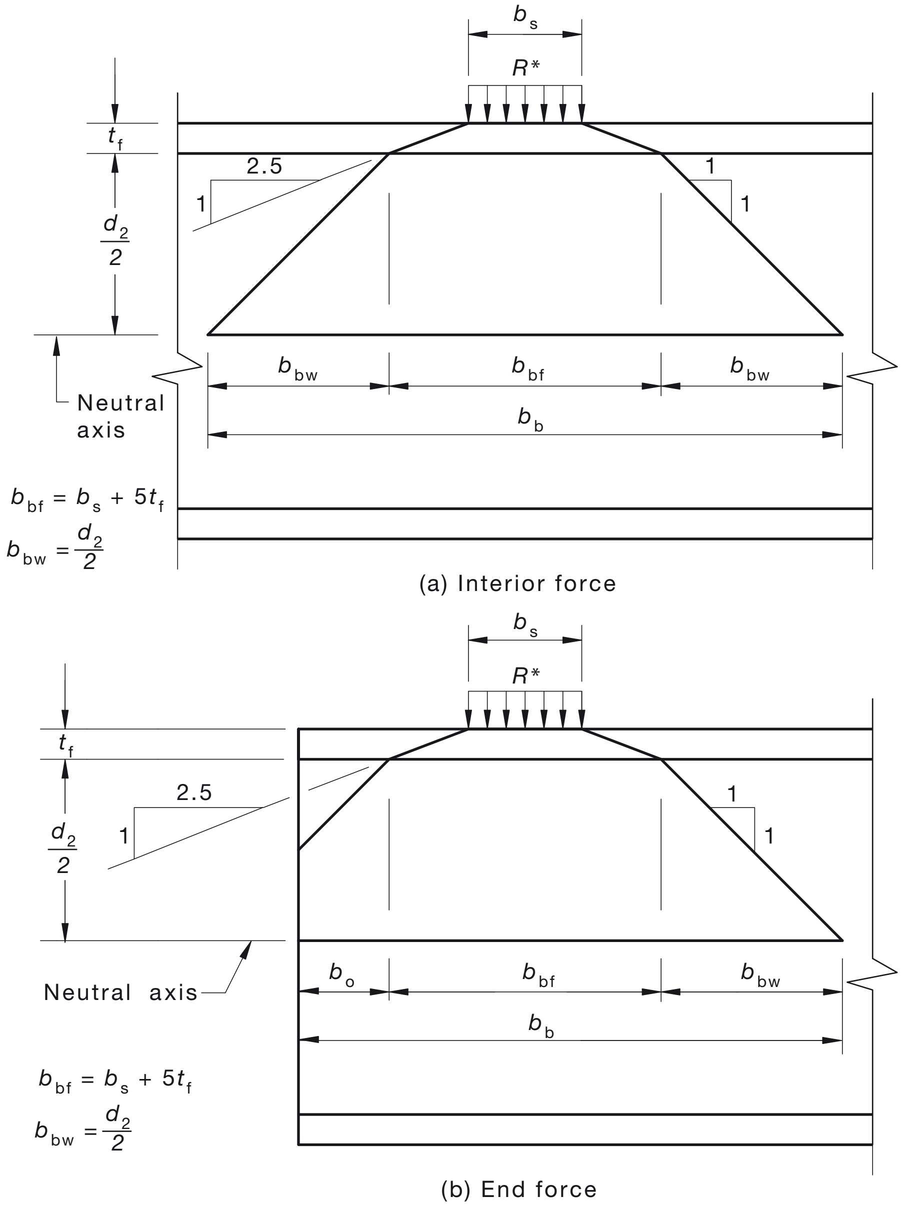


FIGURE 5.13.1.1 DISPERSIONS OF FORCE THROUGH FLANGE AND WEB

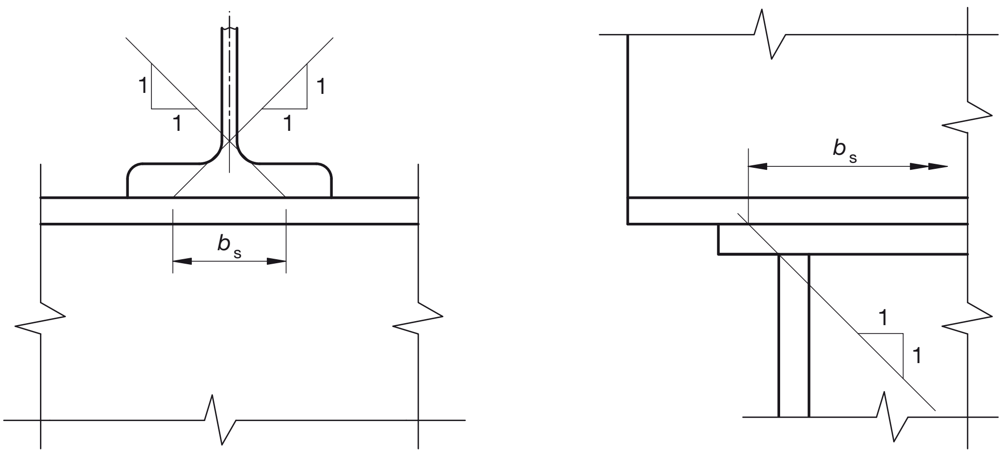


FIGURE 5.13.1.2 STIFF BEARING LENGTH ON FLANGE

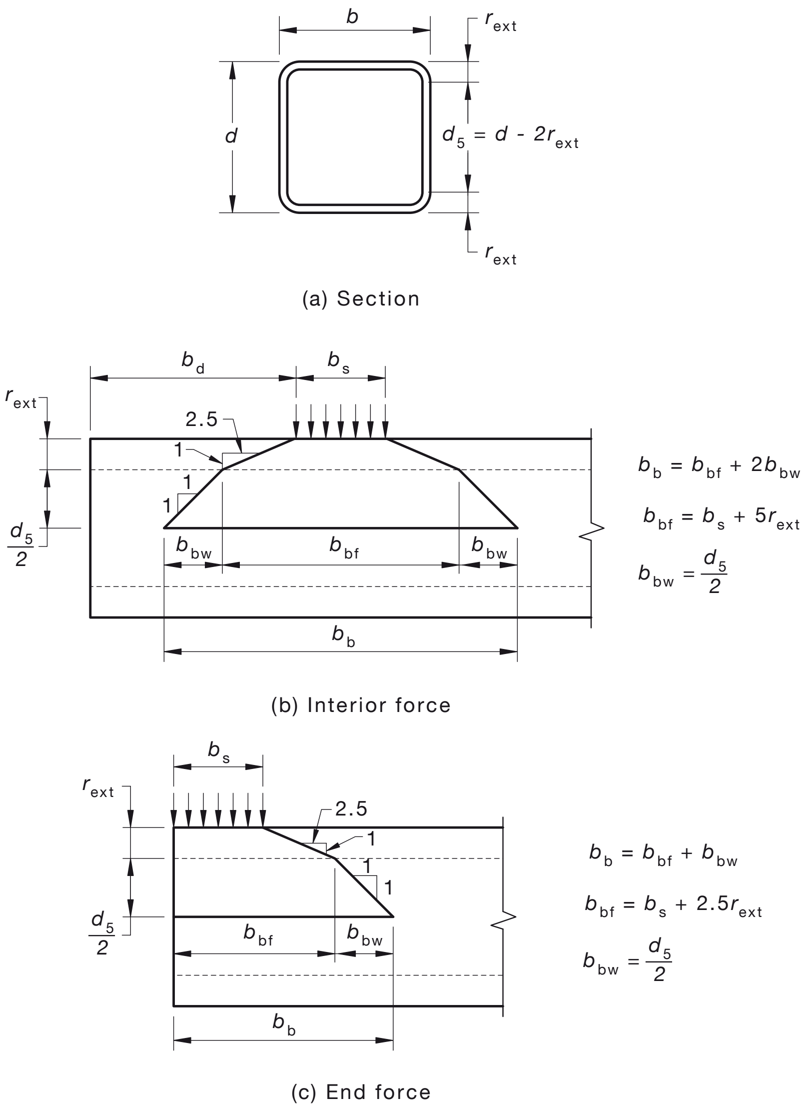


FIGURE 5.13.1.3 RECTANGULAR AND SQUARE HOLLOW SECTIONS—DISPERSION OF FORCE THROUGH FLANGE, RADIUS AND WEB

5.14 DESIGN OF LOAD BEARING STIFFENERS

5.14.1 Yield capacity

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

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

where

R^* = the design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener

ϕ = the capacity factor (see Table 3.4)

R_{sy} = the nominal yield capacity of the stiffened web

= $R_{by} + A_s f_{ys}$

R_{by} = the nominal bearing yield capacity (see Clause 5.13.3)

A_s = the area of the stiffener in contact with the flange

f_{ys} = the yield stress of the stiffener

5.14.2 Buckling capacity

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

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

where

ϕ = the capacity factor (see Table 3.4)

R_{sb} = the nominal buckling capacity of the stiffened web, determined in accordance with Section 6 using α_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—

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

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

$$l_e = 0.7 d_1$$

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—

$$b_{es} \le \frac{15 t_s}{\sqrt{\frac{f_{ys}}{250}}}$$

where

t_s = the thickness of the stiffener

f_{ys} = the 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—

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

where

$$\alpha_t = \frac{230}{l_e / r_y} - 0.60 \text{ and } 0 \leq \alpha_t \leq 4$$

R^* = the design reaction at the bearing

F^* = the total design load on the member between supports

t_f = the thickness of the critical flange (see Clause 5.5)

(l_e/r_y)= the 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 (s) of intermediate web stiffeners which 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 (s) of the end panel is reduced so that its shear buckling capacity (V_b) calculated by using α_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—

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

where

α_v = the value determined in accordance with Clause 5.11.5.2

γ = 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

ϕ = capacity factor (see Table 3.4)

V_b = nominal shear buckling capacity specified in Clause 5.11.5.2 for a stiffened web using α_d equals 1.0 and α_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—

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

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

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} is the outstand width of the stiffener from the face of the web, in millimetres, and t_w is the 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—

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

where

α_v = is given in Clause 5.11.5.2

V_w = is 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—

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

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—

$N^* \leq \phi N_s$, and

$N^* \leq \phi N_c$

where

ϕ = the capacity factor (see Table 3.4)

N_s = the nominal section capacity determined in accordance with Clause 6.2

N_c = the 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 = the form factor given in Clause 6.2.2

A_n = the 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 = the effective area

A_g = the 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 (λ_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 = the 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 the supporting plate elements

t = the thickness of the plate

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

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

where

d_o = the outside diameter of the section

t = the 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 (λ_e) given in Clause 6.2.3 and the element yield slenderness limit (λ_{ey}) given in Table 6.2.4.

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

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

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

$$d_e = d_o \sqrt{\left(\frac{\lambda_{ey}}{\lambda_e}\right)} \le d_o, \text{ and}$$

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

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

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

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 Notes)	Yield slenderness limit, λ_{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

NOTES:

1 SR—stress relieved

HR—hot-rolled or hot-finished

CF—cold-formed

LW—lightly welded longitudinally

HW—heavily welded longitudinally

2 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 The 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 The 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.

A1 ### 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 = the nominal section capacity, determined in accordance with Clause 6.2

α_c = the member slenderness reduction factor

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

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

λ = $\lambda_n + \alpha_a\alpha_b$

η = $0.00326(\lambda - 13.5) \geq 0$

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

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

α_b = the appropriate member section constant given in Table 6.3.3(1) or 6.3.3(2)

k_f = the form factor determined in accordance with Clause 6.2.2

Alternatively, values of the member slenderness reduction factor (α_c) may be obtained directly from Table 6.3.3(3) using the value of the modified member slenderness (λ_n) and the appropriate member section constant (α_b) given in Table 6.3.3(1) or 6.3.3(2).

A1 Fabricated monosymmetric and non-symmetric sections other than unlipped angles, tees and cruciform sections, and hot-rolled channels braced about the minor principal axis, shall be designed for flexural torsional buckling according to 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(1)

VALUES OF MEMBER SECTION CONSTANT (α_b) FOR k_f = 1.0

Compression member section constant, α_b	Section description
−1.0	— Hot-formed RHS and CHS — Cold-formed (stress relieved) RHS and CHS
−0.5	— Cold-formed (non-stress relieved) RHS and CHS — Welded H, I and box section fabricated from Grade 690 high strength quenched and tempered plate
0	— Hot-rolled UB and UC sections (flange thickness up to 40 mm) — Welded H and I sections fabricated from flame-cut plates — Welded box sections
0.5	— Tees flame-cut from universal sections, and angles — Hot-rolled channels — Welded H and I sections fabricated from as-rolled plates (flange thickness up to 40 mm) — Other sections not listed in this Table
1.0	— Hot-rolled UB and UC sections (flange thickness over 40 mm) — Welded H and I sections fabricated from as-rolled plates (flange thickness over 40 mm)

A1

TABLE 6.3.3(2)

VALUES OF MEMBER SECTION CONSTANT (α_b) FOR k_f < 1.0

Compression member section constant, α_b	Section description
−0.5	— Hot-formed RHS and CHS — Cold-formed RHS and CHS (stress relieved) — Cold-formed RHS and CHS (non-stress relieved)
0	— Hot-rolled UB and UC sections (flange thickness up to 40 mm) — Welded box sections
0.5	— Welded H and I sections (flange thickness up to 40 mm)
1.0	— Other sections not listed in this Table

TABLE 6.3.3(3)

VALUES OF MEMBER SLENDERNESS REDUCTION FACTOR (α_c)

Modified member slenderness, λ_n	Compression member section constant, α_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	0.989	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
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
A1 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

(*continued*)

TABLE 6.3.3(3) (*continued*)

Modified member slenderness, λ_n	Compression member section constant, α_b				
	−1.0	**−0.5**	**0**	**0.5**	**1.0**
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
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—

(a) the nominal section capacity (N_s) is the minimum value for all cross-sections along the length of the member; and

(b) the modified member slenderness (λ_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:

$$V^* = \frac{\pi\left(\frac{N_s}{N_c} - 1\right)N^*}{\lambda_n} \geq 0.01N^*$$

where

N_s = the nominal section capacity of the compression member given by Clause 6.2.1

N_c = the nominal member capacity of the compression member given by Clause 6.3.3

N^* = the design axial force applied to the compression member

λ_n = the modified member slenderness

The modified member slenderness (λ_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° to 70° for single lacing.

(b) 40° to 50° 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

$\left(\frac{l_e}{r}\right)_m$ = the slenderness ratio of the whole member about the above axis calculated by assuming that the main components act as an integral member

$\left(\frac{l_e}{r}\right)_c$ = the 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. In this case, the edge stiffeners shall 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:

$$M^* = \frac{V^* s_b}{2 n_b}$$

where

V^* = the design transverse shear force specified in Clause 6.4.1

s_b = the longitudinal centre-to-centre distance between the battens

n_b = the number of parallel planes of battens

d_b = the 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 comply with 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:

$$V_l^* = 0.25 V^* \left(\frac{l_e}{r}\right)_c$$

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 comply with 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 which will ensure that N^* equals ϕN_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

ϕ = the capacity factor, see Table 3.4

N_t = the 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 = the gross area of the cross-section

f_y = the yield stress used in design

k_t = the correction factor for distribution of forces determined in accordance with Clause 7.3

A_n = the 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 = the 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 comply with 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.

7.4 TENSION MEMBERS WITH TWO OR MORE MAIN COMPONENTS

7.4.1 General

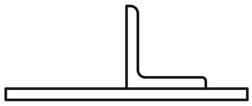
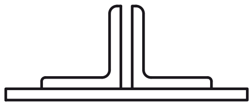
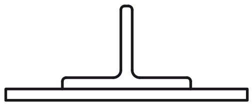
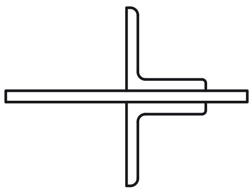
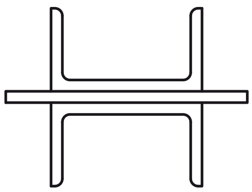
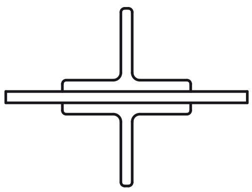
A tension member composed of two or more main components intended to act as a single member shall comply with Clauses 7.4.2 to 7.4.5.

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 considered as divided equally among the connection planes parallel to the direction of force.

TABLE 7.3.2

CORRECTION FACTOR (k_t)

Configuration case		Correction factor, k_t
(i)		0.75 for unequal angles connected by the short leg 0.85 otherwise
(ii)		As for Case (i)
(iii)		0.85
(iv)		0.90
(v)		1.0
(vi)		1.0
(vii)		1.0

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 comply with 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 does not exceed 300; or

 (ii) by connections which comply with 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 comply with 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 comply with 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 9.5. A pin connection in a tension member shall comply with 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° 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 (λ_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 (λ_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} = the nominal section moment capacities about the x- and y-axes respectively, determined in accordance with Clause 5.2

N_s = the 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

ϕ = the capacity factor (see Table 3.4)

M_{rx} = the nominal section moment capacity, reduced by axial force (tension or compression)

$$= M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)$$

A1 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 as appropriate:

(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—

$$M_{rx} = M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)\left[1 + 0.18\left(\frac{82 - \lambda_w}{82 - \lambda_{wy}}\right)\right] \leq M_{sx}$$

λ_w and λ_{wy} are the values of λ_e and λ_{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

ϕ = the capacity factor (see Table 3.4)

M_{ry} = the nominal section moment capacity, reduced by the axial tensile or compressive force

$$= M_{sy}\left[1-\frac{N^*}{\phi N_s}\right]$$

Alternatively, M_{ry} may be calculated by one of the following as appropriate:

(a) For doubly symmetric I-sections which are compact, as defined in Clause 5.2.3—

$$M_{ry} = 1.19 M_{sy}\left[1-\left(\frac{N^*}{\phi N_s}\right)^2\right] \leq M_{sy}$$

A1 (b) For rectangular or square hollow sections to AS/NZS 1163 which are compact, as defined in Clause 5.2.3—

$$M_{ry} = 1.18 M_{sy}\left[1-\left(\frac{N^*}{\phi N_s}\right)\right] \leq M_{sy}$$

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
A1 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 analyzed elastically, or Clause 8.4.3 for a frame analyzed 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 analyzed 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^* = the design bending moment about the principal axis

ϕ = the capacity factor (see Table 3.4)

M_i = the nominal in-plane member moment capacity

$$= M_s\left(1-\frac{N^*}{\phi N_c}\right)$$

M_s = the nominal section moment capacity determined in accordance with Clause 5.2 for bending about the same principal axis as the design bending moment

N^* = the design axial compressive force

N_c = the 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

A1 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:

$$M_i = M_s\left\{\left[1-\left(\frac{1+\beta_m}{2}\right)^3\right]\left(1-\frac{N^*}{\phi N_c}\right)+1.18\left(\frac{1+\beta_m}{2}\right)^3\sqrt{\left(1-\frac{N^*}{\phi N_c}\right)}\right\}$$

$$\leq M_{rx} \text{ or } M_{ry} \text{ as appropriate}$$

where

β_m = the 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

= the value determined in accordance with Clause 4.4.2.2 for members with transverse load

M_{rx} or M_{ry} = the 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:

$$\frac{N^*}{\phi N_s} \le \left[\frac{0.60 + 0.40\beta_m}{\sqrt{(N_s / N_{ol})}}\right]^2 \quad \text{when } \frac{N^*}{\phi N_s} \le 0.15,$$

and

$$\frac{N^*}{\phi N_s} \le \frac{1 + \beta_m - \sqrt{(N_s / N_{ol})}}{1 + \beta_m + \sqrt{(N_s / N_{ol})}} \quad \text{when } \frac{N^*}{\phi N_s} > 0.15,$$

where

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

N_s = the nominal section capacity in axial compression determined in accordance with Clause 6.2

N_{ol} = $\dfrac{\pi^2 EI}{l^2}$

I = the second moment of area for a the axis about which the design moment acts

l = the 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 analyzed 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 analyzed structure to satisfy the requirements of Clause 8.4.2.

8.4.3.4 *Plastic moment capacity*

The design plastic moment capacity (ϕM_{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

ϕ = the capacity factor (see Table 3.4)

M_{ox} = the 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 (α_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} = the nominal member moment capacity without full lateral restraint and with a uniform distribution of design bending moment so that α_m is unity, determined in accordance with Clause 5.6

N_{cy} = the nominal member capacity in axial compression, determined in accordance with Clause 6.3 for buckling about the minor principal y-axis

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

N_{oz} = the 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 = the elastic moduli

A, I_w, I_x, I_y and J = the section constants

l_z = the 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

ϕ = the capacity factor (see Table 3.4)

A1 M_{ox} = the nominal out-of-plane member moment capacity

$$= M_{bx}\left(1+\frac{N^*}{\phi N_t}\right) \leq M_{rx}$$

M_{bx} = the nominal member moment capacity defined in Clause 8.4.4.1

N_t = the nominal section capacity in axial tension determined in accordance with Clause 7.2

M_{rx} = the nominal section moment capacity reduced by axial force determined in accordance with Clause 8.3.2

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:

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} \leq 1$$

where

ϕ = the capacity factor (see Table 3.4)

M_{cx} = the 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 Clauses 8.4.2 and 8.4.4 respectively

M_{iy} = the nominal in-plane member moment capacity, determined in accordance with Clause 8.4.2, 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:

$$\left(\frac{M_x^*}{\phi M_{tx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{ry}}\right)^{1.4} \leq 1$$

where

ϕ = the capacity factor (see Table 3.4)

M_{tx} = the 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} = the 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

Single angle web compression members in trusses which are connected with at least two bolts or welded at their ends and loaded through one leg (see Figure 8.4.6) shall be designed to satisfy Clause 8.3 and either Clause 8.4.5 or the following:

$$\frac{N^*}{\phi N_{ch}} + \frac{M_h^*}{\phi M_{bx} \cos \alpha} \leq 1$$

where

N^* = the design axial compression force in the member

M_h^* = the design bending moment acting about the rectangular h-axis parallel to the loaded leg

ϕ = the capacity factor (see Table 3.4)

N_{ch} = the 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} = the 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 α_m appropriate to the distribution of design bending moment along the member

α = the 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} = the nominal section moment capacity about the x-principal axis, determined in accordance with Clause 5.2

l = the member length

t = the 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).

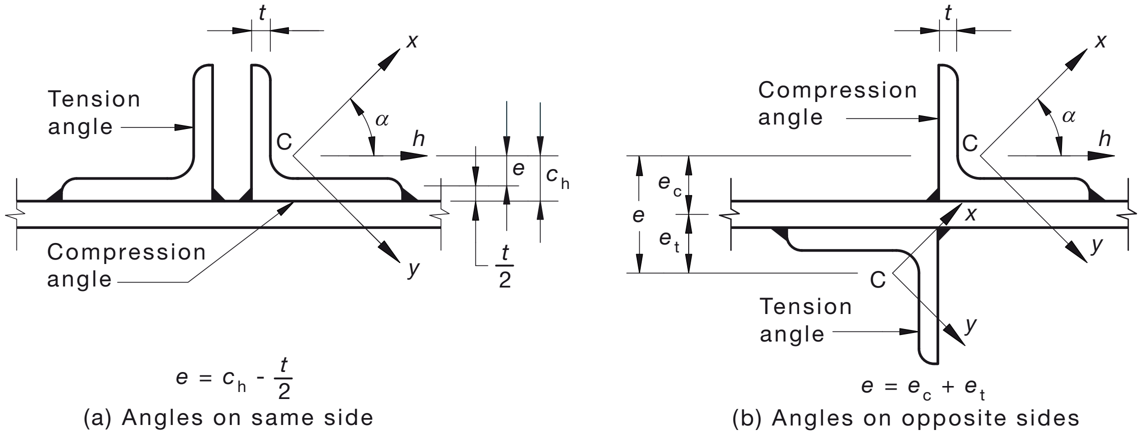


FIGURE 8.4.6 SINGLE ANGLES LOADED THROUGH ONE LEG

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 comply with 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 comply with 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 comply with 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 comply with 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 considered as subject to reaction shear forces acting at an eccentricity appropriate to the connection detailing.

9.1.2.4 *Connections in structures analyzed by the plastic method*

Connections in structures analyzed by the plastic method shall comply with 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 comply with Section 13.

Connections and the adjacent areas of members shall be designed by distributing the design action effects so that they comply with 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

A1 (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 in accordance with Clause 14.4.4.2, 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 hold all parts securely in place. The fasteners shall be sufficient 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

$$M^* = \frac{\delta N^* l_s}{1000}$$

δ = appropriate amplification factor δ_b or δ_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 (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 comply with 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 considered 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), 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), 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 carried 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 carried 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

A1 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 5.11.3.

(b) Connection components subject to tension—using Clause 7.2.

(c) Connection components subject to compression—using Clauses 6.2.1 and 6.3.3.

(d) Connection components subject to bending—using Clause 5.2.1.

A1 (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

ϕ = 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 ($s_p^2 t / 4 s_g$) 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(1))

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(1)). 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(2))

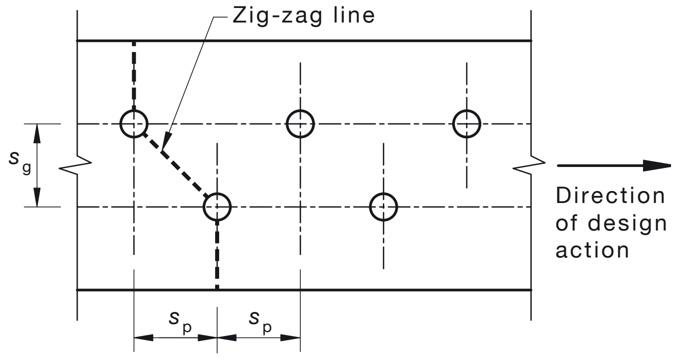


FIGURE 9.1.10.3(1) STAGGERED HOLES

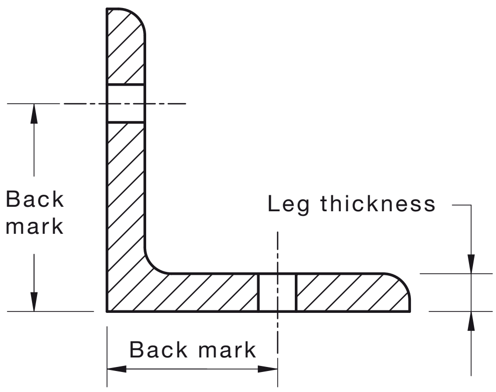


FIGURE 9.1.10.3(2) ANGLES WITH HOLES IN BOTH LEGS

9.1.11 Hollow section connections

When design actions from one member are applied to a hollow section at a connection, consideration shall be given to the local effects on the hollow section.

9.2 DEFINITIONS

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

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.

Friction-type connection—connection effected using high-strength bolts tightened to induce a specified minimum bolt tension so 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.

Full tensioning—a method of installing and tensioning a bolt in accordance with Clauses 15.2.4 and 15.2.5.

In-plane loading—loading for which the design forces and bending moments are in the plane of the connection, such that the design action effects induced in the connection components are shear forces only.

Non-slip fasteners—fasteners which 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.

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.

Pin—an unthreaded fastener manufactured out of round bar.

Prying force—additional tensile force developed as a result of the flexing of a connection component in a connection subjected to tensile force. External tensile 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.

Snug tight—the tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

9.3 DESIGN OF BOLTS

9.3.1 Bolts and bolting category

The bolts and bolting categories listed in Table 9.3.1 shall be designed in accordance with this Clause and Clause 9.4.

A1 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 9.4.

A1

TABLE 9.3.1

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, AS 1110 (series)	8.8	Snug tight	830
8.8/TB	AS/NZS 1252	8.8	Full tensioning	830
8.8/TF (see Note 1)	AS/NZS 1252	8.8	Full tensioning	830

NOTES:

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).

2 f_{uf} is the minimum tensile strength of the bolt as specified in AS 4291.1—2000, except for grade 8.8 bolts less than 16 mm diameter where the minimum tensile strength is 800 MPa.

3 Bolts to AS 1110 (series) and AS 1111 (series) are not suitable for full tensioning.

9.3.2 Bolt strength limit states

9.3.2.1 *Bolt in shear*

A bolt subject to a design shear force (V_f^*) shall satisfy—

$$V_f^* \leq \phi V_f$$

where

ϕ = 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:

$$V_f = 0.62 f_{uf} k_r (n_n A_c + n_x A_o)$$

where

f_{uf} = minimum tensile strength of the bolt as specified in the relevant Standard (see Table 9.3.1)

k_r = reduction factor given in Table 9.3.2.1 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 9.3.2.1

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

9.3.2.2 *Bolt in tension*

A bolt subject to a design tension force (N^*_{tf}) shall satisfy—

$$N^*_{tf} \leq \phi N_{tf}$$

where

ϕ = 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.

9.3.2.3 *Bolt subject to combined shear and tension*

A bolt required to resist both design shear $\left(V^*_f\right)$ and design tensile forces $\left(N^*_{tf}\right)$ 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

ϕ = capacity factor (see Table 3.4)

V_f = nominal shear capacity calculated in accordance with Clause 9.3.2.1

N_{tf} = nominal tensile capacity calculated in accordance with Clause 9.3.2.2

9.3.2.4 *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

ϕ = 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:

$$V_b = 3.2 d_f t_p f_{up} \qquad \text{. . . 9.3.2.4(1)}$$

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 9.3.2.4(1) and that given by Equation 9.3.2.4(2)—

$$V_b = a_e t_p f_{up} \qquad \text{. . . 9.3.2.4(2)}$$

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

9.3.2.5 *Filler plates*

A1 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 9.3.2.1 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.

9.3.3 Bolt serviceability limit state

9.3.3.1 *Design*

For friction-type connections (bolting category 8.8/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

ϕ = 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:

$$V_{sf} = \mu n_{ei} N_{ti} k_h \qquad \text{. . . 9.3.3.1}$$

where

μ = slip factor as specified in Clause 9.3.3.2

n_{ei} = number of effective interfaces

N_{ti} = minimum bolt tension at installation as specified in Clause 15.2.5.1

k_h = factor for different hole types, as specified in Clause 14.3.5.2

= 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 9.3.2.

9.3.3.2 *Contact surfaces*

Where the surfaces in contact are clean 'as-rolled' surfaces, the slip factor (μ) 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 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 (see Clause 14.3.6.3).

9.3.3.3 *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

ϕ = capacity factor (see Clause 3.5.5)

V_{sf} = nominal shear capacity of the bolt as specified in Clause 9.3.3.1

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}

A1 | where N_{ti} is the minimum bolt tension at installation as specified in Table 15.2.5.1.

The strength limit state shall also be separately assessed in accordance with Clause 9.3.2.3.

9.4 ASSESSMENT OF THE STRENGTH OF A BOLT GROUP

9.4.1 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 considered 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 9.3.2.1 using the capacity factor (ϕ) for a bolt group (see Table 3.4) and the ply in bearing shall satisfy Clause 9.3.2.4.

9.4.2 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 comply with Clauses 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the capacity factor (ϕ) for a bolt group (see Table 3.4), and the ply in bearing shall comply with Clause 9.3.2.4.

9.4.3 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 9.4.1 and 9.4.2.

Each bolt shall comply with Clauses 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the capacity factor (ϕ) for a bolt group (see Table 3.4), and the ply in bearing shall comply with Clause 9.3.2.4.

9.5 DESIGN OF A PIN CONNECTION

9.5.1 Pin in shear

A pin subject to a design shear force (V_f^*) shall satisfy—

$$V_f^* \leq \phi V_f$$

where

ϕ = 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

9.5.2 Pin in bearing

A pin subject to a design bearing force (V_b^*) shall satisfy—

$$V_b^* \leq \phi V_b$$

where

ϕ = 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

9.5.3 Pin in bending

A pin subject to a design bending moment (M^*) shall satisfy—

$$M^* \leq \phi M_p$$

where

ϕ = 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

9.5.4 Ply in bearing

A ply subject to a design bearing force (V_b^*) due to a pin in shear shall satisfy Clause 9.3.2.4.

9.6 DESIGN DETAILS FOR BOLTS AND PINS

9.6.1 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 9.3.2.4.

9.6.2 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 14.3.5.2) shall be as given in Table 9.6.2, 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 9.6.2, 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 9.6.2

MINIMUM EDGE DISTANCE

A1

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 Clause 9.3.2.4.

9.6.3 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.

9.6.4 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.

9.6.5 Holes

Holes for bolts shall comply with Clause 14.3.5 and holes for pins shall comply with Clause 14.3.7.

9.7 DESIGN OF WELDS

9.7.1 Scope

9.7.1.1 *General*

A1 Welding shall comply with AS/NZS 1554.1, AS/NZS 1554.2, AS/NZS 1554.4 or AS/NZS 1554.5, as appropriate.

9.7.1.2 *Weld types*

For the purpose of this Standard, welds shall be butt, fillet, slot or plug welds, or compound welds.

9.7.1.3 *Weld quality*

A1 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 11.1.5, weld quality conforming with AS/NZS 1554.5 shall be used. Weld quality shall be specified on the design drawings.

9.7.2 Complete and incomplete penetration butt welds

9.7.2.1 *Definitions*

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

Complete penetration butt weld—a butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

Incomplete penetration butt weld—a butt weld in which fusion exists over less than the complete depth of the joint.

A1 *Prequalified weld preparation*—a joint preparation prequalified in terms of AS/NZS 1554.1 or AS/NZS 1554.4, as appropriate.

9.7.2.2 *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.

9.7.2.3 *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 A1 otherwise provided in (iii), as specified in AS/NZS 1554.1 or AS/NZS 1554.4, as appropriate.

   (ii) Non-prequalified preparation for incomplete penetration butt weld except as provided in (iii)—

      (A) where $\theta < 60°$ . . . $(d - 3)$ mm, for single V weld;
      $[(d_3 + d_4) - 6]$ mm, for double V weld

      (B) where $\theta > 60°$ . . . 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)

   θ = 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 9.7.3.4 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.

9.7.2.4 *Effective length*

The effective length of a butt weld shall be the length of the continuous full size weld.

9.7.2.5 *Effective area*

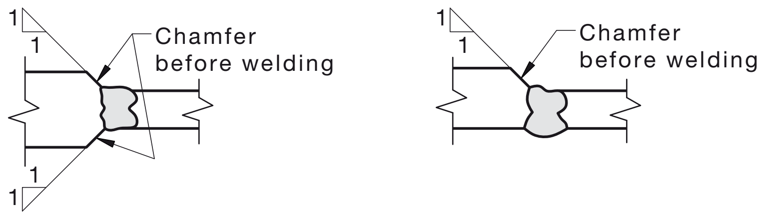
The effective area of a butt weld shall be the product of the effective length and the design throat thickness.

9.7.2.6 *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 9.7.2.6.

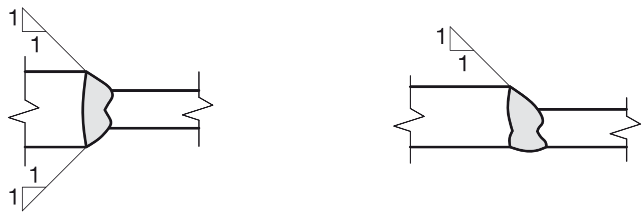
The transition slope between the parts shall not exceed 1:1. However, the provisions of Section 11 require a lesser slope than this or a curved transition between the parts for some fatigue detail categories.



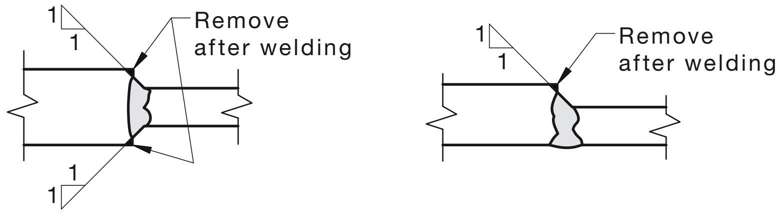
Centre-line alignment
(Particularly applicable to web plates)

Offset alignment
(Particularly applicable to flange plates)

(i) Transition by chamfering thicker part

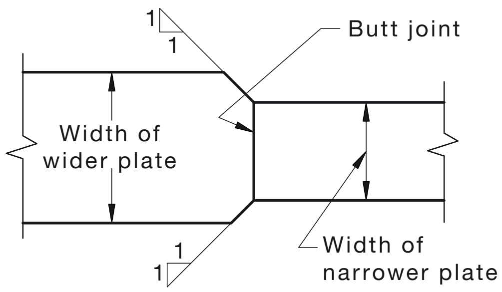


(ii) Transition by sloping weld surface



(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 9.7.2.6 TRANSITIONS OF THICKNESS OR WIDTH FOR BUTT WELDS SUBJECT TO TENSION

9.7.2.7 *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 (ϕ) for butt welds given in Table 3.4, provided that the welding procedures are qualified in accordance with A1 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.

(b) *Incomplete-penetration butt weld* The design capacity of an incomplete-penetration butt weld shall be calculated as for a fillet weld (see Clause 9.7.3.10) using the design throat thickness determined in accordance with Clause 9.7.2.3(b).

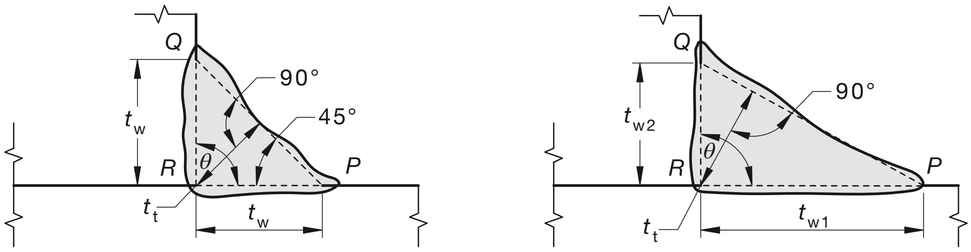
9.7.3 Fillet welds

9.7.3.1 *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 9.7.3.1(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 9.7.3.1(c).

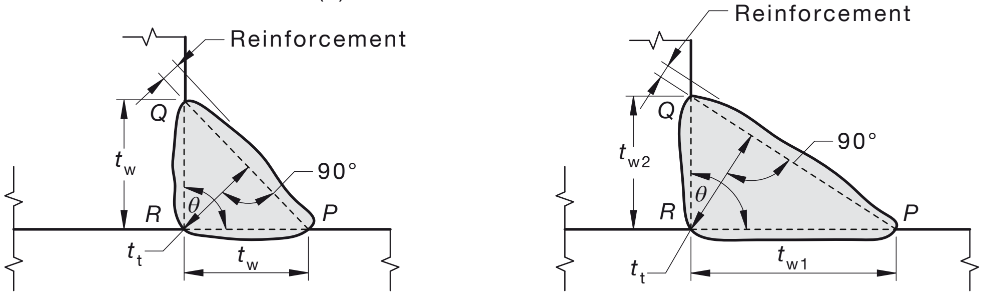
NOTE: The preferred sizes of a fillet weld less than 15 mm are—3, 4, 5, 6, 8, 10 and 12 mm.



Equal leg fillet weld

Unequal leg fillet weld

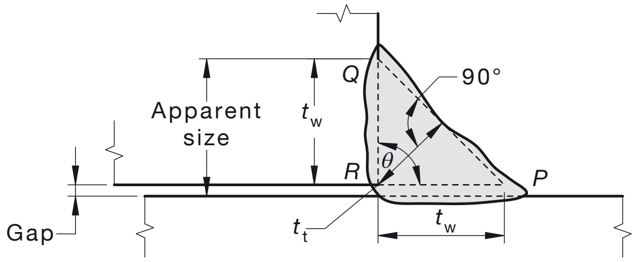
(a) Concave fillet welds



Equal leg fillet weld

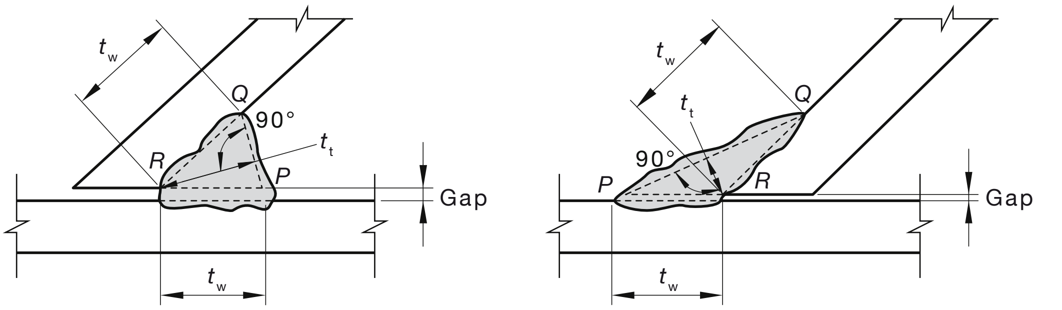
Unequal leg fillet weld

(b) Convex fillet welds



(c) Fillet weld with root gap

A1



(d) Fillet weld at angled connection - Acute angle side

(e) Fillet weld at angled connection - Obtuse angle side

LEGEND:

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 9.7.3.1 FILLET WELD SIZE

9.7.3.2 *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 9.7.3.2, except that the size of the weld need not exceed the thickness of the thinner part joined.

TABLE 9.7.3.2

MINIMUM SIZE OF A FILLET WELD

Thickness of thickest part, t (mm)	Minimum size of a fillet weld, t_w (mm)
$t \leq 7$	3
$7 < t \leq 10$	4
$10 < t \leq 15$	5
$15 < t$	6

9.7.3.3 *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 9.7.3.3(a)); and

(b) for material 6 mm or more in thickness (see Figure 9.7.3.3(b)), unless the weld is designated on the drawing to be built out to obtain the design throat thickness (see Figure 9.7.3.3(c)), 1 mm less than the thickness of the material.

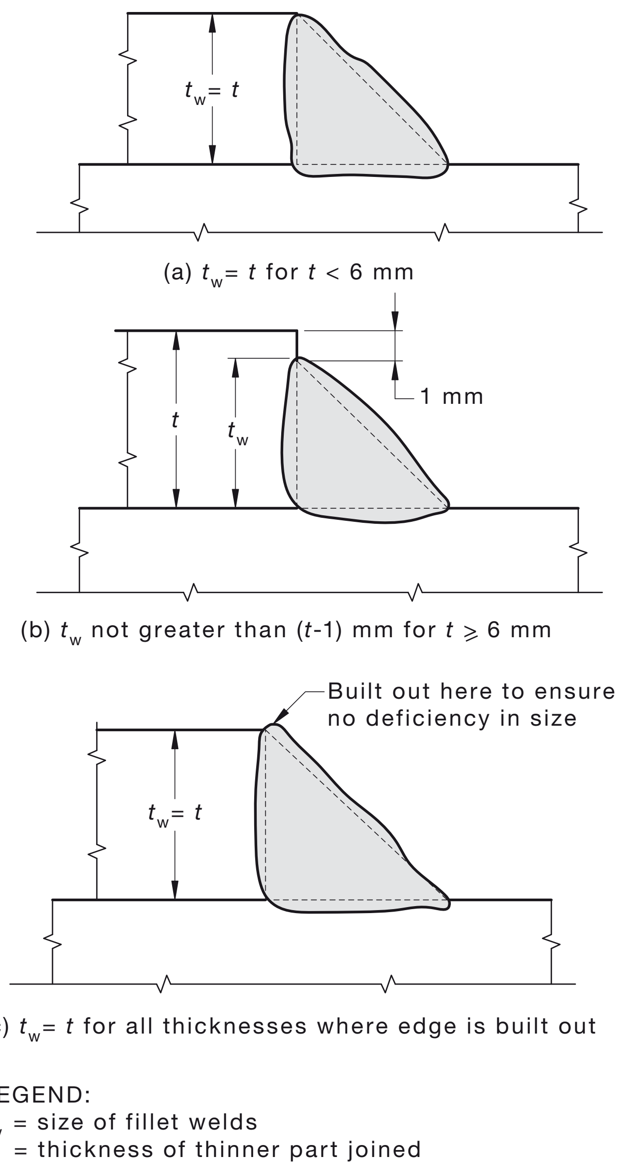


FIGURE 9.7.3.3 MAXIMUM SIZES OF FILLET WELDS ALONG EDGES

9.7.3.4 *Design throat thickness*

The design throat thickness (t_t) of a fillet weld shall be as shown in Figure 9.7.3.1.

For a weld made by an automatic arc welding process, an increase in design throat thickness may be allowed as shown in Figure 9.7.3.4, 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.

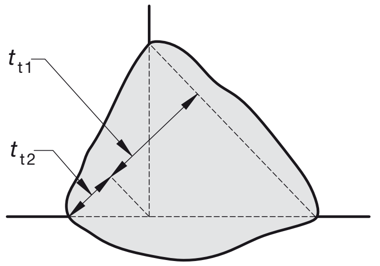


FIGURE 9.7.3.4 DEEP PENETRATION WELD

9.7.3.5 *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 comply with 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.

9.7.3.6 *Effective area*

The effective area of a fillet weld shall be the product of the effective length and the design throat thickness.

9.7.3.7 *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.

9.7.3.8 *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.

9.7.3.9 *Built-up members—intermittent fillet welds*

If intermittent fillet welds connect components forming a built-up member, the welds shall comply with 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.

9.7.3.10 *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

ϕ = 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 9.7.3.10(1))

t_t = design throat thickness

k_r = reduction factor given in Table 9.7.3.10(2) to account for the length of a welded lap connection (l_w). For all other connection types, k_r equals 1.0

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TABLE 9.7.3.10(1)
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}
Structural steel welding to AS/NZS 1554.1 and AS/NZS 1554.5—Steel Types 1–8C					
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
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

Manual metal arc (AS/NZS 4855, AS/NZS 4857)	Submerged arc (AS 1858.1 AS 1858.2)	Flux cored arc (AS/NZS ISO 17632 AS/NZS ISO 18276)	Gas metal arc (AS/NZS 2717.1) (ISO 14341, ISO 16834)	Gas tungsten arc (ISO 636, ISO 16834)	Nominal tensile strength of weld metal, f_{uw}
Structural steel welding to ASNZS 1554.4—Steel Types 8Q–10Q					
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

NOTES:

1 The minimum tensile strength of the European type A classification series consumables is slightly higher than that shown in this Table.

2 The B–E57XX, B–E59XX, B-E78XX and equivalent strength consumables for other welding processes, may be difficult to source commercially.

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 9.7.3.10(2)

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

9.7.4 Plug and slot welds

9.7.4.1 *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 9.7.3.5, and a nominal capacity as defined in Clause 9.7.3.10. The minimum size shall be as for a fillet weld (see Clause 9.7.3.2).

9.7.4.2 *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 considered as 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

ϕ = 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:

$$V_w = 0.60 f_{uw} A_w$$

9.7.4.3 *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.

9.7.5 Compound weld

9.7.5.1 *Description of a compound weld*

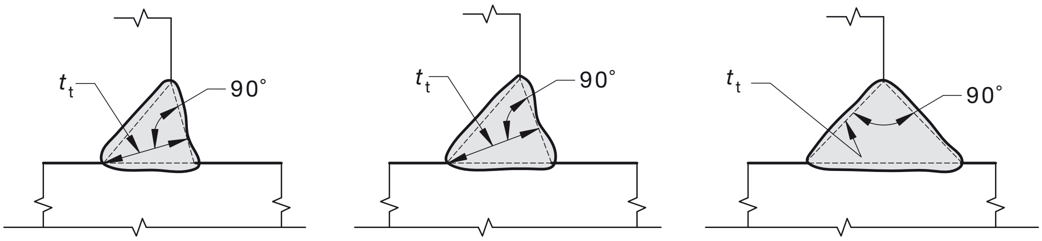
A compound weld, defined as a fillet weld superimposed on a butt weld, shall be as specified in AS 1101.3.

9.7.5.2 *Design throat thickness*

The design throat thickness of a compound weld, for use in design calculations, shall be—

(a) for a complete penetration butt weld, the size of the butt weld without reinforcement; and

(b) 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 9.7.5.2).



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 sketches above illustrate this concept.

FIGURE 9.7.5.2 DESIGN THROAT THICKNESSES OF COMPOUND WELDS

9.7.5.3 *Strength limit state*

The weld shall satisfy the requirements of Clause 9.7.2.7.

9.8 ASSESSMENT OF THE STRENGTH OF A WELD GROUP

9.8.1 Weld group subject to in-plane loading

9.8.1.1 *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 considered 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 recognised 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 9.7.3.10 at all points in the fillet weld group using the appropriate capacity factor (ϕ) 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.

9.8.1.2 *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 9.7.3.10 at all points in the fillet weld group using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.2 Weld group subject to out-of-plane loading

9.8.2.1 *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 considered in isolation from the connected element; and

(b) The design force per unit length in the fillet weld resulting from a design bending moment shall be considered 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 considered to be uniformly distributed over the length of the fillet weld group.

A fillet weld shall satisfy the requirements of Clause 9.7.3.10 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.2.2 *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 9.7.3.10 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.3 Weld group subject to in-plane and out-of-plane loading

9.8.3.1 *General method of analysis*

The design force per unit length as determined from analyses in accordance with Clauses 9.8.1.1 and 9.8.2.1 shall satisfy Clause 9.7.3.10 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.3.2 *Alternative analysis*

The design force per unit length as determined from analyses in accordance with Clauses 9.8.1.2 and 9.8.2.2 shall satisfy Clause 9.7.3.10 at all points in the fillet weld group, using the appropriate capacity factor (ϕ) for a weld group (see Table 3.4).

9.8.4 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.

9.9 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.

A1 **'Text deleted'**

10.3 DESIGN SERVICE TEMPERATURE

A1 ### 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.

A1 ### 10.3.2 Basic design temperature

Lowest one-day mean ambient temperature (LODMAT) isotherms for Australia are given in Figure 10.3. 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 for a significant time to 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.

A1 ### 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.

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TABLE 10.4.1

PERMISSIBLE SERVICE TEMPERATURES ACCORDING TO STEEL TYPE AND THICKNESS

Steel type (see Table 10.4.4)	Permissible service temperature, °C (see Note 1)					
	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

NOTES:

1 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.

2 This Table is based on available statistical data on notch toughness characteristics of steels currently made in Australia or New Zealand. Care should be taken in applying this Table to imported steels as verification tests may be required. For a further explanation, see WTIA Technical Note 11.

3 (—) indicates that material is not available in these thicknesses.

NOTES:

1 Lowest one day mean ambient temperature.

2 Based on records 1957 to 1971 supplied by Australian Bureau of Meteorology.

3 Isotherms in degree Celsius.

FIGURE 10.3 LODMAT ISOTHERMS

10.4.2 Limitations

Table 10.4.1 shall only be used without modification for members and components which comply with the fabrication and erection provisions of Sections 14 and 15, and with the
A1 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

A1 **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.

A1 **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.

A1 **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 may be found in AS 4458.

A1 **10.4.3.4** *Non-complying conditions*

Steels, for which the permissible service temperature (as modified where applicable) is not known or is warmer than the design service temperature specified by the designer, shall not be used, unless compliance with 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.

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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	HA1 HA3 HA4N HA200 HA250 HA250/1 HU250 HA300 HA300/1 HU300 HU300/1	200 250 300	300	—
2	C250L0	—	—	300L0	—
2S	—	—	250S0 300S0	300S0	—
3		XF300	250L15 250L20 250Y20 250L40 250Y40 300L15 300L20 300Y20 300L40 300Y40	300L15	—
4	C350	HA350 HA400 HW350	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

(*continued*)

TABLE 10.4.4 (*continued*)

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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
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

A1 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 comply with the requirements of this Standard for the strength and serviceability limit states.

11.1.2 Definitions

For the purposes of this Section, the definitions below apply.

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).

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).

Design life—period over which a structure or structural element is required to perform its function without repair.

Design spectrum—sum of the stress spectra from all of the nominal loading events expected during the design life.

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

NOTES:

1 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.

2 The detail category number is defined by the fatigue strength at 2×10^6 cycles on the S-N curve (see Figures 11.6.1 and 11.6.2).

Discontinuity—an absence of material, causing a stress concentration.

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

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

*Fatigue loading—s*et of nominal loading events described by the distribution of the loads, their magnitudes and the numbers of applications of each nominal loading event.

Fatigue strength—the stress range defined in Clause 11.6 for each detail category varying with the number of stress cycles (see Figures 11.6.1 and 11.6.2).

Miner's summation—cumulative damage calculation based on the Palmgren-Miner summation or equivalent.

Nominal loading event—the loading sequence for the structure or structural element.

NOTE: 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.

S-N curve—curve defining the limiting relationship between the number of stress cycles and stress range for a detail category.

Stress cycle—one cycle of stress defined by stress cycle counting.

Stress cycle counting method—any rational method used to identify individual stress cycles from the stress history.

Stress range—algebraic difference between two extremes of stress.

Stress spectrum—histogram of the stress cycles produced by a nominal loading event.

11.1.3 Notation

For the purposes of this Section—

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 (5×10^6 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 even I, producing f_i^*

n_r = reference number of stress cycles (2×10^6 cycles)

n_{sc} = number of stress cycles

t_f = flange thickness

t_p = plate thickness

α_s = inverse of the slope of the S-N curve

ϕ = capacity factor

11.1.4 Limitation

In all stress cycles, the magnitude of the design stress shall not exceed f_y and the stress range shall not exceed $1.5f_y$.

11.1.5 Designation of weld category

The welds in the welded details given in Tables 11.5.1(2) and (4) 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.

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The welds in the welded details given in Table 11.5.1(2) for Detail Category 125 shall have a weld quality conforming to that defined in AS/NZS 1554.5.

11.1.6 Method

For the reference design condition, the capacity factor (ϕ) 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 will 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 (ϕ) shall be reduced when any of the above conditions do not apply.

For non-redundant load paths, the capacity factor (ϕ) shall be less than or equal to 0.70.

11.1.7 Thickness effect

The thickness correction factor (β_{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 β_{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 loading used in the fatigue assessment shall be the actual service loading, including dynamic effects.

The fatigue loading shall be obtained from the referring Standards where applicable. These include—

A1

AS		
1418	Cranes, hoists and winches	
1418.1	Part 1:	General requirements
1418.3	Part 3:	Bridge, gantry, portal (including container cranes) and jib cranes
1418.5	Part 5:	Mobile cranes
1418.18	Part 18:	Crane runways and monorails
5100	Bridge design	
5100.1	Part 1:	Scope and general principles
5100.2	Part 2:	Design loads

In other cases, the fatigue loading used in design shall be the expected service loading.

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(1) to 11.5.1(4). 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(1) to 11.5.1(4) 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—

$l/d_x > 40$; or

$l/d_y > 40$

as appropriate.

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:

(a) For a truss connection involving circular hollow sections, the stress range shall be multiplied by the appropriate factor given in Table 11.3.1(1).

(b) 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(2).

(c) The design throat thickness of a fillet weld shall be greater than the wall thickness of the connected member.

TABLE 11.3.1(1)

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(2)

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(1) to (4).

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(1))

Group 2: Welded details—not in hollow sections. (See Table 11.5.1(2))

Group 3: Bolts. (See Table 11.5.1(3))

Group 4: Welded details—in hollow sections. (See Table 11.5.1(4))

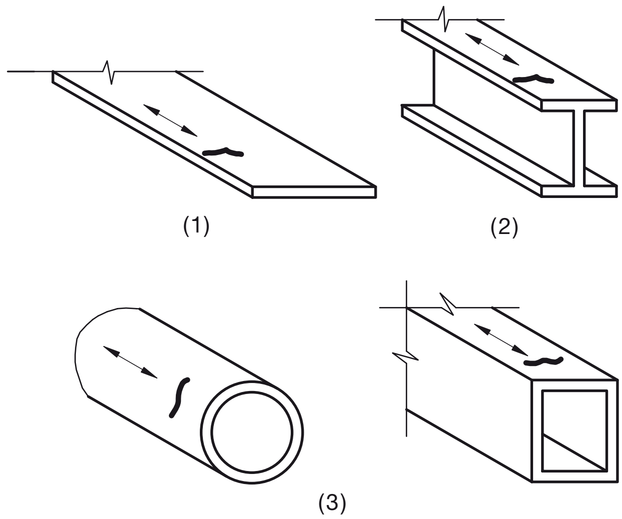
Details not classified in Tables 11.5.1(1) to (4) shall be treated as the lowest detail category of a similar detail, unless a superior fatigue strength is proved by testing or by analysis and testing.

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(2) (Descriptions 39 and 40) and in Table 11.5.1(3) (Description 41).

TABLE 11.5.1(1)

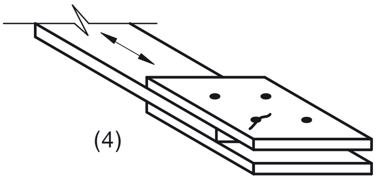
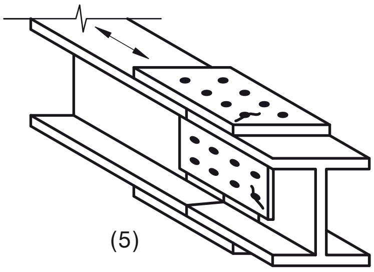
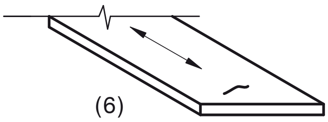
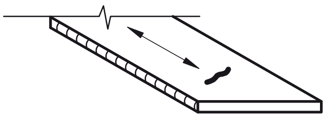
DETAIL CATEGORY CLASSIFICATION GROUP 1 NON-WELDED DETAILS

Detail category	Constructional details	
	Illustration (see Note)	**Description**
160		**ROLLED AND EXTRUDED PRODUCTS** (1) Plates and flats (2) Rolled sections (3) Seamless tubes Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress

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.

(*continued*)

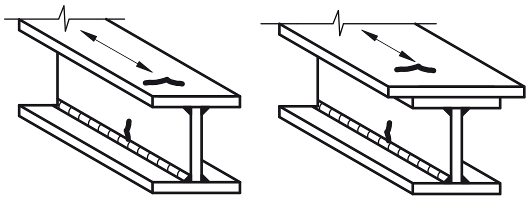
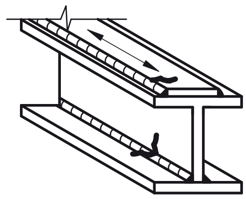
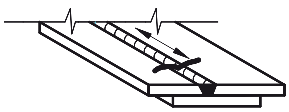
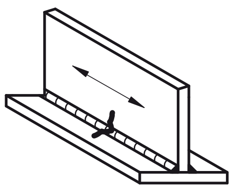
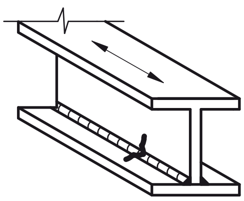
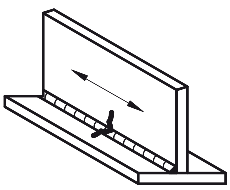
TABLE 11.5.1(1) (*continued*)

Detail category	Constructional details	
	Illustration (see Note)	**Description**
140	 (4)  (5)  (6)	**BOLTED CONNECTIONS** (4) & (5) Stress range calculated on the gross section for 8.8/TF bolting category and on the net section otherwise. Unsupported one-sided coverplate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses. **MATERIAL WITH GAS-CUT OR SHEARED EDGES WITH NO DRAGLINES (6)** All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.
125	 (7)	**MATERIAL WITH MACHINE GAS-CUT EDGES WITH DRAGLINES OR MANUAL GAS-CUT MATERIAL (7)** Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.

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(2)

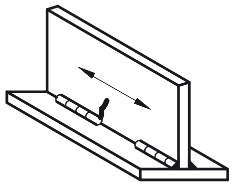
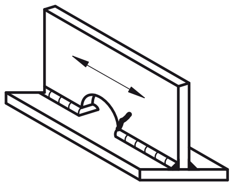
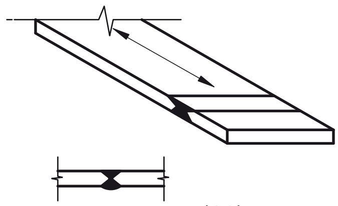
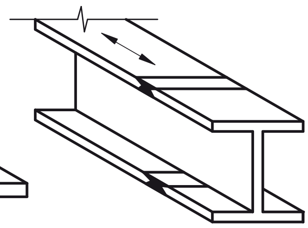
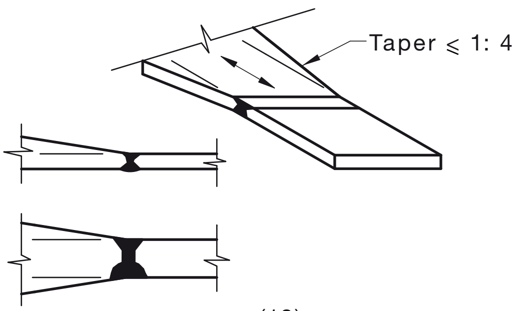
DETAIL CATEGORY CLASSIFICATION GROUP 2 WELDED DETAILS—NOT IN HOLLOW SECTIONS

Detail category	Constructional details	
	Illustration (see Note)	**Description**
125	 (8)  (9)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (8) & (9) Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having unrepaired stop-start positions. NOTE: See Clause 11.1.5 regarding weld category.
112	 (10)  (11)  (12)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (10) & (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. (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.
90	 (13)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (13) Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.

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.

(*continued*)

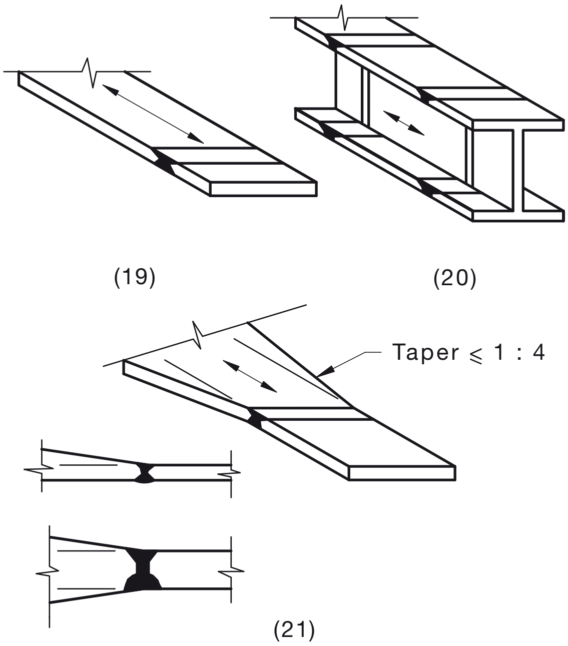
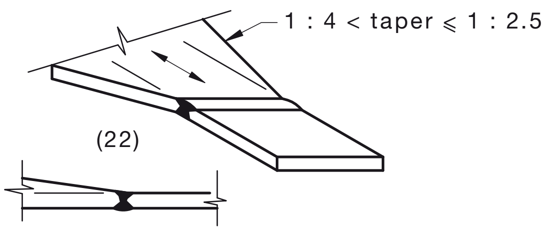
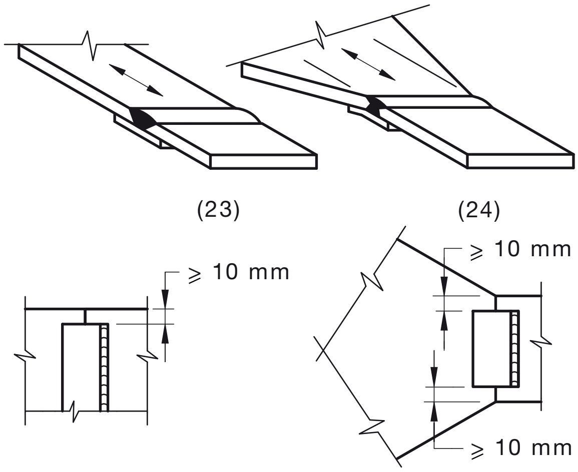
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details	
	Illustration (see Note)	**Description**
80	 (14)	**INTERMITTENT LONGITUDINAL WELDS** (14) Zones of intermittent longitudinal welds.
71	 (15)	**INTERMITTENT LONGITUDINAL WELDS** (15) Zones containing cope holes in longitudinally welded T joints. Cope hole not to be filled with weld.
112	 (16)  (17)  (18)	**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** 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. (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. (17) Plate girders welded as (16) before assembly. (18) Transverse splices as (16) with radiused or tapered transition with taper ≤ 1:4.

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.

(*continued*)

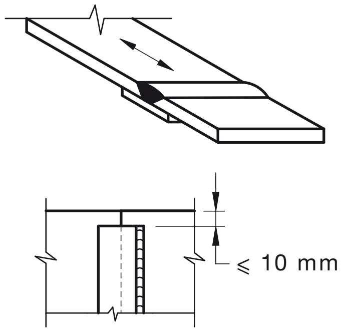
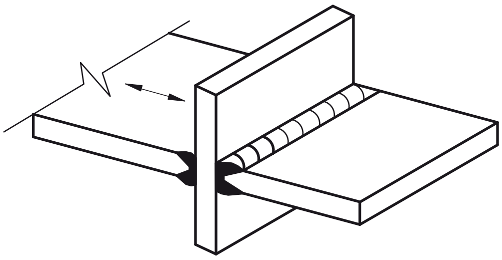
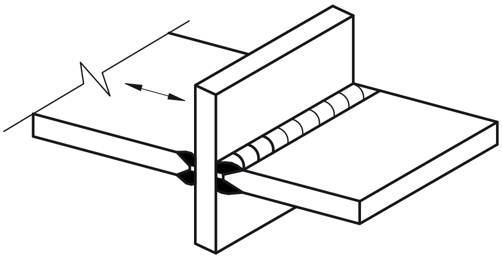
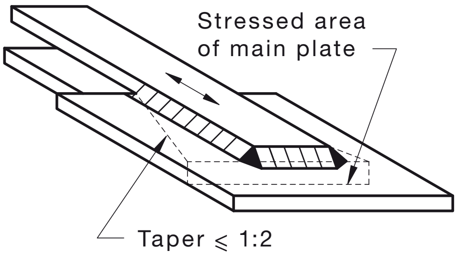
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details: Illustration (see Note)	Constructional details: Description
90		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** 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. (19) Transverse splices of plates, rolled sections, or plate girders. (20) Transverse splices of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 71, as for (15). (21) Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤ 1:4.
80		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** 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. (22) Transverse splices as for (21) with taper in width or thickness > 1:4 and ≤ 1:2.5.
71		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (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. (24) Transverse butt welds as for (23) with taper on width or thickness < 1:2.5.

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.

(*continued*)

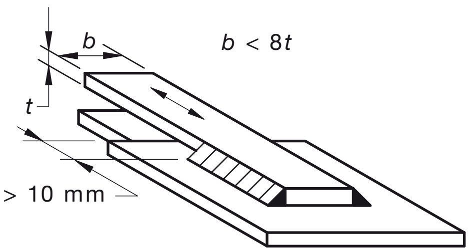
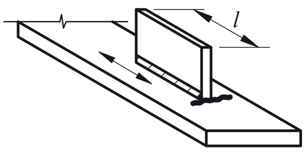
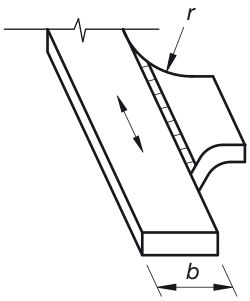
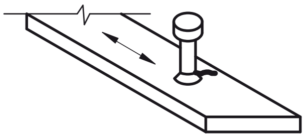
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details — Illustration (see Note)	Constructional details — Description
50	 (25)	**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (25) Transverse butt welds as (23) where fillet welds end closer than 10 mm to plate edge
71	 (26)	**CRUCIFORM JOINTS WITH LOAD-CARRYING WELDS** (26) Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be < 0.15 times the thickness of intermediate plate
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	 (29)	**OVERLAPPED WELDED JOINTS** (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

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.

(*continued*)

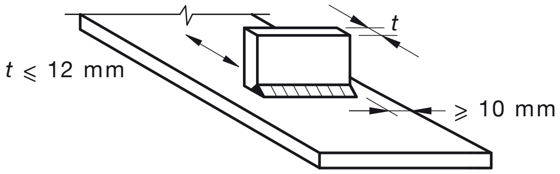
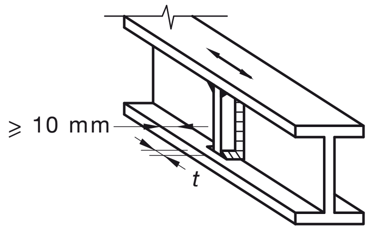
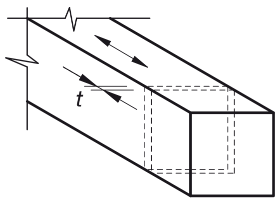
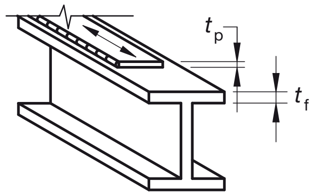
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details			
	Illustration (see Note)			Description
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
90	(32)	(33)	(32) (33)	**WELDED ATTACHMENTS (NON-LOAD CARRYING WELDS) LONGITUDINAL WELDS**
	—	$\frac{1}{3} \leq \frac{r}{b}$		(32) Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted.
80	$l \leq 50$ mm	—		(33) Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r) formed by machining or flame-cutting plus grinding. Class of detail varies according to r/b ratio as noted.
71	$50 < l$ ≤ 100 mm	$\frac{1}{6} \leq \frac{r}{b} < \frac{l}{3}$		
50	100 mm $< l$	—		
45	—	$\frac{r}{b} < \frac{1}{6}$		
80	(34)			**WELDED ATTACHMENTS** (34) Shear connectors on base material (failure in base material)

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.

(*continued*)

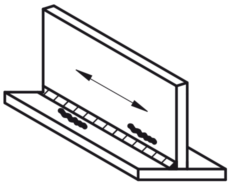
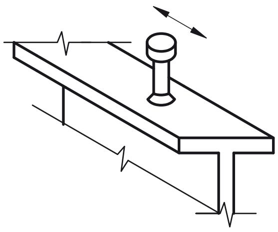
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details		Description
		Illustration (see Note)	
80	$t \leq 12$ mm	 (35)  (36)  (37)	**TRANSVERSE WELDS** (35) Transverse fillet welds with the end of the weld ≥ 10 mm from the edge of the plate.
71	$t > 12$ mm		(36) Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses. (37) Diaphragms of box girders welded to the flange or web by continuous or intermittent welds.
50	t_f and $t_p \leq 25$ mm	 (38)	**COVER PLATES IN BEAMS AND PLATE GIRDERS** (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.)
36	t_f and $t_p > 25$ 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.

(*continued*)

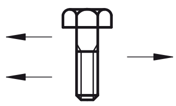
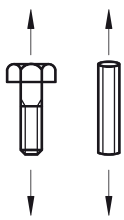
TABLE 11.5.1(2) (*continued*)

Detail category	Constructional details	
	Illustration (see Note)	**Description**
80	 (39)	**WELDS LOADED IN SHEAR** (39) Fillet welds transmitting shear. Stress range to be calculated on weld throat area.
	 (40)	(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).

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(3)

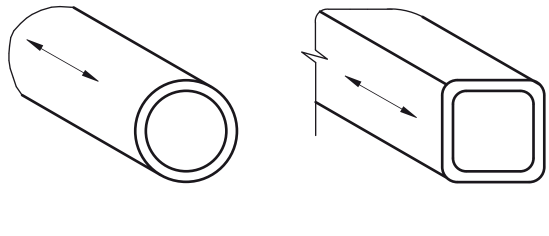
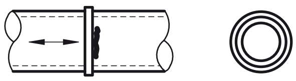
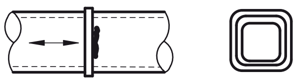
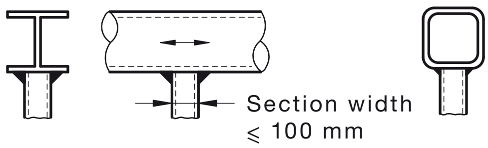
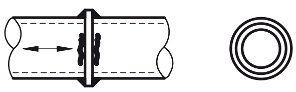
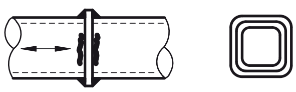
DETAIL CATEGORY CLASSIFICATION GROUP 3 BOLTS

Detail category	Constructional details	
	Illustration (see Note)	**Description**
100	 (41)	(41) **BOLTS IN SHEAR (8.8/TB BOLTING CATEGORY ONLY)** Shear stress range calculated on the minor diameter area of the bolt (A_c) NOTE: If the shear on the joint is insufficient to cause slip of the joint (see Clause 9.3.3), the shear in the bolt need not be considered in fatigue.
36	 (42)	(42) **BOLTS AND THREADED RODS IN TENSION** (tensile stress to be calculated on the tensile stress area A_s). 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. NOTE: In connections with tensioned bolts (see Clauses 15.2.4 and 15.2.5), 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: 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(4)

DETAIL CATEGORY CLASSIFICATION GROUP 4 WELDED DETAILS—IN HOLLOW SECTIONS

Detail category	Constructional details	
	Illustration (see Note)	**Description**
140	 (43)	**CONTINUOUS AUTOMATIC LONGITUDINAL WELDS** (43) No stop-starts, or as manufactured
90 ($t \geq 8$ mm) 71 ($t < 8$ mm)	 (44)	**TRANSVERSE BUTT WELDS** (44) Butt-welded end-to-end connection of circular hollow sections
71 ($t \geq 8$ mm) 56 ($t < 8$ mm)	 (45)	(45) Butt-welded end-to-end connection of rectangular hollow sections
56 ($t \geq 8$ mm) 50 ($t < 8$ mm)	 (46)	**BUTT WELDS TO INTERMEDIATE PLATE** (46) Circular hollow sections, end-to-end butt welded with an intermediate plate
50 ($t \geq 8$ mm) 41 ($t < 8$ mm)	 (47)	(47) Rectangular hollow sections, end-to-end butt welded with an intermediate plate
71	 (48)	**WELDED ATTACHMENTS** (Non-load-carrying) (48) Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction ≤ 100 mm
A1 45 ($t \geq 8$ mm) 40 ($t < 8$ mm)	 (49)	**FILLET WELDS TO INTERMEDIATE PLATE** (49) Circular hollow sections, end-to-end fillet welded with an intermediate plate
40 ($t \geq 8$ mm) 36 ($t < 8$ mm)	 (50)	(50) Rectangular hollow sections, end-to-end fillet welded with an intermediate plate

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—

$$f_f^3 = \frac{f_{fn}^3 \times 2 \times 10^6}{n_{sc}} \qquad \text{when } n_{sc} \leq 5 \times 10^6$$

$$f_f^5 = \frac{f_5^5 \times 10^8}{n_{sc}} \qquad \text{when } 5 \times 10^6 < n_{sc} \leq 10^8$$

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}).

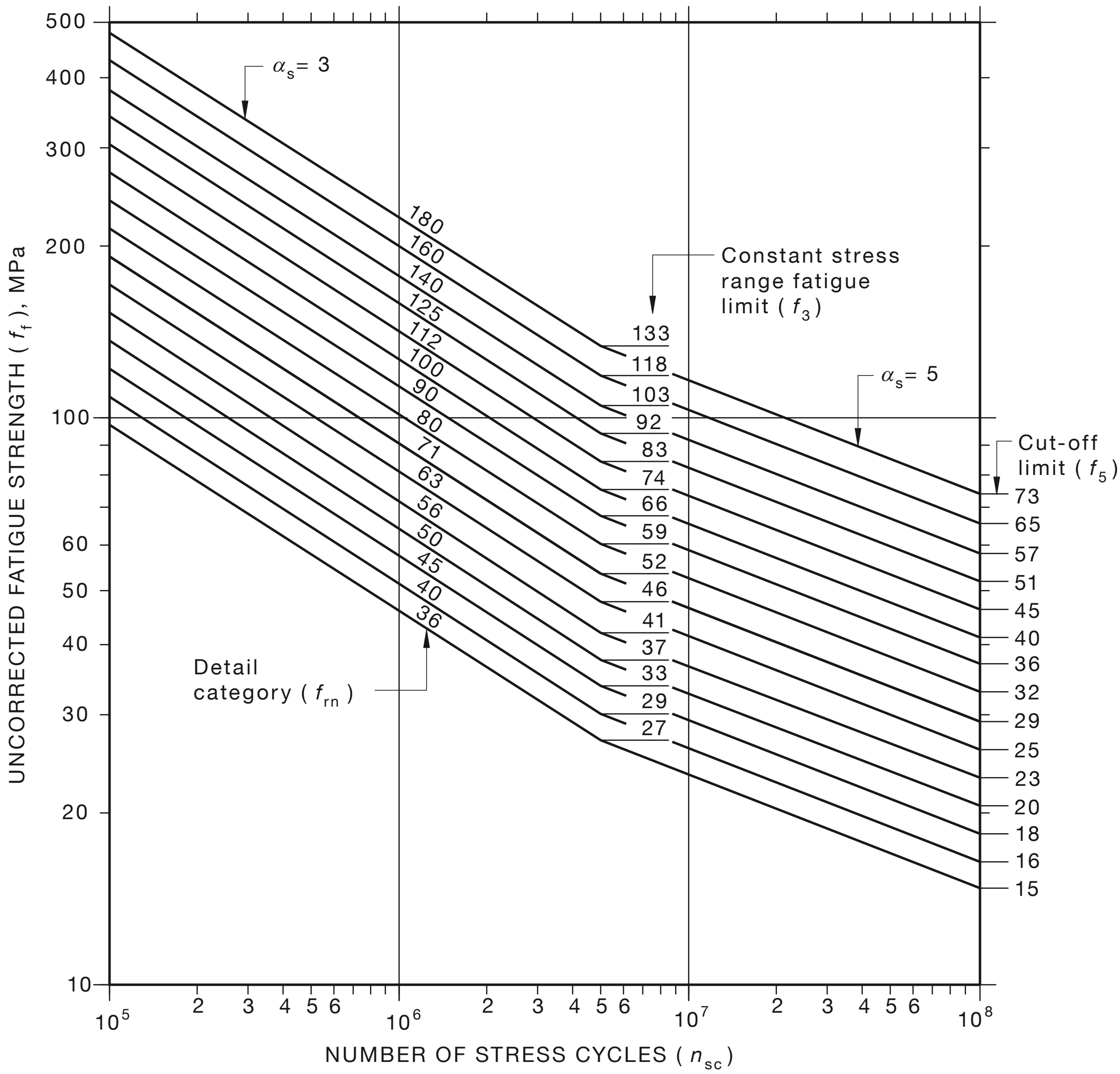


FIGURE 11.6.1 S-N CURVE FOR NORMAL STRESS

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—

$$f_f^5 = \frac{f_{rs}^5 \times 2 \times 10^6}{n_{sc}} \qquad \text{when } n_{sc} \leq 10^8$$

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}).

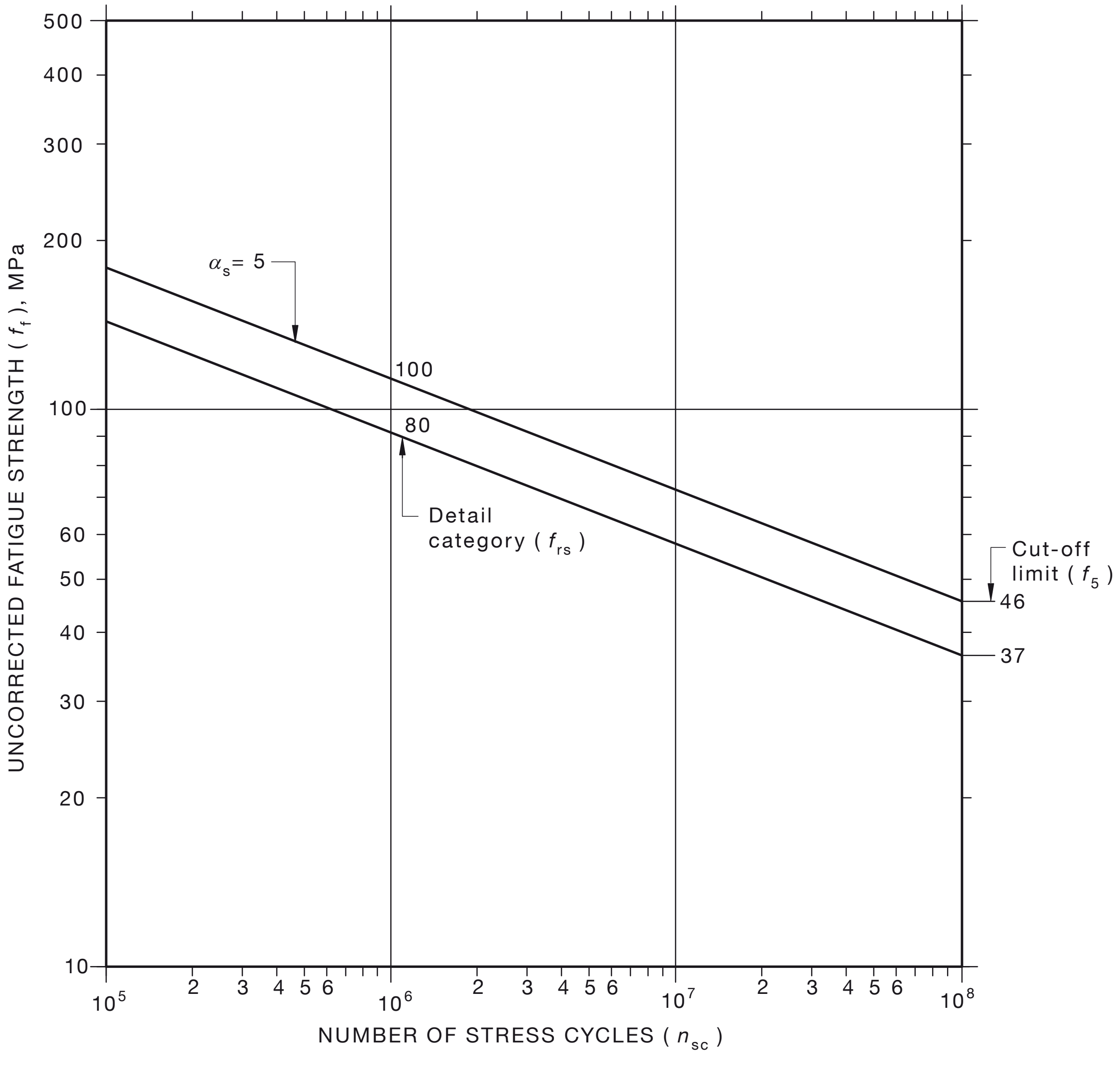


FIGURE 11.6.2 S-N CURVE FOR SHEAR STRESS

11.7 EXEMPTION FROM FURTHER ASSESSMENT

At any point in the structure at which the design stress range (f^*) is less than (ϕf_{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:

$$\frac{\sum_i n_i \left(f_i^*\right)^3}{5 \times 10^6 \left(\phi f_{3c}\right)^3} + \frac{\sum_j n_j \left(f_j^*\right)^5}{5 \times 10^6 \left(\phi f_{3c}\right)^5} \leq 1.0$$

(b) for shear stresses:

$$\frac{\sum_k n_k \left(f_k^*\right)^5}{2 \times 10^6 \left(\phi f_{rsc}\right)^5} \leq 1.0$$

where

(i) the summation $\sum_i$ is for *i* design stress ranges (f_i^*) for which $\phi f_{3c} \leq f_i^*$;

A1 (ii) the summation $\sum_j$ is for *j* design stress ranges (f_j^*) for which $\phi f_{5c} \leq (f_j^*) < \phi f_{3c}$; and

(iii) the 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 period of structural adequacy (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—the 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.

NOTES:

1 Three-sided fire exposure condition is to be considered separately unless otherwise specified in Clause 12.9.

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*—a steel member exposed to fire on all sides.

Fire protection system—the fire protection material and its method of attachment to the steel member.

Fire-resistance level (FRL)—the 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)—the time (t), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test.

Prototype—a test specimen representing a steel member and its fire protection system which is subjected to the standard fire test.

Standard fire test—the fire-resistance test specified in AS 1530.4.

Stickability—the 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—the 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 period of structural adequacy (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:

$$\frac{f_y(T)}{f_y(20)} = 1.0 \qquad \text{when } 0°\text{C} < T \leq 215°\text{C; and}$$

$$= \frac{905 - T}{690} \qquad \text{when } 215°\text{C} < T \leq 905°\text{C}$$

where

$f_y(T)$ = yield stress of steel at T°C

$f_y(20)$ = yield stress of steel at 20°C

T = temperature of the steel in °C

This relationship is shown by Curve 1 in Figure 12.4.

12.4.2 Variation of modulus of elasticity with temperature

The influence of temperature on the modulus of elasticity of steel shall be taken as follows:

$$\frac{E(T)}{E(20)} = 1.0 + \left[\frac{T}{2000\left[\ln\left(\frac{T}{1100}\right)\right]}\right] \qquad \text{when } 0°\text{C} < T \leq 600°\text{C; and}$$

$$= \frac{690\left(1 - \frac{T}{1000}\right)}{T - 53.5} \qquad \text{when } 600°\text{C} < T \leq 1000°\text{C}$$

where

$E(T)$ = modulus of elasticity of steel at T°C

$E(20)$ = modulus of elasticity of steel at 20°C

This relationship is shown by Curve 2 in Figure 12.4.

12.5 DETERMINATION OF LIMITING STEEL TEMPERATURE

The limiting steel temperature (T_l) shall be calculated as follows:

$$T_l = 905 - 690 r_f$$

where r_f is the ratio of the design action on the member under the design load for fire
A1 specified in Section 4 of AS/NZS 1170.0 to the design capacity of the member (ϕR_u) at room temperature.

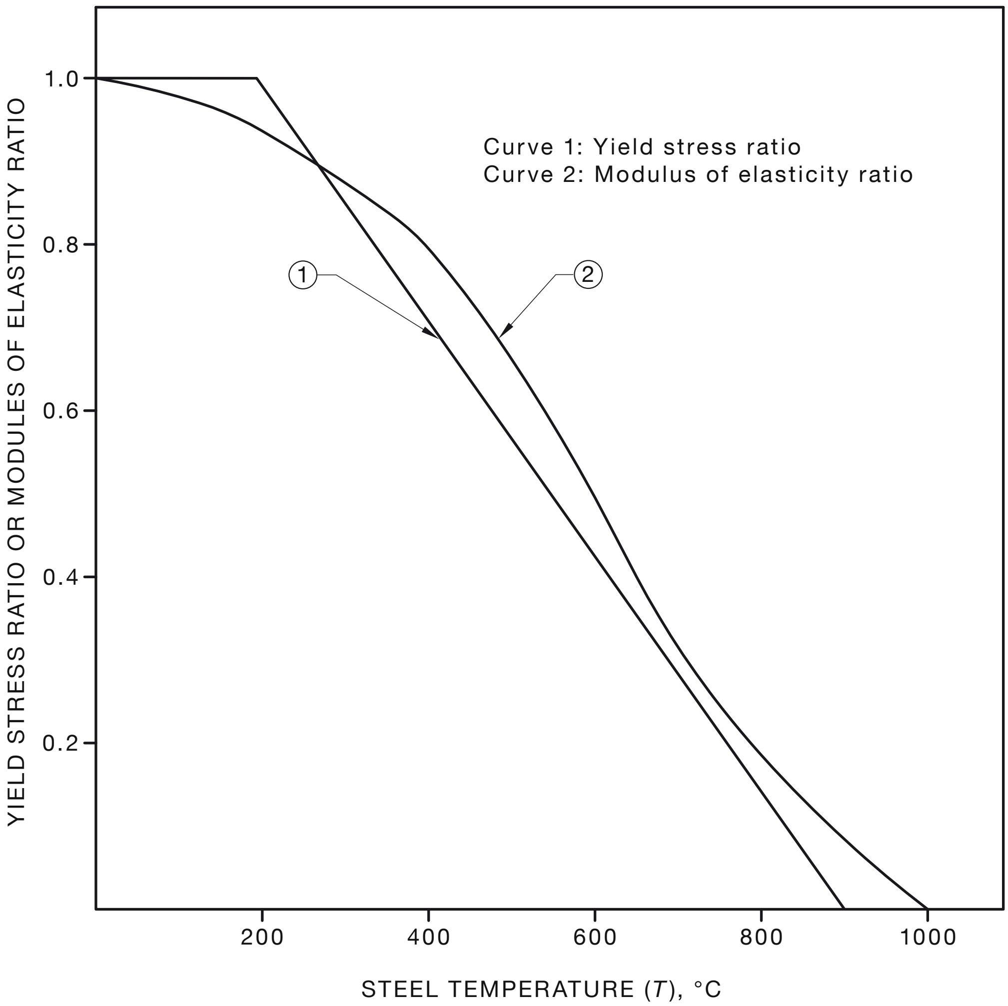


FIGURE 12.4 VARIATION OF MECHANICAL PROPERTIES OF STEEL WITH TEMPERATURE

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.

A1 Alternatively, recognized methods of assessment in accordance with ENV 13381-4 and EN 13381-8 may be used.

12.6.2 Temperature based on test series

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 12.6.2.1 subject to the limitations and conditions of Clause 12.6.2.2.

12.6.2.1 *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

12.6.2.2 *Limitations and conditions on use of regression analysis*

Test data to be utilized in accordance with Clause 12.6.2.1 shall satisfy the following:

(a) Steel members shall be protected with board, sprayed blanket or similar insulation materials having a dry density less than 1000 kg/m^3.

A1 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.

(b) All tests shall incorporate the same fire protection system.

(c) All members shall have the same fire exposure condition.

(d) The test series shall include at least nine tests.

(e) The test series may include prototypes which have not been loaded provided that stickability has been demonstrated.

(f) 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 12.6.2.2.

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.

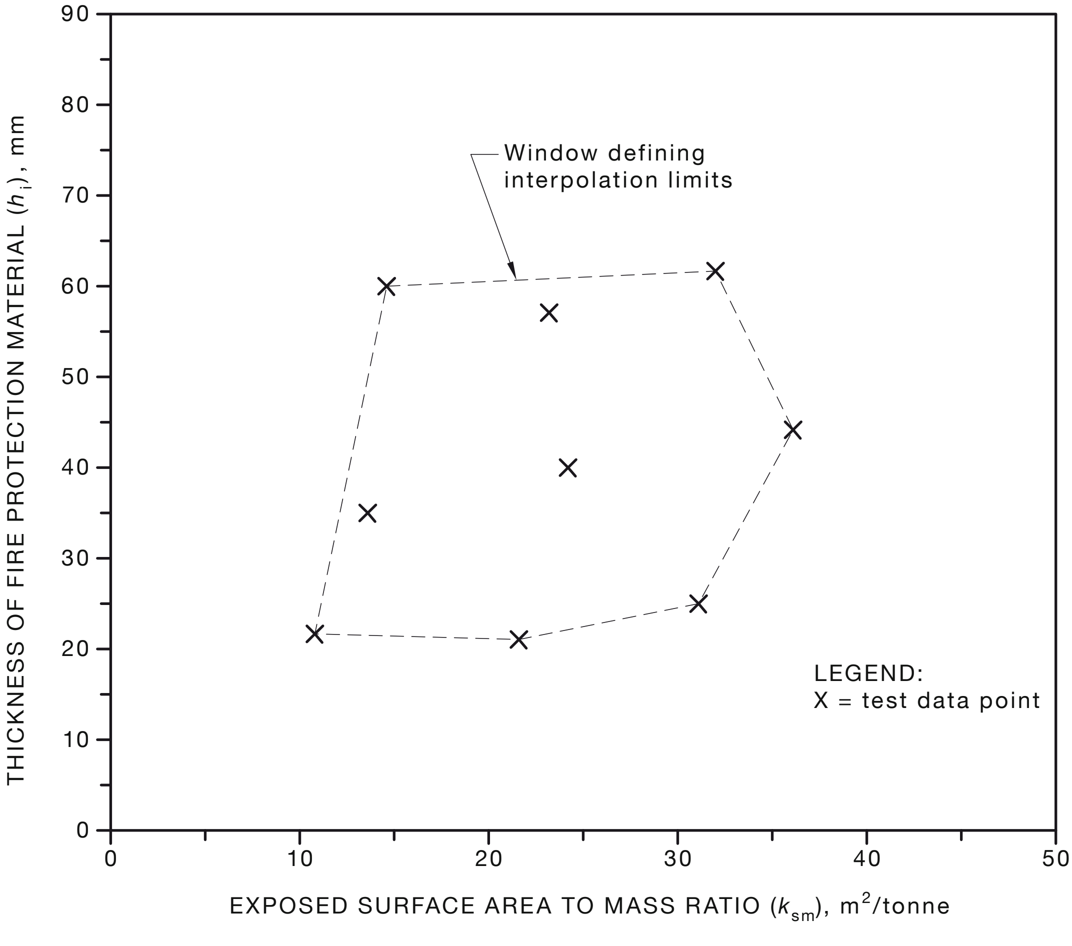


FIGURE 12.6.2.2 DEFINITION OF WINDOW FOR INTERPOLATION LIMITS

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°C ≤ T ≤ 750°C

k_{sm} = exposed surface area to mass ratio, 2 m^2/tonne ≤ k_{sm} ≤ 35 m^2/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 period of structural adequacy (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 considered 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 either be—

(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

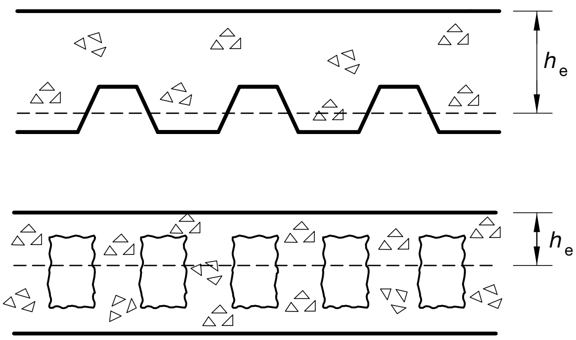
The thickness of fire protection material at and adjacent to web penetrations shall be the greatest of that required for—

(a) the area above the penetration considered as a three-sided fire exposure condition (k_{sm1}) (see Figure 12.10.2);

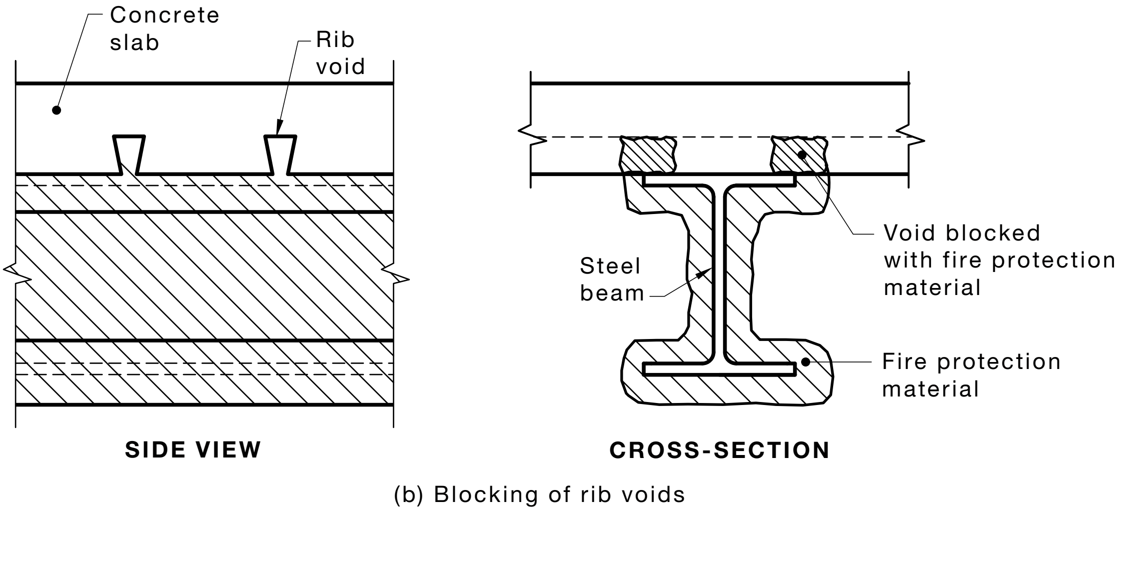
(b) the area below the penetration considered as a four-sided fire exposure condition (k_{sm2}) (see Figure 12.10.2); and

(c) 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.



(a) Effective thickness



(b) Blocking of rib voids

FIGURE 12.9 THREE-SIDED FIRE EXPOSURE CONDITION REQUIREMENTS

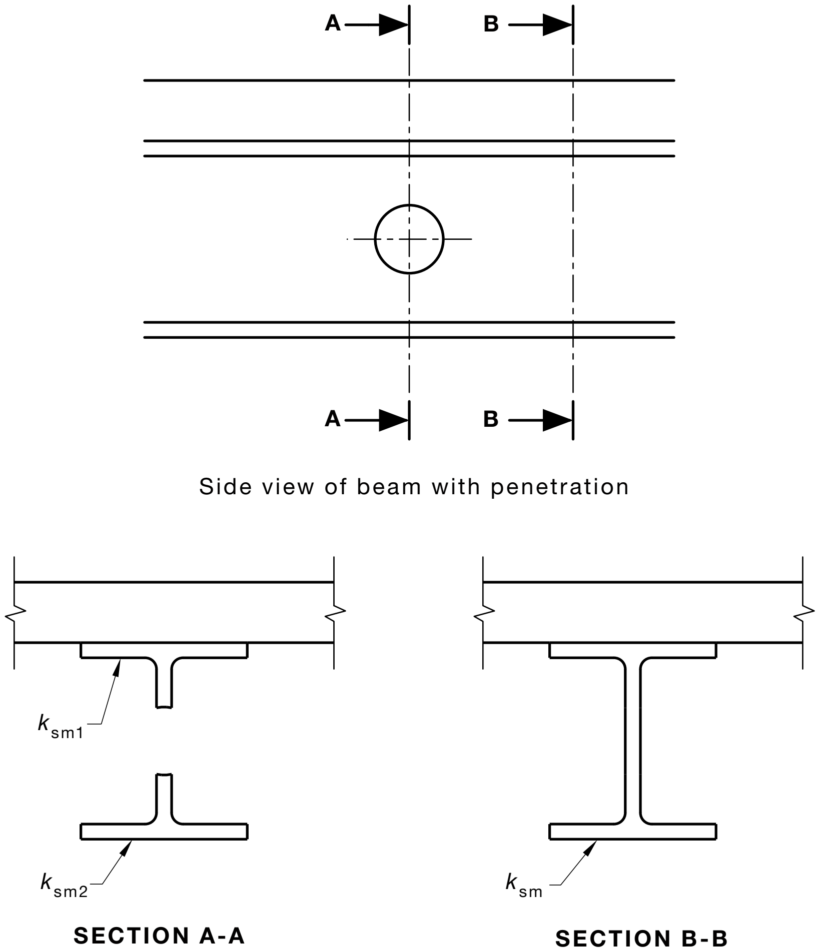


FIGURE 12.10.2 WEB PENETRATIONS

SECTION 13 EARTHQUAKE

A1

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 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 comply with Clause 13.3.5.

Moderately ductile steel structures shall comply with Clause 13.3.6.

Fully ductile steel structures shall comply with 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.

A1

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 (μ) 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 (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p)—STEEL STRUCTURES

Description of structural system	μ	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 Clause 13.3.7).

13.3.5 Requirements for 'limited ductile' steel structures (μ = 2)

Limited ductile steel structures shall comply with 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' structures (μ = 3)

13.3.6.1 *General*

The minimum yield stress specified for the grade of steel shall not exceed 350 MPa.

A1 **13.3.6.2** *Bearing wall and building frame systems*

Concentrically braced frames in bearing wall and building frame systems shall comply with 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 comply 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 comply with 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 comply with 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 μm (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 (μ >3)

A steel structure which 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 either 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 Methods

All material shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values used in design. Steel may be bent or pressed to the required shape by either hot or cold processes.

Local application of heat or mechanical means may be used to introduce or correct camber, sweep and out-of-straight. The temperature of heated areas shall not exceed 650°C.

14.3.2 Full contact splices

Full contact splices may be produced by cold saw cutting or machining.

The surfaces of such splices shall be such that, when the ends of the two members are abutted, the alignment of the members and the gap shall be within the tolerances specified in Clause 14.4.4.2.

14.3.3 Cutting

A1 Cutting may be by sawing, shearing, cropping, machining, thermal cutting (including laser cutting and plasma cutting) or water cutting processes, as appropriate.

Shearing of items over 16 mm thick shall not be carried out when the item is to be galvanized and subject to tensile force or bending moment unless the item is stress relieved subsequently.

Any cut surface not incorporated in a weld shall have a roughness not greater than the appropriate value given in Table 14.3.3. A cut surface to be incorporated in a weld shall comply with AS/NZS 1554.1, AS/NZS 1554.4 or AS/NZS 1554.5, as appropriate. A1

TABLE 14.3.3

MAXIMUM CUT SURFACE ROUGHNESS

Application	Maximum roughness, CLA μm
Normal applications, i.e. where the face and edges remain as-cut or with minor dressing	25
Fatigue applications (Detail categories as defined in Clause 11.5)	
detail category ≥ 80 MPa	12
detail category < 80 MPa	25

NOTES:

1 Roughness values may be estimated by comparison with surface replicas, such as the WTIA. Flame Cut Surface replicas.

2 Suitable techniques of flame cutting are given in WTIA Technical Note 5.

3 CLA ≡ *Centre Line Average Method.*

Cut surface roughness exceeding these values shall be repaired by grinding to give a value less than the specified roughness. Grinding marks shall be parallel to the direction of the cut.

Notches and gouges, not closer than $20t$ (where t is the component thickness) and not exceeding 1% of the total surface area on an otherwise satisfactory surface, are acceptable provided that imperfections greater than $t/5$ but not exceeding 2 mm in depth are removed by machining or grinding. Imperfections outside the above limits shall be repaired by welding in accordance with AS/NZS 1554.1, AS/NZS 1554.4 or AS/NZS 1554.5, as appropriate. A1

A re-entrant corner shall be shaped notch free to a radius of at least 10 mm.

14.3.4 Welding

A1 Welding shall comply with AS/NZS 1554.1, AS/NZS 1554.4 or AS/NZS 1554.5 as appropriate (see Clause 11.1.5), and welding of studs shall comply with AS/NZS 1554.2.

14.3.5 Holing

14.3.5.1 *General*

A1 A round hole for a bolt shall either be machine cut, or drilled full size, or subpunched 3 mm undersize and reamed to size, or punched full size.

A slotted hole shall be either machine cut, or punched in one operation, or formed by drilling two adjacent holes and completed by machine cutting.

Hand cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

A punched hole shall only be permitted in material whose yield stress (f_y) does not exceed 360 MPa and whose thickness does not exceed ($5600/f_y$) mm.

14.3.5.2 *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 comply with 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 $1.33d_f$ 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 $2.5d_f$ 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.

14.3.6 Bolting

14.3.6.1 *General*

All bolts and associated nuts and washers shall comply with the appropriate bolt material Standard specified in Clause 2.3.1. All material within the grip of the bolt shall be steel and no compressible material shall be permitted in the grip.

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.)

14.3.6.2 *Tensioned bolt*

A tensioned high strength bolt when installed during fabrication shall be installed in accordance with Clauses 15.2.4 and 15.2.5. The contact surfaces of a joint using a tensioned bolt shall be prepared in accordance with Clause 14.3.6.3.

14.3.6.3 *Preparation of surfaces in contact*

Preparation of surfaces in contact shall be as follows:

(a) *General* All oil, dirt, loose scale, loose rust, burrs, fins and any other defects on the surfaces of contact which will prevent solid seating of the parts in the snug-tight condition shall be removed.

NOTES:

1 If cleaning is necessary to meet these requirements, reference should be made to AS 1627.7.

2 A clean 'as-rolled' surface with tight mill scale is acceptable without further cleaning.

3 Snug-tight is defined in Clause 15.2.5.2.

(b) *Friction-type connection* For a friction-type connection, the contact surfaces shall be clean 'as-rolled' surfaces or equivalent and, in addition to satisfying the provisions of Item (a), shall be free from paint, lacquer, galvanizing or other applied finish unless the applied finish has been tested in accordance with Appendix J to establish the friction coefficient (see Clause 9.3.3.2).

In a non-coated connection, paint including any overspray shall be excluded from areas closer than one bolt diameter to any hole but not less than 25 mm from the edge of any hole and all areas within the bolt group.

(c) *Bearing-type connection* For a bearing-type connection, an applied finish on the contact surfaces shall be permitted.

14.3.7 Pinned connection

Pins and holes shall be finished so that the forces are distributed evenly to the joint plies.

14.4 TOLERANCES

14.4.1 General

The tolerance limits of this Clause shall be satisfied after fabrication is completed and any corrosion protection has been applied. Unless otherwise specified, the tolerance on all structural dimensions shall be ±2 mm.

14.4.2 Notation

For the purpose of this Clause—

a_0, a_1 = out-of-square dimensions of flanges

a_2, a_3 = diagonal dimensions of a box section

b = lesser dimension of a web panel

b_f = width of a flange

d = depth of a section

d_o = overall depth of a member including out-of-square dimensions

d_1 = clear depth between flanges ignoring fillets or welds

e = web off-centre dimension

l = member length

Δ_f = out-of-flatness of a flange plate

Δ_v = deviation from verticality of a web at a support

Δ_w = out-of-flatness of a web

14.4.3 Cross-section

A1 After fabrication, the tolerances on any cross-section of a rolled section or a welded I-section shall be those specified in AS/NZS 3679.1 or AS/NZS 3679.2, as appropriate, in respect of depth, flange width, flange thickness, web thickness, out-of-square, and web off-centre.

For any built-up section, the deviations from the specified dimensions of the cross-section shall not exceed the following:

(a) Depth of a section (d) (see Figure 14.4.3(1))

for $d \leq 900$, ±3 mm

for $900 < d \leq 1800$, $\pm\left[3 + \frac{(d - 900)}{300}\right]$ mm

for $d > 1800$, ±6 mm

(b) Width of a flange (b_f) (see Figure 14.4.3(1)

for all b_f, ±6 mm

(c) Out-of-square of an individual flange (a_0 or a_1) (see Figure 14.4.3(1))

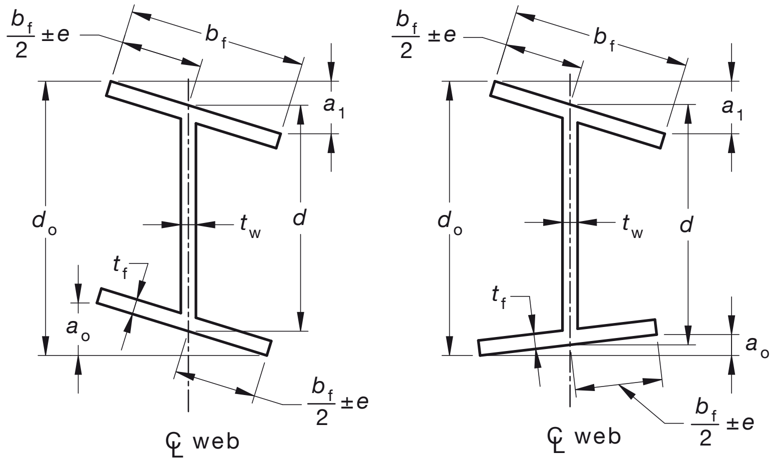
for $b_f \leq 600$ mm, ±3 mm

for $b_f > 600$ mm, $\pm\left(\frac{b_f}{200}\right)$ mm

(d) Total out-of-square of two flanges ($a_0 + a_1$) (see Figure 14.4.3(1))

for $b_f \leq 600$ mm, ± 6 mm

for $b_f > 600$ mm, $\pm\left(\frac{b_f}{100}\right)$ mm



NOTES:

1 Dimensions d, d_o, a_o and a_1 are measured parallel to the centreline of the web. Dimensions b_f and $(0.5b_f \pm e)$ are measured parallel to the plane of the flange.

2 Dimension d is measured at the centreline of the web.

FIGURE 14.4.3(1) TOLERANCES ON A CROSS-SECTION

(e) Out-of-flatness a of web (Δ_w) (see Figure 14.4.3(2))

$d_1/150$ mm for unstiffened web,

$b/100$ mm for stiffened web with intermediate stiffeners,

measured on a gauge length in the direction of d_1 or b, as appropriate.

(f) Deviation from verticality of a web at a support (Δ_v) (see Figure 14.4.3(2))

for $d \leq 900$ mm, ± 3 mm

for $d > 900$ mm, $\pm\left(\frac{d}{200}\right)$ mm

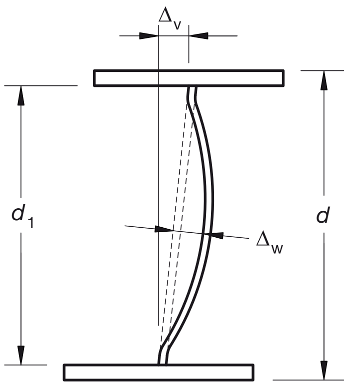


FIGURE 14.4.3(2) TOLERANCES ON A WEB

(g) Tolerance on shape of a built-up box section (see Figure 14.4.3.(3))

A built-up box section shall not deviate at the diaphragm from the prescribed shape by more than ±5 mm or $\pm[(a_2 + a_3)/400]$ mm, whichever is greater, unless connection requirements necessitate more stringent tolerances.

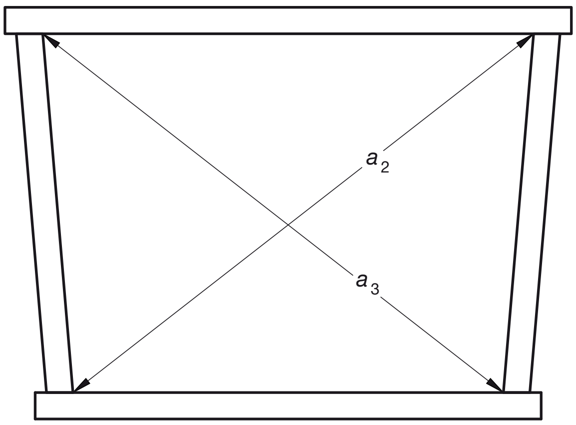


FIGURE 14.4.3(3) TOLERANCE ON SHAPE OF A BOX SECTION

(h) Off-centre of a web (e) (see Figure 14.4.3(4)) ±6 mm

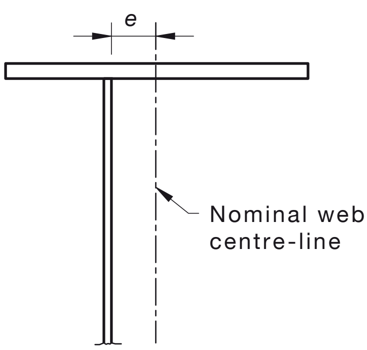


FIGURE 14.4.3(4) TOLERANCE ON OFF-CENTRE OF A WEB

(j) Out-of-flatness of a flange (Δ_f) (see Figure 14.4.3(5))

A1

$$\text{for } b_f \leq 450 \text{ mm} \pm \left(\frac{b_f}{150}\right) \text{mm}$$

$$\text{for } b_f > 450 \text{ mm} \pm 3 \text{ mm}$$

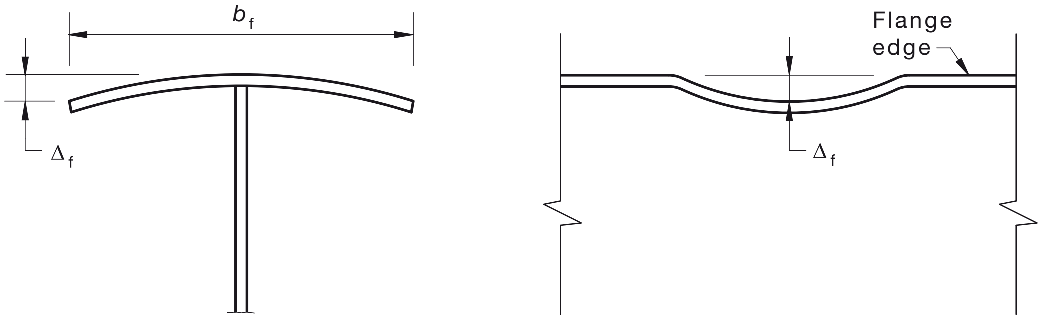


FIGURE 14.4.3(5) TOLERANCE ON OUT-OF-FLATNESS OF A FLANGE

14.4.4 Compression member

14.4.4.1 *Straightness*

A member shall not deviate about either principal axis from a straight line drawn between end points by an amount exceeding $l/1000$ or 3 mm whichever is the greater.

14.4.4.2 *Full contact splice*

If the ends of two butting lengths of a member, or the end of a member and the contact face of an adjoining cap plate or baseplate, are required to be in full contact, such a requirement shall be deemed to be satisfied if the bearing surfaces are prepared so that when the abutting member length or lengths are aligned to within the tolerance specified in Clause 15.3.3, the maximum clearance between the abutting surfaces shall not exceed 1 mm, and shall also not exceed 0.5 mm over at least 67% of the contact area.

14.4.4.3 *Length*

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

14.4.5 Beam

14.4.5.1 *Straightness*

A beam shall not deviate from a straight line drawn between the ends of the beam by more than the following:

(a) *Camber*—measured with the web horizontal on a test surface (see Figure 14.4.5.1(a)). The tolerance on specified camber shall not exceed $l/1000$ or 10 mm whichever is the lesser.

(b) *Sweep*—measured with the web vertical (see Figure 14.4.5.1(b)). The sweep in plan shall not exceed $l/1000$ or 3 mm whichever is the greater.

14.4.5.2 *Length*

The length of a beam shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

14.4.6 Tension member

14.4.6.1 *Straightness*

A member shall not deviate from a straight line drawn between end points by more than $l/500$, where l is the length between end points.

14.4.6.2 *Length*

The length of a tension member shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

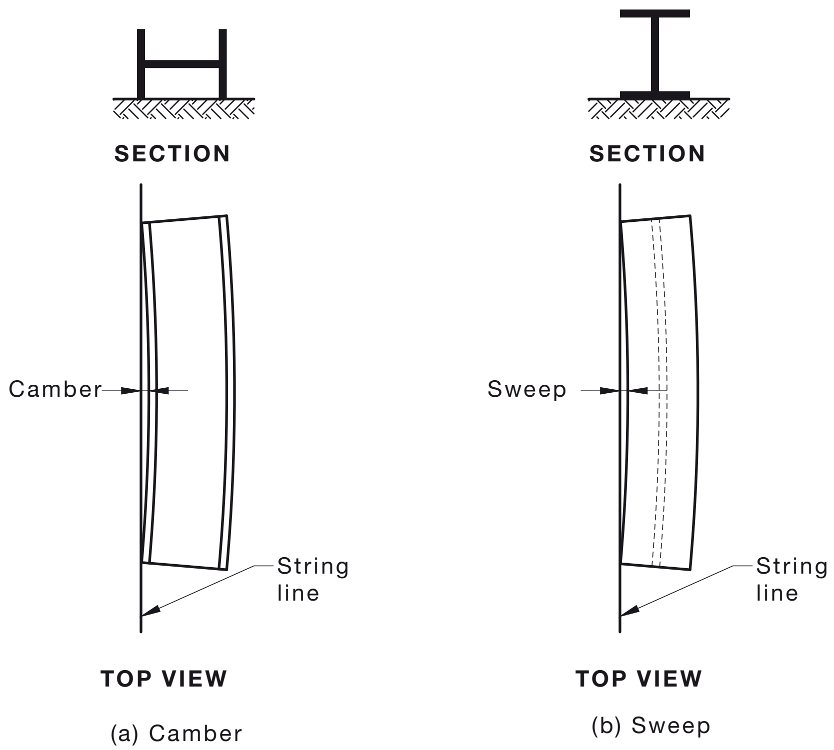


FIGURE 14.4.5.1 MEASUREMENT OF CAMBER AND SWEEP

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 comply with Clauses 14.3.6, 15.2.3, 15.2.4 and 15.2.5, unless it can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby.

Grouting at supports which does not satisfy the requirements of Clause 15.5 shall be rejected.

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.

15.1.3 Equipment support

Equipment supported on partly erected steelwork shall not induce actions in the steel greater than the design capacities permitted in this Standard.

15.1.4 Reference temperature

Dimensions shall be set out on the basis of a reference temperature of 20 degrees Celsius.

15.2 ERECTION PROCEDURES

15.2.1 General

The requirements specified in Clause 14.3 shall also be observed during the erection of the steel frame and during any modifications to the steelwork in the course of erection.

This requirement shall apply to the following:

(a) Full contact splices (see Clause 14.3.2).

(b) Cutting (see Clause 14.3.3).

(c) Welding (see Clause 14.3.4).

(d) Holing (see Clause 14.3.5).

(e) Bolting (see Clause 14.3.6).

Throughout the erection of the structure, the steelwork shall be securely bolted or fastened to ensure that it can adequately withstand all loadings liable to be encountered during erection, including, where necessary, those from erection plant and its operation. Any temporary bracing or temporary restraint shall be left in position until such time as erection is sufficiently advanced as to allow its safe removal.

All connections for temporary bracing and members to be provided for erection purposes shall be made in such a manner as not to weaken the permanent structure or to impair its serviceability. All welding of such connections and their removal shall be in accordance A1 with AS/NZS 1554 (series).

15.2.2 Delivery, storage and handling

Members, components and fasteners shall be handled and stacked in such a way that damage is not caused to them. Means shall be provided to minimize damage to the corrosion protection on the steelwork.

All work shall be protected from damage in transit. Particular care shall be taken to stiffen free ends, prevent permanent distortion, and adequately protect all surfaces prepared for full contact splices. All bolts, nuts, washers, screws, small plates and articles generally shall be suitably packed and identified.

15.2.3 Assembly and alignment

All matching holes shall align with each other so that a gauge or drift, equal in diameter to that of the bolts, shall pass freely through the assembled contact faces at right angles to them. Drifting to align holes shall be done in a manner that will not distort the metal nor enlarge the holes.

Each part of the structure shall be aligned as soon as practicable after it has been erected. Permanent connections shall not be made between members until sufficient of the structure has been aligned, levelled, plumbed and temporarily connected to ensure that members will not be displaced during subsequent erection or alignment of the remainder of the structure.

Each bolt and nut shall be assembled with at least one washer. A washer shall be placed under the rotating component. 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 suitable tapered washer shall be used against the sloping surface. The non-rotating component shall be placed against the tapered washer.

Bolting categories 4.6/S and 8.8/S shall be installed to the snug-tight condition specified in Clause 15.2.5.2(a).

Hardened or plate washers shall be used under both the bolt head and nut for any slotted and oversize holes as specified in Clause 14.3.5.2(b).

15.2.4 Assembly of a connection involving tensioned bolts

15.2.4.1 *Placement of a nut*

The nut shall be placed so that the mark specified in AS/NZS 1252 to identify a high strength nut is visible after tightening.

15.2.4.2 *Packing*

Packing shall be provided wherever necessary to ensure that the load-transmitting plies are in effective contact when the connection is tightened to the snug-tight condition defined in Clause 15.2.5.2(a). All packing shall be steel with a surface condition similar to that of the adjacent plies.

15.2.4.3 *Tightening pattern*

Snug-tightening and final tensioning of the bolts in a connection shall proceed from the stiffest part of the connection towards the free edges.

High strength structural bolts that are to be tensioned may be used temporarily during erection to facilitate assembly, but if so used they shall not be finally tensioned until all bolts in the connection have been snug-tightened in the correct sequence.

15.2.4.4 *Retensioning*

Retensioning of bolts which have been fully tensioned shall be avoided, except that if retensioning is carried out, it shall only be permitted once and only where the bolt remains in the same hole in which it was originally tensioned and with the same grip.

Retensioning of galvanized bolts shall not be permitted.

Under no circumstances shall bolts which have been fully tensioned be reused in another hole.

Touching up or retensioning of previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as retensioning.

15.2.5 Methods of tensioning

15.2.5.1 *General*

The method of tensioning shall be in accordance with either Clause 15.2.5.2 or Clause 15.2.5.3.

In the completed connection, all bolts shall have at least the minimum bolt tension specified in Table 15.2.5.1 when all bolts in the bolt group are tightened.

TABLE 15.2.5.1

MINIMUM BOLT TENSION

Nominal diameter of bolt	Minimum bolt tension kN
M16	95
M20	145
M24	210
M30	335
M36	490

A1 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, as specified in AS 4291.1.

15.2.5.2 *Part-turn method of tensioning*

Tensioning of bolts by the part-turn method shall be in accordance with the following procedure:

(a) On assembly, all bolts in the connection shall be first tightened to a snug-tight condition to ensure that the load-transmitting plies are brought into effective contact.

Snug-tight is the tightness attained by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

(b) After completing snug-tightening, location marks shall be established to mark the relative position of the bolt and the nut and to control the final nut rotation.

Observation of the final nut rotation may be achieved by using marked wrench sockets, but location marks shall be permanent when required for inspection.

(c) Bolts shall be finally tensioned by rotating the nut by the amount given in Table 15.2.5.2. During the final tensioning, the component not turned by the wrench shall not rotate.

TABLE 15.2.5.2

NUT ROTATION FROM THE SNUG-TIGHT CONDITION

Bolt length (underside of head to end of bolt)	Disposition of outer face of bolted parts (see Notes 1, 2, 3 and 4)		
	Both faces normal to bolt axis	One face normal to bolt axis and other sloped	Both faces sloped
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters (see Note 5)	2/3 turn	5/6 turn	1 turn

NOTES:

1 Tolerance on rotation: for 1/2 turn or less, one-twelfth of a turn (30°) over and nil under tolerance; for 2/3 turn or more, one-eighth of a turn (45°) over and nil under tolerance.

2 The bolt tension achieved with the amount of nut rotation specified in Table 15.2.5.2 will be at least equal to the minimum bolt tension specified in Table 15.2.5.1.

3 Nut rotation is the rotation relative to the bolt, regardless of the component turned.

4 Nut rotations specified are only applicable to connections in which all material within the grip of the bolt is steel.

5 No research has been performed to establish the turn-of-nut procedure for bolt lengths exceeding 12 diameters. Therefore, the required rotation should be determined by actual test in a suitable tension measuring device which simulates conditions of solidly fitted steel.

15.2.5.3 *Tensioning by use of direct-tension indication device*

Tensioning of bolts using a direct-tension indication device shall be in accordance with the following procedure:

(a) The suitability of the device shall be demonstrated by testing a representative sample of not less than three bolts for each diameter and grade of bolt in a calibration device capable of indicating bolt tension. The calibration test shall demonstrate that the device indicates a tension not less than 1.05 times the minimum bolt tension specified in Table 15.2.5.1.

(b) On assembly, all bolts and nuts in the connection shall be first tightened to a snug-tight condition defined in Clause 15.2.5.2(a).

(c) After completing snug-tightening, the bolt shall be tensioned to provide the minimum A1 bolt tension specified in Table 15.2.5.1. This shall be indicated by the tension indication device.

NOTE: Tensioning of bolts using a direct-tension indication device should also be in accordance with the manufacturer's specification.

15.3 TOLERANCES

15.3.1 Location of anchor bolts

Anchor bolts shall be restrained in position both in a vertical and a horizontal direction during all setting-in operations.

Anchor bolts shall be set out in accordance with the erection drawings. They shall not vary from the positions shown on the erection drawings by more than the following: (See Figure 15.3.1.)

(a) 3 mm centre-to-centre of any two bolts within an anchor bolt group, where an anchor bolt group is defined as the set of anchor bolts which receives a single fabricated steel member.

(b) 6 mm centre-to-centre of adjacent anchor bolt groups.

(c) Maximum accumulation of 6 mm per 30 000 mm along an established column line of multiple anchor bolt groups, but not to exceed a total of 25 mm. The established column line is the actual field line most representative of the centres of the as-built anchor bolt groups along a line of columns.

(d) 6 mm from the centre of any anchor bolt group to the established column line through that group.

Anchor bolts shall be set perpendicular to the theoretical bearing surface, threads shall be protected and free of concrete and nuts shall run freely on the threads.

The projection of the end of the anchor bolt from the theoretical bearing surface shall not be more than 25 mm longer nor 5 mm shorter than that specified.

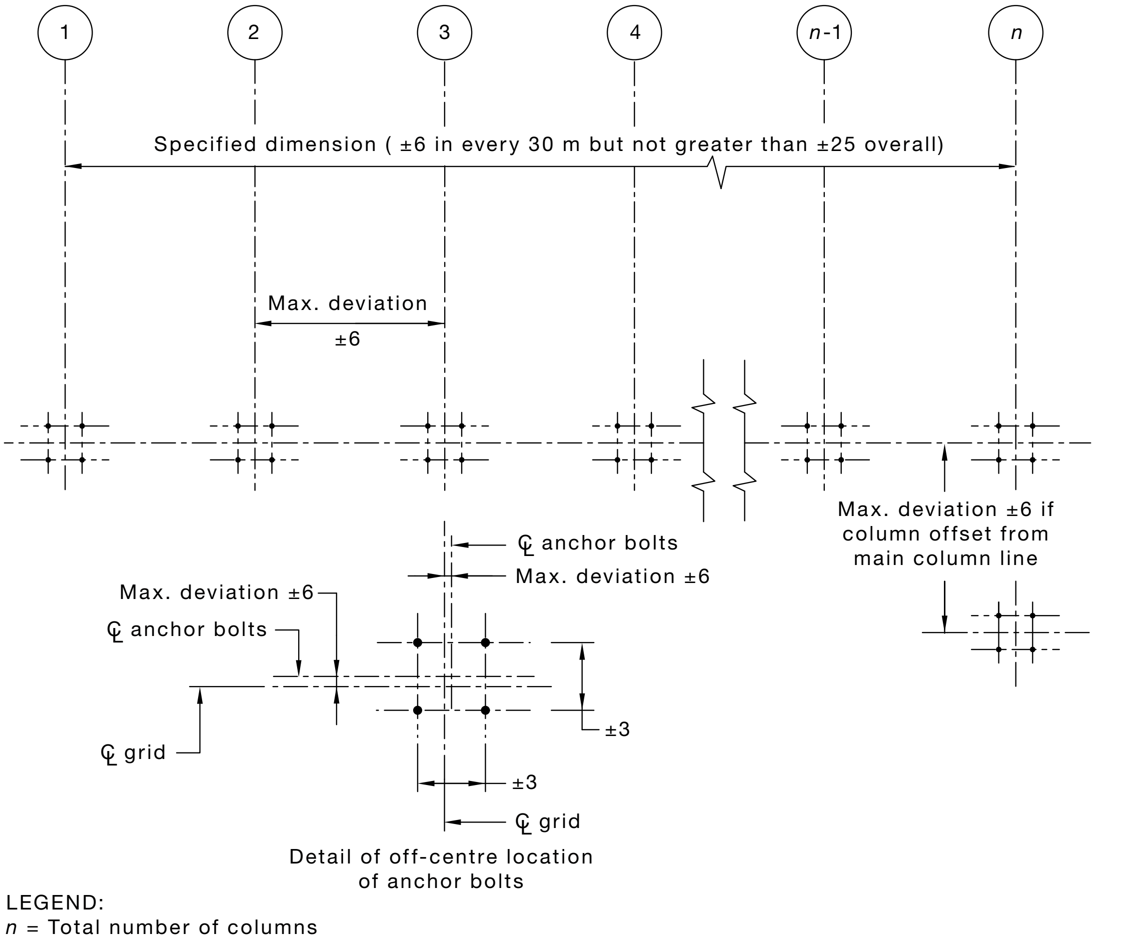


FIGURE 15.3.1 TOLERANCES IN ANCHOR BOLT LOCATION

15.3.2 Column base

15.3.2.1 *Position in plan*

The position in plan of a steel column base shall not deviate from its correct value by more than 6 mm along either of the principal setting out axes.

15.3.2.2 *Level*

The level of the underside of a steel base plate shall not deviate from its correct value by more than ±10 mm.

15.3.2.3 *Full contact*

If full contact is specified, the requirements of Clause 14.4.4.2 shall be satisfied, unless shims are used to reduce the measureable gaps to values specified in Clause 14.4.4.2.

Packs, shims and other supporting devices shall be flat and of the same steel grade as the member. If such packings are to be subsequently grouted, they shall be placed so that the grout totally encloses them with a minimum cover of 50 mm.

15.3.3 Plumbing of a compression member

The alignment and plumbing of a compression member shall be in accordance with both of the following requirements:

(a) The deviation of any point above the base of the compression member from its correct position shall not exceed height/500 or as follows, whichever is the lesser:

(i) For a point up to 60 m above the base of the member ....25 mm.

(ii) For a point more than 60 m above the base of the member ....25 mm plus 1 mm for every 3 m in excess of 60 m up to a maximum of 50 mm.

(b) The deviation of the top of the compression member from its correct position relative to the bottom of the member from one storey to the next shall not exceed storey height/500.

15.3.4 Column splice

A column splice shall conform to the following requirements:

(a) The level of the centre-line of a column splice shall not deviate from its correct level by more than ±10 mm.

(b) The position in plan of a column splice shall be in accordance with the plumbing tolerances specified in Clause 15.3.3.

(c) The plan position of each spliced member relative to the other shall not deviate by more than 2 mm from its correct position along either of the principal setting-out axes.

15.3.5 Level and alignment of a beam

In erecting a structure, a beam shall be deemed to be correctly positioned when—

(a) all connections including splices are completed;

(b) the maximum sweep in the beam is less than $l_b/500$, where l_b is the length between points of effective bracing or restraint;

(c) a beam is within ±10 mm of its correct level at connections to other members; and

(d) a web of a beam is within ±3 mm horizontally of its correct position at connections to other members.

15.3.6 Position of a tension member

A tension member shall not deviate from its correct position relative to the members to which it is connected by more than 3 mm along any setting-out axis.

15.3.7 Overall building dimensions

The overall building dimensions shall not deviate from the correct values by more than the following:

(a) Length (see Figure 15.3.7.1)

$$\text{for } \Sigma l_c \leq 30\text{ m}, \Sigma\Delta l_c \leq \pm 20\text{ mm}$$

$$\text{for } \Sigma l_c > 30\text{ m}, \Sigma\Delta l_c \leq \pm\{20\text{ mm} + 0.25(\Sigma l_c - 30)\text{ mm}\}$$

(b) Height (see Figure 15.3.7.2)

$$\text{for } \Sigma h_b \leq 30\text{ m}, \Sigma\Delta h_b \leq \pm 20\text{ mm}$$

$$\text{for } \Sigma h_b > 30\text{ m}, \Sigma\Delta h_b \leq \pm\{20\text{ mm} + 0.25(\Sigma h_b - 30)\text{ mm}\}$$

provided that—

(i) the distance between adjacent steel column centres (l_c) at every section does not deviate by more than ±15 mm from the correct length;

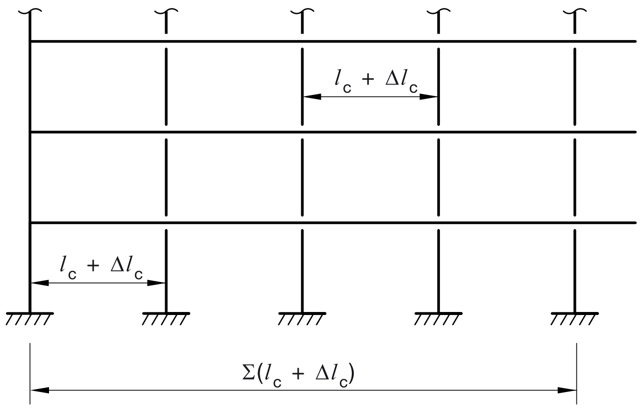
(ii) the vertical distance between tops of beams (h_b) at every section does not deviate by more than ±20 mm from the correct values; and

(iii) all other tolerances in this Section are complied with.

For the purposes of this Clause—

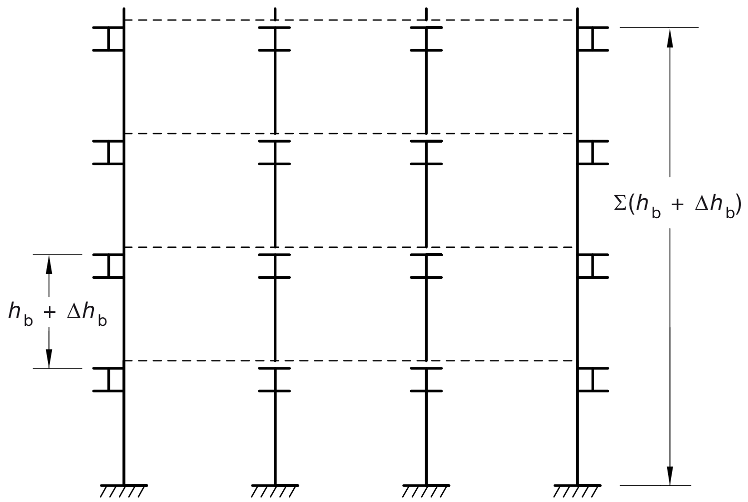
Σl_c = the correct overall length of steelwork, being the centre-to-centre distance of the extreme columns as shown in Figure 15.3.7.1, at any location along the building, and

Σh_b = the correct overall height of steelwork, being the vertical distance from underside of column baseplate to the top of the finished floor level shown in Figure 15.3.7.2, at any location along the building



LEGEND:
l_c = Distance between columns
Δl_c = Deviation from l_c
Σl_c = Correct overall length of steelwork
$\Sigma \Delta l_c$ = Deviation from Σl_c

FIGURE 15.3.7.1 DEVIATIONS IN LENGTH (VERTICAL SECTION)



LEGEND:
h_b = Distance between top of beams
Δh_b = Deviation from h_b
Σh_b = Correct overall length of steelwork
$\Sigma \Delta h_b$ = Deviation from Σh_b

FIGURE 15.3.7.2 DEVIATIONS IN HEIGHT (VERTICAL SECTION)

15.4 INSPECTION OF BOLTED CONNECTIONS

15.4.1 Tensioned bolts

The methods of tensioning specified in Clause 15.2.5 shall comply with the following requirements:

(a) *Part-turn tensioning* The correct part-turn from the snug-tight position shall be measured or observed.

(b) *Direct-tensioning indication device* The minimum tension developed in the bolt shall be indicated directly by the device.

NOTES:

1 The manufacturer's recommendations for inspection procedures should be followed when using a direct-tensioning indication device.

2 The use of a torque wrench for inspection is considered suitable only to detect gross undertensioning. A procedure for such use is detailed in Appendix K.

15.4.2 Damaged items

Bolts, nuts and washers which, on visual inspection, show any evidence of physical defects shall be removed and replaced by new items.

15.5 GROUTING AT SUPPORTS

15.5.1 Compression member base or beam

Bedding under a compression member base plate or a bearing of a beam on masonry and concrete shall be provided by grout or mortar.

Grouting or packing shall not be carried out until a sufficient portion of the structure (for multistorey buildings, a sufficient number of bottom column lengths) has been aligned, levelled and plumbed and adequately braced by other structural members which have been levelled and are securely held by their permanent fastenings. Steel packing or levelling nuts on the anchor bolts shall be under the base plate to support the steelwork. The space under the steel shall be thoroughly cleaned and be free from moisture immediately before grouting.

15.5.2 Grouting

Grout shall completely fill the space to be grouted and shall either be placed under pressure or placed by ramming against fixed supports.

Grout shall comply with AS 3600.

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.

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.

16.3 CLEANING

Surfaces of existing material, which are to be strengthened, repaired or welded, shall be cleaned of dirt, rust, and other foreign matter except adherent surface protection. The portions of such surfaces that are to be welded shall be cleaned thoroughly of all foreign matter, including paint film, for a distance of 50 mm from each side of the outside lines of the welds.

16.4 SPECIAL PROVISIONS

16.4.1 Welding and cutting

The capacity of a member to carry loads while welding or oxygen cutting is being performed on it shall be determined in accordance with to the provisions of this Standard, taking into consideration the extent of cross-section heating of the member which results from the operation that is being performed.

16.4.2 Welding sequence

The welding sequence shall be chosen so as to minimize distortion of the member and ensure that its straightness remains within the appropriate straightness limits of Clauses 14.4.3, 14.4.4, 14.4.5 and 14.4.6.

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—the 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—the 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 comply with the requirements for strength if it is able to sustain the strength limit state test load for at least 15 minutes. 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 considered 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 comply with Sections 2 and 14, respectively. Any additional requirements of a manufacturing specification shall be complied with 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 comply with the requirements for strength if it is able to sustain the strength limit state test load for at least 5 minutes.

(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

No 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

REFERENCED DOCUMENTS

(Normative)

The following documents are referred to in this Standard:

A1

AS		
1101	Graphical symbols for general engineering	
1101.3	Part 3:	Welding and non-destructive examination
1110	ISO metric hexagon bolts and screws	
1110.1	Part 1:	Product grades A and B—Bolts
1110.2	Part 2:	Product grades A and B—Screws
1111	ISO metric hexagon bolts and screws	
1111.1	Part 1:	Product grade C—Bolts
1111.2	Part 2:	Product grade C—Screws
1112	ISO metric hexagon nuts	
1112.1	Part 1:	Style 1—Product grades A and B
1112.2	Part 2:	Style 2—Product grades A and B
1112.3	Part 3:	Product grade C
1112.4	Part 4:	Chamfered thin nuts—Product grades A and B
1170	Structural design actions	
1170.4	Part 4:	Earthquake actions in Australia
1210	Pressure vessels	
1275	Metric screw threads for fasteners	
1391	Metallic materials—Tensile testing at ambient temperature	
1418	Cranes, hoists and winches	
1418.1	Part 1:	General requirements
1418.3	Part 3:	Bridge, gantry and portal (including container cranes) and jib cranes
1418.5	Part 5:	Mobile cranes
1418.18	Part 18:	Crane runways and monorails
1530	Methods for fire tests on building materials, components and structures	
1530.4	Part 4:	Fire-resistance test of elements of construction
1657	Fixed platforms, walkways, stairways and ladders—Design, construction and installation	
1735	Lifts, escalators and moving walks	
1735.1	Part 1:	General requirements
1858	Electrodes and fluxes for submerged-arc welding	
1858.1	Part 1:	Carbon steels and carbon manganese steels
2074	Cast steels	
2205	Methods of destructive testing of welds in metal	
2205.2.1	Part 2.1:	Transverse butt tensile test
2327	Composite structures	
2327.1	Part 1:	Simply supported beams

A1

AS		
2670	Evaluation of human exposure to whole-body vibration	
2670.1	Part 1:	General requirements
2670.2	Part 2:	Continuous and shock-induced vibration in buildings (1 to 80 Hz)
3597	Structural and pressure vessel steel—Quenched and tempered plate	
3600	Concrete structures	
4291	Mechanical properties of fasteners made of carbon steel and alloy steel	
4291.1	Part 1:	Bolts, screws and studs
4291.2	Part 2:	Nuts with specified proof load values—Coarse thread
4458	Pressure equipment—Manufacture	
5100	Bridge design	
5100.1	Part 1:	Scope and general principles
5100.2	Part 2:	Design loads
5100.6	Part 6:	Steel and composite construction
AS/NZS		
1163	Cold-formed structural steel hollow sections	
1170	Structural design actions	
1170.0	Part 0:	General principles
1170.1	Part 1:	Permanent, imposed and other actions
1170.2	Part 2:	Wind actions
1170.3	Part 3:	Snow and ice actions
1252	High strength steel bolts with associated nuts and washers for structural engineering	
AS/NZS		
1554	Structural steel welding	
1554.1	Part 1:	Welding of steel structures
1554.2	Part 2:	Stud welding (steel studs to steel)
1554.4	Part 4	Welding of high strength quenched and tempered steels
1554.5	Part 5:	Welding of steel structures subject to high levels of fatigue loading
1559	Hot-dip galvanized steel bolts with associated nuts and washers for tower construction	
1594	Hot-rolled steel flat products	
1873	Powder-actuated (PA) hand-held fastening tools (series)	
2717	Welding—Electrodes—Gas metal arc	
2717.1	Part 1:	Ferritic steel electrodes
3678	Structural steel—Hot-rolled plates, floorplates and slabs	
3679	Structural steel	
3679.1	Part 1:	Hot-rolled bars and sections
3679.2	Part 2:	Welded I sections
4600	Cold-formed steel structures	
4855	Welding consumables—Covered electrodes for manual metal arc welding of non-alloy and fine grain steels—Classification	
4857	Welding consumables—Covered electrodes for manual metal arc welding of high-strength steels—Classification	

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NZS		
1170	Structural design actions	
1170.5	Part 5:	Earthquake actions—New Zealand
3404	Steel structures Standard	
ISO		
636	Welding consumables—Rods, wires and deposits for tungsten inert gas welding of non-alloy and fine-grain steels—Classification	
14341	Welding consumables—Wire electrodes and weld deposits for gas shielded metal arc welding of non alloy and fine grain steels—Classification	
16834	Welding consumables—Wire electrodes, wires, rods and deposits for gas-shielded arc welding of high strength steels—Classification	
17632	Welding consumables—Tubular cored electrodes for gas shielded and non-gas shielded metal arc welding of non-alloy and fine grain steels—Classification	
18276	Welding consumables—Tubular cored electrodes for gas-shielded and non-gas-shielded metal arc welding of high-strength steels—Classification	
EN		
13381	Test methods for determining the contribution to the fire resistance of structural members	
13381-4	Part 4:	Applied passive protection products to steel members
13381-8	Part 8:	Applied reactive protection to steel members
BS		
7910	Guide to methods for assessing the acceptability of flaws in metallic structures	

APPENDIX B

SUGGESTED DEFLECTION LIMITS

(Informative)

B1 SUGGESTED VERTICAL DEFLECTION LIMITS FOR BEAMS

The vertical deflection of beams may be controlled using the suggested limits given in Table B1. Alternatively, the guidance given in Appendix C of AS/NZS 1170.0 may be used, where appropriate.

A1

TABLE B1

SUGGESTED LIMITS ON CALCULATED VERTICAL DEFLECTIONS OF BEAMS

Type of beam	Deflection to be considered	Deflection limit (Δ) for span (l) (see Note 1)	Deflection limit (Δ) 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 $\frac{\Delta}{l} \leq \frac{1}{1000}$	$\frac{\Delta}{l} \leq \frac{1}{250}$ where provision is made to minimize the effect of movement, otherwise $\frac{\Delta}{l} \leq \frac{1}{500}$
All beams	The total deflection	$\frac{\Delta}{l} \leq \frac{1}{250}$	$\frac{\Delta}{l} \leq \frac{1}{125}$

NOTES:

1 Suggested deflection limits in this Table may not safeguard against ponding.

2 For cantilevers, the values of Δ/l given in this Table apply, provided that the effect of the rotation at the support is included in the calculation of Δ.

B2 SUGGESTED HORIZONTAL DEFLECTION LIMITS

A1

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.

A1 (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 may be used where appropriate.

APPENDIX C

CORROSION PROTECTION

(Informative)

C1 SCOPE

This Appendix applies to the corrosion protection of steel members and connection components.

C2 SYSTEMS

A1 Recommendations for the painting of steelwork installed within building structures and not exposed to rain or sun may be found in AS/NZS 2311. For other steelwork, including steelwork requiring long-life protection, recommendations on protection systems may be found in AS/NZS 2312.

The type of coating and surface preparation should be specified, after proper account has been taken of the use of the structure, climatic or other local conditions, maintenance provisions, and of the effects of the fabrication processes on previously applied coatings.

C3 STANDARDS

All steelwork which is to be painted after fabrication and before erection should be prepared and painted in accordance with the relevant Standards. A list of such Standards may be found in Paragraph C7.

C4 INACCESSIBLE SURFACES

Surfaces which will be in contact or near contact after fabrication or erection should receive their specified surface preparation and treatment prior to assembly. Such surfaces should be dry before assembly.

A1 This Paragraph does not apply to the interior of hollow sections conforming to AS/NZS 1163, or box sections, or connection surfaces for joints with friction type bolting category where bare steel interfaces are specified.

C5 PROTECTION DURING TRANSPORT AND HANDLING AFTER CORROSION PROTECTION

Structural members should be adequately protected during handling and transport to prevent damage to the corrosion protection. Units which are transported in nested bundles should be separable without damage to the units or their coatings. Care should be taken when handling long units or bundles. Consideration should be given to the use of lifting beams with appropriately spaced lifting points and slings, or to lifting with properly spaced fork-lift tines.

C6 REPAIRS TO CORROSION PROTECTION

Corrosion protection which has been damaged by welding or other causes should be restored before the structure is put into service. The damaged area should be dry and clean, free from dirt, grease, loose or heavy scale or rust before the corrosion protection is applied. The corrosion protection should be applied as soon as practicable and before noticeable oxidation of cleaned surfaces occurs. Damaged zinc coating should be restored by a suitable zinc paint.

C7 RELEVANT STANDARDS

A1

AS

1192	Electroplated coatings—Nickel and chromium	
1214	Hot-dip galvanized coatings and threaded fasteners (ISO metric coarse thread series)	
1627	Metal finishing—Preparation and pretreatment of surfaces	
1627.0	Part 0:	Method selection guide
1627.1	Part 1:	Removal of oil, grease and related contamination
1627.2	Part 2:	Power tool cleaning
1627.4	Part 4:	Abrasive blast cleaning of steel
1627.5	Part 5:	Pickling
1627.6	Part 6:	Chemical conversion treatment of metals
1627.9	Part 9:	Pictorial surface preparation standards for painting steel surfaces
1789	Electroplated zinc (electrogalvanized) coatings on ferrous articles (batch process)	
1856	Electroplated coatings—Silver	
1897	Electroplated coatings on threaded components (metric coarse series)	
1901	Electroplated coatings—Gold and gold alloys	
2239	Galvanic (sacrificial) anodes for cathodic protection	
2832	Cathodic protection of metals	
2832.3	Part 3:	Fixed immersed structures
2832.4	Part 4:	Internal surfaces
2832.5	Part 5:	Steel in concrete structures
3730	Guide to the properties of paints for buildings (series)	
4169	Electroplated coatings—Tin and tin alloys	

AS/NZS

1580	Paints and related materials—Methods of test (series)	
2311	Guide to the painting of buildings	
2312	Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings	
3750	Paints for steel structures	
3750.6	Part 6:	Full gloss polyurethane (two-pack)
3750.10	Part 10:	Full gloss epoxy (two-pack)
3750.11	Part 11:	Chlorinated rubber—High-build and gloss
3750.12	Part 12:	Alkyd/micaceous iron oxide
3750.13	Part 13:	Epoxy primer (two-pack)
3750.15	Part 15:	Inorganic zinc silicate paint
3750.17	Part 17:	Etch primers (single pack and two-pack)
3750.22	Part 22:	Full gloss enamel—Solvent-borne
4534	Zinc and zinc/aluminium alloy coatings on steel wire	
4680	Hot-dip galvanized (zinc) coatings on fabricated ferrous articles	
4792	Hot-dip galvanized (zinc) coatings on ferrous hollow sections, applied by a continuous or specialized process	

APPENDIX D

ADVANCED STRUCTURAL ANALYSIS

(Normative)

D1 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.

D2 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 recognize that the earthquake loads calculated in accordance with AS 1170.4 shall be assumed to correspond to the load at which the first significant plastic hinge forms in the structure.

APPENDIX E
SECOND ORDER ELASTIC ANALYSIS
(Normative)

E1 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 (λ_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.

E2 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 δ_b is the moment amplification factor for a braced member determined in accordance with Clause 4.4.2.2.

APPENDIX F

MOMENT AMPLIFICATION FOR A SWAY MEMBER

(Normative)

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}^* = the design end bending moment obtained from a first-order elastic analysis of the frame with sway prevented (i.e. a braced frame), and

$M_{fs}^* = M_f^* - M_{fb}^*$

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 δ_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 δ_b is the moment amplification factor for a braced member (see Clause 4.4.2.2).

APPENDIX G

BRACED MEMBER BUCKLING IN FRAMES

(Normative)

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 γ_1 and γ_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:

$$\gamma = \frac{\left(\dfrac{I}{l}\right)_m}{\sum \beta_e \alpha_{sr} \left(\dfrac{I}{l}\right)_r}$$

$(I/l)_m$ = 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)

β_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

α_{sr} = theoretical stability function multiplier, or the approximation shown in Figure G1 to account for the effect of the design axial force (N_r^*) in the braced restraining member on its flexural stiffness

In Figure G the value of ρ is calculated as follows:

$$\rho = \frac{N_r^*}{N_{ol\,r}}$$

where

$$N_{ol\,r} = \frac{\pi^2 EI_r}{l_r^2}$$

For a braced restraining member in tension, α_{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.

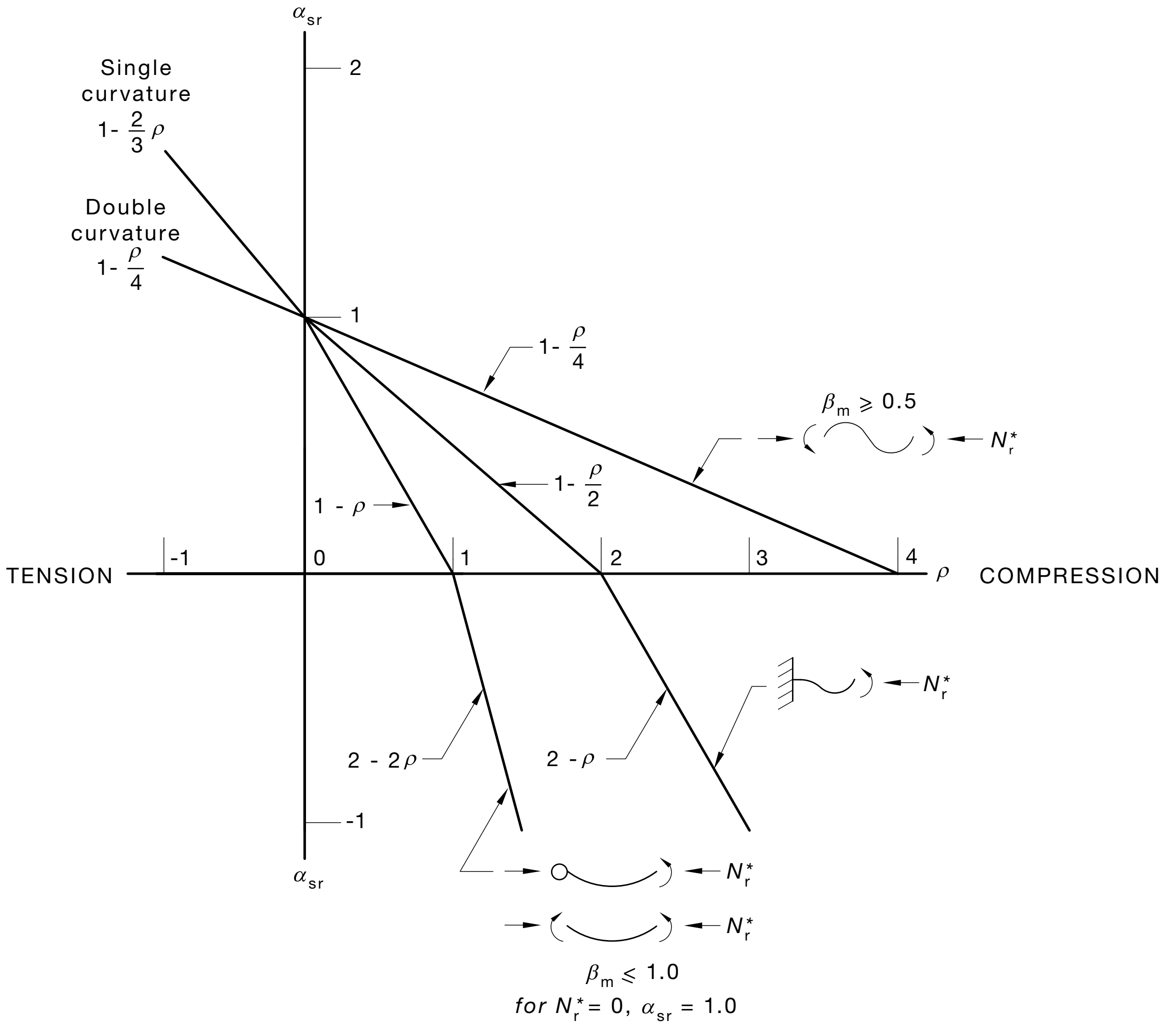


FIGURE G1 STABILITY FUNCTION MULTIPLIERS

APPENDIX H

ELASTIC RESISTANCE TO LATERAL BUCKLING

(Informative)

H1 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.

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 Paragraphs H2, H3 and H5. They may be used in conjunction with the method of design by buckling analysis of Clause 5.6.4.

H2 SEGMENTS RESTSRAINED 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:

$$M_{ob} = \alpha_m \alpha_l M_o \qquad \text{. . . H2(1)}$$

where

α_m is given in Clause 5.6.1(a), or may be approximated in accordance with Clause 5.6.4

$$\alpha_l = \sqrt{\left\{1+\left[\frac{0.4\alpha_m y_L}{M_o}\left(\frac{\pi^2 EI_y}{l^2}\right)\right]^2\right\}} + \left[\frac{0.4\alpha_m y_L}{M_o}\left(\frac{\pi^2 EI_y}{l^2}\right)\right] \qquad \text{. . . H2(2)}$$

M_o is given by Equation H4(2)

y_L = 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—

$$M_{oa} = M_o + \left[0.4\alpha_m y_L \left(\frac{\pi^2 EI_y}{l^2} \right) \right] \quad \text{. . . H2(3)}$$

H3 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:

$$M_{ob} = \alpha_{mc}\alpha_{lc} M_o \quad \text{. . . H3(1)}$$

where

$$\alpha_{mc} = \frac{(C_3 + C_4 K)}{\pi\sqrt{(1+K^2)}} \quad \text{. . . H3(2)}$$

$$\alpha_{lc} = 1 + \frac{\left(\frac{2 y_L K}{d_f 2} \right)}{\sqrt{\left[1 + \left(\frac{2 y_L K}{d_f 2} \right)^2 \right]}} \quad \text{. . . H3(3)}$$

M_o is given by Equation H4(2)

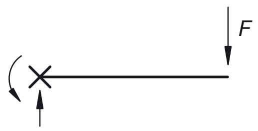
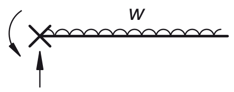
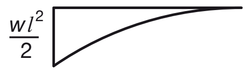
K is given by Equation H4(3)

C_3, C_4 are given in Table H3

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 equals 0 in Equation H3(2).

TABLE H3

FACTORS (C_3) AND (C_4) FOR BEAMS UNRESTAINED 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

NOTE: ✕ ≡ Full or partial restraint

H4 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:

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{l^2}\right)}\left\{\sqrt{\left[GJ + \frac{\pi^2 EI_w}{l^2} + \left(\frac{\beta_x^2}{4}\frac{\pi^2 EI_y}{l^2}\right)\right]} + \frac{\beta_x}{2}\sqrt{\left(\frac{\pi^2 EI_y}{l^2}\right)}\right\} \quad \text{. . . H4(1)}$$

For sections bent about an axis of symmetry, β_x equals 0, and Equation H4(1) simplifies to—

$$M_o = \left(\frac{\pi\sqrt{(EI_y GJ)}}{l}\right)\sqrt{\left(1 + K^2\right)} \quad \text{. . . H4(2)}$$

where

$$K = \sqrt{\left(\frac{\pi^2 EI_w}{GJl^2}\right)} \quad \text{. . . H4(3)}$$

E = 200 000 MPa

G ≈ 80 000 MPa

$I_w = \frac{I_y (d_f)^2}{4}$ for doubly-symmetric I-section,

$= I_{cy} d_f^2 \left(1 - \frac{I_{cy}}{I_y}\right)$ for a monosymmetric I-section

$$= \frac{b_f^3 t_f b_w^2}{48}\left(8 - \frac{3 b_f t_f b_w^2}{I_x}\right)$$ for a thin-walled channel section,

$$= 0$$ for an angle section, a tee-section, or a narrow rectangular section, and may be taken as 0 for a hollow section

$$J \approx \sum\left(\frac{bt^3}{3}\right)$$ for an open section

$$\approx \frac{4A_e^2}{\sum\left(\frac{b}{t}\right)}$$ for a hollow section where A_e is the area enclosed by the hollow section,

$$\beta_x = \frac{1}{I_x}\int\left(x^2 y + y^3\right)\mathrm{d}A - 2y_o$$

$$\approx 0.8 d_f\left(\frac{2I_{cy}}{I_y} - 1\right)$$ for a monosymmetric I-section

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.

H5 EFFECTS OF END RESTRAINTS

H5.1 Torsional end restraints

The approximations given in Paragraphs H2 and H3 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 β_t depends on the elastic stiffness (α_{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—

$$\beta_t \approx \frac{\alpha_{rz} l / GJ}{5(1+2K^2)/(1+K^2)}$$

H5.2 End restraints against lateral rotation

H5.2.1 *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 H4(2) and H4(3).

H5.2.2 *Segments unrestrained at one end*

The approximate elastic buckling moments obtained by using the values of C_3 and C_4 of Table H3 in Equations H4(1) to H4(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 H3(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 α_{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).

H6 REFERENCES

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APPENDIX I

STRENGTH OF STIFFENED WEB PANELS UNDER COMBINED ACTIONS

(Informative)

I1 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 I1) should satisfy the yielding criterion—

$$\left(\frac{R^*_w}{\phi b_{bf} t_w}\right)^2 - \frac{f^*_w}{\phi}\left(\frac{R^*_w}{\phi b_{bf} t_w}\right) + \left(\frac{f^*_w}{\phi}\right)^2 + \left(\frac{V^*_w}{0.6\phi A_w}\right)^2 \le \left(f_y\right)^2$$

where

f^*_w = $\dfrac{N^*_w}{A_w} + \left(0.77\dfrac{M^*_w}{Z_{we}}\right)$

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)

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

= $\dfrac{t_w (d_p)^2}{6}$

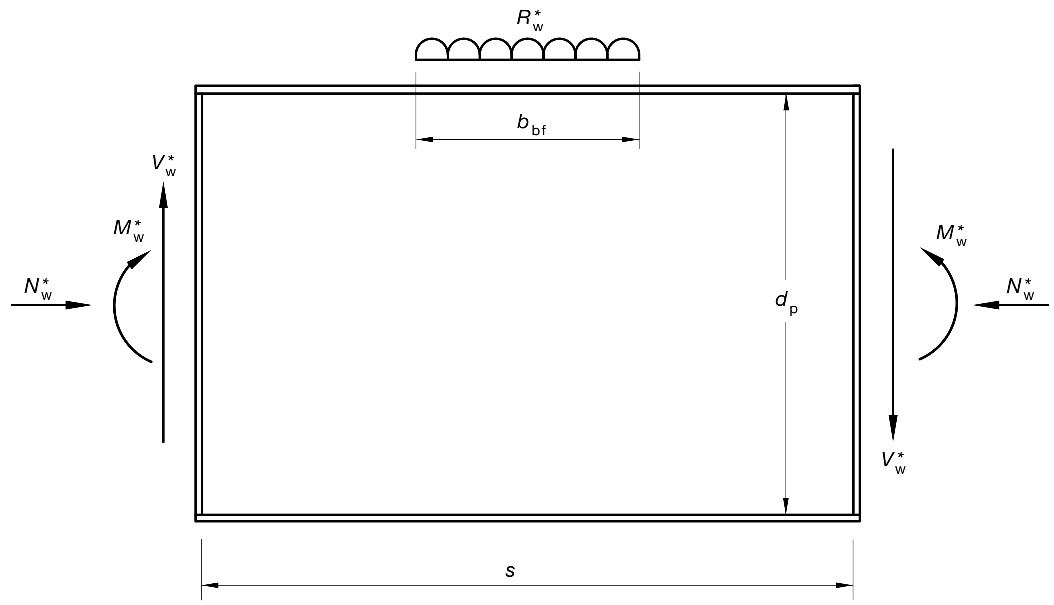


FIGURE I1 ACTIONS ON WEB PANEL

I2 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—

$$\left(\frac{R_w^*}{\phi R_{sb}}\right)+\left(\frac{N_w^*}{\phi N_{wo}}\right)+\left(\frac{V_w^*}{\phi V_v}\right)^2+\left(\frac{M_w^*}{\phi M_w}\right)^2 \leq 1$$

where

N_{wo} = nominal axial load capacity of the web panel if the web panel resisted axial load alone

$$= \frac{45 A_w f_y}{\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}} \leq A_w f_y$$

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

$$= \beta_w b_{bf} t_w f_y$$

$$\beta_w = 0.10 + \frac{20}{\left(\frac{d_e}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}}$$

$$d_e = \frac{1.9\sqrt{(b_{bf} d_p)}}{\alpha_w}$$

$$\alpha_w = \left[3.4+\left(\frac{2.2 d_p}{s}\right)\right]\left[0.4+\left(\frac{0.5 b_{bf}}{s}\right)\right]$$

APPENDIX J

STANDARD TEST FOR EVALUATION OF SLIP FACTOR

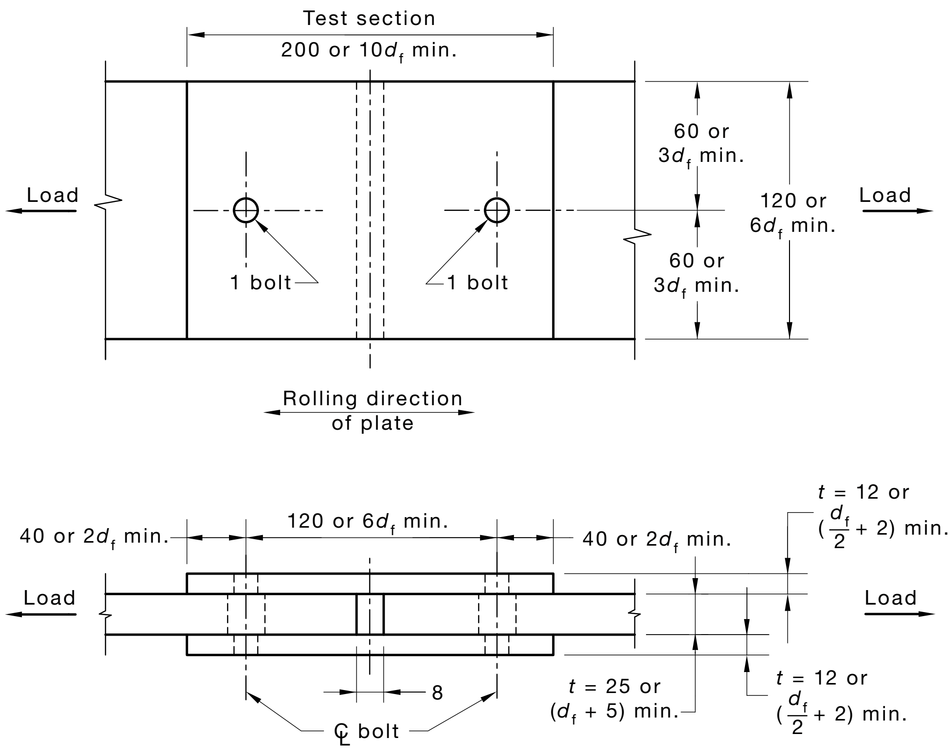
(Normative)

J1 TEST SPECIMENS

J1.1 Form

The standard test specimen shall be a symmetrical double coverplated butt connection as shown in Figure J1. 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.



NOTES:

1 d_f is the diameter of the bolt.

2 t is the thickness of the plate.

3 Holes in plates:

Cover plates . .. . 22 mm or d_f + 2 mm

Inner plates . . . . 23 mm or d_f + 3 mm

4 Length and width of inner plates outside of test section may be increased to suit the laboratory's testing facilities.

5 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.

DIMENSIONS IN MILLIMETRES

FIGURE J1 STANDARD TEST SPECIMEN

J1.2 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 15.2.5.1.

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:

$$N_{ti} = \frac{E\Delta \times 10^{-3}}{\dfrac{a_o}{A_o} + \left[\dfrac{a_t + \dfrac{t_n}{2}}{A_s}\right]} \qquad \ldots \text{J1}$$

where

N_{ti} = tension induced in the bolt, in kilonewtons

E = Young's modulus of elasticity, 200 000 MPa

Δ = 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.

J1.3 Number of specimens

Tests on at least three specimens shall be undertaken, but five is preferred as a practical minimum number.

J2 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 J2 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.

J3 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.

NOTES:

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.

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.

J4 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.

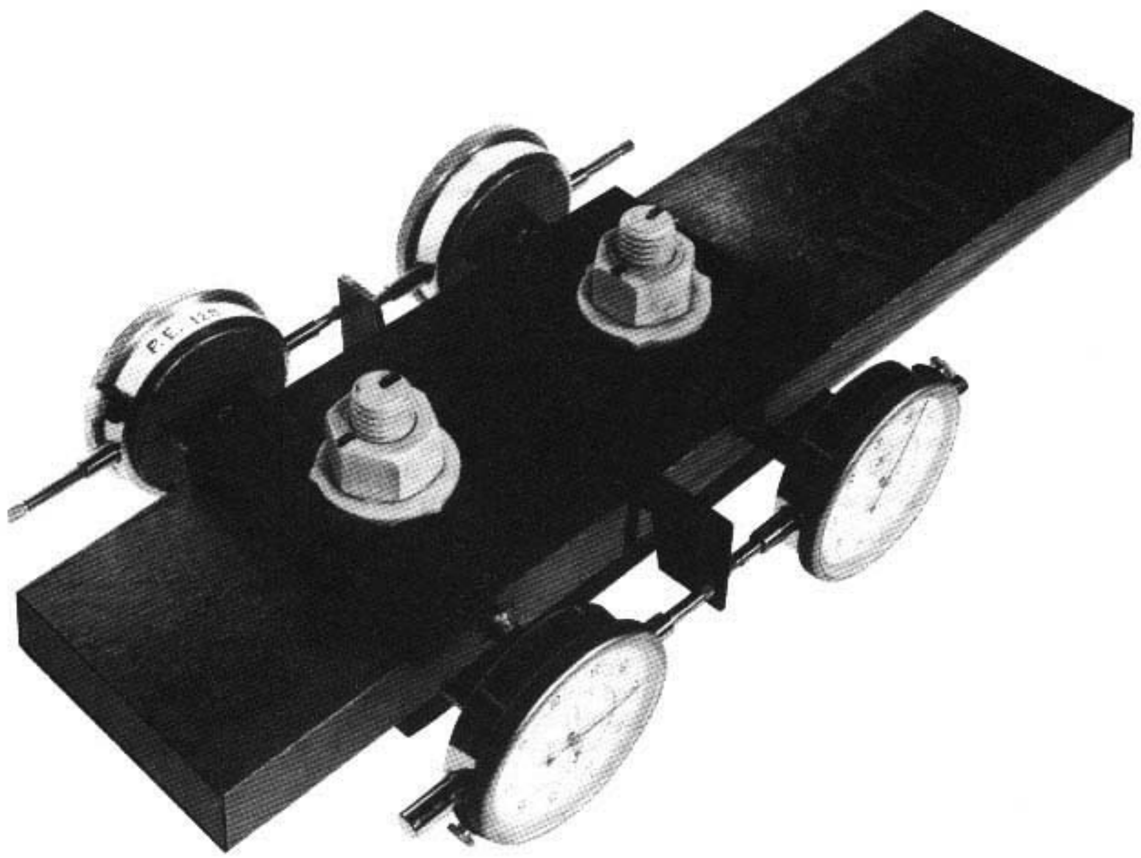


FIGURE J2 TYPICALLY INSTRUMENTED TEST ASSEMBLY

J5 SLIP FACTOR

The slip factor (μ) 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

μ_m = mean value of slip factor for all tests

δ = 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 = the number of specimens tested, each providing two estimates of μ

V_{si} = the measured slip load at the position of the ith bolt

N_{ti} = the tension induced in the ith bolt by the tensioning as calculated from Equation J1

However, if the calculated value of μ is less than the lowest of all values of μ_i, then μ may be taken as equal to the lowest value of μ_i.

APPENDIX K

INSPECTION OF BOLT TENSION USING A TORQUE WRENCH

(Informative)

K1 GENERAL

The correlation between the torque required to fully tension a calibration specimen and that which will be required on a bolt-nut assembly installed in a structural connection, will be materially affected by such factors as—

(a) the actual condition of the thread and the bearing face surface and their lubrication;

(b) the occurrence of galling during tensioning; and

(c) the time lapse between tensioning and inspection.

With due regard for these limitations, the procedure given in this Appendix is considered the most practical method for an independent assessment of whether gross undertensioning exists.

K2 CALIBRATION

The inspection wrench may be either a hand-operated or an adjustable-torque power-operated wrench. It should be calibrated at least once per shift, or more frequently if the need to closely simulate the condition of the bolts in the structure so demands.

The torque value determined during calibration may not be transferred to another wrench.

At least three bolts, desirably of the same size (the minimum length may have to be selected to suit the calibrating device) and condition as those under inspection, should be placed individually in a calibrating device capable of indicating bolt tension. A hardened washer should be placed under the part turned.

Each calibration specimen should be tensioned in the calibrating device by any convenient means to 1.05 times the minimum bolt tension specified for that diameter in Table 15.2.5.1. The inspection wrench then should be applied to the tensioned bolt, and the torque necessary to turn the nut or bolt head 5° (approximately 25 mm at 300 mm radius) in the tensioning direction should be determined. The average torque measured in the tests of at least three bolts should be taken as the job inspection torque.

K3 INSPECTION

Bolts represented by the sample which have been tensioned in the structure should be inspected by applying, in the tensioning direction, the inspection wrench with its job inspection torque to such proportion of the bolts in the structure as prescribed.

NOTE: For guidance, it is suggested that a suitable sample size would be 10% of the bolts, but not less than two bolts in each connection.

K4 ACTION

Where no further rotation occurs under the torque applied by the inspection wrench, the connection should be accepted as properly tensioned.

Where any nut or bolt head is turned by the application of the job-inspection torque, this torque should then be applied to all other bolts in the connection and all bolts whose nut or head is turned by the job inspection torque should be tensioned and re-inspected. Alternatively, the Fabricator or Erector may retension all of the bolts in the connection and then resubmit the connection for inspection.

INDEX

A1

'Text deleted'

AMENDMENT CONTROL SHEET

AS 4100—1998

Amendment No. 1 (2012)

REVISED TEXT

SUMMARY: This Amendment applies to the Preface, Clauses 1.1.1, 1.1.2, 1.3, 1.4, 2.2.1, 2.2.2, 2.3.1, 2.3.3, 2.3.4, 3.2.1, 3.2.3, 3.2.4, 3.2.5(new), 3.3, 3.11, 4.1.1, 5.13.1, 5.13.3, 5.13.4, 5.13.5, 6.3.3, 8.3.2, 8.3.3, 8.3.4, 8.4.2.2, 9.1.4, 9.1.9, 9.3.1, 9.3.2.5, 9.3.3.3, 9.7.1.1, 9.7.1.3, 9.7.2.1, 9.7.2.3, 9.7.2.7, 10.2, 10.3.1, 10.3.2, 10.3.3(new), 10.4.2, 10.4.3.1, 10.4.3.2, 10.4.3.3, 10.4.3.4(new), 10.5, 11.1.5, 11.2, 11.8.2, 12.5, 12.6.1, 12.6.2.2, Section 13, Clauses 14.3.3, 14.3.4, 14.3.5.1, 14.3.5.2, 14.4.3, 15.2.1 and 15.2.5.3, Appendices A, B and C, Tables 2.1, 3.4, 5.6.1, 6.3.3(1), 6.3.3(3), 9.3.1, 9.6.2, 9.7.3.10(1), 10.4.1, 10.4.4, 11.5.1(4) and 15.2.5.1, Figures 4.6.3.2 and 9.7.3.1, and Index.

Published on 29 February 2012.

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NOTES

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ISBN 0 7337 1981 3