

AS/NZS 1664.2
Supplement 1:1997

Aluminium structures

**Part 2: Allowable stress design—
Commentary**

**(Supplement 1 to
AS/NZS 1664.2:1997)**

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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 C 1 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.

S E C T I O N C 2 D E S I G N P R O C E D U R E

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—

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

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_r .

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:

- Determine the elastic torsional-flexural buckling stress according to the torsional-flexural theory.
- Determine the equivalent slenderness ratio using Equation 3.4.8.3(1).
- 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).