CHAPTER 10 DIRECT STRENGTH METHOD

10.1 Introduction

The design methods used throughout this book to account for local and distortional buckling of thin-walled members in compression and bending are based on the effective width concept for stiffened and unstiffened elements introduced in Chapter 4. The effective width method is an elemental method since it looks at the elements forming a cross-section in isolation. It was originally proposed by Von Karman (Ref. 4.4) and calibrated for cold-formed members by Winter (Refs 4.5 and 4.6). It was initially intended to account for local buckling but has been extended indirectly to distortional buckling of stiffened elements with an intermediate stiffener in AS/NZS 4500 Clause 2.5 and edge stiffened elements in Clause 2.4. It accounts for post-buckling by using a reduced (effective) plate width at the design stress.

As sections become more complex with additional edge and intermediate stiffeners such as those shown in Figs. 1.1 and 1.2, the computation of the effective widths becomes more complex. Interaction between the elements also occurs so that consideration of the elements in isolation is less accurate. To overcome these problems, a new method has been developed by Schafer and Peköz (Ref. 10.1) and is called the "Direct Strength Method". It uses elastic buckling solutions for the entire member rather than the individual elements, and strength curves for the entire member.

The method had its genesis in the design method for distortional buckling of thin-walled sections developed by Hancock, Kwon and Bernard (Ref. 10.2). This method was incorporated in AS/NZS 4600 Clauses 3.3.3.3 and 3.4.6 and has been used successfully to predict the distortional buckling strength of both flexural and compression members since 1996. However, the Direct Strength Method goes one step further and assumes that local buckling behavior can also be predicted using the elastic local buckling stress of the whole section with an appropriate strength design curve for local instability. The method has the advantage that calculations for complex sections are very simple, as demonstrated in the examples following, provided elastic buckling solutions are available. The AISI has produced a Direct Strength Design Guide (Ref. 1.16)

10.2 Elastic Buckling Solutions

The finite strip method of buckling analysis described in Chapter 3 of this book provides elastic buckling solutions suitable for use with the Direct Strength Method and serves as a useful starting point. For the lipped channels shown in compression in Fig. 3.6 and bending in Fig. 3.12, there are 3 basic buckling modes: These are:

1. Local buckling
2. Flange-distortional buckling
3. Overall buckling

The <u>local mode</u> involves only plate flexure in the buckling mode with the line junctions between adjacent plates remaining straight. It can occur for lipped channels as shown in Figs 3.6 and 3.12 or unlipped channels as shown at Point A in Fig. 3.3. The mode has a strong post-buckling reserve and occurs at short half-wavelengths.

The <u>flange-distortional mode</u> involves membrane bending of the stiffening elements such as the edge stiffeners shown in Figs 3.6 and 3.12. Plate flexure also occurs so that the mode has a moderate post-buckling reserve. It occurs at intermediate half-wavelengths.

The <u>overall mode</u> involves translation of cross-sections of the member without section distortion. It may consist of simple column (Euler) buckling as shown at Point C in Fig. 3.3, torsional-flexural buckling as shown at Point D in Figs 3.3 and 3.6 for columns, or lateral buckling as shown at Point C in Fig. 3.12 for beams. It occurs at longer half-wavelengths and has very little post-buckling reserve. The overall mode may be restrained by bracing or sheathing as shown at Point D in Fig. 3.12. The resulting lateral-distortional mode at longer half-wavelengths is not regarded as a distortional mode in the Direct Strength Method but should be treated as a type of hybrid overall mode.

The Direct Strength Method uses the following solutions. For local buckling, the buckling stress (f_{ol}) is the minimum point for the local mode on a graph of stress versus half-wavelength as shown in Figs 3.3, 3.6 and 3.12. The buckling stress may be replaced by a load for compression or a moment for bending to simplify the calculations. The interaction between the different elements is accounted for so that simple elastic local buckling coefficients such as k = 4 as shown in Table 4.1 for a simple stiffened element in compression no longer apply. Elastic buckling solutions for simple sections of the type given by Bulson (Ref. 4.2) could be used instead of the finite strip method.

For flange-distortional buckling, the buckling stress (f_{od}) is the minimum point for the flange-distortional mode on a graph of buckling stress versus half-wavelength as shown at Point B in Figs 3.6 or 3.12. The buckling stress may be replaced by a load for compression or a moment for bending to simplify the calculations. The interaction between the different elements is automatically accounted for as it should be for such complex modes. Elastic buckling solutions for edge stiffened sections are given for compression members in Lau and Hancock (Ref. 3.8), and for flexural members in Schafer and Peköz (Ref. 10.3) and Hancock (Ref. 10.4), and can be used instead of the finite strip method.

For the overall modes, the elastic buckling stresses (f_{oc}) predicted by the simple formulae in Section 3 of the AS/NZS 4600 are used. The reason for using the AS/NZS 4600 rather than the finite strip analysis is that boundary conditions other than simple supports are not accounted for in the finite strip method. Further, for flexural members, moment gradient cannot be accounted for in the finite strip method. By comparison, the design formulae in the AS/NZS 4600 can easily take account of end boundary conditions using effective length factors, and moment gradient using C_b factors as described in Clause 3.3.3 of AS/NZS 4600 and Chapter 5 of this book.

10.3 Strength Design Curves

10.3.1 Local Buckling

Local buckling direct strength curves for individual elements have already been discussed and were included in Fig. 4.5 for stiffened compression elements and Fig. 4.6 for unstiffened compression elements. The limiting stress on the full plate element has been called the effective design stress in these figures. The concept is that at plate failure, either the effective width can be taken to be at yield or the full width can be taken to be at the effective design stress. This concept can be generalized for sections so that a limiting stress on the gross section, either in compression or bending, can be defined for the local buckling limit state. The resulting method is the Direct Strength Method. The research of Schafer and Peköz (Ref. 10.1) has indicated that the limiting stress (f_{cl}) for local buckling of a full section is given by Eqs (10.1) to (10.3).

$$f_{c\ell} = f_y \text{ for } \lambda_\ell \leq 0.776 \tag{10.1}$$

$$f_{c\ell} = \left(1-0.15\left(\frac{f_{o\ell}}{f_y}\right)^{0.4}\right)\left(\frac{f_{o\ell}}{f_y}\right)^{0.4} f_y \text{ for } \lambda_\ell > 0.776 \tag{10.2}$$

where

$$\lambda_\ell = \sqrt{\frac{f_y}{f_{ol}}} \tag{10.3}$$

The 0.4 exponent in Eqs (10.1) and (10.2) rather than 0.5 as used in the Von Karman and Winter formulae discussed in Chapter 4 reflect a higher post-local-buckling reserve for a complete section when compared with an element. A comparison of local buckling moments in laterally braced beams is shown by the crosses (x) in Fig. 10.1 compared with Eqs (10.1) to (10.3).

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

by

Gregory J. Hancock BSc BE PhD DEng

Bluescope Steel Professor of Steel Structures
Dean
Faculty of Engineering & Information Technologies
University of Sydney

fourth edition - 2007

CONTENTS