

Australian/New Zealand Standard™

Bridge design

Part 6: Steel and composite construction



AS/NZS 5100.6:2017

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Part 6: Steel and composite construction

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PREFACE

This Standard was prepared by the Standards Australia Committee BD-090, Bridge Design, Sub-Committee 6, Steel and Composite Construction, to supersede AS 5100.6—2004.

This Standard is also designated as AUSTROADS publication AP-G51.6-17.

The objectives of the AS(AS/NZS) 5100 series are to provide nationally acceptable requirements for—

- (a) the design of road, rail, pedestrian and cyclist path bridges;
- (b) the specific application of concrete, steel, composite construction and timber construction methods, which embody principles that may be applied to other materials in association with relevant Standards;
- (c) the assessment of the load capacity of existing bridges; and
- (d) the strengthening and rehabilitation of existing bridges.

The objective of this Part (AS/NZS 5100.6) is to provide design rules for steel and steel-concrete composite bridges, or member within bridges.

Whereas earlier editions of the *Bridge design* were essentially administered by the infrastructure owners and applied to their own inventory, an increasing number of bridges are being built under the design-construct-operate principle and being handed over to the relevant statutory authority after several years of operation. This Standard includes clauses intended to facilitate the specification to the designer of the functional requirements of the owner to ensure the long-term performance and serviceability of the bridge and associated structure.

Significant differences between this edition and the 2004 edition are the following:

- (i) New fatigue provisions.
- (ii) New welding provisions.
- (iii) New Appendix (New Zealand-only) dealing with other steels.

Appendix H is for New Zealand only. Differences identified by the technical committee between the Standards referenced in Appendix H and Clause 2.2 were unable to be addressed in this revision so the Appendix allows for the existing practices in New Zealand to remain in place. Appendix J provides a commentary of the differences identified.

In line with Standards Australia policy, the words ‘shall’ and ‘may’ are used consistently throughout this Standard to indicate respectively, a mandatory provision and an acceptable or permissible alternative.

Statements expressed in mandatory terms in Notes to tables are deemed to be requirements of this Standard.

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

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STANDARDS AUSTRALIA/STANDARDS NEW ZEALAND

Australian/New Zealand Standard
Bridge design

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SECTION 1 SCOPE AND GENERAL**1.1 SCOPE**

This Standard sets out minimum requirements for the design, fabrication and erection of the structural steelwork in bridges. Wrought and cast iron structures may be checked in accordance with this Standard, using the appropriate material properties and capacity reduction factors.

The Standard also includes requirements for the design of other steel components of bridges including steel piers, steel railings, sign structures, and structural interaction between steel and concrete composite members.

1.2 EXCLUSIONS

This Standard does not cover the steelwork of the following structures, members and materials:

- (a) Bridges with orthotropic plate decks.
- (b) Cold-formed members other than those complying with AS/NZS 1163.
- (c) Steel members for which the value of yield stress (f_y) used in design exceeds 690 MPa.
- (d) Steel elements, other than packers, less than 3 mm thick.

1.3 APPLICATION

The requirements for bridges, members and materials specified in Items (a) to (d) of Clause 1.2, and for new and unusual bridge types shall be designated by the relevant authority.

In the design of steel-concrete composite members, the general requirements of AS 5100.5 pertaining to the design of concrete shall apply, where relevant, in addition to the requirements of this Standard.

NOTE: In New Zealand, the concrete structures design may be designated by the relevant authority.

The design of structural elements using non-ferrous metals such as aluminium alloys is not covered in this Standard. Where such components are to carry calculated stress, the applicable specification to be used in their design shall be designated by the relevant authority.

1.4 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

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

AS

- | | |
|---------------|---|
| 1110 | ISO metric hexagon bolts and screws—Products A and B—Bolts |
| 1111 | ISO metric hexagon bolts and screws—Product C—Bolts |
| 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 |
| 1210 | Pressure vessels |
| 1237 | Plain washers for metric bolts, screws and nuts for general purposes |
| 1237.1 | Part 1: General plan |
| 1237.2 | Part 2: Tolerances |
| 1275 | Metric screw threads for fasteners |
| 1391 | Metallic materials—Tensile testing at ambient temperature |
| 1420 | ISO metric hexagon socket head cap screws |
| 1831 | Ductile cast iron |
| 2074 | Cast steels |
| 2327 | Composite structures |
| 2327.1 | Part 1: Simply supported beams |
| 3597 | Structural and pressure vessel steel—Quenched and tempered plate |
| 4100 | Steel structures |
| 4312 | Atmospheric corrosivity zones in Australia |
| 5100 | Bridge design |
| 5100.1 | Part 1: Scope and general principles |
| 5100.2 | Part 2: Design loads |
| 5100.5 | Part 5: Concrete |
| 5100.7 | Part 7: Rating of existing bridges |
| 5104 | General principles on reliability for structures |
| AS/NZS | |
| 1163 | Structural steel hollow sections |
| 1170 | Structural design actions |
| 1170.0 | Part 0: General principles |
| 1252 | High strength steel bolts with associated nuts and washers for structural engineering |
| 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 |
| 1594 | Hot-rolled steel flat products |
| 3678 | Structural steel—Hot-rolled plates, floorplates and slabs |

AS/NZS

- 3679 Structural steel
- 3679.1 Part 1: Hot-rolled bars and sections
- 3679.2 Part 2: Welded I sections
- 5131 Structural steelwork—Fabrication and erection

AS/NZS ISO

- 3834 Quality requirements for fusion welding of metallic materials
- 3834.2 Part 2: Comprehensive quality requirements
- 3834.3 Part 3: Standard quality requirements
- 4671 Steel reinforcing materials

NZS

- 3404.1 Steel Structures Standard

AASHTO

- LRFD Bridge Design Specifications

ANSI/AWS

- D1.1 Structural Welding Code—Steel

ISO

- 7040 Prevailing torque type hexagon nuts (with non-metallic insert), style 1—Property classes 5.8 and 10
- 7041 Prevailing torque type hexagon nuts (with non-metallic insert), style 2—Property classes 9 and 12
- 7042 Prevailing torque type all metal hexagon nuts, style 2—Property classes 5, 8, 10 and 12.
- 10511 Prevailing torque type hexagon thin nuts (with non-metallic insert)
- 13918 Welding—Studs and ceramic ferrules for arc stud welding

1.5 NOTATION

Symbols used in this Standard are listed in Table 1.5.

Unless a contrary intention is given, the following applies:

- (a) Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.
- (b) The dimensional units for length, force and stress in all expressions or equations are to be taken as millimetres (mm), Newtons (N) and megapascals (MPa) respectively, unless specifically noted otherwise.
- (c) An asterisk (*) placed after a symbol as a superscript denotes a design action effect due to the design load for the ultimate limit state specified in AS 5100.2.

1.6 MATTERS FOR RESOLUTION BEFORE DESIGN COMMENCES

The matters for resolution listed below shall be confirmed as accepted by the relevant authority or owner of a bridge or associated structure before commencing the design process.

- 1 Requirements for bridges, members and materials specified in Items (a) to (d) of Clause 1.2, and for new and unusual bridges.
- 2 Design requirements for structural elements using non-ferrous metals (see Clause 1.3).

- 3 Requirements for steels for machined parts and for uses in other than structural member or elements (see Clause 2.2.4).
- 4 Requirements for structures, members and materials (see Clause 1.2).
- 5 Requirements for the fatigue design assessment method (see Clause 13.6).

1.7 WORKMANSHIP

The minimum requirements for fabrication and erection shall be in accordance with AS/NZS 5131.

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

1.8 ADDITIONAL REQUIREMENTS—NEW ZEALAND ONLY

In New Zealand, requirements for steel other than specified in Section 2 shall be in accordance with Appendix H.

NOTE: For purchasing guidelines for steels other than specified in Section 2, see Appendix J.

TABLE 1.5
NOTATION

Symbol	Definition
A	area of a cross-section
A_{bs}	area of bottom slab reinforcement, shown in Figure 6.8.5.2.1
A_{bt}	area of bottom transverse reinforcement, shown in Figure 6.8.5.2.1
A_{bv}	cross-sectional area of the transverse reinforcement per unit length of a beam placed in the bottom of a slab or haunch
A_c	area of concrete in a cross-section; <i>or</i> minor diameter area of the bolt
A_d	deducted area for fastener holes based on the gross area of a hole in the plane of its axis and not that of the fastener
A_e	effective area of the cross-section; <i>or</i> effective area of a diaphragm and flanges at the vertical cross-section through the reference point; <i>or</i> effective area of a vertical cross-section of the diaphragm and flanges, at the point under consideration; <i>or</i> area enclosed by the hollow section
A_{ep}	area of the end plate
A_{fb}	area of the bottom flange
A_{fc}	flange area at the critical cross-section
A_{ff}	area per unit width of the flange of a box girder (including longitudinal stiffeners)
A_{fg}	gross area of a flange
A_{fm}	flange area at the minimum cross-section; <i>or</i> lesser of the flange effective areas
A_{fn}	net area of a flange
A_g	gross area of the cross-section of the member or element
A_{he}	effective horizontal shear area of a diaphragm plate only
A_n	sum of the net areas of the flanges and the gross area of the web; <i>or</i> net area of the cross-section
A_o	nominal plain shank area of the bolt

Symbol	Definition
A_p	cross-sectional area of the pin
A_r	cross-sectional area of the reinforcement
A_{rs}	area of slab reinforcement within the effective width of the slab
A_s	area of the stiffener; <i>or</i> cross-sectional area of the structural steel section; <i>or</i> tensile stress area of a bolt
A_{se}	area of effective stiffener section; <i>or</i> effective cross-sectional area of a group of bearing stiffeners; <i>or</i> effective cross-sectional area of the stiffener; <i>or</i> effective area of the diaphragm and web junction; <i>or</i> effective area of the horizontal stiffeners; <i>or</i> effective cross-sectional area of a continuity member
A_t	area of the section to one side of the shear plane under consideration
A_{tf}	total area of both steel flanges and the longitudinal reinforcement included in the section
A_{ts}	effective cross-sectional area of the transverse reinforcement per unit length of beam, which is fully anchored in accordance with AS 5100.5, and crosses a shear plane in Figure 6.8.5.2.1 that is assumed to resist shear failure along that plane
A_{tt}	area of top transverse reinforcement, shown in Figure 6.8.5.2.1
A_{ve}	effective vertical shear area of a diaphragm plate only
A_{vea}	minimum value of the effective vertical shear area, shown in Figure 7.6.5.2.1
A_{veb}	minimum value of the effective vertical shear area, shown in Figure 7.6.5.2.1
A_w	area of the web; <i>or</i> effective shear area of a plug or slot weld; <i>or</i> gross sectional area of the web
A_z	sum of the areas of all holes in a zig-zag line extending progressively across a member or part of a member
a	a distance or dimension as defined in the text length between transverse stiffeners; <i>or</i> length of a panel; <i>or</i> clear distance between transverse web stiffeners; <i>or</i> transverse member spacing (or mean of adjacent spacings); <i>or</i> length of the plate panel (that is, the distance between transverse stiffeners); <i>or</i> panel dimension adjacent to an inclined web; <i>or</i> length of the longer side of a plate panel
a_e	minimum distance from the edge of a hole to the edge of a ply measured in the direction of the component of the force, plus half the diameter of the bolt
$a_{max.}$	maximum spacing of vertical stiffeners; <i>or</i> one-half of the sum of the panel widths on each side of the stiffener
a_o, a_1	out-of-square dimensions of flanges
a_2, a_3	diagonal dimensions of a box section
a_1, a_2, a_3	dimensions of bearing stress, shown in Figure 7.4.3.1
B	spacing between main beams of a U-frame; <i>or</i> spacing of the main beam webs at the level of the transverse member
B_c	length of the cantilever portion of a transverse member
B_f	total width of a stiffened flange between main beam webs
b	distance or dimension as defined in the text; <i>or</i> clear width of an element outstand from the face of a supporting plate element or the clear width of the element between the faces of the supporting plate elements; <i>or</i> distance between centres of adjacent beams or maximum distance when the beams are not exactly parallel; <i>or</i> total length of the section; <i>or</i>

Symbol	Definition
b (cont.)	width of panel; <i>or</i> mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration; <i>or</i> width of a diaphragm taken as the average of the widths at the top and bottom flange levels for boxes with sloping webs; <i>or</i> spacing of stiffeners or the distance between a stiffener and a beam flange and web boundary, as appropriate; <i>or</i> width of the concrete flange of the composite section; <i>or</i> lesser dimension of a web panel; <i>or</i> length of the shorter side of a plate panel; <i>or</i> 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
b_b	total width of the bearing obtained by dispersions at 1:1 from b_{bf} to the neutral axis, shown in Figure 5.12.1(A)
b_{bf}	width of the bearing obtained by dispersion at 1:2.5 from b_s to the underside of the flange, shown in Figure 5.12.1(A)
b_{bw}	width of the bearing obtained at the neutral axis to either side of the width b_{bf} , shown in Figure 5.12.1(A)
b_d	distance from a stiff bearing to the end of the member, shown in Figure 5.12.1(C); <i>or</i> width of bottom flange of a box girder, shown in Figure 7.6.1
b_e	width of a diaphragm at the point under consideration; <i>or</i> effective width of a plate element
$b_{eff,i}$	total effective width
b_{es}	outstand width of a stiffener from the face of a web
b_f	width of the flange
b_{fe}	effective width of a flange for checking the web restraint
b_{fo}	value used in calculating flange restraint factor; <i>or</i> half the clear distance between the centre-lines of adjacent webs or the distance from the centre-line of a web to the free edge of the flange or width of the top flange of a steel section
b_o	trunkated value of b_{bw} , shown in Figure 5.12.1(A)
b_s	length of a stiff bearing; <i>or</i> width of an angle stiffener
b_{so}	width of the flange outstand of a stiffener
b_w	depth of the web
b_1, b_2	greater and lesser leg lengths of an angle section; <i>or</i> dimension across a U stiffener or between U stiffeners
C_d	factor to allow for the effects of boundary shears
C_3, C_4	factors for beams unrestrained at one end
C_{4r}	reduced value of C_4
c	width between supports at a diaphragm of box girder, shown in Figure 7.6.1; <i>or</i> distance between centres of bearings
c_h	perpendicular distance to the centroid of an angle section from the face of the loaded leg of the angle
c_m	factor for unequal moments
d	depth of the steel section; <i>or</i> depth of web of a box girder, shown in Figure 7.6.1; <i>or</i> depth of the diaphragm; <i>or</i> depth of preparation for incomplete penetration butt weld
d_b	lateral distance between the centroids of the welds or fasteners
d_{bs}	nominal shank diameter

Symbol	Definition
d_c	depth of a section at the critical cross-section
d_e	net depth of a diaphragm at the point under consideration; <i>or</i> effective outside diameter of a circular hollow section; <i>or</i> factor for buckling capacity
d_f	distance between flange centroids; <i>or</i> nominal diameter of fastener
d_g	distance from the centroid of a steel beam to the centroid of a composite concrete slab
d_h	depth of a compression block
d_m	depth of a section at the minimum cross-section
d_n	distance from the outer fibre of the bottom flange to the centroidal axis of the section (or of the transformed composite section)
d_o	outside diameter of a circular hollow section; <i>or</i> overall section depth of the segment; <i>or</i> overall depth of a member including out-of-square dimensions
d_p	clear transverse dimension of a web plate; <i>or</i> greatest panel depth in the length; <i>or</i> depth of the deepest web panel
d_s	thickness of a concrete slab forming the flange of a composite beam
d_{s1}, d_{s2}	depth to the centre-line of the flange of the stiffener; <i>or</i> widths of the walls of the stiffeners
d_x, d_y	distances of the extreme fibres from the neutral axes
d_w	depth of the steel web
d_1	distance between flanges of an I beam; <i>or</i> distance from the centroid of the compression flange to the nearer face of the cross-member of a U-frame; <i>or</i> distance from the centroid of the compression flange to the nearest surface of the structural deck; <i>or</i> clear depth between flanges, ignoring fillets or welds; <i>or</i> dimension of an opening, shown in Figure 7.6.2.3
d_2	distance from the centroid of the compression flange to the centroidal axis of the cross-member of a U-frame; <i>or</i> distance from the centroid of the compression flange to the centroidal axis of the deck; <i>or</i> twice the clear distance from the neutral axis to the compression flange; <i>or</i> dimension of an opening, shown in Figure 7.6.2.3
d_3, d_4	value of d for each side of the weld
d_5	depth of section excluding corner radii, shown in Figure 5.12.1(C)
E	modulus of elasticity of steel; <i>or</i> modulus of elasticity of reinforcement
E_c	modulus elasticity of concrete at the age being considered
$(EI)_e$	effective elastic flexural stiffness of a column
e	distance between an end plate and a loadbearing stiffener; <i>or</i> eccentricity of bearing reaction along the span; <i>or</i> eccentricity of loading; <i>or</i> out-of-plane eccentricity; <i>or</i> web off-centre dimension
e_c, e_t	eccentricities of the centroids of compression and tension angles
e_f	greatest offset of the flange plate from a straight line of a length L_2 due to a specified camber or curvature
F	fully restrained
F	a force

Symbol	Definition
F^*	total design load on a member between supports
F_c^*	additional horizontal force per unit length resulting from interaction between the bending of cross-members and vertical stiffeners
F_h^*	total horizontal shear on connectors
F_n^*	design force normal to the web
F_p^*	design force parallel to the web panel
F_u^*	nominal horizontal force acting normal to the compression flange at the level of its centroid
f	flexibility of the joint between a cross-member and the verticals of a U-frame
f_a	longitudinal stress at the centroid of the effective section of the stiffener
f_b	bending strength for bending about the x -axis
f_{ci}	equivalent buckling stress in the flange
f_{ds}	characteristic shear capacity of a shear connector acting as an element of a group of n shear connectors
f_{fl}	lesser of the yield stress used in design or the design stress in either flange
f_{ks}	characteristic shear capacity of the connector
f_{Ls}	limiting stiffener stress
f_{ry}	yield stress used in design of a transverse reinforcement; <i>or</i> nominal yield stress of the reinforcement
f_{sy}	nominal yield stress of tensile reinforcement
f_{tc}	tensile stress at the top surface of the concrete slab of a composite beam at supports
f_u	tensile strength used in design
f_{uc}	ultimate tensile strength of the shear connector
f_{uf}	minimum tensile strength of a bolt
f_{up}	tensile strength of a ply
f_{uw}	nominal tensile strength of weld metal
f_y	yield stress used in design
f_{yb}	nominal yield stress of the bottom flange material
f_{yd}	nominal yield stress of the diaphragm plate
f_{ye}	effective yield stress
f_{yf}	nominal yield stress of the flange material
f_{yp}	yield stress of the pin used in design
f_{ys}	nominal yield stress of the stiffener; <i>or</i> nominal yield stress of a diaphragm and web junction
f_{yw}	nominal yield stress of the web material
f^*	design stress (action effect); <i>or</i> design stress range; <i>or</i> maximum longitudinal stress due to in-plane bending of the individual panel after any assumed redistribution of stresses
f_a^*	total vertical stress in a bearing stiffener
f_b^*	maximum longitudinal stress due to in-plane bending of an individual panel
f_{bs}^*	bending stress in a bearing stiffener due to an out-of-plane moment
f_{b2}^*	bending stress induced in a stiffener

Symbol	Definition
f_c^*	concrete compressive stress in regions of sagging moment
f_{d1}^*	vertical stress in stiffened diaphragm
f_{d2}^*	horizontal stress in stiffened diaphragm
f_f^*	longitudinal stress at mid-plane of the flange plate; <i>or</i> longitudinal compressive stress in the flange of a main beam (box girder), averaged across the width of the segment between the webs; <i>or</i> maximum compressive stress in the flange of a U-frame
f_i^*	design stress range for loading event 'i'
f_{in}^*	design stress range for loading event 'i' for normal stresses
f_{is}^*	design stress range for loading event 'i' for shear stresses
f_{jn}^*	design stress range for loading event 'j' for normal stresses
f_{ks}^*	design stress range for loading event 'k' for shear stresses
f_n^*	design stress range for normal stresses
f_q^*	stresses in diaphragm stiffeners
f_{Rv}^*	reference value of in-plane shear stress in unstiffened diaphragm
f_{R1}^*	reference value of in-plane vertical stress in unstiffened diaphragm
f_{R2}^*	reference value of in-plane horizontal stress in unstiffened diaphragm
f_s^*	summation of stresses in each flange; <i>or</i> longitudinal stress, including any redistribution of stresses from the web, positive when compressive, at the centroid of the effective section of the stiffener; <i>or</i> design stress range for shear stresses
f_{se}^*	equivalent axial stress in the stiffener for buckling
f_v^*	average design shear stress due to the applied shear force and, in a closed section, due to the applied torsional moment; <i>or</i> average shear stress in the panels on either side of the stiffener
f_{va}^*	average design shear stress in a web
f_{vf}^*	shear stress in the flange due to torsion and shear force on the beam
f_{vh}^*	shear stress which would occur in the plating adjacent to a stiffener if the opening had been fully plated
f_{vj}^*	shear stress in the flange at the junction with the web of the beam due to shear force on the beam
f_{vm}^*	maximum design shear stress in a web
f_{vt}^*	in-plane shear stress in the flange due to torsion
f_w^*	equivalent design stress on the web panel
f_x^*	summation of the stresses about the x-axis
f_y^*	summation of stresses about the y-axis
f_l^*	mean longitudinal in-plane stress on a cross-section of the panel after any assumed redistribution of stresses; <i>or</i> longitudinal stress along the stiffener connection centre-line derived without distribution
f_{le}^*	effective longitudinal in-plane stress in a plate panel
f_{ls}^*	vertical stress in a bearing stiffener
f_{lT}^*	vertical stress from torsional reaction on a single bearing

Symbol	Definition
f_2^*	in-plane transverse stress at the mid-plane of the flange plate; <i>or</i> in-plane transverse stress
f_{2b}^*	horizontal bending stress in a stiffened diaphragm
$f_{2b \max.}^*$, $f_{2b \min.}^*$	maximum and minimum values of f_{2b}^* within the length L_s
f_{2s}^*	average value of f_{d2}^* within the middle-third of the length L_s
f_{2v}^*	horizontal stress in the horizontal component of a girder shear in a stiffened diaphragm when the webs are inclined
f'_c	concrete compressive stress in regions of sagging moment; <i>or</i> characteristic compressive strength of concrete at 28 days
f'_{cy}	characteristic strength of the concrete at the age being considered
G	shear modulus of elasticity of steel; <i>or</i> length of a gauge
h_h	dimension of the opening parallel to the diaphragm stiffener
h_s	depth of a stiffener
I	second moment of area
I_{bc}	average effective second moment of area of the cantilever portion of a transverse member
I_{be}	average second moment of area of the effective transverse member between main beam (box girder) webs
I_c	second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment; <i>or</i> second moment of area of the uncracked concrete section
I_{cy}	second moment of area of the compression flange about the section principal y -axis
I_{ff}	second moment of area per unit width of the flange of a box girder (including longitudinal stiffeners), taken about the centroidal axis of the flange
I_r	second moment of area of the reinforcement
I_s	second moment of area of a pair of stiffeners about the centre-line of a web; <i>or</i> second moment of area about the face of a web; <i>or</i> second moment of area of the structural steel section
I_{se}	second moment of area of the effective section of the stiffener or a group of stiffeners
I_t	second moment of area of a transformed composite cross-section
I_w	warping constant for a cross-section
I_x	second moment of area of a cross-section about the principal x -axis; <i>or</i> section constant
I_{xse}	effective second moment of area of the stiffener group about the centroidal axis of the stiffener group (axis x - x in Figure 7.6.1)
I_y	second moment of area of the section about the principal y -axis
I_{yd}	second moment of area of the diaphragm plate of width j , excluding cut-outs, about the y -axis
I_{yse}	effective second moment of area of the stiffener group about the centroidal axis of the stiffener group (axis y - y in Figure 7.6.1)
I_1	second moment of area of the effective section of the vertical about its axis of bending
I_2	second moment of area of the cross-member of the U-frame about an axis perpendicular to the plane of the U-frame; <i>or</i> second moment of area of the deck per unit length about its axis of bending, with the gross concrete area being transformed in terms of steel
i	number of loading event

Symbol	Definition
J	torsion constant for a cross-section
j	contact width of a stiffened diaphragm above a bearing, shown in Figure 7.6.1; <i>or</i> width of contact of a bearing pad plus 1.5 times the thickness of the bottom flange at each end if available
K	a coefficient for calculating elastic buckling
K_b	plate buckling coefficient under in-plane bending
K_c	plate buckling coefficient under direct compression
K_D	diaphragm buckling coefficient
$K_{D1}, K_{D2}, K_{D3}, K_{D4}, K_{D5}$	coefficients used for checking buckling of diaphragms (K_D)
K_T	buckling coefficient for transverse members
K_v	shear coefficient
K_1	axial coefficient
K_2	transverse coefficient
k	coefficient given by the lesser of y/b or 0.77; <i>or</i> coefficient
k_b	elastic buckling coefficient for a plate element
k_{bo}	basic value of k_b
k_{co}	cohesion coefficient given in Table 6.8.5.2.1
k_d	a factor for buckling of diaphragms
k_e	member effective length factor
k_f	form factor for members subject to axial compression
k_h	factor for different hole types
k_L	load height effective length factor
k_{L1}, k_{L2}	values of the reduction factor k_L for longitudinal flange stiffeners
k_n	load-sharing factor given as a function of n
k_p	factor for pin rotation
k_r	lateral rotation restraint factor; <i>or</i> reduction factor to account for the length of a bolted lap connection; <i>or</i> reduction factor to account for the length of a welded lap connection
k_s	ratio used to calculate α_p and α_{pm} ; <i>or</i> coefficient used in determining effective stiffener stress
k_{s1}, k_{s2}	coefficients used for the design of longitudinal stiffeners
k_t	twist restraint factor; <i>or</i> correction factor defined for distribution of forces
k_v	ratio of the flat width of a web (d_s) to the thickness (t) of the section
k_1	U-frame restraint effective length factor
k'_s	coefficient used in determining effective stiffener stress
L	laterally restrained
L	length of a stringer; <i>or</i> member length from centre-to-centre of its intersections with supporting members; <i>or</i> length of a segment; <i>or</i> overhang from the outer edge of a bearing to the box; <i>or</i> member length; <i>or</i> clear length of stiffener between adjacent transverse stiffeners, cross-frames, cantilevers or diaphragms

Symbol	Definition
L_b	span length of the beam between vertical supports; <i>or</i> length between points of effective bracing or restraint
L_e	effective length of a member
(L_e/r)	geometrical slenderness ratio
$(L_e/r)_{bn}$	slenderness ratio of a battened compression member about the axis normal to the plane of the battens
$(L_e/r)_{bp}$	slenderness ratio of a battened compression member about the axis parallel to the plane of the batten
$(L_e/r)_c$	maximum slenderness ratio of the main component in a laced or battened compression member; <i>or</i> slenderness ratio of the main component between the interconnections
$(L_e/r)_m$	slenderness ratio of the whole battened compression member
L_f	horizontal distance from the reference point to the nearest edge of the bottom flange
L_j	length of a bolted lap splice connection
L_R	distance to reference point from bottom flange, shown in Figure 7.6.5.2.1
L_r	length of a segment over which the cross-section is reduced
L_s	spacing of cross-beams or diaphragms, or both, which restrain longitudinal stiffeners; <i>or</i> length of the stiffener between points of effective restraint; <i>or</i> total length of a diaphragm and web junction; <i>or</i> span of the stiffener between supporting members
L_{se}	effective length of longitudinal stiffeners on flanges not stiffened transversely
L_u	distance between U-frames
L_w	greatest internal dimension of an opening in the web; <i>or</i> length of a welded lap connection
M_b	nominal member moment capacity
M_{bx}	nominal member moment capacity about the principal x -axis
M_{bxo}	nominal member capacity about the principal x -axis without full lateral restraint and with a uniform distribution of design bending moment
M_{cx}	lesser of the nominal in-plane member moment capacity (M_{ix}) and the nominal out-of-plane member moment capacity (M_{ox})
M_{ds}	nominal moment capacity of a steel section allowing for interaction between plasticity and elastic lateral distortional buckling
M_{dx}	total bending moment capacity about the principal x -axis when N^* is acting on the section
M_{dy}	total bending moment capacity about the principal y -axis when N^* is acting on the section
M_f	nominal moment capacity calculated for the flanges alone
M_i	nominal in-plane member moment capacity
M_{ix}	nominal in-plane member moment capacity about the principal x -axis
M_{iy}	nominal in-plane member moment capacity about the principal y -axis
M_o	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 bending moment
M_{obr}	decreased resistance of the elastic buckling bending moment for elastic torsional end restraint
M_{od}	elastic lateral distortional buckling moment
M_{oo}	reference elastic buckling moment obtained using $L_e = L$
M_{os}	elastic buckling moment for a segment, fully restrained at both ends, unrestrained against lateral rotation and loaded at the shear centre

Symbol	Definition
M_{ox}	nominal out-of-plane member moment capacity about the principal x -axis
M_p	nominal section moment capacity determined by simple plastic theory; <i>or</i> nominal moment capacity of a pin
M_{rx}	nominal section moment capacity about the principal x -axis reduced by axial force tension or compression
M_{ry}	nominal section moment capacity about the principal y -axis reduced by axial force tension or compression
M_s	nominal section moment capacity
M_{sh}	nominal section moment capacity for an angle bent about the rectangular h -axis parallel to the loaded leg
M_{ss}	nominal section moment capacity of the steel section in the support region
M_{sx}	nominal section moment capacity about the principal x -axis
M_{sy}	nominal section moment capacity about the principal y -axis
M_{tx}	lesser of M_{rx} and M_{ox}
M_w	nominal section moment capacity of a web panel if the web panel resisted bending alone
M_y	section moment capacity (elastic) when the web yielding is disregarded
M_{yr}	section moment capacity (elastic) when the web yielding is accounted for
M^*	design bending moment; <i>or</i> in-plane primary moment on a diaphragm
M_e^*	second order (or amplified) end bending moment
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_s^*	portion of the out-of plane moment carried by a group of bearing stiffeners
M_w^*	design bending moment acting on a web panel
M_x^*	design bending moment about the principal x -axis
M_y^*	design bending moment about the principal y -axis
m_b, m_c, m_v, m_1, m_2	factors for checking the plate panel interaction buckling criterion
m_{fw}	factor used in calculating the web plate restraint
N	number of shear connectors per unit length
N_c	nominal member capacity in compression
N_{ch}	nominal member capacity in axial compression
N_{cr}	elastic critical load
N_{cy}	nominal member capacity in axial compression for buckling about the principal y -axis
N_{om}	elastic flexural buckling load of a member in axial compression
N_{omb}	elastic flexural buckling load of a braced member
N_{oms}	elastic flexural buckling load of a sway member
N_{oz}	nominal elastic torsional buckling capacity of a member
N_s	nominal section capacity of a compression member
N_t	nominal section capacity in tension
N_{tf}	nominal tensile capacity of a bolt
N_{ti}	minimum bolt tension at installation
N_u	design axial tension on the shear stud at the strength limit state

Symbol	Definition
N_{uc}	nominal member capacity for a member in axial compression; <i>or</i> nominal member capacity of a composite member
N_{us}	nominal section capacity for a member in axial compression; <i>or</i> nominal section capacity of a concentrically loaded composite compression member
N_{ow}	nominal axial capacity of a web panel if the web panel resisted axial load alone
N^*	design axial tensile or compressive force
N_e^*	effective axial force resulting from out-of-plane eccentricity of the design axial force (N^*)
N_s^*	design axial load in the shear connector
N_t^*	minimum tensile force per unit length of a beam in the transverse reinforcement in the top of a slab due to transverse bending in the slab
N_{tf}^*	design tensile force on a bolt
N_u^*	design axial tension on the shear stud
N_w^*	design axial force acting on a web panel
n	number of shear connectors per unit length
n_b	number of parallel planes of battens
n_{ei}	number of effective interfaces
n_n	number of shear planes with threads intercepting the shear plane in a bolted connection
n_s	number of longitudinal stiffeners in width B_f ; <i>or</i> number of full width horizontal stiffeners; <i>or</i> number of shear planes
n_w	number of webs
n_x	number of shear planes without threads intercepting the shear plane
n'	number of shear connectors per unit length placed within 200 mm of the centre-line of the web being considered
P	partially restrained
P^*	local load on a beam, shown in Figure 7.4.3.1; <i>or</i> local load applied to the top of a diaphragm; <i>or</i> force on the effective diaphragm and web junction section
P_d^*	total vertical load applied to the top of a diaphragm
P_i^*	locally applied deck load between the section under consideration and the web
P_s^*	total vertical force in a group of bearing stiffeners
Q^*	design transverse force
Q_{bv}^*	total vertical force transmitted to the diaphragm by the portion of the bottom flange between the inner edges of the bearings when there is a change in flange slope
Q_c^*	vertical component of any cross-beam or cantilever shear
Q_{fv}^*	vertical force transmitted to the diaphragm by the portion of the bottom flange over a width L_f when there is a change of flange slope
Q_h^*	shear force due to transverse horizontal loads on the bridge transmitted from the top flange to the diaphragm
Q_T^*	vertical component of torsional shear transmitted into the diaphragm from one web
Q_v^*	total vertical component of symmetric shear transmitted into the diaphragm from one web
q^*	shear flow at a the section of a diaphragm under consideration
R_b	nominal bearing capacity of a web
R_{bb}	nominal bearing buckling capacity
R_{by}	nominal bearing yield capacity

Symbol	Definition
R_{sb}	nominal buckling capacity of a stiffened web and stiffener; <i>or</i> nominal buckling capacity of a transversely stiffened web panel in bearing alone
R_{sy}	nominal yield capacity of the stiffened web
R_u	nominal capacity
R^*	design bearing force on a web; <i>or</i> design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener; <i>or</i> design reaction at the bearing
R_v^*	total vertical load transmitted by the diaphragm to one bearing, including the effects of torque on twin bearings
R_w^*	design bearing force acting on a web panel
r	radius of gyration
r_e	ratio of the distance from the elastic neutral axis to the compression edge of the web to the depth of the web
r_{ext}	outside radius of a section, shown in Figure 5.12.1(C)
r_n	hybrid factor
r_p	ratio of the distance from the plastic neutral axis to the compression edge of the web to the depth of the web
r_t, r_s	ratio
r_{se}	radius of gyration of the effective section of a longitudinal stiffener about the centroidal axis parallel to the flange plate; <i>or</i> radius of gyration of the effective stiffener section about an x - x axis parallel to the web; <i>or</i> radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm; <i>or</i> radius of gyration of the effective diaphragm and web junction about its centroidal axis parallel to the web
r_y	radius of gyration about the principal y -axis; <i>or</i> radius of gyration of the compression flange about the principal y -axis
r_l	radius of a concave bearing seating
S	plastic section modulus of a section
S^*	design action effect
s	longitudinal dimension of the web plate; <i>or</i> constant stress for design of longitudinal shear
s_b	longitudinal centre-to-centre distance between battens
s_g	gauge distance, measured at right angles to the direction of the design action in a member, centre-to-centre of holes in consecutive lines
s_p	staggered pitch, which is the distance measured parallel to the direction of the design action in a member, centre-to-centre of holes in consecutive lines
s_s	spacing of stiffeners, shown in Figure 7.6.1; <i>or</i> distance between stiffener centroids on a diaphragm
T^*	torque transmitted to the diaphragm in shear through the box walls and from cross-beams or cantilever loading, or both
T_b^*	torsional reaction at a single central bearing
T_s^*	moment in the plane of a diaphragm on a group of bearing stiffeners
t	thickness of a material; <i>or</i> element thickness; <i>or</i> thickness of a holed material; <i>or</i> thickness of the thicker part; <i>or</i> thickness of a thinner part joined

Symbol	Definition
t_d	plate thickness of a diaphragm
t_f	thickness of the flange; <i>or</i> thickness of the compression flange plate; <i>or</i> top flange thickness of a steel beam
t_p	thickness of the thinner ply element; <i>or</i> thickness of the thinner part joined; <i>or</i> thickness of a plate
t_s	thickness of a stiffener
t_{so}	average thickness of a flange outstand of width b_{so}
t_t	design throat thickness of a weld
t_w	thickness of the web of the beam; <i>or</i> size of a fillet weld
t_{w1}, t_{w2}	leg lengths of a fillet weld
U	unrestrained
u	factor for a U-frame restraint; <i>or</i> length of a shear plane
V_b	nominal shear buckling capacity of the web; <i>or</i> nominal shear buckling capacity for a stiffened web; <i>or</i> nominal bearing capacity of a ply; <i>or</i> nominal bearing capacity of a pin
V_f	nominal shear capacity of the bolt; <i>or</i> nominal shear capacity of the pin
V_{sf}	nominal shear capacity of a bolt for a friction-type connection
V_u	nominal shear capacity of a web with uniform shear stress distribution
V_v	nominal shear capacity of a web; <i>or</i> nominal shear capacity of a web panel if the web panel resisted shear alone
V_{vm}	nominal shear capacity of a web in the presence of bending moment
V_w	nominal shear yield capacity of the web; <i>or</i> nominal shear capacity of a weld
V^*	design shear force at the cross-section under consideration; <i>or</i> design transverse shear force
V_b^*	design bearing force of a ply; <i>or</i> design bearing force of a pin
V_{eff}^*	effective design longitudinal shear force on a shear connector
V_f^*	design shear force on a bolt or on a pin
V_{Lx}^*	design longitudinal shear force on a connector at a distance x from the centre-line of the web
V_{sf}^*	design shear force in the plane of the interfaces of a friction-type connection
V_w^*	design shear force acting on a weld; <i>or</i> design shear force acting on a web panel
v_L^*	design longitudinal shear force per unit length of a composite beam
v_{Lp}^*	design longitudinal shear force per unit length of the beam on the particular shear plane considered
v_w^*	design shear force per unit length; <i>or</i> design force per unit length of weld
v_{Ls}	permissible longitudinal shear design force per unit length at the serviceability limit state
v_w	nominal capacity of a fillet weld per unit length
W_d^*	total uniformly distributed load applied to the top of a diaphragm

Symbol	Definition
w	distributed load; <i>or</i> width of the load P^* , shown in Figure 7.4.3.1; <i>or</i> actual width of the load P^* on a diaphragm
w_h	width of a stiffener loop in a box girder, shown in Figure 7.6.1; <i>or</i> width of a cut-out for a stiffener in a diaphragm, shown in Figure 7.6.5.2.1
x	dimension along x -axis; <i>or</i> major axis coordinate; <i>or</i> horizontal distance of a stiffener under consideration from the centroidal axis, normal to the plane of a diaphragm, of a stiffener group
x_b	distance from the section under consideration to the inner edge of the nearest bearing
x_c	horizontal distance from the section under consideration to the root of the cross-beam or cantilever
x_i	distance from the section under consideration to the locally applied deck load P_i^*
x_R	distance parallel to the bottom flange from the reference point to the web mid-point
x_w	horizontal distance from the section under consideration to the mid-point of the web
y	dimension along y -axis; <i>or</i> minor axis coordinate; <i>or</i> perpendicular distance from the point being considered to the longitudinal centre-line of the panel; <i>or</i> distance of the extreme fibre of a stiffener under consideration from the centroidal axis, parallel to the plane of the diaphragm, of a stiffener group
y_{Bs}	distance from the centroid of the effective stiffener section to the neutral axis of the effective cross-section of the beam
y_c	distance from the neutral axis of the composite cross-section to the centroid of area A_i
y_L	distance of the gravity loading below the centroid
y_o	coordinate of the shear centre; <i>or</i> distance from the centroid of the effective stiffener section to the point on the stiffener furthest from the plate
y_t	distance from the axis of zero stress to the extreme tension fibre of a section
y_z	distance from the centroid of the effective stiffener section to the mid-plane of the flange plate
Z	elastic section modulus
Z_e	effective section modulus; <i>or</i> effective section modulus of a diaphragm and flanges at the vertical cross-section through the reference point; <i>or</i> effective section modulus of a vertical cross-section of a diaphragm and flanges at the point under consideration
Z_{ec}	effective section modulus of the transformed composite section
Z_{en}	effective elastic section modulus of a not compact section
Z_{enc}	effective elastic section modulus of the section transformed to steel
Z_{enx}	effective elastic section modulus of the tension and compression flange, as appropriate
Z_{ex}	effective section modulus about the principal x -axis
Z_{se}	lowest section modulus of the effective junction section about the centroidal axis parallel to the web
Z_{we}	elastic section modulus of the web panel
α	angle between the x - and h -axes; <i>or</i> dynamic load allowance in AS 5100.2 Table 7.7.2
α_a	compression member factor
α_b	compression member section constant

Symbol	Definition
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; <i>or</i> distortional buckling reduction factor
α_f	flange restraint factor for web shear buckling
α_L, α_{Lc}	factor for bending
α_m	moment modification factor for bending
α_{mc}	factor for bending
α_n	a factor for interaction curve
α_p	coefficient used to calculate the nominal bearing yield capacity; <i>or</i> value of S/Z for compact sections
α_{pm}	coefficient used to calculate α_p
α_{ry}	elastic stiffness of the flexural end restraint
α_{rz}	elastic stiffness of a torsional end restraint; <i>or</i> elastic stiffness
α_s	slenderness reduction factor; <i>or</i> steel contribution factor; <i>or</i> inverse of the slope of the S-N curve
α_{st}	reduction factor
α_T	coefficient of thermal expansion for steel
α_t	factor for torsional end restraint
α_u	magnification factor
α_v	shear buckling coefficient for the web
α_w	a factor for buckling capacity
β	ratio of web to bottom flange area for a hybrid girder; <i>or</i> inclination of a box web to the vertical, shown in Figure 7.6.1
β_c	modifying factor which accounts for end conditions at the far ends of beam members
β_m	ratio of the smaller to the larger bending moment at the ends of a member
β_t	measure of elastic stiffness of torsional end restraint
β_{tf}	size correction factor for plate thicknesses
β_w	a factor for buckling capacity
β_x	monosymmetry section constant
γ	factor for transverse stiffener arrangement; <i>or</i> index
η_1, η_2	ratio of the compression member stiffness to the end restraint stiffness
Δ	deflection
Δ_f	out-of-flatness of a flange plate
Δ_{sx}, Δ_{sy}	maximum deviations from flatness within a specified gauge length, measured perpendicular to and parallel to the plate surface, respectively
Δ_u	lateral deflection which would occur in a U-frame at the level of the centroid of the flange being considered
Δ_v	deviation from verticality of a web at a support
Δ_w	out-of-flatness of a web
Δ_x	maximum deviation from flatness within a specified gauge length

Symbol	Definition
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_s	moment amplification factor for a sway member
ε_{nom}	nominal strain
ε_{true}	true strain
η	compression member imperfection factor ($\eta = \text{eta}$)
$\eta_1, \eta_2, \eta_{10}, \eta_{20}$	coefficients used to calculate N_{us} of a circular member with an increase in concrete strength
θ	rotation in radians of a cross-member at its junction with the main beam under consideration; <i>or</i> angle of preparation of an incomplete penetration butt weld
λ	slenderness ratio
λ_c	elastic buckling load factor
λ_d	non-dimensional slenderness for elastic distortional buckling
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_{ms}	elastic buckling load factor for a member
λ_η	modified compression member slenderness
λ_r	relative slenderness for a composite column in a given plane of bending
λ_s	section slenderness
λ_{sp}	section plasticity slenderness limit
λ_{sy}	section yield slenderness limit
λ_w	slenderness of a web
λ_{wy}	yield slenderness limit of a web
μ	slip factor for friction type bolted joints ($\mu = \text{mu}$); <i>or</i> coefficient of friction given in Table 6.8.5.2.1
ν	Poisson's ratio 0.25 ($\nu = \text{nu}$)
π	constant
ξ	compression factor ($\xi = \text{xi}$)
ρ	ratio of yield stresses of web to bottom flange for a hybrid girder; <i>or</i> proportion of stress redistributed from an unrestrained web panel; <i>or</i> proportion of stress assumed to be redistributed from the panel
ρ_c	density of plain concrete, as given in AS 5100.5
σ_{cp}	average intensity of effective prestress in concrete
σ_{nom}	nominal stress
σ_{true}	true stress
τ_{bond}	design shear strength
τ_u	longitudinal shear strength on the surface considered
ΣR_v^*	total vertical force transmitted by the diaphragm to the bearings
Σw_h	sum of the widths of any cut-outs for stiffeners within the width j at the level immediately above the flange
ϕ	capacity reduction factor

Symbol	Definition
ϕ_c	capacity factor for concrete in compression members
ϕ_p	aspect ratio a/b , as shown in Figure 7.4.2
$\phi^{V_{Ls}}$	shear capacity
ϕR_u	design capacity
ψ	ratio of section depths for a hybrid girder ($\psi = \psi_i$)

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. Additionally for New Zealand, the yield stress shall be in accordance with Appendix H.

2.1.2 Tensile strength

The tensile strength used in design (f_u) shall not exceed that given in Table 2.1. Additionally for New Zealand, the tensile strength shall be in accordance with Appendix H.

2.2 STRUCTURAL STEEL

2.2.1 Compliance

Except as otherwise permitted in Clause 2.2.4, all structural steel covered by the scope of this Standard shall, before fabrication, comply with the requirements of the following Standards, as appropriate:

- (a) AS 3597.
- (b) AS/NZS 1163.
- (c) AS/NZS 1594.
- (d) AS/NZS 3678.
- (e) AS/NZS 3679.1.
- (f) AS/NZS 3679.2.

Additionally, for New Zealand only, other certified steels shall comply with Appendix H.

2.2.2 Acceptance of steel

Mill 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.

NOTE: A mill certificate includes both test and inspection certificates.

2.2.3 Unidentified steel

Where steel does not satisfy the requirements of Clause 2.2.2, it shall be classed as unidentified steel. Unidentified steel shall not be used.

2.2.4 Other steels

Except where permitted with the specific approval of the relevant authority, steels for machined parts and for uses in other than structural members or elements shall comply with the relevant Standard.

2.2.5 Properties of steel

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

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

2.2.6 Through-thickness deformation properties

Through-thickness deformation properties shall be as specified to guarantee adequate deformation capacity perpendicular to the surface to provide ductility and toughness against lamellar tearing. For product in accordance with AS/NZS 3678, the minimum values for reduction of area for the specified quality class shall comply with the relevant requirements of AS/NZS 3678 and as given in Table 2.2.6. For flat products, the minimum values for reduction of area shall apply to the whole product.

TABLE 2.2.6
MINIMUM REDUCTION IN AREA

Grade suffix	%
Z15	15
Z25	25
Z35	35

2.3 CONCRETE, REINFORCEMENT AND PRESTRESSING STEELS

Concrete, reinforcement and prestressing steels shall meet the requirements of AS 5100.5.

In analysing a structure to determine the load effects, the material properties associated with the unfactored characteristic strength shall be used irrespective of the limit state being considered.

2.4 FASTENERS

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

- (a) AS 1110.
- (b) AS 1111.
- (c) AS 1112.1.
- (d) AS 1112.2.
- (e) AS 1112.3.
- (f) AS 1237.1.
- (g) AS 1237.2.
- (h) AS/NZS 1252.
- (i) ISO 7040.
- (j) ISO 7041.
- (k) ISO 10511.

2.5 WELDS

All welding consumables and deposited weld metal shall comply with AS/NZS 1554 series as appropriate. 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 weld is required by Section 13, weld quality conforming with AS/NZS 1554.4 category FP or AS/NZS 1554.5, as appropriate, shall be used.

Welding shall be undertaken in accordance with AS/NZS ISO 3834.2 and AS/NZS ISO 3834.3.

NOTE: Where welding of wrought iron is proposed, specialized metallurgist advice should be sought.

2.6 WELDED STUD SHEAR CONNECTORS

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

2.7 STEEL CASTINGS

All steel castings shall comply with AS 2074.

2.8 WROUGHT IRON

Where the properties of wrought iron in a structure are unknown, the material shall be tested in accordance with AS 1391, and sufficient samples of the material shall be tested to provide a reasonable statistical distribution of its properties. The ductility of the wrought iron shall be checked to ensure that it has a minimum characteristic elongation at failure of 5%. Provided that its ductility meets this limit, the capacity of the wrought iron structure may be checked using the rules of this Clause for steel, but with particular consideration taken of the fatigue strength of critical details.

NOTE: For material characteristics of typical existing wrought iron, refer to AS 5100.7.

2.9 RIVETS

Where 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 the material is tested in accordance with AS 1391, and sufficient samples of the material are tested to provide a reasonable statistical basis for assessing its properties, 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.

2.10 CAST IRON

The minimum tensile strength (f_u) of cast iron shall be taken from AS 1831 for the grade selected, and the properties of cast iron in a structure shall be tested in accordance with AS 1391. Sufficient samples of the material shall be tested to provide a reasonable statistical distribution of its properties. Full account shall be taken of the brittleness of cast iron and potential casting defects.

NOTE: For material characteristics of typical existing cast iron, refer to AS 5100.7.

2.11 DUCTILITY REQUIREMENTS

For steels and cast iron a minimum ductility is required, which shall be expressed in terms of limits for:

- $f_u/f_y \geq 1.2$, where f_u is the specified minimum ultimate tensile strength and f_y is the specified minimum yield strength;
- elongation at failure not less than 15% on a gauge length of $5.65\sqrt{S_0}$, where S_0 is the cross-sectional area of the test piece before testing; and
- the ultimate strain $\epsilon_u \geq 6\epsilon_y$, where ϵ_u corresponds to the ultimate strength f_u and ϵ_y is the yield strain ($\epsilon_y = f_y/E$).

Steel conforming with one of the steel grades listed in Table 2.1 shall be accepted as satisfying these requirements.

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 3) AND AS 3597

Steel Standard	Form	Steel grade	Thickness of material (<i>t</i>) mm	Yield stress (<i>f_y</i>) MPa	Tensile strength (<i>f_u</i>) 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 HU300	All	300	400
		HA250 HU250	All	250	350
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
		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

(continued)

TABLE 2.1 (continued)

Steel Standard	Form	Steel grade	Thickness of material (t) mm	Yield stress (f_y) MPa	Tensile strength (f_u) MPa
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
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.
- 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 GENERAL

3.1.1 Aim

The aim of structural design is to provide a structure that does not reach any of the limit states defined in AS 5100.1, which requires that the structure be durable, serviceable and be adequately strong while serving its intended function. In addition, the structure has to satisfy other relevant requirements, such as robustness, fatigue, brittle fracture, corrosion resistance and protection, fire resistance, ease of construction and economy.

3.1.2 Design for ultimate limit states

The structure as a whole and its components shall be designed for the requirements of all the ultimate limit states specified in AS 5100.1.

3.1.3 Design for serviceability limit states

The structure and its components shall be designed for serviceability by controlling deflection, cracking and vibration, as appropriate, in accordance with the relevant requirements of Clauses 3.3 to 3.8.

3.2 DESIGN FOR ULTIMATE (STRENGTH) LIMIT STATE

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

- (a) The design action effects (S^*) resulting from the design loads at the ultimate limit state shall be determined by an analysis in accordance with Section 4.
- (b) The design capacity (ϕR_u) shall be determined from the nominal capacity (R_u) determined from Sections 5 to 12, as appropriate, where the capacity reduction factor (ϕ) shall not exceed the appropriate value given in Table 3.2.
- (c) 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 3.2
CAPACITY REDUCTION FACTORS (ϕ) FOR ULTIMATE (STRENGTH) LIMIT STATES

Design capacity for	Clauses	Capacity reduction factor (ϕ)	
(a) Structural steel:			
(i) Members subject to bending—			
– full lateral support	5.1, 5.11, 8.4 and App. B	0.90	
– segment without full lateral support	5.1 and 5.11	0.90	
– web in shear	5.10 and 5.11	0.90	
– web in bearing	5.12 and 4.13	0.90	
– stiffener	5.12, 5.13 and 5.14	0.90	
(ii) Members subject to axial compression—			
– section capacity	10.1 and 10.6	0.90	
– member capacity	10.1 and 10.6	0.90	
(iii) Members subject to axial tension	9.1	0.90	
(iv) Members subject to combined actions—			
– section capacity	11.3	0.90	
– member capacity	11.4	0.90	
(b) Composite members:			
(i) Members subject to bending—			
– full lateral support	6.2, 7.3 and 7.6	0.90	
(ii) Composite compression members—		Steel	Concrete
– section capacity	10.6	0.9	0.65
– relative stiffness	10.6.2.3	1.0	0.20
– combined axial and flexural actions	11.5	0.90	
(c) <i>Connections</i> Connection component other than bolt, pin, rivet or weld	12.3.6	0.90	
(i) Bolted connections—			
– bolt in shear	12.5	0.80	
– bolt in tension	12.5	0.80	
– bolt subject to combined shear and tension	12.5	0.80	
– ply in bearing	12.5	0.90	
– bolt group	12.5	0.80	
(ii) Riveted connections—			
– power driven rivets	As for bolted connections	0.80	
– hand driven rivets	As for bolted connections	0.60	
(iii) Pin connections—			
– pin in shear	12.5	0.80	
– pin in bearing	12.5	0.80	
– pin in bending	12.5	0.80	
– ply in bearing	12.5	0.90	

(continued)

TABLE 3.2 (continued)

Design capacity for	Clauses	Capacity reduction factor (ϕ)	
(iv) Welded connections—		SP Category	GP Category
– complete penetration butt weld	12.6	0.90	0.60
– fillet weld and incomplete penetration butt weld	12.6	0.80	0.60
– weld group	12.6	0.80	0.60
– plug or slot weld	12.6	0.80	0.60
(d) Shear connection:			
– Shear connectors	6.8.3	0.8	
– Transverse reinforcement	6.6	1.00	
(e) Wrought iron	As for structural steel	0.85	
(f) <i>Cast iron</i> Factors based on minimum tensile strength (f_u):			
– Axial and flexural tension	5.1 and 9.1	0.33	
– Axial compression in short columns and flexural compression	10.1 and 10.6	0.41	
– Shear	5.10	0.21	
(g) Bearings:			
– Steel and cast iron components of bearings	AS 5100.4	1.00	

3.3 DESIGN FOR SERVICEABILITY LIMIT STATE

3.3.1 General

The structure and its components shall be designed for the SLS by controlling or limiting deflection, vibration, crack control, bolt slip, as appropriate, in accordance with the relevant requirements of Clauses 3.3.2 to 3.3.7, using the capacity reduction factors (ϕ) given in Table 3.3.

3.3.2 Deflection limits for beams

The deflection of beams and box girders under service conditions shall comply with the deflection limits as specified in AS 5100.2.

For cantilevers, the calculated deflection shall include effects due to rotation at the support.

Deflections shall be calculated by an elastic analysis in accordance with Clause 4.2.

3.3.3 Vibration of beams

The design of beams for vibration, including box girders, under service conditions shall comply with the vibration requirements specified in AS 5100.2.

3.3.4 Shear connection

In composite beams, the longitudinal shear connection between the steel member and the concrete slab shall be designed in accordance with Section 6.

3.3.5 Steel reinforcement

The design and detailing of the steel reinforcement for composite members shall be carried out in accordance with Clause 6.2.3 and AS 5100.5.

3.3.6 Connections

Bolts in friction-type connections, in which slip under serviceability loads shall be avoided, shall be designed in accordance with Clause 12.5.4.

3.3.7 Steel and cast iron components for bridge bearings

The design of steel and cast iron bridge bearings shall be in accordance with AS 5100.4.

TABLE 3.3
CAPACITY REDUCTION FACTORS (ϕ) FOR SERVICEABILITY LIMIT STATES

Design capacity for	Clauses	Capacity reduction factor (ϕ)
(a) Connections:		
Bolted connection	12.5.4	0.70
– bolts in friction type connection checked for serviceability limit state		
(b) Cast steel:		
– members subject to axial tension	15.3	1.00
– members subject to axial compression	15.3	1.00
– members subject to shear	15.3	1.00
(c) Shear connection:		
Shear connectors	6.6	1.00

3.4 DESIGN FOR STRENGTH AND SERVICEABILITY BY LOAD TESTING

Notwithstanding the requirements of Clause 3.2 or Clause 3.3, 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 15. If this alternative procedure is adopted, Clauses 3.5 to 3.8, as appropriate, shall also apply.

3.5 AVOIDANCE OF BRITTLE FRACTURE AND LAMELLAR TEARING

To avoid failure by brittle fracture, the selection of the parent material shall be made in accordance with Section 14.

3.6 FATIGUE

For members subjected to fatigue, a fatigue assessment shall be carried out in accordance with Section 13.

3.7 CORROSION

3.7.1 Resistance and protection

All steelwork shall be designed and detailed to minimize the risk of corrosion (see Notes 1, 3, 4 and 5).

Where necessary, the effects of possible stray current corrosion shall be considered (see Note 2).

NOTES:

- 1 The basis for the design of components contained in this Standard makes no allowance for any loss of material due to corrosion.
- 2 A corrosion allowance is not required for structural steel in Corrosivity Categories C1 and C2 where the coatings are appropriately specified, applied, and maintained to AS/NZS 2312.1 or the structural steel is hot dip galvanized to AS/NZS 4680 with design details from AS/NZS 2312.2. For Corrosivity Categories C3, C4, C5 and CX, the selection of a coating system and the maintenance strategy should be the subject of professional advice.
- 3 An estimate of the durability of a coating system can be obtained from AS/NZS 2312.
- 4 For corrosion maps for Australia and New Zealand, see Appendix G.

- 5 Guidance for applying AS/NZS 2312.1 and AS/NZS 2312.2 in combinations with the corrosion maps in Appendix G can be found in: *Australian steelwork corrosion and coatings guide*, Australian Steel Institute (ASI), ASI Number 870; and, in New Zealand, *New Zealand Steelwork Corrosion and Coatings Guide*, Heavy Engineering Research Association (HERA), HERA R4-133.

3.7.2 Allowances of weathering steel

To cater for loss of structurally effective material due to the development of rust patina during the life of the bridge, the following corrosion allowances shall be made on each exposed surface, representing a loss of thickness of material used for structural purposes:

- (a) For AS 4312 atmospheric corrosivity categories C1 and C2, the corrosion allowance shall be 1.0 mm per surface. For AS 4312 atmospheric corrosivity category C3, the corrosion allowance shall be 1.5 mm per surface.

NOTE: Caution is needed for the use of weathering steel in corrosivity Category C3 in environments in which levels of chlorides, sulfur dioxide and any other type of contaminants are high.

- (b) For AS 4312 atmospheric corrosivity categories C4 and C5, weathering steel shall be allowed, provided the steel is protected.

NOTE: Weathering steel with an applied corrosion protective coating is deemed to perform the same as ordinary carbon steel with an identical applied corrosion protective coating.

- (c) For 'interior' surfaces of box sections, the allowance shall be 0.5 mm.

The allowances stated in Items (a), (b) and (c) shall apply to fillet welds and partial penetration butt welds.

3.8 DESIGN FOR FIRE RESISTANCE

In instances where it is considered necessary for a bridge to be designed for fire resistance, for example, rail bridges near stations, the relevant provisions of AS 4100, AS 2327.1 and AS 5100.2 shall apply.

3.9 PARTICULAR DESIGN REQUIREMENTS—GENERAL

3.9.1 Web-to-flange welds

For rail bridges, and road and pedestrian composite box girder bridges, the total effective thickness of web-to-flange welds shall be not less than the thickness of the web. Where used, fillet welds shall be provided on both sides of the connecting flange or web plate.

3.9.2 Beam restraint at piers of rail bridges

Longitudinal beams of rail bridges shall be braced with cross-frames or diaphragms at support points. These shall be designed to provide both adequate strength and stiffness to the structure.

3.10 PARTICULAR DESIGN REQUIREMENTS—RAIL BRIDGES

3.10.1 Beam restraint at intermediate locations of rail bridges

A cross-frame or diaphragm shall be used at the centre of a span if the length of longitudinal beams is greater than the permissible spacing of cross-frames or diaphragms as given in this Clause.

Cross-frames or diaphragms shall be used at intervals not greater than the following:

- (a) For open deck construction 5.5 m.
- (b) Where steel plate, timber or precast concrete decking is utilized in ballasted or direct fixed deck construction without top lateral bracing 3.5 m.

- (c) Where steel plate, timber or precast concrete decking is utilized in ballasted or direct fixed deck construction with top lateral bracing 5.5 m.
- (d) Where cast in situ concrete decking is used in ballasted or direct fixed deck construction 7 m.

Beams restrained by U-frames designed in accordance with Clauses 5.6.3 and 8.4.6, and beams continuously restrained by a deck not at compression flange level, which have been designed in accordance with Clauses 5.6.4 and 8.4.7, shall be deemed to comply with this Clause.

3.10.2 Slenderness ratio for rail bridges

The slenderness ratio of length to the least radius of gyration in rail bridges shall be not greater than—

- (a) 100 for main compression members;
- (b) 120 for wind and sway bracing in compression;
- (c) 140 for single lacing members;
- (d) 200 for double lacing members; and
- (e) 200 for tension members,

unless a rigorous fatigue analysis is carried out.

3.10.3 Bracing for rail bridges

Bracing for rail bridges complying with Clause 3.10.2 shall be assessed for fatigue by allowing two times the calculated effective number of load cycles in the fatigue assessment.

3.10.4 Rail truss bridges

Rail truss bridge members and lattice girder members complying with Clause 3.10.2 shall be assessed for fatigue by allowing 1.4 times the calculated effective number of load cycles in the fatigue assessment.

3.10.5 End connections of floor members in rail bridges

End connections of floor members in rail bridges shall be designed as follows:

- (a) Stringers shall have end connection angles to ensure the necessary flexibility in the connection to cross-girders. Cross-girders and beams in solid floor construction shall have end connection angles, where practicable. Welding shall not be used to connect the flexing legs.
- (b) Where applicable, the flexing legs of the connection angles shall be not less than 100 mm in width and 12 mm in finished thickness.
- (c) Where applicable for stringers, the gauge from back of angle to the first line of the fasteners in the flexing legs of the connection angles over the top third of the stringer depth shall be not less than—

$$\sqrt{\frac{Lt}{8}} \quad \dots 3.10.5$$

where

L = length of a stringer

t = thickness of angle

3.10.6 Minimum moment on end connections in rail bridges

For stringers and cross-girders with end connections complying with Clause 3.10.5, the end connections shall be designed for a minimum hogging moment that is 33% of the calculated centre moment, assuming the member to be simply supported.

Bolted or riveted end connections not complying with Clause 3.10.5 shall be designed for a minimum hogging moment of 50% of the calculated centre moment, assuming the member to be simply supported.

3.10.7 Welded stiffeners and cross-girder to through girder connections for rail bridges

To achieve satisfactory fatigue life, stiffeners, other than end bearing stiffeners, at bracing connections on deck spans or through spans, at the connection of cross-girders to through girders, and at the connection of stringers to cross-girders, shall have the following details:

- (a) The end of the stiffener shall be bolted to the tension flange. Welding shall not be permitted.
- (b) The end 40 mm of stiffener to web weld shall be fatigue life enhanced.
NOTE: For methods of enhancement, refer to 'TGN-D-02, Introduction to Fatigue of Welded Steel Structures and Post-Weld Improvement Techniques,' Welding Technology Institute of Australia.
- (c) At cross-girder to through girder connections, the cross-girder bottom flange shall be bolted to the through girder bottom flange.

3.10.8 Transom top rail bridge

For transom top spans where transoms load either girders or trusses, full account of the eccentricity of transom loading shall be taken in the following cases:

- (a) Local bending under transoms of top flanges of girders or top chords of trusses.
- (b) Torsional loading of girders or top chords of trusses.

3.10.9 Thickness of material for rail bridges

Metal, except for fillers, shall be not less than 8 mm thick.

NOTE: Parts used in highly corrosive environment should have their thickness increased as appropriate, or else protected against such influences.

3.11 DESIGN FOR EARTHQUAKE

This Section presents minimum design and detailing requirements of steel and composite members and structures for the earthquake loading provisions.

For Australia, when $\mu \leq 2$, the rules of AS 5100.2 and AS/NZS 5100.6 shall be applied. For cases when $\mu > 2$, the steel members and connections for such structures shall be designed and detailed in accordance with NZS 3404.1 (see Figure 3.11).

For New Zealand, when the loadings given in the NZTA Bridge manual are applied, the steel members and connections shall be designed and detailed in accordance with NZS 3404.1 (see Figure 3.11).

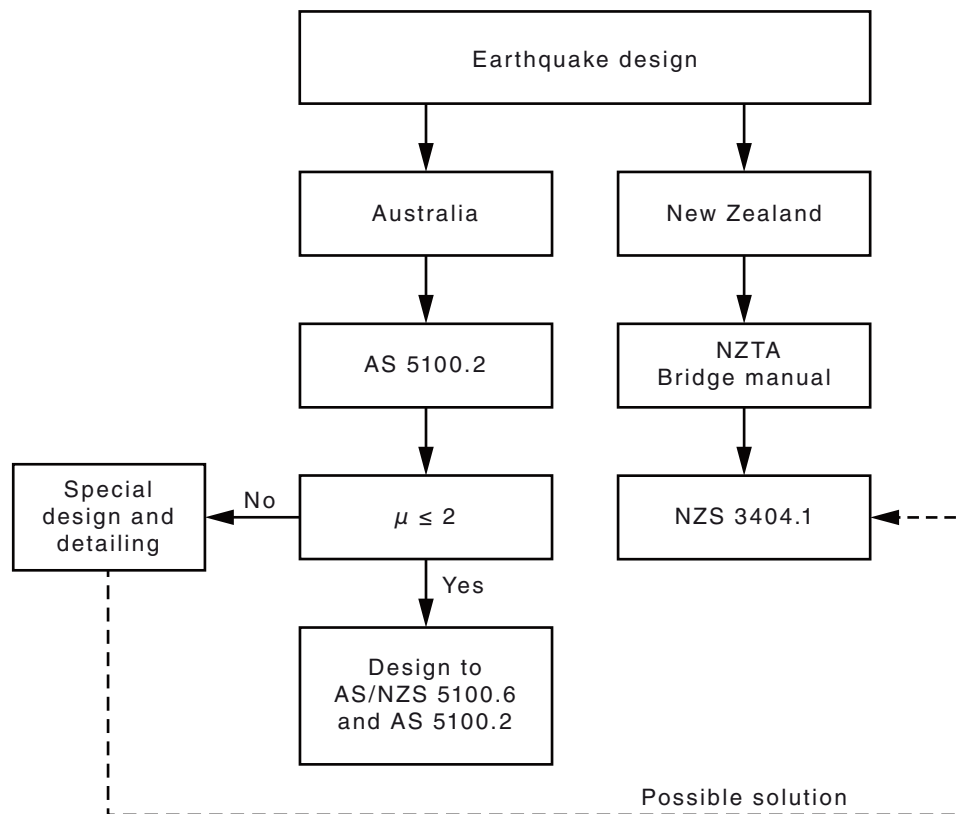


FIGURE 3.11 FLOWCHART FOR DESIGN OF STRUCTURES FOR EARTHQUAKES

3.12 RELIABILITY MANAGEMENT

For the application of AS/NZS 5131, the selection of Construction Category should be selected in accordance with Appendix L of this Standard.

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

SECTION 4 METHODS OF STRUCTURAL ANALYSIS

4.1 METHODS OF DETERMINING ACTION EFFECTS

4.1.1 General

For the purpose of complying with the requirements for the limit states of stability, strength and serviceability specified in Section 3, the design action effects in a structure and its members and connections caused by the design loads shall be determined by structural analysis using one of the following methods:

- (a) Elastic analysis, in accordance with Clause 4.2.
- (b) Rigorous structural analysis, in accordance with Clause 4.10.

In addition, where appropriate, the requirements of Clauses 4.4 to 4.9 shall be satisfied.

4.1.2 Definitions

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

4.1.2.1 *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 deck slabs or by bracing systems parallel to the plane of buckling of the member.

4.1.2.2 *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 that depend on flexural action to limit the sway.

4.2 ELASTIC ANALYSIS

4.2.1 General

4.2.1.1 *Assumptions*

Individual members shall be assumed to remain elastic for the purpose of analysis, which shall apply to both the ultimate and serviceability 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.

4.2.1.2 *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 4.2.2.2, provided the moment amplification factors for a braced member or sway member (δ_b or δ_s) are not greater than 1.4; or
- (b) a second-order elastic analysis in accordance with Appendix C.

4.2.2 First-order elastic analysis

4.2.2.1 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 the methods of moment amplification of Clause 4.2.2.2 or Clause 4.2.2.3 as appropriate, except that where the calculated moment amplification factor (δ_b or δ_s) is greater than 1.4, a second-order elastic analysis in accordance with Appendix C shall be carried out.

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

4.2.2.2 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:

$$M^* = M_m^* \quad \dots 4.2.2.2(1)$$

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:

$$M^* = \delta_b M_m^* \quad \dots 4.2.2.2(2)$$

where

M_m^* = maximum calculated design bending moment along the length of a member or in a segment

δ_b = moment amplification factor for a braced member, calculated as follows:

$$\delta_b = \frac{c_m}{1 - \left(\frac{N^*}{N_{omb}} \right)} \geq 1 \quad \dots 4.2.2.2(3)$$

N_{omb} = elastic flexural buckling load of a braced member, determined in accordance with Clause 4.3.1 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 moments only, the factor for unequal moments (c_m) shall be calculated as follows:

$$c_m = 0.6 - 0.4 \beta_m \leq 1.0 \quad \dots 4.2.2.2(4)$$

where

β_m = ratio of the smaller to the larger moment at the ends of the member, taken as positive when the member is bent in reverse curvature, except that for column members being designed for load combinations which include earthquake loads

$$= 0$$

The same expression for c_m shall be used for a braced member with transverse load applied to it, with the values of β_m approximated by the value obtained by matching the distribution of moment along the member with one of the typical distributions of bending moment shown in Figure 4.2.2.2. For members where the maximum moment occurs within the length of the member, β_m may conservatively be taken as -1.0 .

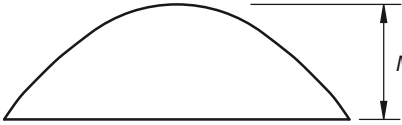
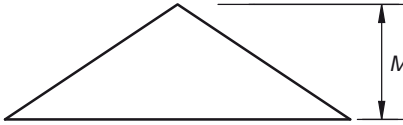
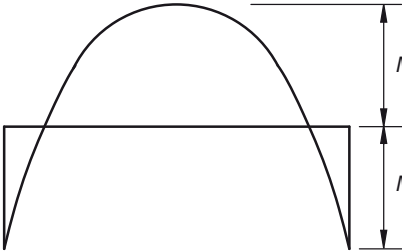
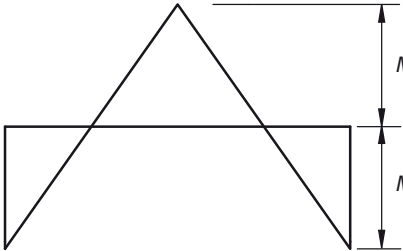
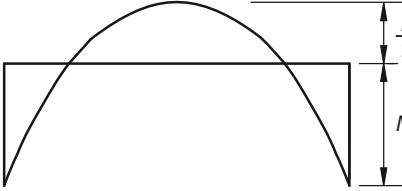
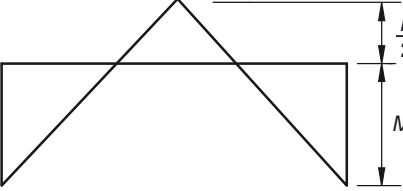
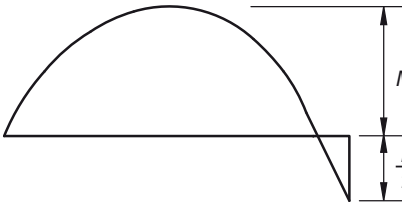
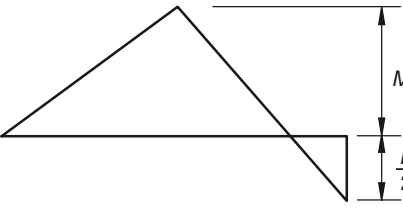
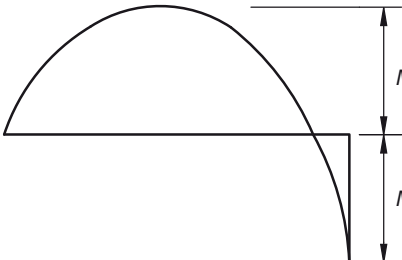
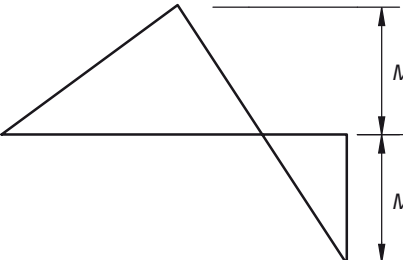
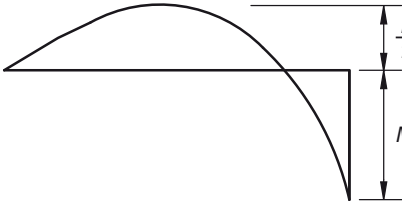
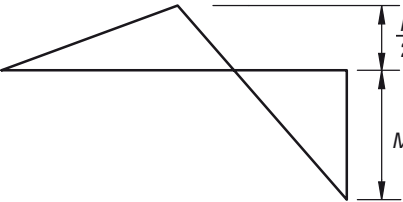
Moment distribution	β_m	Moment distribution	β_m
	-1.0		-1.0
	+0.2		+0.5
	+0.6		+1.0
	-0.5		+0.4
	+0.2		+0
	+0.2		+0.5

FIGURE 4.2.2.2 (in part) VALUES OF β_m FOR VARIOUS DISTRIBUTIONS OF BENDING MOMENT

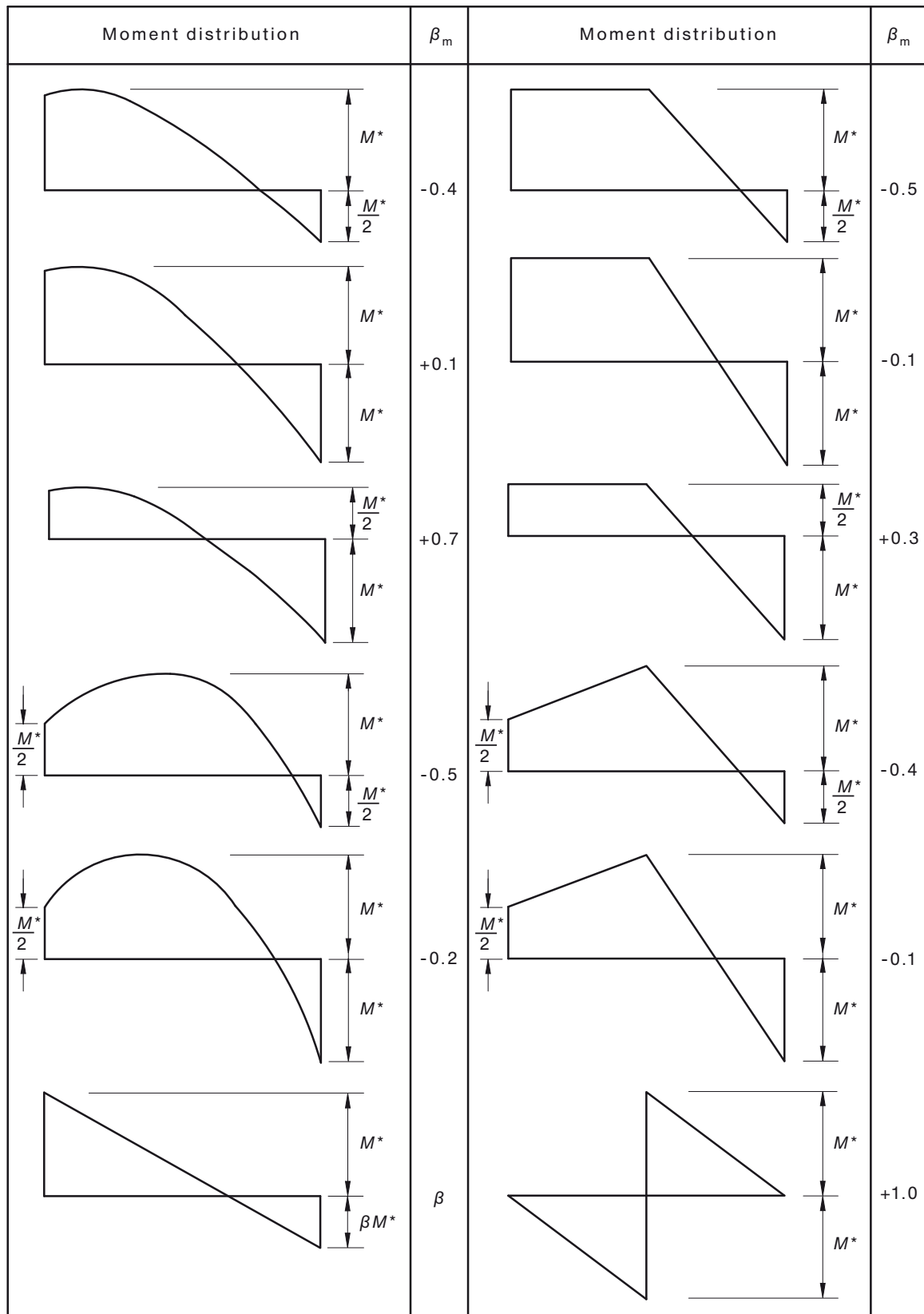


FIGURE 4.2.2.2 (in part) VALUES OF β_m FOR VARIOUS DISTRIBUTIONS OF BENDING MOMENT

4.2.2.3 Moment amplification for a sway member

For a sway member, the design bending moment (M^*) shall be calculated as follows:

$$M^* = \delta_m M_m^* \quad \dots 4.2.2.3(1)$$

The moment amplification factor (δ_m) shall be taken as the greater of—

δ_b = moment amplification factor for a braced member, determined in accordance with Clause 4.2.2.2; and

δ_s = moment amplification factor for a sway member, determined as follows:

- (a) *Sway members in rectangular frames* The amplification factor (δ_s) shall be calculated as follows:

$$\delta_s = \frac{1}{1 - \left(\frac{1}{\lambda_{ms}} \right)} \quad \dots 4.2.2.3(2)$$

The elastic buckling load factor (λ_{ms}) for the member under consideration shall be determined as follows:

$$\lambda_{ms} = \frac{\sum \left(\frac{N_{oms}}{L} \right)}{\sum \left(\frac{N^*}{L} \right)} \quad \dots 4.2.2.3(3)$$

where

N_{oms} = member buckling load for each column determined in accordance with Clause 4.3.1, and the summations include all columns at the same level

N^* = member design axial force, with tension taken as negative

- (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:

$$\delta_s = \frac{1}{1 - \left(\frac{1}{\lambda_c} \right)} \quad \dots 4.2.2.3(4)$$

where the elastic buckling load factor (λ_c) shall be determined from a rational buckling analysis of the whole frame.

NOTE: λ_c is the ratio of the elastic buckling load set of the frame to the design load set for the frame, and hence will vary with each design load set.

4.2.3 Residual stresses

The design strengths determined by the methods defined in this Standard may be assumed to allow for residual stresses due to rolling, handling, transportation and normal welding procedures. Allowance may however need to be made for the effects of residual stresses in the calculation of initial deflections.

4.3 MEMBER BUCKLING ANALYSIS

4.3.1 Member elastic buckling load

The elastic flexural buckling loads of braced (N_{omb}) and sway (N_{oms}) members shall be determined as follows:

$$N_{omb}, N_{oms} = \frac{\pi^2 EI}{(k_e L)^2} \quad \dots 4.3.1$$

where

k_e = member effective length factor, determined in accordance with Clause 4.3.2

L = member length from centre-to-centre of its intersections with supporting members

For composite compression members, the value of $(EI)_e$ determined in accordance with Clause 10.6.2.3 shall be used for EI .

4.3.2 Member effective length factor

4.3.2.1 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.

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

- Clause 4.3.2.2 for members with idealized end restraints;
- Clause 4.3.2.3 for braced members in frames; or
- Clause 4.3.2.3 for sway members in rectangular frames with regular loading and negligible axial forces in the beams.

4.3.2.2 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 4.3.2.2.

	Braced member			Sway member		
Buckled shape						
Effective length factor (K_e)	0.70	0.85	1.00	1.20	2.20	2.20

FIGURE 4.3.2.2 EFFECTIVE LENGTH FACTORS FOR MEMBERS FOR IDEALIZED CONDITIONS OF END RESTRAINT

4.3.2.3 Members in frames

For a compression member that forms part of a rigid-jointed frame, the member effective length factor (k_e) shall be obtained from Figure 4.3.2.3(A) for a braced member and from Figure 4.3.2.3(B) for a sway member. In Figures 4.3.2.3(A) and 4.3.2.3(B), γ_1 and γ_2 are the ratios of the compression member stiffness to the end restraint stiffness. The γ -values shall be obtained from the following equation:

$$\gamma = \frac{\sum \left(\frac{I}{L} \right)_c}{\sum \beta_e \left(\frac{I}{L} \right)_b} \quad \dots 4.3.2.3$$

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;

where

$\Sigma (I/L)_c$ = 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

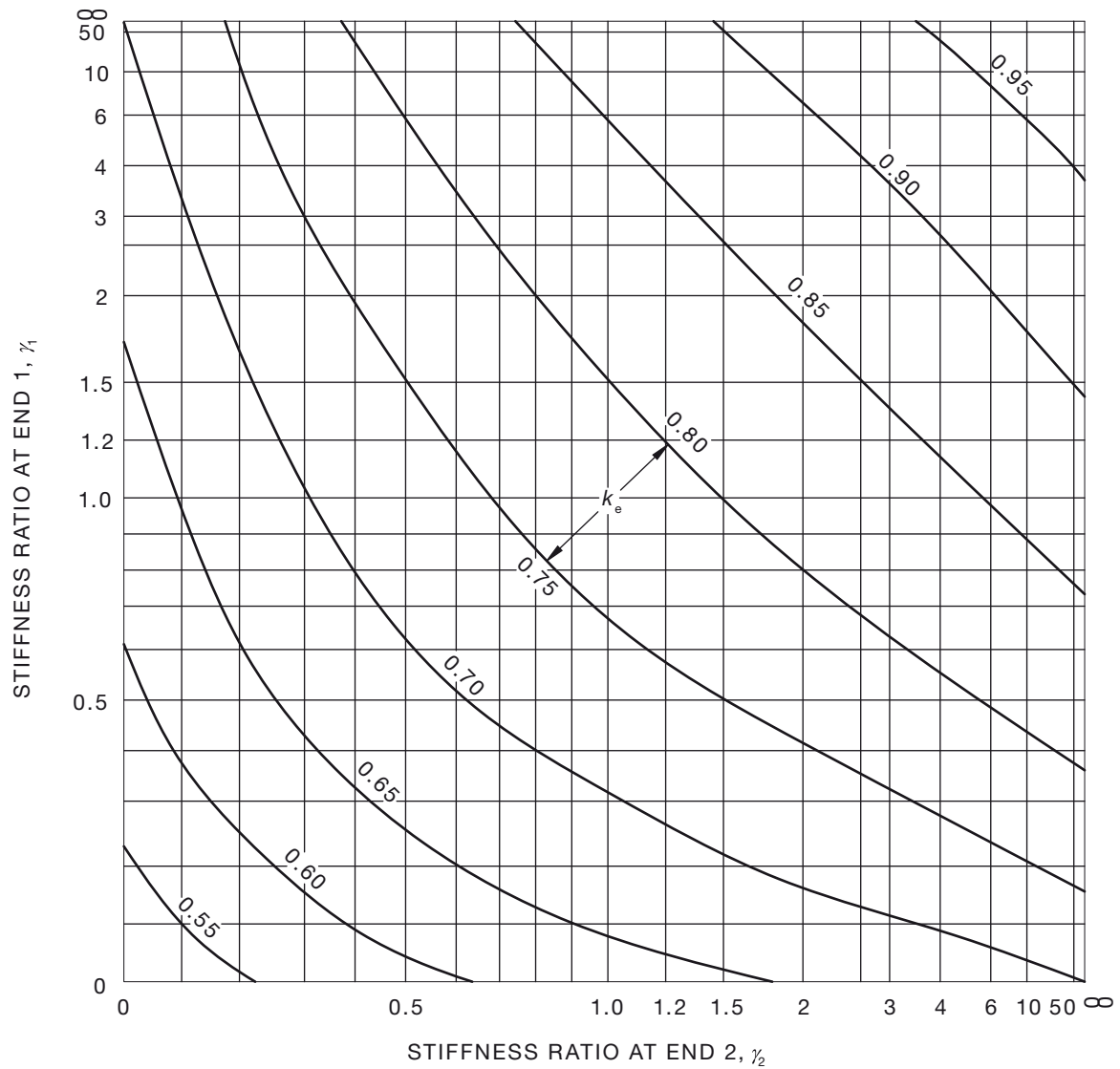
$\Sigma \beta_e (I/L)_b$ = sum of the stiffnesses in the plane of bending for all the beams rigidly connected at the end of the member under consideration

The contributions of any beams pin-connected to the member shall be neglected

β_e = modifying factor that accounts for end conditions at the far ends of the beams, as given in Table 4.3.2.3

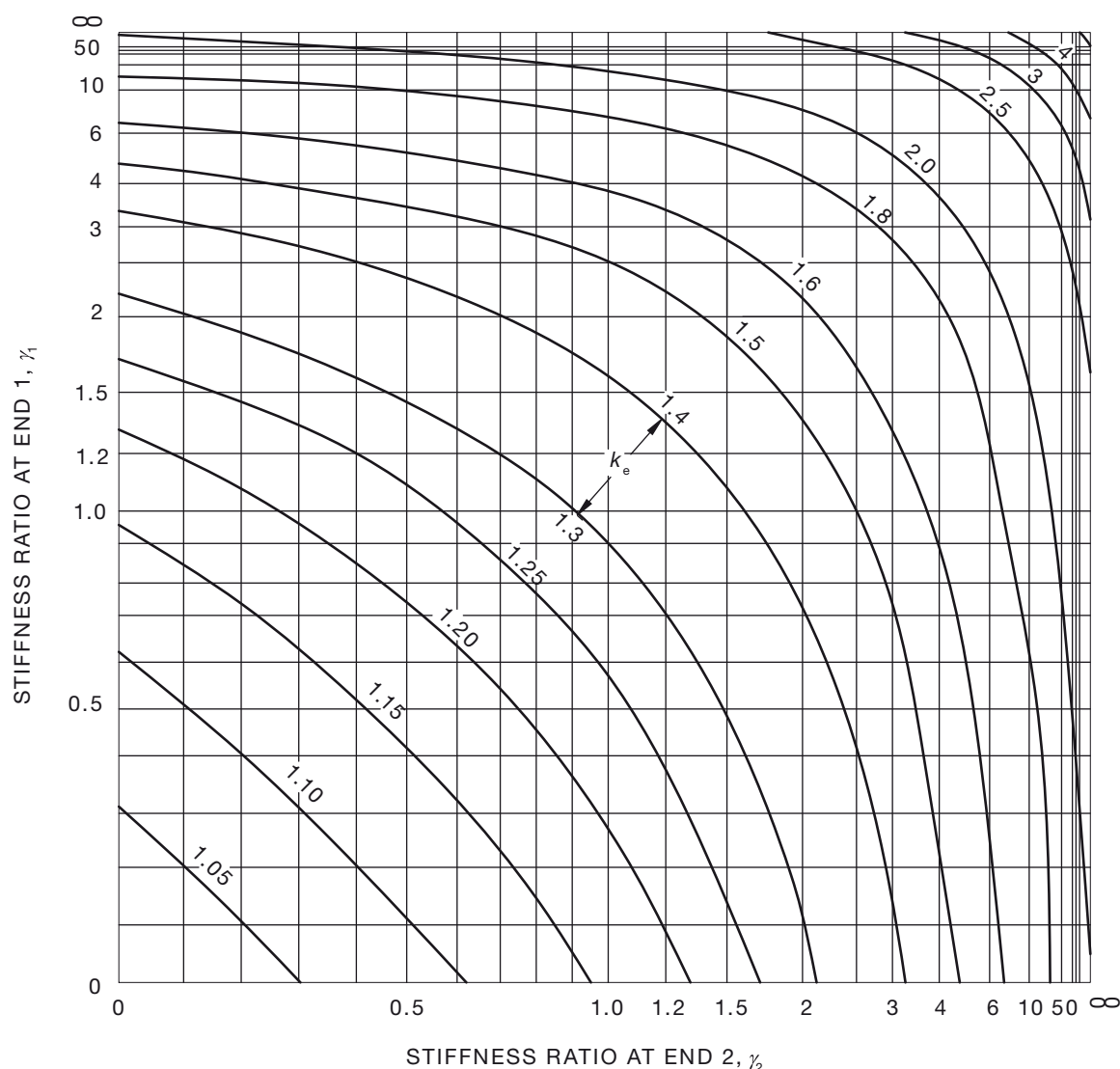
TABLE 4.3.2.3
STIFFNESS MODIFYING FACTORS (β_e)

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



NOTE: For a braced member, the translational restraint stiffness has been assumed to be infinite.

FIGURE 4.3.2.3(A) EFFECTIVE LENGTH FACTORS FOR BRACED MEMBERS



NOTE: For a sway member, the translational restraint stiffness has been assumed to be zero.

FIGURE 4.3.2.3(B) EFFECTIVE LENGTH FACTORS FOR SWAY MEMBERS

4.4 ANALYSIS OF COMPOSITE BEAMS, GIRDERS AND COLUMNS

4.4.1 Effective width of flanges for shear lag

Allowance shall be made for the flexibility of composite flanges affected by shear in their plane (shear lag) either by means of rigorous analysis, or by using an effective width of flange (see Clause 4.4.2).

4.4.2 Effective width of concrete flanges—Simplified method

The effective width of concrete flanges for shear lag shall be determined in accordance with the following:

- When elastic global analysis is used, an effective width shall be assumed over the whole of each span. This value shall be taken as the value $b_{\text{eff},1}$ at midspan for a span supported at both ends, or the value $b_{\text{eff},2}$ at the support for a cantilever.
- At midspan or an internal support, the total effective width ($b_{\text{eff},i}$) (see Figure 4.4.2), shall be determined as follows:

$$b_{\text{eff},i} = b_0 + \sum b_{\text{ei}}$$

... 4.4.2(1)