CHAPTER 7 COMPRESSION MEMBERS

7.1 General

The design of cold-formed members in compression is generally more complex than conventional hot-rolled steel members. This is a result of the additional modes of buckling deformation which commonly occur in thin-walled structural members. These are:

(a) local buckling and post-local buckling of stiffened and unstiffened compression elements, as described in Chapter 4,

(b) flexural, torsional and flexural-torsional modes of buckling of the whole compression member as shown in Fig. 1.17(b), and

(c) distortional buckling as shown in Fig. 1.18(a).

The design for (a) determines the nominal section compression capacity (N_s) described in Section 7.3 of this book. The design for (b) and (c) gives the nominal member compression capacity (N_c) given in Section 7.4 of this book. As specified in Table 1.6 of AS/NZS 4600, the capacity reduction factor (ϕ_c) for computing the design section or member capacity from N_s or N_c is 0.85.

The elastic buckling stress for flexural, torsional and flexural-torsional buckling of members in compression is discussed in Section 7.2.1 of this book. The elastic distortional buckling stress of members in compression was discussed in Section 5.3.1.1 of this book.

Section 3.4 of AS/NZS 4600 includes rules which enable the designer to account for these additional buckling modes as well as the normal failure modes of flexural buckling and yielding commonly considered for doubly-symmetric hot-rolled compression members.

7.2 Elastic Member Buckling

7.2.1 Flexural, Torsional and Flexural-Torsional Buckling

The elastic critical load for flexural, torsional and flexural-torsional buckling of a thin-walled member of general cross-section, as shown in Fig. 7.1(a), was derived by Timoshenko and is explained in detail in Timoshenko and Gere (Ref. 3.6) and also by Trahair in Ref. 5.2. For a member subjected to uniform compression and restrained at its ends by simple supports which prevent lateral displacement of the section perpendicular to its longitudinal axis, as well as twisting rotation, the elastic critical load (N_{oc}) is given by solution of the following three simultaneous equations in the displacement amplitudes A_1, A_2 in the x-, y-directions respectively and twist angle A_3 as follows:

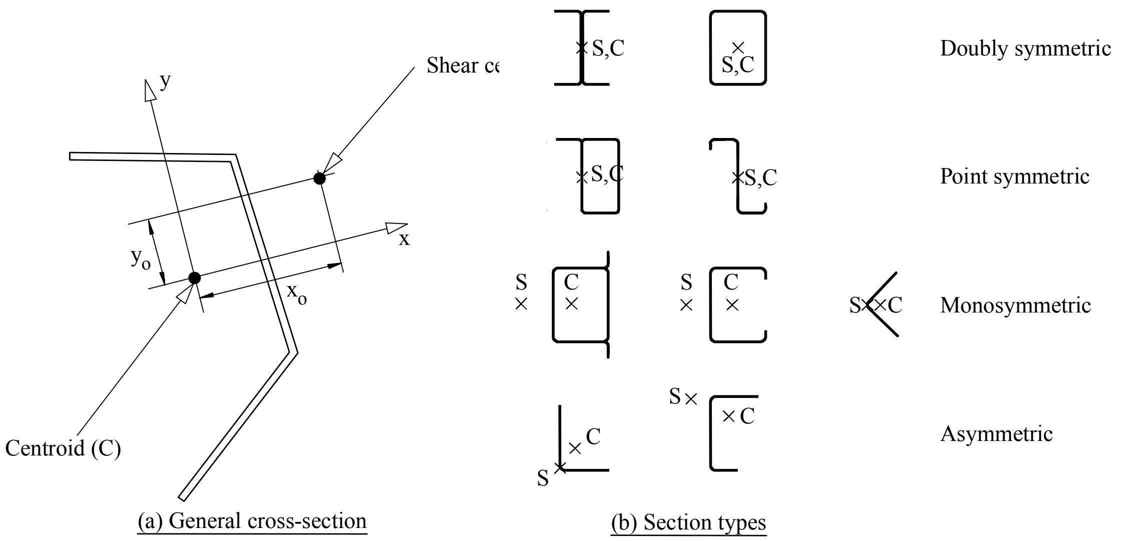

Fig. 7.1 Thin-walled sections

$$(N_{oy} - N_{oc})A_1 - N_{oc}\, y_o\, A_3 \quad = \quad 0 \qquad (7.1)$$

$$(N_{ox} - N_{oc})A_2 + N_{oc}\, x_o\, A_3 \quad = \quad 0 \qquad (7.2)$$

$$-N_{oc}\, y_o\, A_1 + N_{oc}\, x_o\, A_2 + (r_{o1})^2 (N_{oz} - N_{oc})A_3 \quad = \quad 0 \qquad (7.3)$$

where

$$N_{ox} = \pi^2 \frac{EI_x}{l^2} \qquad (7.4)$$

$$N_{oy} = \pi^2 \frac{EI_y}{l^2} \qquad (7.5)$$

$$N_{oz} = \frac{GJ}{r_{o1}^2}\left(1 + \frac{\pi^2 EI_w}{GJl^2}\right) \qquad (7.6)$$

$$r_{o1}^2 = \frac{(I_x + I_y)}{A} + x_o^2 + y_o^2 \qquad (7.7)$$

The terms x_o and y_o are the shear centre coordinates shown in Fig. 7.1(a). The modes of buckling for flexure about both principal axes, and twist angle are sinusoidal with amplitudes of A_1, A_2 and A_3 respectively.

Two solutions to Eqs (7.1) - (7.3) exist. Either the column does not buckle and $A_1 = A_2 = A_3 = 0$, or the column buckles and the determinant of the matrix of coefficients of A_1, A_2 and A_3 in Eqs (7.1) - (7.3) is zero. In this case, the resulting cubic equation to be solved for the critical load (N_{oc}) is:

$$N_{oc}^3\left(r_{o1}^2 - x_o^2 - y_o^2\right) - N_{oc}^2\left[\left(N_{ox} + N_{oy} + N_{oz}\right)r_{o1}^2 - \left(N_{oy}x_o^2 + N_{ox}y_o^2\right)\right]$$
$$+ N_{oc}r_{o1}^2\left(N_{ox}N_{oy} + N_{oy}N_{oz} + N_{oz}N_{ox}\right) - N_{ox}N_{oy}N_{oz}r_{o1}^2 = 0 \qquad (7.8)$$

For general asymmetric sections of the type shown in Fig. 7.1(b), the solution of Eq. (7.8) in its general form is necessary. Eq. (7.8) is given in Clause 3.4.5 of AS/NZS 4600 such that it has been divided by the gross sectional area cubed to convert it to a buckling stress (f_{oc}).

For monosymmetric sections of the type shown in Fig. 7.1(b), for which either x_o or y_o are zero, then simpler solutions exist. In the case of the *x*-axis as the axis of symmetry, then y_o is zero and hence:

$$(N_{oc})_1 = N_{oy} \qquad (7.9)$$

$$(N_{oc})_{2,3} = \frac{(N_{ox} + N_{oz}) \pm \sqrt{(N_{ox} - N_{oz})^2 + 4N_{ox}N_{oz}\left(\frac{x_o}{r_{o1}}\right)^2}}{2\left[1 - \left(\frac{x_o}{r_{o1}}\right)^2\right]} \qquad (7.10)$$

Eq. (7.9) gives the flexural buckling load about the *y*-axis and the smaller of the two values computed using Eq. (7.10) gives the flexural-torsional buckling load. Eqs (7.9) and (7.10), converted to buckling stresses (f_{oc}) by dividing by the gross area (A) form the basis of Clause 3.4.3 of AS/NZS 4600.

Eq. (7.9) for the unlipped channel section in Fig. 3.2 produces identical results with the curve through C in Fig. 3.3 for column lengths greater than 1000 mm where local buckling effects have no influence. Similarly evaluation of Eq. (7.10) for the channel section in Fig. 3.2 produces identical results with the curve through D in Fig. 3.3 for column lengths greater than 1000 mm.

In Figs 3.6, 3.7 and 3.9, the curves shown as the dashed line labelled "Timoshenko flexural-torsional buckling formula" were calculated using the lower root of Eq. (7.10).

For doubly symmetric or point symmetric sections where x_o and y_o are both zero, then the three solutions of Eq. (7.8) are simply

$$(N_{oc})_1 = N_{ox} \tag{7.11}$$

$$(N_{oc})_2 = N_{oy} \tag{7.12}$$

$$(N_{oc})_3 = N_{oz} \tag{7.13}$$

For doubly-symmetric sections, closed cross-sections and any other sections that can be shown not to be subject to torsional or flexural-torsional buckling, Clause 3.4.2 of AS/NZS 4600 uses the lesser N_{oc} derived from Eqs (7.11) and (7.12) divided by the gross area (A) to give the flexural buckling stress (f_{oc}) (Eq. 3.4.2(1) of AS/NZS 4600). For point symmetric sections, where torsional buckling can also occur, Clause 3.4.4 of AS/NZS 4600 uses N_{oc} derived from Eq. 7.13 divided by the gross area (A) to give the torsional buckling stress (f_{oz}). The lesser of f_{oc} from Clause 3.4.2 and f_{oz} from Clause 3.4.4 must be used for the design of point symmetric sections.

In Eqs (7.4) - (7.6), the length (l) is the unbraced length between the simply supported ends of the column. However the length (l) in Eqs (7.4), (7.5) and (7.6) has been replaced by l_{ex}, l_{ey} and l_{ez} respectively in Clause 3.3.3.2.1 of AS/NZS 4600. The lengths l_{ex}, l_{ey} and l_{ez} are the effective lengths about the x-, y- and z-axes respectively as defined in Clause 3.3.3.2.1. The use of different lengths in the computation of the flexural-torsional buckling load by Eqs (7.8) or (7.10) is not theoretically justifiable. However it usually produces a conservative estimate of the flexural-torsional buckling load of columns which have different flexural and torsional effective lengths. This procedure has been justified experimentally for the uprights of steel storage rack columns as described in Ref. 7.1.

In Section 5 of the Australian Steel Storage Racking Standard (Ref. 1.21), effective length factors are specified for flexural buckling in the direction perpendicular to the upright frame (typically $l_{ez} / l = 1.7$ for racking not braced against side sway), for flexural buckling in the plane of the upright frame (typically $l_{ey} / l = 1.0$) and for torsional buckling (typically $l_{ez} / l = 0.8$) provided twisting of the upright is prevented at the brace points.

For the design of studs in the walls of steel framed housing, effective lengths have been suggested in Section 5.3.2.1(ii) of Ref. 7.2. The torsional effective length (l_{ez}) is recommended as *0.8* times the noggin space where bracing is provided at the noggin and the ends according to Clause 4.4(a) of AS/NZS 4600. The effective length for flexure (l_{ex}) is recommended as *0.65* times the height of the stud for flat-ended wall studs restrained from flexure normal to the wall at both ends.

7.2.2 Distortional Buckling

The calculation of the elastic distortional buckling stress (f_{od}) for flange distortional buckling was discussed in detail in Section 5.3.1.1 of this book as an introduction to flange distortional buckling in flexure described in Section 5.3.1.2. Detailed formulations for computing the elastic distortional buckling stress (f_{od}) are given for a general channel in compression in Appendix D1 of AS/NZS 4600, and for a simple lipped channel in compression in Appendix D2 of AS/NZS 4600.

7.3 Section Capacity in Compression

The strength of short length columns compressed between rigid end platens is governed mainly by the yield strength of the material and the slenderness of the plate elements forming the cross-section. For sections with stocky plate elements, the strength is simply equal to the

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(To Australian/New Zealand Standard
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