For eccentric compression with the bending about the axis of symmetry, the response of the column involves biaxial bending and twisting even if the column contains no geometric imperfections. The approximate design approach adopted in AS/NZS 4600 uses the conventional beam-column interaction formulae given in Section 8.2, with the buckling stress for concentric axial compression of a monosymmetric section based on Eq. (7.10) and the buckling stress for pure bending about the axis of symmetry based on Eq. (5.11). In AS/NZS 4600, Eq. (5.11) has been rearranged in terms of the elastic buckling stresses (f_{oy}, f_{oz}) to be:

$$M_{ob} = A\, r_{o1} \sqrt{f_{oy} f_{oz}} \tag{8.9}$$

Eq. (8.9) is the same as Eq. (3.3.3.2(8)) of AS/NZS 4600 except that $C_b = 1.0$ in Eq. (3.3.3.2(8)) when computing M_o for use in the combined bending and compression interaction equations.

The experimental justification of this method is given in Refs 8.5 and 8.6. This justification includes the use of the linear interaction formulae (Eqs (8.1) – (8.3)) for sections with slender elements.

8.4 Combined Axial Tensile Load and Bending

The design rules for members subject to combined axial tensile load and bending are new, having been added for the first time to the most recent edition of the AISI Specification (Ref. 1.14) and also included for the first time in Australia in AS/NZS 4600:1996. The design rules consist of two checks. These are a lateral buckling check as given by Eq. (8.10), and a section capacity in tension check given by Eq. (8.11).

$$\frac{M_x^*}{\phi_b M_{bx}} + \frac{M_y^*}{\phi_b M_{by}} - \frac{N^*}{\phi_t N_t} \le 1.0 \tag{8.10}$$

$$\frac{N^*}{\phi_t N_t} + \frac{M_x^*}{\phi_b M_{sxf}} + \frac{M_y^*}{\phi_b M_{syf}} \le 1.0 \tag{8.11}$$

where M_{bx}, M_{by} = nominal member moment capacity about the *x*- and *y*-axes respectively of the effective section as given in Clause 3.3.3

M_{sxf}, M_{syf} = nominal section yield moment capacity of the full section about the *x*- and *y*-axes respectively and equals the section modulus of the full section for the extreme tension fibre multiplied the yield stress

The nominal section capacity of a member in tension (N_t) is specified in Clause 3.2 of AS/NZS 4600 as the lesser of:

$$N_t = A_g f_y \tag{8.12}$$

and

$$N_t = 0.85\ k_t A_n f_u \tag{8.13}$$

where A_g = gross area of the cross-section, and

A_n = net area of the cross-section

The term (k_t) allows for connections which do not provide uniform force distribution and is described fully in Clause 3.2 of AS/NZS 4600.

When bending is combined with axial tension, the effect of bending on the tension section capacity is accounted for by the interaction in Eq. (8.11) which lowers the design axial tensile force when bending is included. The bending terms in Eq. (8.11) are based on yield in the extreme tension fibre of the section and therefore use the section modulus of the full unreduced section based on the extreme tension fibre. The effect of bending on the lateral buckling capacity is accounted for by Eq.

(8.10) which increases the design bending moment by subtracting the tension term from the bending terms. Care must be taken with terms of this type to ensure that a design situation cannot occur where the tension may go to zero although the bending moment remains non-zero. If this were to occur, then Eq. (8.8) would be unconservative.

8.5 Examples

8.5.1 Unlipped Channel Section Beam-Column Bent in Plane of Symmetry

Problem

Calculate the maximum design axial compressive load in the channel shown in Fig. 7.11 assuming that the channel is loaded with an axial force on the line of the *x*-axis at a point in line with the flange tips. The following clause numbers refer to AS/NZS 4600.

A. Monosymmetry Parameter (β_y)

From Fig. 8.5 and Example 7.6.2A

$a = 148.8$ mm $b = 48.4$ mm $t = 3.2$ mm

$$\bar{x} = \frac{b^2}{a+2b} = 9.538 \text{ mm} \quad \text{(Table E1)}$$

$$x_{os} = \bar{x} + \frac{3b^2}{a+6b} = 25.54 \text{ mm} \quad \text{(Table E1)}$$

$$I_{ys} = \frac{2b^3t}{12} + 2bt\left(\frac{b}{2}-\bar{x}\right)^2 + at\bar{x}^2 = 1.704 \times 10^5 \text{ mm}^4$$

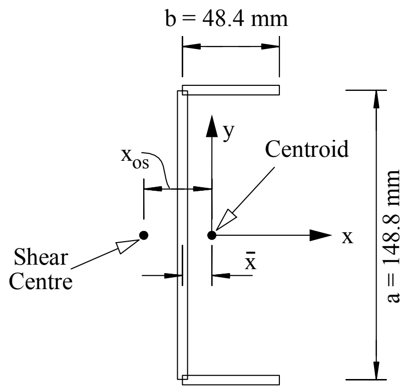

Fig. 8.5 Idealised section ignoring corner radii

Note that the distances (x_{os} and $\bar{x}$) in Fig. 8.5 are in the negative *x*-direction. The formulae in Appendix E2 give magnitudes only so that

$\bar{x} = -\bar{x} = -9.538$ mm $\qquad x_{os} = -x_{os} = -25.539$ mm

$$\beta_w = \frac{t\bar{x}a^3}{12} + t\bar{x}^3a = -8.793 \times 10^6 \text{ mm}^5 \quad \text{(Table E1)}$$

$$\beta_f = \frac{1}{2}t\left[\left(b+\bar{x}\right)^4 - \bar{x}^4\right] + \frac{1}{4}a^2t\left[\left(b+\bar{x}\right)^2 - \bar{x}^2\right] = 2.878 \times 10^7 \text{ mm}^5 \quad \text{(Table E1)}$$

Design of Cold-Formed Steel Structures
(To Australian/New Zealand Standard
AS/NZS 4600:2005)

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