

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 local mode 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 flange-distortional mode 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 overall mode 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).

$$f_{cl} = f_y \text{ for } \lambda_\ell \leq 0.776 \quad (10.1)$$

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

where
$$\lambda_\ell = \sqrt{\frac{f_y}{f_{ol}}} \quad (10.3)$$

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)

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CONTENTS

	Page
PREFACE TO THE 4 th EDITION	viii
CHAPTER 1 INTRODUCTION	1
1.1 Design Standards and Specifications for Cold-Formed Steel	1
1.1.1 General	1
1.1.2 History of Australian Cold-Formed Steel Structures Standards and USA Specifications	1
1.1.3 New Developments in the 2005 Edition	2
1.2 Common Section Profiles and Applications of Cold-Formed Steel	4
1.3 Manufacturing Processes	10
1.4 Special Problems in the Design of Cold-Formed Sections	12
1.4.1 Local Buckling and Post-local Buckling of Thin Plate Elements	12
1.4.2 Propensity for Twisting	13
1.4.3 Distortional Buckling	14
1.4.4 Cold Work of Forming	14
1.4.5 Web Crippling under Bearing	15
1.4.6 Connections	15
1.4.7 Corrosion Protection	16
1.4.8 Inelastic Reserve Capacity	16
1.4.9 Fatigue	16
1.5 Loading Combinations	17
1.6 Limit States Design	17
1.7 Computer Analysis	19
1.8 References	20
CHAPTER 2 MATERIALS AND COLD WORK OF FORMING	22
2.1 Steel Standards	22
2.2 Typical Stress-Strain Curves	23
2.3 Ductility	25
2.4 Effects of Cold Work on Structural Steels	29
2.5 Corner Properties of Cold-Formed Sections	30
2.6 Fracture Toughness	32
2.6.1 Background	32
2.6.2 Measurement of Critical Stress Intensity Factors	32
2.6.3 Evaluation of the Critical Stress Intensity Factors for Perforated Coupon Specimens	34
2.6.4 Evaluation of the Critical Stress Intensity Factors for Triple Bolted Specimens	35
2.7 References	36
CHAPTER 3 BUCKLING MODES OF THIN-WALLED MEMBERS IN COMPRESSION AND BENDING	37
3.1 Introduction to the Finite Strip Method	37
3.2 Monosymmetric Column Study	38
3.2.1 Unlipped Channel	38
3.2.2 Lipped Channel	41
3.2.3 Lipped Channel (Fixed Ended)	44
3.3 Purlin Section Study	45
3.3.1 Channel Section	45
3.3.2 Z-Section	46



3.4	Tubular Flange Sections	47
3.4.1	Hollow Flange Beam in Bending	47
3.4.2	LiteSteel Beam Section in Bending	48
3.5	References	49
CHAPTER 4 STIFFENED AND UNSTIFFENED COMPRESSION ELEMENTS		50
4.1	Local Buckling	50
4.2	Postbuckling of Plate Elements in Compression	51
4.3	Effective Width Formulae for Imperfect Elements in Pure Compression	52
4.4	Effective Width Formulae for Imperfect Elements under Stress Gradient	56
4.4.1	Stiffened Elements	56
4.4.2	Unstiffened Elements	56
4.5	Effective Width Formulae for Elements with Stiffeners	57
4.5.1	Edge Stiffened Elements	57
4.5.2	Intermediate Stiffened Elements with One Intermediate Stiffener	58
4.5.3	Edge Stiffened Elements with Intermediate Stiffeners, and Stiffened Elements with more than One Intermediate Stiffener	58
4.5.4	Uniformly Compressed Edge Stiffened Elements with Intermediate Stiffeners	59
4.6	Examples	59
4.6.1	Hat Section in Bending	59
4.6.2	Hat Section in Bending with Intermediate Stiffener in Compression Flange	63
4.6.3	C-Section Purlin in Bending	68
4.7	References	75
CHAPTER 5 BEAMS, PURLINS AND BRACING		76
5.1	General	76
5.2	Flexural-Torsional (Lateral) Buckling	77
5.2.1	Elastic Buckling of Unbraced Simply Supported Beams	77
5.2.2	Continuous Beams and Braced Simply Supported Beams	81
5.2.3	Bending Strength Design Equations	85
5.3	Distortional Buckling	86
5.3.1	Flange Distortional Buckling	86
5.3.2	Lateral-Distortional Buckling	89
5.4	Basic Behaviour of Purlins	89
5.4.1	Linear Response of Channel and Z-sections	89
5.4.2	Stability Considerations	92
5.4.3	Sheeting and Connection Types	94
5.5	Design Methods for Purlins	95
5.5.1	No Lateral and Torsional Restraint Provided by the Sheeting	95
5.5.2	Lateral Restraint but No Torsional Restraint	95
5.5.3	Lateral and Torsional Restraint	96
5.6	Bracing	98
5.7	Inelastic Reserve Capacity	101
5.7.1	Sections with Flat Elements	101
5.7.2	Cylindrical Tubular Members	102
5.8	Examples	102
5.8.1	Simply Supported C-Section Purlin	102
5.8.2	Distortional Buckling Stress for C-Section	107
5.8.3	Continuous Lapped Z-Section Purlin	108
5.8.4	Z-Section Purlin in Bending	116
5.9	References	122



CHAPTER 6	WEBS	125
6.1	General	125
6.2	Webs in Shear	125
6.2.1	Shear Buckling	125
6.2.2	Shear Yielding	127
6.3	Webs in Bending	127
6.4	Webs in Combined Bending and Shear	129
6.5	Web Stiffeners	130
6.6	Web Crippling (Bearing) of Open Sections	130
6.6.1	Edge Loading Alone	130
6.6.2	Combined Bending and Edge Loading	133
6.7	Webs with Holes	134
6.8	Examples	136
6.8.1	Combined Bending and Shear at the End of the Lap of a Continuous Z-Section Purlin	136
6.8.2	Combined Bearing and Bending of Hat Section	138
6.9	References	139
CHAPTER 7	COMPRESSION MEMBERS	141
7.1	General	141
7.2	Elastic Member Buckling	141
7.2.1	Flexural, Torsional and Flexural-Torsional Buckling	141
7.2.2	Distortional Buckling	143
7.3	Section Capacity in Compression	143
7.4	Member Capacity in Compression	144
7.4.1	Flexural, Torsional and Flexural-Torsional Buckling	144
7.4.2	Distortional Buckling	146
7.5	Effect of Local Buckling	147
7.5.1	Monosymmetric Sections	147
7.5.2	High Strength Steel Box Sections	149
7.6	Examples	151
7.6.1	Square Hollow Section Column	151
7.6.2	Unlipped Channel Column	153
7.6.3	Lipped Channel Column	157
7.7	References	164
CHAPTER 8	MEMBERS IN COMBINED AXIAL LOAD AND BENDING	165
8.1	Combined Axial Compressive Load and Bending - General	165
8.2	Interaction Equations for Combined Axial Compressive Load and Bending	166
8.3	Monosymmetric Sections under Combined Axial Compressive Load and Bending	167
8.3.1	Sections Bent in a Plane of Symmetry	167
8.3.2	Sections Bent about an Axis of Symmetry	169
8.4	Combined Axial Tensile Load and Bending	170
8.5	Examples	171
8.5.1	Unlipped Channel Section Beam-Column Bent in Plane of Symmetry	171
8.5.2	Unlipped Channel Section Beam-Column Bent about Plane of Symmetry	174
8.5.3	Lipped Channel Section Beam-Column Bent in Plane of Symmetry	176
8.6	References	180



CHAPTER 9	CONNECTIONS	182
9.1	Introduction to Welded Connections	182
9.2	Fusion Welds	184
9.2.1	Butt Welds	184
9.2.2	Fillet Welds subject to Transverse Loading	184
9.2.3	Fillet Welds subject to Longitudinal Loading	185
9.2.4	Combined Longitudinal and Transverse Fillet Welds	186
9.2.5	Flare Welds	186
9.2.6	Arc Spot Welds (Puddle Welds)	187
9.2.7	Arc Seam Welds	190
9.3	Resistance Welds	190
9.4	Introduction to Bolted Connections	190
9.5	Design Formulae and Failure Modes for Bolted Connections	192
9.5.1	Tearout Failure of Sheet (Type I)	193
9.5.2	Bearing Failure of Sheet (Type II)	193
9.5.3	Net Section Tension Failure (Type III)	194
9.5.4	Shear Failure of Bolt (Type IV)	196
9.6	Screw Fasteners and Blind Rivets	196
9.7	Rupture	200
9.8	Examples	201
9.8.1	Welded Connection Design Example	201
9.8.2	Bolted Connection Design Example	205
9.9	References	208
CHAPTER 10	DIRECT STRENGTH METHOD	209
10.1	Introduction	209
10.2	Elastic Buckling Solutions	209
10.3	Strength Design Curves	210
10.3.1	Local Buckling	210
10.3.2	Flange-distortional buckling	212
10.3.3	Overall buckling	213
10.4	Direct Strength Equations	213
10.5	Examples	215
10.5.1	Lipped Channel Column (Direct Strength Method)	215
10.5.2	Simply Supported C-Section Beam	216
10.6	References	218
CHAPTER 11	STEEL STORAGE RACKING	219
11.1	Introduction	219
11.2	Loads	220
11.3	Methods of Structural Analysis	221
11.3.1	Upright Frames - First Order	222
11.3.2	Upright Frames - Second Order	223
11.3.3	Beams	223
11.4	Effects of Perforations (Slots)	224
11.4.1	Section Modulus of Net Section	224
11.4.2	Minimum Net Cross-Sectional Area	225
11.4.3	Form Factor (Q)	225
11.5	Member Design Rules	225
11.5.1	Flexural Design Curves	225
11.5.2	Column Design Curves	226



11.5.3	Distortional Buckling	227
11.6	Example	227
11.7	References	235
SUBJECT INDEX BY SECTION		236

