AS/NZS 1664.2
Supplement 1:1997

Aluminium structures

Part 2: Allowable stress design—Commentary

(Supplement 1 to AS/NZS 1664.2:1997)

This Joint Australian/New Zealand Standard was prepared by Joint Technical Committee BD/50, Aluminium Structures. It was approved on behalf of the Council of Standards Australia on 27 June 1997 and on behalf of the Council of Standards New Zealand on 11 July 1997. It was published on 5 September 1997.

The following interests are represented on Committee BD/50:

The Association of Consulting Engineers, Australia
Aluminium Development Council
Australian Building Codes Board
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Part 2: Allowable stress design—Commentary

(Supplement 1 to AS/NZS 1664.2:1997)

Originated in Australia as part of AS 1664—1975.
Previous Australian edition AS 1664—1979.
Jointly revised and designated AS/NZS 1664.2 Supplement 1:1997.

Incorporating:
Amdt 1—1999

PUBLISHED JOINTLY BY:

STANDARDS AUSTRALIA
1 The Crescent,
Homebush NSW 2140 Australia

STANDARDS NEW ZEALAND
Level 10, Radio New Zealand House,
155 The Terrace,
Wellington 6001 New Zealand

ISBN 0 7337 1285 1

PREFACE

This Commentary was prepared by the Standards Australia/Standards New Zealand Committee BD/50, Aluminium Structures. It is intended to be read in conjunction with AS/NZS 1664.2, *Aluminium Structures*, Part 2: *Allowable stress design*, but it does not form an integral part of that Standard.

The objective of this Commentary is to provide background material to the requirements of AS/NZS 1664.2.

The clause numbers and titles used in this Commentary are the same as those in AS/NZS 1664.2 except that they are prefixed by the letter C.

Gaps in the numerical sequence of this Commentary's clause numbering means that no explanation of or background to the missing clause(s) is necessary.

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SECTION C1 GENERAL

C1.1 SCOPE These specifications apply to normal ambient temperature uses of aluminium alloys. For higher temperatures, strengths and other properties (such as corrosion resistance) of different alloys are affected to varying degrees. For information regarding properties at elevated temperatures, the supplier should be consulted.

C1.2 MATERIALS The alloys covered in the specifications are those that are normally used for general structural purposes. Most are covered by applicable Australian/New Zealand specifications. Additional information on these alloys, the temper designations, and the products available is published in Reference 1.

C1.3 SAFETY FACTORS The specifications for aluminium structures in Reference 19 are not limited as to type of structure. The general formulas in Table 3.4(C) can be applied to any structure, with appropriate values substituted for the factors of safety n_y and n_u. The values of factors of safety are given in Table 3.4(A) for 'Building type structures' and for 'Bridge structures'. 'Building type structures' include highway signs, luminaires and traffic signals. The 'Bridge structures' cover bridges that are not designed according to Reference 2.

C1.4 REFERENCED DOCUMENTS The Standards listed in Clause 1.4 are subject to revision from time to time. A check should be made with Standards Australia or Standards New Zealand, as appropriate, as to the currency of any document referenced in the text.

SECTION C2 DESIGN PROCEDURE

C2.2 PROCEDURE Calculated stresses in the members resulting from external loading are compared with the appropriate allowable stresses. Alternatively, the provisions of Section 8, Testing, can be used.

C2.3 LOADING AS/NZS 1664.2 permits the use of reduced wind or seismic loads or combined loads involving wind or seismic loads. This reduction is only applicable in Australia because for New Zealand NZS 4203:1984 already allows for such a reduction.

SECTION C3 GENERAL DESIGN RULES

C3.4 ALLOWABLE STRESSES

C3.4.2 Tension, axial, net section In general, the allowable tensile stress for building structures is the lower of two values that results from applying a factor of safety of 1.65 to the yield strength or 1.95 to the tensile strength. The corresponding factors of safety used to determine allowable tensile stresses for bridge structures are 1.85 and 2.2. These factors of safety are the same as those that were used in References 4 and 5.

In the general formula for determining allowable tensile stress on the basis of the ultimate tensile strength, the factor of safety n_u is multiplied by a factor k_t. For regions farther than 25 mm from a weld, this factor is 1.0 for all the alloys that appear in the specifications, except for alloy 2014-T6. The value of k_t for 2014-T6 is 1.25. This factor is introduced to take account of the fact that this high-strength alloy is somewhat more notch sensitive than the other alloys listed in the specifications. The resulting allowable tensile stress for bridge structures of 2014-T6 is the same as that used in specifications for structures of this alloy published in Reference 6.

C3.4.3 Tension in extreme fibres of beams—structural shapes bent about strong axis, rectangular tubes This allowable tensile stress is the same as that specified for axial tension. This Clause is intended to apply to a wide variety of shapes, including trapezoidal corrugated sheet. As a result, no attempt is made to take advantage of the shape factor which would apply to standard structural shapes and which justifies somewhat higher allowable tensile stresses.

C3.4.4 Tension in extreme fibres of beams—round or oval tubes The allowable tensile stresses for round or oval tubes subjected to bending are somewhat higher than for structural shapes. Analysis and tests (Ref. 7) have demonstrated that yielding or failure of tubular beams does not occur until the bending moment considerably exceeds the yield moment predicted by the ordinary flexure formula. This results from the non-linear distribution of stress in the inelastic range. Yielding does not become apparent as soon as the calculated stress in the extreme fibre reaches the yield strength because the less highly stressed fibres near the centre of the beam are still in the elastic range. The constants 1.17 and 1.24 can be considered as shape factors for yielding and ultimate strength, respectively.

These constants were picked from curves of yield strengths at 0.2 percent offset for tubes of representative proportions. The shape factors on ultimate strength were deduced from apparent and actual stress-strain curves at a stress corresponding to tensile strength of the material.

C3.4.5 Tension in extreme fibres of beams—shapes bent about weak axis, rectangular bars, solid round bars and plates As in the case of round tubes, theory and tests have shown that aluminium alloy members of these shapes can undergo bending moments that are considerably higher than those predicted on the basis of the ordinary flexure formula (Ref. 8). In this case, the shape factors used for yielding and ultimate strength, respectively, are 1.30 and 1.42. That these factors are conservative can be noted from the fact that the shape factor for fully plastic action is 1.50 for a rectangular section.

C3.4.6 Bearing on rivets and bolts Bearing yield and ultimate strengths are defined by means of a test on a pin in a hole, as described in ASTM E238 (Ref. 9). The bearing yield stress is the stress at an offset of 2 percent of the hole diameter on a bearing stress-deformation curve. The ratio of edge distance to fastener diameter in this test is normally 2.0, where edge distance is the distance measured from the centre of the hole to the edge of the material in the direction of applied stress. Bearing tests (Ref. 10) show that for ratios of edge distance to fastener diameter as small as 1:5, it is conservative to reduce the allowable bearing stress by the ratio of the edge distance to twice the fastener diameter. The specifications do not allow ratios of edge distance to fastener diameter smaller than 1:5.

Tests (Ref. 11) have demonstrated that a relatively even distribution of load among the fasteners is achieved before ultimate failure of mechanically fastened joints in structural aluminium alloys. Nevertheless, the factor of safety normally applied to ultimate strength is increased by 20 percent since this safety can normally be obtained with relatively little cost.

C3.4.7 Bearing on flat surfaces and pins and on bolts in slotted holes The bearing strength for flat surfaces, elements with pins in holes and elements with pins or bolts in elongated holes is 2/3 the bearing strength of elements joined by properly fitting rivets and bolts. This requirement originally was adopted from steel specifications. A lower bearing strength appears to be reasonable in these cases because the applied pressure can be much more concentrated than that in riveted or bolted joints, because the diameter of the loading element (pin) can be small compared to the diameter of the opening in the element that is being loaded. Good practice in bolted and riveted joints requires a reasonable fit between fastener and hole diameter.

C3.4.8 Compression in columns, axial, gross section

C3.4.8.1 *General* The formulas in this Clause for values of kL/r exceeding S_1 approximate the column strength given by the tangent modulus column formula. The tangent modulus formula is—

$$F_{cr} = \frac{\pi^2 E_t}{(kL/r)^2} \qquad \text{. . . C3.4.8.1(1)}$$

where

F_{cr} = column strength

E_t = tangent modulus (slope of stress strain curve) corresponding to F_{cr}

kL = effective length of column

r = least radius of gyration of column

In the elastic range, this formula is simply the Euler column formula, which is used as a basis for allowable stresses for values of kL/r exceeding S_2. For values of kL/r between S_1 and S_2 the tangent modulus formula is approximately closely by the straight line (Ref. 8), which is used as a basis for the allowable stress formula.

Numerous tests have shown that these formulas closely predict the strength of essentially straight columns (Refs. 8 and 12). To ensure adequate safety in the presence of accidental eccentricity and initial crookedness, which may reduce the strength of practical columns (Refs. 13 and 14), the factor of safety n_u rather than n_y is applied to column strength.

The effective length of columns is normally defined as a factor k times the length of the column between lateral support. The designer is given the freedom to chose the proper value of k. The background for this change can be found in Reference 15.

For values of kL/r less than S_1, the allowable compressive strength in columns is based on the compressive yield strength. The factor of safety on yielding n_y is increased by multiplying it by the quantity k_c in order to provide a range of slenderness ratios in the short column region for which the allowable stress is independent of the slenderness ratio. This was done to simplify the proportioning of very short members whose failures are more controlled by yielding rather than buckling.

A great deal of background information relating to column specifications and other buckling problems can be found in Reference 16.

C3.4.8.3 *Doubly or singly symmetric sections subject to torsional or torsional-flexural buckling* Based on data in Reference 17, Reference 18 shows that the column design equations of Clause 3.4.8.1 can be used for torsional-flexural buckling if an equivalent slenderness ratio $\left(\frac{kL}{r}\right)_e$ is defined. The redefinition is based on the elastic torsional-flexural buckling stress. The inelastic torsional-flexural buckling stress is then calculated using the column design equations used for flexural buckling.

For point symmetric sections such as cruciforms, torsional buckling is the most likely mode of failure and F_e becomes equal to F_t.

C3.4.8.4 *Nonsymmetric sections subject to torsional or torsional-flexural buckling* Non-symmetric sections that are subject to torsional or torsional-flexural buckling may be designed as follows:

(a) Determine the elastic torsional-flexural buckling stress according to the torsional-flexural theory.

(b) Determine the equivalent slenderness ratio using Equation 3.4.8.3(1).

(c) Determine the limiting or allowable stress with the equations of Clause 3.4.8.1.

C3.4.9 Uniform compression in components of columns—buckling axis

C3.4.9.1 *Uniform compression in components of columns whose buckling axis is an axis of symmetry—flat plates supported along one edge* Items 3.4.9.1(a) and 3.4.9.1(b) are the same as those in Reference 19. However, Item 3.4.9.1(c) is changed and is now based on the post-buckling strength rather than the buckling strength of unstiffened plate elements. This change may result in calculated capacities that are much larger than those according to Reference 19. Justifications for this change can be found in Reference 20. Tests performed on stub-columns with cruciform cross-sections show that the local buckling equation used in Reference 19 is too conservative and the post-buckling equation given in this Standard represents the behaviour better. These provisions apply to wide flange shapes buckling about either axis and channels buckling in the strong direction.

C3.4.9.2 *Uniform compression in components of columns whose buckling axis is not an axis of symmetry-flat plates supported along one edge* In columns buckling about a principal axis that is not an axis of symmetry the centroid of the stresses may not be the same as that for the full section. This is due to the non-linear stress distribution in the post-buckling range of the flat plate elements of the section. In such cases, though some post-buckling strength may exist, it may not be as large as that if the buckling axis were an axis of symmetry. For this reason the provisions of this section limits the strength to local buckling strength. Column sections such as channels buckling about the weak axis are covered by these provisions.

C3.4.10 Uniform compression in components of columns, gross section—flat plates

C3.4.10.1 *Uniform compression in components of columns, gross section—flat plates with both edges supported* The ultimate strength of a plate supported on both edges may be appreciably higher than the local buckling strength. Thus the allowable stress is obtained by applying the factor of safety n_u to a formula that gives a conservative approximation to the ultimate strength of the plate (Ref. 21).

In the inelastic stress range, the ultimate strength is the same as local buckling strength, so the allowable stress is based on the local buckling formula with an equivalent slenderness ratio of 1.6 b/t and a factor of safety n_u.

The coefficient 1.6 is approximately the value that applies to a plate simply supported on two longitudinal edges.

C3.4.10.2 *Uniform compression in components of columns—flat plates with one edge supported and other edge with stiffener* Clause 4.4 of Reference 19 contained provisions which could be very conservative. Equation 3.4.10.2(2) provides a transition between the allowable stress in an unstiffened plate element and the allowable stress in an edge stiffened plate element with a fully adequate stiffener. This Equation removes the conservatism mentioned. The predicted capacities using the provisions in this Section correlate well with the experimental capacities obtained from test on stub columns with edge stiffeners. Development of the new design procedure is given in Reference 20.

Equations 3.4.10.2(3) to 3.4.10.2(5) are the r_s/R_a ratios for different ranges of the (b/t) ratios where r_s is the radius of gyration of an edge stiffener about the plate mid-thickness surface and R_a is the radius of gyration of a stiffener adequate to make the flange being stiffened function as a plate element supported on both longitudinal edges. Equations for R_a are given by the denominators of Equations 3.4.10.2(4) and 3.4.10.2(5). The equations for determining R_a are adapted from Reference 22 and compared with the equation proposed in Reference 24. The elastic buckling analysis in Reference 24 shows that an edge stiffener is adequate if $r_s = 6t$. Elastic buckling begins at a (b/t) ratio equal to S where S is the limiting (b/t) ratio at which a stiffened element is fully effective. At this value of (b/t) ratio, the value of R_a obtained from Equation 3.4.10.2(4) is identical to the value of r_s derived in Reference 24. A linear relationship is assumed between R_a and (b/t) ratio if the (b/t) ratio is between $S/3$ and S.

The value of r_s necessary to be considered as an adequate edge stiffener is larger than $6t$ in the post-buckling range of the element being stiffened. Post-buckling strength exists in an edge stiffened plate element with a (b/t) ratio exceeding S. Equation 3.4.10.2(5) is valid for values of the (b/t) ratios between S and $2S$. Sufficient test data does not exist to develop an equation for R_a when the (b/t) ratio exceeds $2S$.

The limitation on the D_s/b ratio prevents any adverse interaction between the local buckling of the lip stiffener and the flange.

It should be noted that F_c determined according to Equations 3.4.10.2(1) and (2) should not exceed the value of F_c determined for the stiffening lip according to Clause 3.4.9.1.

In this Clause as well as in some of the subsequent clauses, it is stated that if the inside corner radius exceeds 4 times the thickness then the inside radius shall be assumed equal to 4 times the thickness in calculating b. This rule was reached on the basis that a radius that is too large would be detrimental to the post buckling strength of the element and that the flat element width would be too unconservative to take in calculating the strength.

C3.4.10.3 *Uniform compression in components of columns—flat plates with both edges supported and with an intermediate stiffener* The provisions in this Clause are based on Reference 24 which is discussed further in Clause 3.4.19.

C3.4.11 Uniform compression in components of columns, gross section—curved plates supported on both edges, walls of round or oval tubes In theory, the elastic buckling strength of an ideal cylindrical shell loaded in compression can be determined by substituting an equivalent slenderness ratio of 4.0 R/t into the column formula. The buckling strength of actual shells, however, is strongly affected by imperfections in the geometry and end conditions of the shells. Tests indicate that this effect tends to increase with increasing R/t. This effect of imperfections is taken into account by the formulas in this Clause, which are conservative when compared with the results of numerous tests on tubes and cylinders in References 7 and 25. The formulas of this Clause are based on local buckling strength, since severe deformations occur at this load.

The strength of circumferentially welded tubes has been shown to be given accurately by the same equations as those for unwelded tubes for cases in which $R/t \leq 20$ (approximately). For circumferentially welded cylinders with much higher R/t, recent studies show that the provisions may be very unconservative (Ref. 18). Thus the restriction of $R/t \leq 20$ for tubes with circumferential welds was introduced in this Standard.

C3.4.12 Compression in beams, extreme fibre, gross section—single web beams bent about strong axis The allowable compressive stresses in single-web structural shapes and built-up sections bent about the strong axis are based on the lateral, torsional buckling strength of beams with a factor of safety n_y. In the inelastic stress range the formulas employ the straight line approximation to the tangent modulus buckling curve that is also used for columns. Tests have shown this curve to be conservative for beams (Ref. 8). The basis for the lateral torsional buckling of single web beams about their strong axis is in Reference 26.

A simple span beam restrained against movement laterally and vertically at the supports, but free to rotate about the vertical and horizontal axes at the ends, is assumed. A symmetrical section and uniform moment are also assumed. The expressions derived for lateral buckling (Ref. 26) were rather complicated. To simplify calculations an approximate method for estimating lateral buckling strength was developed. An effective slenderness ratio $L/1.2r_y$ was found to provide conservative answers for standard aluminium shapes. Because of the conservatism of the approximate method, Clause 4.9 allows the designer to calculate a more precise value for r_y based on the ‘exact’ solution.

The factor of safety applied to beam buckling is n_y rather than the value used for columns, n_u. The assumptions on restraint at ends and at loads are conservative. In addition, continuous beams can redistribute moment and beams attached at their ends can carry some load in membrane actions. All the assumptions err on the conservative side, and thus the lower factor of safety was used.

C3.4.13 Compression in beams, extreme fibre, gross section—round or oval tubes For values of R_b/t below the slenderness limit S_1, the allowable stress is increased over the basic allowable compressive design stress for single web beams, since tests have demonstrated that a shape factor of 1.17 can be applied to the yielding of round tubes. For values of R_b/t between S_1 and S_2, the allowable stress is based on a formula that gives a close approximation to experimental values of buckling strength for round tubes in bending (Ref. 7).

The value of S_2 in this Clause is the value of R_b/t at which the curve for bending strength intersects the curve for buckling stress under axial compression. For greater values of R_b/t, the conservative assumption is made that the allowable stress in bending is the same as that in direct compression. The limitation that the equations apply for $R/t \leq 20$ for tubes with circumferential welds is the same as that applied in Clause 3.4.11.

C3.4.14 Compression in beams, extreme fibre, gross section—solid rectangular and round section beams If a solid rectangular beam is laterally unsupported and is sufficiently narrow in cross-section, it can fail by lateral torsional buckling. This type of failure is taken into account, using $2.3(d/t)\sqrt{L_b/d}$ as the equivalent slenderness ratio. If the beam is sufficiently wide, it will not buckle, and the allowable stress is controlled by the yield strength. When $2.3(d/t)\sqrt{L_b/d} < S_1$ a shape factor of 1.3 for yielding is assumed as for Clause 3.4.5. In the intermediate slenderness ratio range, the buckling strength is considerably affected by a redistribution of stress that accompanies plastic yielding, so that the apparent stresses at buckling are appreciably higher than values for single web beams. The formula used to represent buckling strength has been shown to agree well with the results of buckling tests on rectangular beams (Ref. 8).

The formulas are based on the conditions of a uniform moment on a single span beam, simply supported, with the ends prevented from lateral deflection, but free to rotate about the vertical axis.

The factor of safety applied to beam buckling is n_y, as in Clause 3.4.12. Experience indicates this factor of safety is adequate.

C3.4.15 Compression in beams, extreme fibre, gross section—rectangular tubes, box sections and beams having sections containing tubular portions The allowable stresses in this Clause are based on the lateral, torsional buckling strength of box beams. The factor of safety used is n_y, for the same reasons as discussed under Clause 3.4.12. The expression used for equivalent slenderness ratio of a box beam is $1.6\sqrt{\dfrac{L_b Z_c}{0.5\sqrt{I_y J}}}$.

This relationship is more accurate than the slenderness ratio of $1.6\sqrt{L_b Z_c / I_c}$ used previously, which was based on References 4 and 5. It was derived from the more complex theoretical equation for lateral buckling strength by ignoring the term that represents the warping resistance of the beam, since this term is small in comparison to the term that represents St. Venant torsion, and by taking note of the fact that lateral buckling will govern the design only for relatively deep narrow beams for which the torsion constant J is roughly proportional to I_y. Since this Standard may be used for a wide variety of extruded or formed shapes, the conservative assumption was made that the shape factor for yielding is 1.0.

This Section also allows replacing $0.5\sqrt{I_y J}$ by I_y, for narrow rectangular tubes in which warping is insignificant. The equations in this Clause give conservative results and are accurate when the warping constant C_w is small compared to 0.01 $J\,(k_y L_b)^2$. If C_w is not small compared to 0.01 $J\,(k_y L_b)^2$ the use of an effective value calculated according to Clause 4.9.4 gives more accurate results.

Torsional constant J for a closed section is—

$$J = \frac{4A_m^2}{\int \frac{ds}{t}} \qquad \text{. . . C3.4.15(1)}$$

where

A_m is the mean of the areas between the inner and outer boundaries and ds is the incremental length along the perimeter of thickness t. For uniform thickness t, this equation becomes—

$$J = \frac{4A_m^2 t}{s} \qquad \text{. . . C3.4.15(2)}$$

where

s is the length of the boundary at mid-thickness. The expression for a hollow rectangular tube is—

$$J = \frac{2t_2 t_1 (a - t_2)^2 (b - t_1)^2}{a t_2 + b t_1 - t_2^2 - t_1^2} \qquad \text{. . . C3.4.15(3)}$$

The dimensional notation is illustrated in Figure C3.4.15.

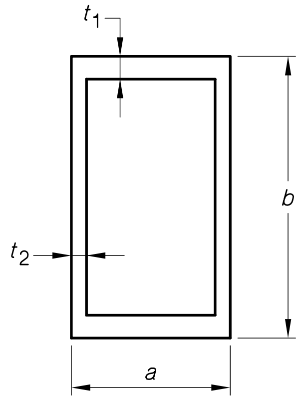


FIGURE C3.4.15 CROSS-SECTIONAL NOTATION

C3.4.16 Compression in components of beams (component under uniform compression), gross section—flat plates supported along one edge Allowable stresses for values of *b/t* exceeding S_1 were obtained by applying the factor of safety n_y to the ultimate strength of an outstanding flange simply supported on one edge (Ref. 21). If this Clause were to be applied only to standard structural shapes, it would have been possible to assume some restraint against rotation at the supported edge of the flange, which would have resulted in somewhat higher allowable stresses. However, this Clause also covers other extruded shapes and formed sheet members, in which the web may offer little restrain against flange rotation. Therefore, the conservative assumption of simple support was made.

This Clause permits the designer to take advantage of the fact that the ultimate strength may exceed the local buckling strength for very thin sections.

Equations 3.4.16(2) and (3) are based on the ultimate strength of an outstanding flange simply supported on one edge.

C3.4.17 Compression in components of beams (component under uniform compression)—flat plates with both edges supported This is similar to Clause 3.4.10 for components of columns, except that the factor of safety used is n_y rather than n_u because the strength prediction of beams and beam elements are thought to be more conservative than those of compression members.

Equations 3.4.17(2) and 3.4.17(3) are based on the ultimate strength of a plate simply supported on both edges.

C3.4.18 Compression in components of beams—curved plates with both edges supported These expressions for curved sections are taken from Reference 27. They apply to curved components of beams other than round or oval tubes, which are covered in Clause 3.4.13. For values of R_b/t between S_1 and S_2 the stresses allowed by Clause 3.4.18 are somewhat lower than those allowed by Clause 3.4.13 because tests have shown that not all beams with curved sections of these proportions can sustain the high apparent stresses developed by round or oval tubes.

C3.4.19 Uniform compression in components of beams—flat plates with one edge supported and other edge with stiffener The provisions in this Clause are similar to that in Clause 3.4.10.2. The commentary on Clause 3.4.10.2 is equally applicable to this Clause.

The predicted capacities using the provisions in this Clause, in conjunction with the weighted allowable stress approach, correlate well with the experimental capacities obtained from beam tests (Ref. 20).

C3.4.20 Uniform compression in components of beams—flat plates with both edges supported and with an intermediate stiffener The provisions in this Clause are based on work performed by Sharp (Ref. 24). The equivalent slenderness ratio (λ_s) to be used with the column buckling Equations 3.4.20(2) and 3.4.20(3) is detailed in Clause 3.4.20. The predicted capacities using the provisions in this Clause, in conjunction with the weighted allowable stress approach, correlate well with the experimental capacities obtained from beam tests as shown in Reference 20.

C3.4.21 Compression in components of beams (component under bending in own plane), gross section—flat plates with compression edge free, tension edge supported The coefficients in the formula for inelastic buckling strength were assumed to be the same as for rectangular beams (see Clause C3.4.14) because calculations and tests have shown that the apparent stress (M_c/I) at which the yield strength is reached in the outer fibre of sections such as tees, angles and channels is even higher than for rectangular beams. The equivalent slenderness ratio was assumed to be 3.5 b/t, which implies partial restraint, against rotation at the supported edge.

This is based on elastic buckling strength. This type of component is assumed to have negligible post-buckling strength.

C3.4.22 Compression in components of beams (component under bending in own plane), gross section—flat plate with both edges supported The comments in Clause C3.4.21 concerning shape factor and buckling formula constants apply here also. The equivalent slenderness ratio was assumed to be 0.67 h/t, which applies to a plate in bending with both edges simply supported. Simple support was assumed because the boundary conditions at the compression edge are more important than those at the tension edge and it is possible that compression elements supporting the compression flange may buckle at the same time as the web.

C3.4.23 Compression in components of beams (component under bending in own plane), gross section—flat plates with horizontal stiffener, both edges supported Comments made in Clauses C3.4.21 and C3.4.22 apply here also. The equivalent slenderness ratio is 0.29 h/t based on simple support at the edges and at the stiffener (Ref. 28).

C3.4.24 Shear in webs—unstiffened flat webs Allowable shear stresses in unstiffened flat webs are determined by applying the factor of safety n_y to the calculated buckling strength for a web with partial restraint against rotation at the attachment to the flanges. The corresponding value of the equivalent slenderness ratio is 1.25 h/t (Refs 28 and 29). The formulas for the buckling coefficients in the inelastic range were developed originally for shear buckling of tubes (Ref. 7) but they apply also to flat plates in shear.

C3.4.25 Shear in webs—stiffened flat webs A stiffened flat web that has buckled in shear can continue to carry load by diagonal tension action in the web (Refs 30, 31 and 32). Thus it is not necessary to use the same factor of safety against shear buckling of the stiffened web as is used for an unstiffened web in which local buckling could bring about collapse. However, it was assumed that it would not be desirable to have local buckling of webs at design loads, both from the standpoint of appearance and because of the possibility of fatigue failure. Thus, the factor of safety n_a was applied to the local buckling strength of stiffened flat webs in shear. This factor of safety is used to ensure that stresses at design loads are less than the local buckling stress. The edges were assumed to be partially restrained against rotation, giving an equivalent slenderness ratio of—

$$1.25 a_1 \Big/ \left(t \sqrt{1 + 0.7 \left(\frac{a_1}{a_2} \right)^2} \right)$$

SECTION C4 SPECIAL DESIGN RULES

C4.1 COMBINED COMPRESSION AND BENDING The provisions on combined compression and bending in Reference 19 have been consolidated into this Clause. The provisions have also been changed to agree with the allowable stress design versions of other metal structural specifications (Ref. 22).

C4.1.1 Combined tension and bending The provisions in this Clause are the same as those used in other metal structural specifications (Ref. 22).

C4.2 TORSION AND SHEAR IN TUBES The equation for equivalent *h/t* is based on the theoretical elastic buckling strength of cylinders in torsion. Tubes loaded in torsion are not as sensitive to the effect of initial imperfections in the geometry as are tubes loaded in axial compression. The theoretical buckling strength has been found to give good agreement with the results of tests on thin cylinders that fail in the elastic range (Ref. 33) and the use of this expression with the inelastic buckling equations employed in the specifications also gives good agreement with experimental results in the inelastic stress range (Ref. 7).

C4.4 COMBINED SHEAR, COMPRESSION AND BENDING The formula for interaction of combined stresses in walls of curved surfaces or round tubular members is based on investigations reported in References 16, 29 and 34. Likewise, the interaction equation for combined stresses in webs of rectilinear shapes and plates of built-up girders or similar members is based on the buckling strength of these members (Refs 16 and 28).

C4.5 HORIZONTAL STIFFENERS FOR WEBS This Clause requires that if a horizontal stiffener is used on a beam web, it shall be located so that the distance from the toe of the compression flange to the centroid of the stiffener is 0.4 of the distance from the toe of the compression flange to the neutral axis of the girder. This is the optimum location for increasing the buckling strength of the web under the influence of compressive bending stresses in the web. The resulting increase in allowable compressive stress in the web is reflected in Clause 3.4.23 (Ref. 28). The formula for stiffener moment of inertia which is the same as that used in earlier specifications published by ASCE (Refs 4 and 5), agrees closely with the size recommended on the basis of theoretical considerations (Ref. 28) and is also in good agreement with the results of tests (Ref. 23). The factor α takes account of the effect of eccentricity for a stiffener on one side of the web only (Ref. 35).

C4.6 VERTICAL STIFFENERS FOR SHEAR WEBS The stiffener size recommended is sufficient to limit local buckling of shear webs to the panels between stiffeners and to provide considerable post-buckling strength in the web. These formulas were also used in References 4 and 5. They agree well with the results of tests in Reference 36 and are conservative in comparison with stiffener sizes derived from theoretical considerations (Ref. 37). Background for these provisions is discussed further in References 38 and 39.

The Clause requires that the moment of inertia of a stiffener at a point of bearing should be equal to the sum of the moment of inertia required to resist the tendency of the web to buckle and the moment of inertia required for the stiffener to carry the bearing load as a column with the length equal to the height of the web.

C4.7 EFFECTS OF LOCAL BUCKLING ON MEMBER PERFORMANCE These specifications apply to either thin or heavy-gauge construction. In some cases, consideration is given to the design of members that incorporate elements having relatively large ratios of width to thickness. In the following Clauses such elements are referred to as 'thin', meaning that they are thin relative to their width, even though the thickness itself may be any value.

C4.7.1 Local buckling stresses In Clauses 3.4.9, 3.4.10.1, 3.4.10.2, 3.4.16, 3.4.17, 3.4.19, 3.4.22 and 3.4.23 for thin plate elements, namely, elements having b/t ratios in excess of S_2, the ultimate load carrying capacity is based on the post buckling strength which can be quite significantly higher than the local buckling strength.

For these cases where the post buckling strength is the basis for design, the local buckling stresses are needed in certain situations. All the equations for local buckling stresses except those in Clauses 3.4.10.2 and 3.4.19 are based on plate or stiffener buckling theories. In Clauses 3.4.10.2 and 3.4.19, the local buckling stress is based on the derivation given below. Limiting the stresses to the local buckling stress divided by a factor of safety of 1.2 would limit the appearance of buckling at allowable loads.

One can visualize the post buckling strength in terms of the effective width approach, as is done for deflection calculations. Using the effective width approach, the ultimate axial load capacity of a plate element supported by webs on both longitudinal edges is determined as follows:

$$P_{ult} = tb_e F_{cy} \quad \text{. . . C4.7.1(1)}$$

where F_y is the yield stress, b_e is the effective width and t is the thickness of the plate.

Using the average stress approach as is done in Clause 4.7.2, the load capacity of the plate can be determined as follows:

$$P_{ult} = tbn_y F_c \quad \text{. . . C4.7.1(2)}$$

where

b = plate width

n_y = factor of safety

F_c = allowable stress

Setting Equations C4.7.1(1) and (2) equal, the following expression for the effective width at ultimate load is obtained:

$$b_e = b\frac{n_y F_c}{F_{cy}} \quad \text{. . . C4.7.1(3)}$$

The effective width according to the effective width equations in Clause 4.7.6 may be written as—

$$b_e = b\sqrt{\frac{F_{cr}}{F_{cy}}} \quad \text{. . . C4.7.1(4)}$$

where F_{cr} is the plate buckling stress.

Setting Equations C4.7.1(3) and (4) equal, the following expressing for F_{cr} is obtained:

$$F_{cr} = \frac{(n_y F_c)^2}{F_{cy}} \quad \text{. . . C4.7.1(5)}$$

Equation C4.7.1(5) is the equation used for the case of Clause 3.4.19.

For cases where post buckling strength is used, the allowable compressive stresses given may result in visible local buckling, even though an adequate margin of safety is provided against ultimate failure.

In applications where any appearance of buckling must be avoided, the stresses for thin sections should not exceed the value of F_{cr} given divided by 1.2. The factor 1.2 is based on experience.

C4.7.2 Weighted average allowable compressive stress It has been found that the ultimate compressive strength of a member consisting of a number of thin, buckled elements can be estimated by simply adding up the ultimate or buckling strengths of the individual elements (Ref. 40). Tests show that the same concept can be applied to the compression flanges of formed sheet beams (Ref. 21).

Weighted average allowable compressive stresses for beam flanges may be calculated in the same way, where the allowable stresses for individual elements are determined from Clauses 3.4.16 to 3.4.23.

The weighted average allowable compressive stress for a trapezoidal formed sheet beam, calculated according to this Clause, is—

$$F_{ba} = \frac{F_{bf} + F_{bh}\left(\frac{h}{3b}\right)}{1 + \frac{h}{3b}} \quad \text{. . . C4.7.2(1)}$$

where

F_{ba} = weighted average allowable compressive stress for beam flange

F_{bf} = allowable stress for flange proper based on Clause 3.4.17

F_{bh} = allowable stress for webs based on Clauses 3.4.22 or 3.4.23

h = height of shear web measured along the web as shown in Figure C4.7.2

b = clear width of compression flange

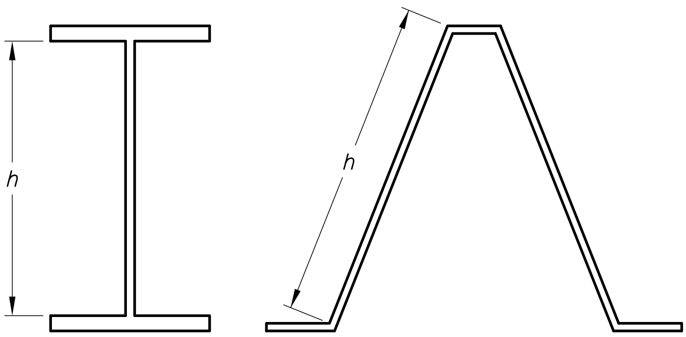


FIGURE C4.7.2 CROSS-SECTIONAL NOTATION

C4.7.3 Weighted average allowable tensile stress The provisions of Clause 4.7.2 may also be applied to the allowable tensile stress in trapezoidal formed sheet beams, if the designer wishes to take full advantage of the strength of the section. In regions of positive bending moment (load applied to concave side of beam—the usual situation at the centre of a span), F_{ba} is the weighted average allowable tensile stress, F_{bf} is determined from Clause 3.4.3 and F_{bh} is given by Clause 3.4.5.

In regions of negative bending moment (for example, at interior supports of multiple span beams), the allowable tensile stress on the tension flange of a formed sheet beam shall not exceed the compressive stress that would be allowed on the same flange if it were in compression.

This provision is required to take account of the effects of flange curling, the tendency of the tension flange to bend toward the neutral axis. It governs design only where the tension flange is wider than the compression flange.

C4.7.4 Effect of local buckling on column strength The provisions in this Clause have been put in general terms so that they apply to H-sections as well as box-sections. Clauses 3.4.9 and 3.4.10 take advantage of the post-buckling strength of plate elements, because in general such elements may buckle without causing failure of the member. However, if the local buckling stress of the section is lower than the flexural buckling strength of the column, the reduced stiffness that accompanies local buckling may reduce the allowable column stress as given by Clause 3.4.8. The formula for allowable stress is based on an equation in Reference 41 that has been found to give good agreement with the results of compression tests on H-section and box-section columns incorporating thin elements (Ref. 42).

The local buckling values used in the calculations referenced in Clause 4.7.1 are accurate for shapes such as square boxes and conservative for all other shapes. These values can be quite conservative for sections in which the edge restraint of the elements is much higher than the simply supported cases used.

C4.7.5 Effect of local buckling on beam strength The equations in this Clause have been put into general terms. The provisions of this Clause take into account the effect that the reduced stiffness due to local buckling may have on the lateral buckling strength of single web beams.

The basic relationship that applies to columns has been found to be useful also for beams (Refs 41 and 42). The local buckling values used in the calculations, referenced in Clause C4.7.1, are based on flanges with a simply supported attached edge, and thus can be quite conservative for sections in which the edge restraint is much higher than the simply supported case.

C4.7.6 Effective width for calculation of bending deflection One way to take into account the effect of local buckling on the post-buckling behaviour of structural members is to consider that at stresses above the local buckling stress, only part of the cross-section of the buckled element is effective in carrying load. The formula given here has been found to be generally conservative for aluminium elements (Refs 20 and 21).

As noted in Clause 4.7.1 the allowable compressive stresses may in certain instances result in some local buckling at design loads for very thin sections, even though an adequate margin of safety is provided against ultimate failure. This local buckling may result in increased deflections for sections with plate elements covered by Clauses 3.4.9, 3.4.10, 3.4.16, 3.4.17, 3.4.22, and 3.4.23 with b/t values exceeding 1.65 S_2 where the value of S_2 is obtained for the element in question.

The formulation of Clauses 3.4.10.2 and 3.4.19 is somewhat different and a different criterion is used for deciding when the effective section is to be used.

C4.7.7 Web crippling of flat webs The formulas given in this Clause are based on Reference 43 which is also described in Reference 18. If the edge load is concentrated over a portion of the element length, web crippling needs to be considered. This failure mode is confined to the area of the web under the load. The equation for maximum strength for interior loads is given by Equation 4.7.7(1), and that for end loads is given by Equation 4.7.7(2). The strengths are effectively post-buckled values. Thus thin webs will have lateral displacements at the calculated strengths.

C4.7.8 Combined web crippling and bending for flat webs The formulas given in this Clause are based on Reference 43, which is also described in Reference 18.

C4.8 FATIGUE

C4.8.1 General The provisions of this Clause are modifications of the original fatigue specifications (Ref. 44). The modifications include changes to the fatigue strength curves and the addition of a method to determine life of parts under spectrum loading. The changes are based on recent tests of full-scale welded beams in the United States (Ref. 45) and Europe (Ref. 46).

The analyses consider that the major factors affecting fatigue behaviour are the number of stress cycles, the magnitude of the stress range and the type and location of the member or detail. The fatigue crack will generally grow perpendicular to the plane of maximum stress. This Clause uses a nominal stress range determined by elastic analysis. The effect of stress concentrations are accounted for through the proper selection of fatigue details. Many other factors, including environment, detrimental weld quality, and post-weld mechanical treatment can have an effect, but are not considered within the scope of this document. Special analysis or tests are required for details and conditions not specifically covered by this Clause.

Loads and number of load applications are not covered. If the information exists for structures of other materials, the same values may be used for aluminium structures of the same type. Wind induced vibrations of undamped structures or components can cause large numbers of cycles and high stresses, and thus need to be avoided. Alternatively, vibration dampers may be used to limit wind induced vibrations.

C4.8.2 Constant amplitude loading The equations for allowable stress are based on the 95 percent confidence for 97.7 percent probability of survival. The results of the recent beam tests account for the revision of the previous values. The fatigue limit was assumed to occur at 5×10^6 cycles for each detail. Static strength provisions in other Sections limit the design fatigue strength for low numbers of cycles.

C4.8.3 Variable amplitude loading Real load histories are frequently more complicated than the constant amplitude loading discussed in Clause 4.8.2. Clause 4.8.3 provides a method by which the engineer may design for more random variable amplitude loadings experienced by many structures. The equivalent stress method is based on nominal stress ranges, linear damage accumulation, and no sequencing effects. The designer should also use a standard cycle counting algorithm, such as rainflow counting (Refs 72 and 73) to determine the equivalent stress range.

The equation for the equivalent stress range is derived directly from Miner's rule when the S-N curve is a straight line in log–log space. Miner's rule is given by—

$$\sum \frac{n_i}{N_i} \leq 1.0$$

where

n_i = number of cycles of the i^{th} stress range

N_i = number of cycles constituting failure at the i^{th} stress range

The equation states that when this fraction approaches unity, some of the details within the group have begun to fail. The engineer may wish to use the Miner's rule formulation over the equivalent stress range when assessing the remaining life of an existing structure or when fatigue data is not linear in the log(stress)–log(life) space.

The analysis is made as specified in Clause 4.8.2 except that the fatigue limit is not used. In this case, the equations for allowable stress are also used for a number of cycles greater than 5×10^6 because available data for spectrum loads show continuing decrease at long lives.

C4.9 COMPRESSION IN SINGLE WEB BEAMS AND BEAMS HAVING SECTIONS CONTAINING TUBULAR PORTIONS

C4.9.1 General The formulas of Clause 3.4.12 for single-web beams and girders are based on an approximation in which the term L_b/r_y replaces a more complicated expression involving several different properties of the beam cross-section. Because of this approximation, the formulas give very conservative results for certain conditions, namely for values of L_b/r_y exceeding about 50; for load distributions such that the bending moment near the centre of the beam is appreciably less than the maximum bending moment in the beam; or for beams with transverse loads applied to a flange and in a direction away from the shear centre of the beam. If the designer wishes to compute more precise values of allowable compressive stress for these cases, the value of r_y in Clause 3.4.12 may be replaced by an 'effective r_y' denoted r_{ye} given by one of the formulas of Clause 4.9.

For doubly symmetric sections one has the choice of using either Clause 4.9.2 or Clause 4.9.4. The equations of the latter Clause are more accurate and, in general, they yield higher design stresses. For singly-symmetric sections unsymmetric about the bending axis Clause 4.9.3 or Clause 4.9.4 may be used. The latter Clause is the more accurate of the two.

The increase in lateral buckling capacity due to moment variation over the unbraced length L_b is considered using the factor C_b given in Clause 4.9.5. These values of C_b can also be used in Clauses 3.4.14 and 3.4.15.

This Clause 4.9 also recognizes the possibility of taking advantage of the effect of bracing the tension flange using a method of rational analysis. An example of a rational analysis is given in Reference 47. In this reference an expression for the elastic critical moment M_e for a singly symmetric I-section with the tension flange prevented from lateral displacement but free to rotate is derived. For pure bending M_e is—

$$M_e = \frac{EI_c d\pi^2}{L_b^2} + \frac{GJ}{d} \quad \text{. . . C4.9(1)}$$

r_{ye} can be evaluated for this case using this M_e in Equation 4.9.4(1). Equation C4.9(1), which was derived for uniform moment, is conservative for the case of uniform loading.

In the above equation I_c is the moment of inertia of the compression flange about the web, d, L_b, and J are as defined in Clause 4.9.2.

C4.9.2 Doubly symmetric sections and sections symmetric about the bending axis Allowable stresses are determined at the ends or at the brace points of beams as well as between brace points. At brace or support points of a doubly symmetric beam Equation 4.9.2(1) is to be used to calculate the allowable stress. The same equation is to be used between brace points if the beam is subjected to lateral loads that are applied only at the shear centre of the section. Equation 4.9.2(2) is used to calculate the allowable stress between brace or support points when a transverse load is applied to the top or bottom flange of the beam and the load is free to move laterally with the beam if it should buckle.

The selection of the proper equation for r_{ye} can be illustrated using Figure C4.9.2 At point B, for both beams, Equation 4.9.2(1) is to be used. The same equation is also to be used for point A if the distributed load is applied at the level of the neutral axis. If the distributed load is not applied at the level of the neutral axis then Equation 4.9.2(2) is to be used. The approach for checking the moment at point C will be discussed in connection with the selection of C_b in Clause 4.9.4.

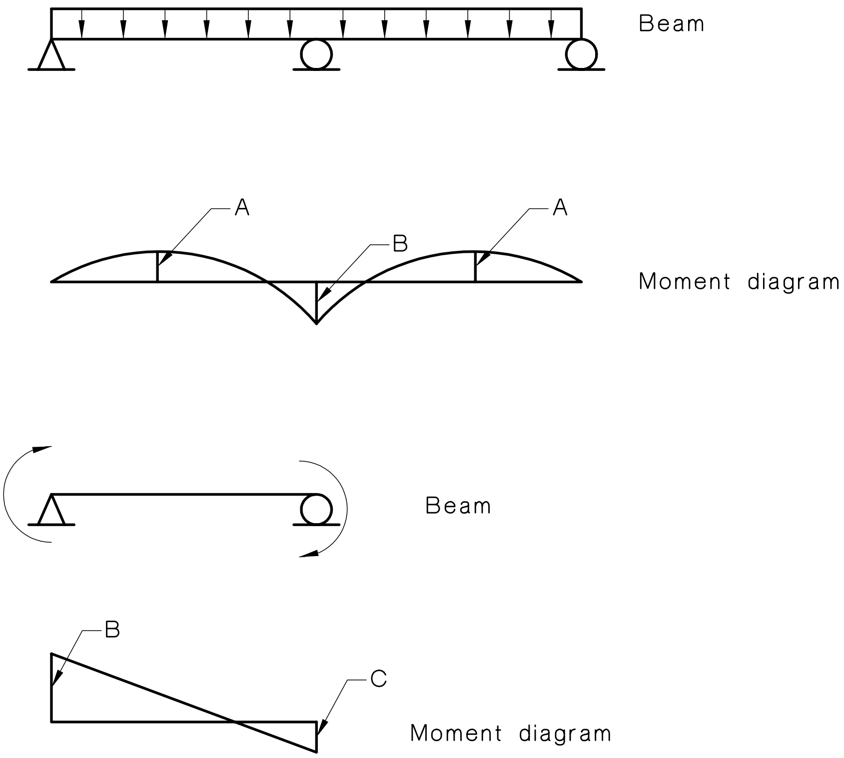


FIGURE C4.9.2 BEAM AND MOMENT DIAGRAM EXAMPLES

C4.9.3 Singly symmetric sections unsymmetric about the bending axis For beams that are unsymmetrical about the x-axis, r_{ye} in Clause 4.9.2 may be calculated approximately by taking r_y, I_y, S_c and J as though both flanges were the same as the compression flange with the overall depth remaining the same. This approximation is always quite conservative when the smaller flange is in compression. The approximation may be somewhat unconservative when the larger flange is in compression. Any unconservatism inherent in assuming a larger than actual section in the case of larger flange in compression, may or may not be compensated by the conservative nature of the equations of Clause 4.9.2.

C4.9.4 Singly symmetric sections symmetric or unsymmetric about the bending axis, doubly symmetric sections and sections without an axis of symmetry This Clause is applicable to any beam bent about the strong axis by moments or by lateral loads applied through the shear centre of the section. Equation 4.9.4(2) is derived in Reference 26 based on the elastic torsional-flexural buckling theory. This expression considers non-symmetry of the section about the bending axis as well as the location of the laterally applied load with respect to the shear centre.

In calculating the section properties as well as the parameter g, it is essential to use the axis orientation specified. The orientation of the axes and the cross-sectional notation are illustrated in Figure C4.9.4(A).

The magnitudes of y_o, torsion constant J and the warping constant C_w can be determined from the expressions given in references such as Reference 48.

The approximate formula for j given in Equation 4.9.4(3) as well as the approach for reverse curvature bending is based on information given by Reference 49. For cases when the areas of the compression and tension flanges are approximately equal, j can also be approximated by $-y_o$.

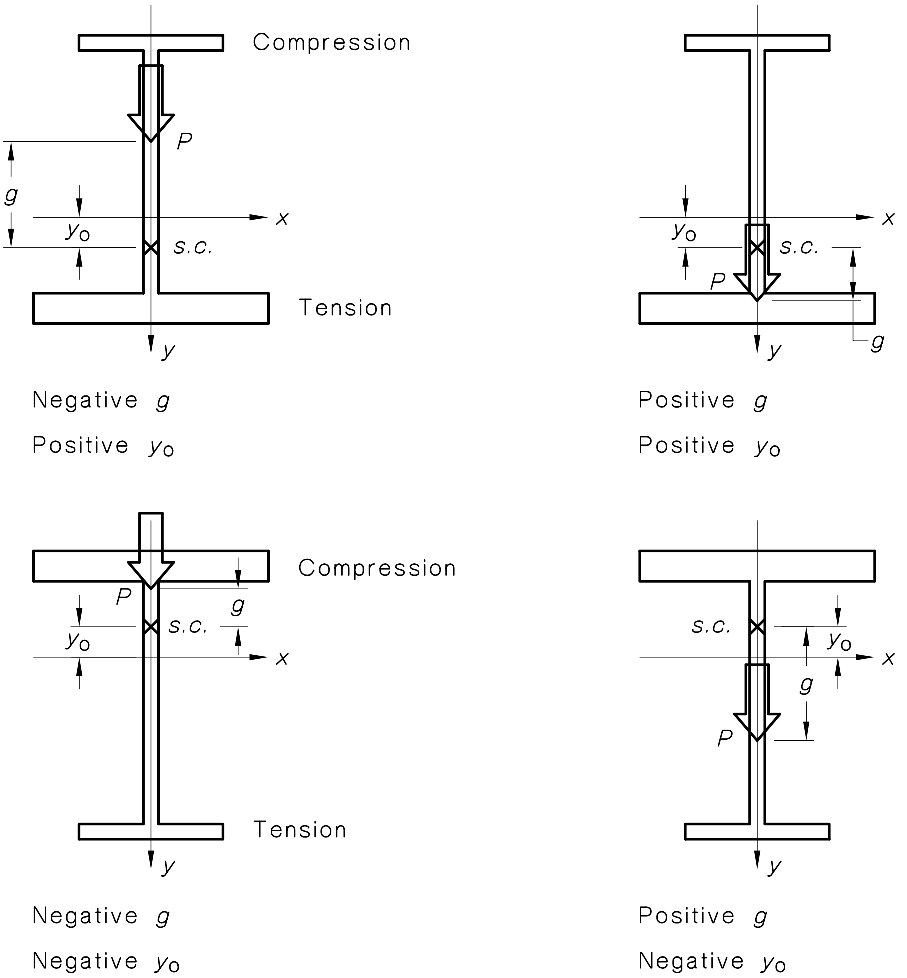


FIGURE C4.9.4(A) ORIENTATION OF THE AXES AND CROSS-SECTIONAL NOTATION

In Clause 4.9, the approach for considering the effect of moment variation over the unbraced length involved modifying the lateral buckling moment for single curvature with equal end moments through the use of coefficients C_b, C_1 and C_2. Clause 4.9.5 gives values of C_b, C_1 and C_2 for various cases.

A somewhat different form of the equation for C_b (Equation 4.9.5.2(1)) was originally proposed by Prof. M Horne. It was later modified by Prof. D. Nethercot. The equation in the form given here is the same as in Reference 50.

The expressions for C_b, C_1 and C_2 for the special cases are based on the work reported in Reference 51. The C_b expressions are somewhat simplified versions of the ones given in the reference.

Application of the C_b factor to singly symmetric sections in the same manner as for doubly symmetric sections has been shown to be unconservative in certain situations by Reference 49. The unconservative cases arise if the C_b factor is applied to the critical moment determined for the case of larger flange in compression, M_L, when it is possible that somewhere in the unbraced segment the smaller flange may be subject to compression. In such cases the proper C_b factor should also be applied to the critical moment determined for the case of smaller flange in compression, M_S.

The application of the coefficients C_b, C_1 and C_2 can be discussed with the help of examples given in Figures C4.9.2 and C4.9.4.(B). In the single span beam of Figure C4.9.2, if the top flange is the smaller flange and M_{MAX} occurs at a section (point B) with the smaller flange in compression, the application of the C_b factor M_S would be used in determining the critical moment.

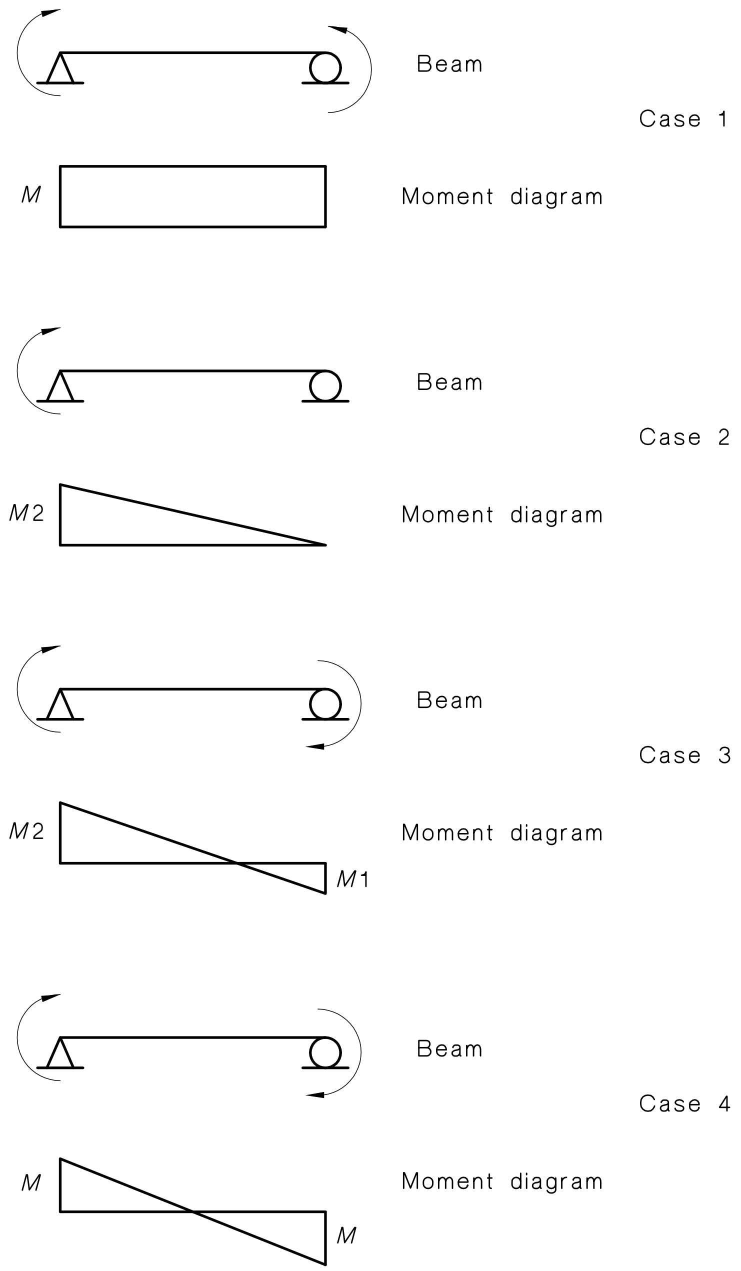


FIGURE C4.9.4.(B) BEAM AND MOMENT DIAGRAM EXAMPLES

If the top flange is the larger flange of the single span beam in Figure C4.9.2, and M_{MAX} occurs at a section with the large flange in compression (at point B), then determining the critical moment as $C_b M_L$ may be unconservative because the presence of a segment with a smaller flange in compression could lead to a lower actual critical moment. A lower bound to the lateral buckling moment at the end with the smaller flange in compression (point C) can be found assuming the moment gradient in the beam to be as shown in Case 2 of Figure C4.9.4.(B) and using the corresponding value of C_b.

The application of the coefficients C_b, C_1 and C_2 to end moment cases can be demonstrated for the four beams shown in Figure C4.9.4.(B). If the top flange is the smaller flange, the C_b factor can be applied to M_S conservatively in each case. The resulting lateral buckling moments are required to be larger than the actual applied maximum moments.

If the top flange is the larger flange, the C_b factor cannot be applied to M_L conservatively in Case 3 without checking to see if a lower lateral buckling moment is possible, due to the fact that over a portion of the beam the smaller flange is in compression. A lower bound to the buckling moment for the case with the smaller flange in compression over a portion of the span can be found by assuming that the smaller flange is subjected to a moment distribution as shown for Case 2 with the small flange in compression, namely $C_b = 1.67$.

For Case 4 where the end moments are equal and opposite, only the smaller flange at the right end needs to be checked. For this check $C_b = 2.27$ according to Equation 4.9.5.2(1).

In summary, C_b can be determined as usual for all cases except when M_{MAX} produces compression on the larger flange and the smaller flange is also subjected to compression in the unbraced length. In this case, the member need also be checked at the location where the smaller flange is subjected to its maximum compression.

It should be noted that if one of the two flanges is small such that I_{cy}/I_y is less than or equal to 0.1 or greater than or equal to 0.9 then C_b shall be taken as 1.0 based on the information given in Reference 49. C_b is also to be taken as 1.0 when the rotational restraint is considered ($k_y < 1$) since Equation 4.9.5.2(1) overestimates C_b when k_y less than 1 is used.

For continuous beams there are no directly derived values of C_1 and C_2. For this reason rational analysis has to be used in estimating the values of these coefficients for such applications. It can be shown that for loading as shown in Figure C4.9.4(A), reasonably conservative results can be obtained by taking—

> $C_1 = 0.41\ C_b$ and $C_2 = 0.47\ C_b$ when the smaller (top) flange is in compression (shown in the top two cases of Figure C4.9.4(A)); and
>
> $C_1 = 0$ and $C_2 = 0$ when the larger (top) flange is in compression (shown in the bottom two cases of Figure C4.9.4(A)).

Alternatively, for the case of continuous beams finite element programs that are shown to be correct for those cases covered in this Clause may be used.

Extensive provisions for cantilevers are not given in this Standard. This is due to the complexity of the subject, particularly for singly symmetric sections. Guidance for the design of such members can be found in References 52, 53, 54 and 55.

C4.10 COMPRESSION IN ELASTICALLY SUPPORTED FLANGES The formula may be used for determining the allowable stress at the centroid of the compression flange of a beam that has lateral stays only at the tension flange where the stays are intermittent, such as stringers, girts, or purlins. This type of analysis is described in Reference 56. If the rotational stiffness of the joint between the stringer and the tension flange is not known, it should be measured experimentally and introduced in the equation for β (Ref. 57).

SECTION C5 MECHANICAL CONNECTIONS

INTRODUCTION The provisions in Clauses 5.1 and 5.2 follow conventional, well established practice for riveted and bolted joints.

C5.1 BOLTED AND RIVETED CONNECTIONS

C5.1.1 General For mechanical connections of pre-painted aluminium structures see paragraph C6.6.1.

C5.1.2 Allowable loads In Clauses 5.1.2.2 and 5.1.2.3 tension and shear calculations are listed separately since the allowable tension cannot be readily calculated for rivets.

In Clause 5.1.2.4, the allowable bearing stress depends on the ratio of edge distance to rivet or bolt diameter where edge distance is the distance from the centre of the rivet or bolt to the edge of the load carrying member toward which the pressure of the rivet or bolt is directed (see Clause 3.4.6).

The use of allowable fastener shear or tension values based on the ultimate strength only, may provide a factor of safety for the fastener material yielding at less than 1.65 for buildings or 1.85 for bridges. The designer may want to investigate joint distortions for undesirable effects on the structure, or perhaps to specify stronger fasteners.

Many useful, proprietary mechanical fasteners made of various combinations of aluminium, stainless steel, plastics, etc. are used in aluminium structures.

Values for minimum strengths of common alloys and hardnesses of other materials approved for use in aluminium structures in Clause 5.1.17, as well as factors of safety for tension and shear on fasteners of these materials may be found in the following documents:

(a) For galvanized, cadmium plated or aluminized steel fasteners—*AISC Specification for structural steel*, *Allowable stress design*, latest edition.

(b) For 300 series stainless steel fasteners—ANSI/ASCE 8-90, *Specification for the design of cold-formed stainless steel structural members*.

(c) For carbon steel or stainless steel fasteners—AAMA TIR-A9, *Metal curtain wall fasteners*.

C5.1.7 Net section Figures C5.1.7(A) and 5.1.7(B) illustrate the notation of this Clause. The net section area for the strap shown in Figure C5.1.7(A) is—

$$A_{net} = \left(b - 2d + \frac{s^2}{4g}\right) t \qquad \text{. . . C5.1.7(1)}$$

where t is the thickness of the strap and d is the diameter of the hole.

In Figure C5.1.7(B), the angle section is flattened out into a strap for the purpose of calculating the net section. The flattened width is $a + b - t$.

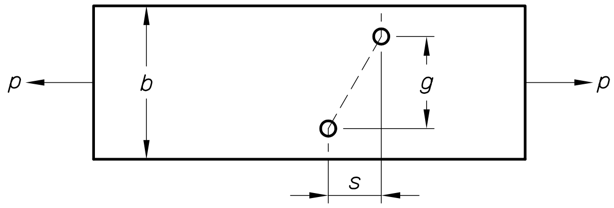


FIGURE C5.1.7(A) STRAP IN TENSION

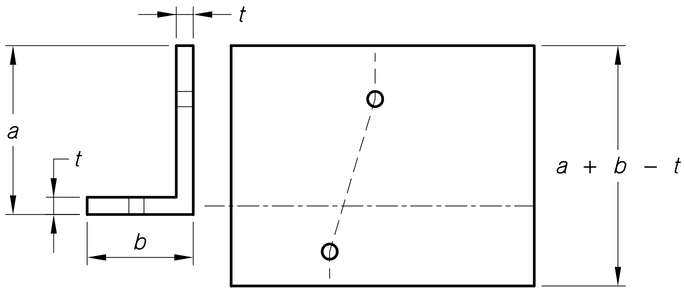


FIGURE C5.1.7(B) ANGLE IN TENSION

C5.2 METAL STITCHING STAPLES Metal stitching staples find an increasing application in joining of light gauge structures. In view of the wide variance of joint strength obtainable with these fastening methods, no standard or uniform allowable loads are recommended.

C5.3 SELF TAPPING SCREW CONNECTIONS

C5.3.1 Notation Results of over 3500 tests on light-gauge steel and aluminium connections worldwide were analyzed to formulate screw connection provisions (Ref. 58). European Recommendations (Ref. 59) and British Standards (Ref. 60) were considered and modified as appropriate. These provisions are intended to be used when a sufficient number of test results is not available for the particular application. A higher degree of accuracy can be obtained by testing any particular application.

Proper installation of screws is important to achieve satisfactory performance. Power tools with adjustable torque controls and driving depth limitations are usually used.

Screw connection tests used to formulate the provisions included single fastener specimens as well as multiple fastener specimens. However, it is recommended that at least two screws should be used to connect individual elements. This provides redundancy against under torquing, over torquing, etc., and limits lap shear connection distortion of flat unformed members such as straps.

C5.3.2 Shear

C5.3.2.1 *General* Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pull-out of the screw and bearing of the joined materials.

Tilting of the screw followed by threads tearing out of the lower sheet reduces the connection shear capacity from that of the typical connection bearing strength.

Diameter and rigidity of the fastener head assembly as well as sheet thickness and tensile strength have a significant effect on the shear failure load of a connection. There are a variety of washers and head styles in use. Washers must be at least 1.25 mm thick to withstand bending forces with little or no deformation.

C5.3.2.2 *Connection shear* Equation 5.3.2.2(2) covers the cases when the screw tilting can lower the strength.

C5.3.3.3 *Pull-over* The pull-over strength equation is based on Reference 18.

SECTION C6 FABRICATION

INTRODUCTION The provisions of this Section are very similar to those in the suggested specifications published by the American Society of Civil Engineers (Refs 4, 5 and 6).

C6.6 PAINTING

C6.6.1 General Mechanical connections of pre-painted aluminium structural members may be affected as follows:

(a) *Connecting by bolts* The total film build between the two painted aluminium members may affect the torque retention of the connecting bolts. The retention torque loss may lead to loose connection and subsequently premature fatigue failure of bolts and aluminium members in the localized join area.

In addition, the corrosion resistance of the localized bolting area may be reduced.

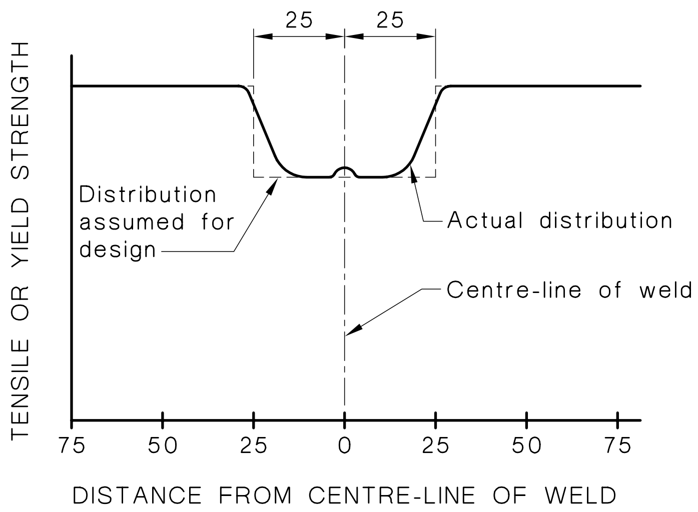
(b) *Connecting by rivets* The surface roughness and film build of the pre-painted aluminium underneath the rivet head may deteriorate due to long-term fretting, in particular when connecting aluminium members are undergoing either intermittent or constant service loading.

The overall result is that loose connection may occur and lead to premature fatigue failure of both rivet and aluminium members. In addition, the corrosion resistance in the localized rivetted area may be reduced.

SECTION C7 WELDED CONSTRUCTION

INTRODUCTION Most of the structural aluminium alloys attain their strength by heat treatment or strain hardening. Welding causes local annealing which produces a zone of lower strength along both sides of the weld bead. The resulting variation in mechanical properties in the vicinity of a weld is illustrated by the typical distribution of yield strength in Figure C7. When designing welded load carrying members this decrease in strength shall be taken into consideration in addition to the design rules as outlined in Section 3.

The effect of heat of welding on mechanical properties of aluminium has been discussed extensively (Refs 61, 62, 63 and 64). For the non heat-treatable alloys, the strength in the heat-affected zone after welding is essentially that of annealed material. The strength of welds in heat-treated alloys, such as 6061-T6, is part way between the annealed strength and that of the original heat-treated material.



DIMENSIONS IN MILLIMETRES

FIGURE C7 DISTRIBUTION OF MECHANICAL PROPERTIES NEAR A WELD

C7.1 ALLOWABLE STRESSES FOR WELDED MEMBERS In determining allowable tensile stresses, the factor of safety on ultimate strength is divided into a quantity equal to 90 percent of the ASME weld qualification test value for ultimate strength across a butt weld. The 90 percent factor was used to take account of the fact that for welds which receive only visual inspection, the minimum strength value based on the weld qualification requirements can probably not be considered to have the same reliability as the minimum mechanical properties of the parent metal (Ref. 65).

The minimum strength values for fillet welds listed in Tables 7.2(A) and 7.2(B) are based on extensive tests (Refs 66 and 67). In obtaining allowable stresses based on these strength values, the usual factor of safety on ultimate strength was increased by 20 percent to account for the difference between the average shear strength developed in the small, standard fillet weld specimen used to establish the strength values and the average strength that may be developed in practical fillet welded joints, which can involve long welds and complex geometry (Ref. 64).

The yield strength across a butt weld for design purposes was considered to be the value corresponding to an offset of 0.2 percent from the initial modulus line measured on a gauge length of 250 mm. The total permanent set across the joint at this value of yield strength (0.50 mm) is about equivalent to the amount of permanent set measured in typical riveted or bolted joints when the stress on the net area equals the yield strength (Ref. 68).

In general, welds have little effect on buckling strength except in the range of slenderness ratios where the allowable stress is controlled by the welded yield strength (Ref. 69). Parent metal values of the buckling formula constants are then used for welded members. An exception is the case of welded tubes, Clauses 3.4.11, 3.4.13, and 3.4.18 for which the buckling formula coefficients are determined from the formulas in Table 3.3(C), using the 250 mm, gauge length compressive yield strength F_{cyw} from Table 3.3(B). Buckling tests on welded tubes have shown this procedure to be conservative (Ref. 7). Another exception is a column with welds at locations other than the ends (or a cantilever column with a weld at the end).

Compressive tests on welded aluminium plates (Refs 63 and 70) have demonstrated that the welds have little effect on post-buckling strength.

C7.2 FILLER WIRE The amount of loss of properties depends to a large extent on the alloy and temper of the components involved. The filler wire used also influences the strength of the weld.

C7.3 MEMBERS WITH LONGITUDINAL WELDS The equation in this Clause is based on the fact that the strength of a cross-section with only part of the area in the heat-affected zone can be estimated by adding up the strength of the softened material in the heat-affected zone and the unaffected material outside this zone (Refs 63 and 68). The provisions apply to tension members and columns.

The version in the present clause is the same as that in Reference 19 except for the last paragraph and the clarified definition of A_w.

The allowable stress for the cross-section assuming all reduced strength material is based upon the tensile strength of the as-welded material when calculating tensile strength, but 0.75 times the as-welded yield strength for the calculation of yield strength. The 0.75 factor on yield strength is an approximate correction factor to convert 250 mm gauge length yield strengths (yield strength across a groove weld) as provided in the Standard to a 50 mm gauge length needed for material in the heat-affected zone (Ref. 18).

For calculating the column buckling strength of the heat-affected material the constants given in Table 3.3(C) are used for all alloys and tempers because they best represent the heat-affected material (Ref. 18).

C7.4 MEMBERS WITH TRANSVERSE WELDS If a column is simply supported at both ends, welds at the ends have little effect on the buckling strength, except in the range of slenderness ratios where the allowable stress is controlled by the welded yield strength. However, if a weld is in a location where it can materially affect the bending strength of the column as it buckles, for example at the centre of a column supported on both ends or at the fixed end of a cantilever column, it may have appreciable effect on the buckling strength. For these cases the strength is calculated as though the entire column has the welded strength as given by the compressive yield strength. This procedure is conservative (Ref. 18) and is generally accurate.

SECTION C8 TESTING

C8.2 TEST REQUIREMENTS A load/deformation curve will serve not only as a check against observational errors, but also to indicate any irregularities in the behaviour of the structure under load. It is desirable that a minimum of 6 points, excluding the zero load point, be obtained to define the shape of the load/deformation curve if the curve is predominantly linear, and a minimum of 10 points if the curve is significantly non-linear.

C8.4 PROCEDURE For a proof load test on a structure to be successful, it is necessary that the structure does not collapse and also that it does not incur permanent structural damage. Suitable methods for detecting the onset of damage vary from one material to another, and include such techniques as the measurement of crack widths and acoustic emissions. One commonly used method is the measurement of recovery of the deformation on unloading the structure. A recovery value of 85 percent is recommended by Reference 74.

For the serviceability test, it is suggested that a 95 percent recovery of deformation after removal of the test load will ensure that the specimen was substantially elastic at the test load.

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