LRFD cold-formed steel design manual

American Iron and Steel Institute
LRFD COLD-FORMED STEEL DESIGN MANUAL

AMERICAN IRON AND STEEL INSTITUTE
LRFD COLD-FORMED STEEL DESIGN MANUAL

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE

This edition of the LRFD Cold-Formed Steel Design Manual is based on the March 16, 1991 Edition of the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members. The Manual includes the following sections:

Part I—Specification
Part II—Commentary
Part III—Supplementary Information
Part IV—Illustrative Examples
Part V—Charts and Tables
Part VI—Computer Aids
Part VII—Test Procedures

The Specification and the Commentary are both also available as separately bound booklets.


American Iron and Steel Institute
December 1991
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute or any other person named herein that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE

The American Iron and Steel Institute allowable stress design specification has long been used for the design of cold-formed steel structural members. The Load and Resistance Factor Design (LRFD) Specification has recently been developed from a research project sponsored by AISI at the University of Missouri-Rolla under the direction of Wei-Wen Yu with consultation of T.V. Galambos and initial contribution of M.K. Ravindra. In this LRFD Specification, separate load and resistance factors are applied to specified loads and nominal resistance to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic methodology as used for the development of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings.

This Specification contains six chapters of the LRFD recommendations for cold-formed steel structural members and connections. The background information for the design criteria is discussed in the Commentary and other related references.

AISI acknowledges the devoted efforts of the members of the Committee on Specifications for the Design of Cold-Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold-formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on this LRFD Specification are:

R. L. Brockenbrough, Chairman
R.B. Haws, Secretary
R.E. Albrecht
R. Bjorhovde
R.E. Brown
C.R. Clauer
D.A. Cuoco
D.S. Ellifrit
S.J. Errera*
E.R. Estes, Jr.
J.M. Fisher
T.V. Galambos
M. Golovin
W.B. Hall
G.S. Harris
R.B. Heagler
N. Iwankiw
A.L. Johnson
D.L. Johnson
T.J. Jones
H. Klein
K. H. Klippstein*
R.A. Laboube
J.N. Macadam
R.R. McCluer
W.R. Midgley
T.J. Morris
J.A. Moses
T.M. Murray
G.G. Nichols
J.N. Nunnery
T.B. Pekoz
C.W. Pinkham
P.G. Schurter
R.M. Schuster
P.A. Seaburg
F.V. Slocum
D.L. Tarlton
D.S. Wolford*
W.W. Yu
A.S. Zakrezewski

*Past Chairman
The activities of the Committee are sponsored by AISI's Light Construction Subcommittee of the Construction Marketing Committee. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
March 16, 1991
# TABLE OF CONTENTS

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS
MARCH 16, 1991 EDITION

<table>
<thead>
<tr>
<th>Section</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>PREFACE</td>
<td>I-3</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td>I-5</td>
</tr>
<tr>
<td>SYMBOLS AND DEFINITIONS</td>
<td>I-8</td>
</tr>
<tr>
<td>A. GENERAL PROVISIONS</td>
<td>I-18</td>
</tr>
<tr>
<td>A1 Limits of Applicability and Terms</td>
<td>I-18</td>
</tr>
<tr>
<td>A1.1 Scope and Limits of Applicability</td>
<td>I-18</td>
</tr>
<tr>
<td>A1.2 Terms</td>
<td>I-18</td>
</tr>
<tr>
<td>A1.3 Units of Symbols and Terms</td>
<td>I-19</td>
</tr>
<tr>
<td>A2 Non-Conforming Shapes and Constructions</td>
<td>I-19</td>
</tr>
<tr>
<td>A3 Material</td>
<td>I-19</td>
</tr>
<tr>
<td>A3.1 Applicable Steels</td>
<td>I-19</td>
</tr>
<tr>
<td>A3.2 Other Steels</td>
<td>I-20</td>
</tr>
<tr>
<td>A3.3 Ductility</td>
<td>I-20</td>
</tr>
<tr>
<td>A3.4 Delivered Minimum Thickness</td>
<td>I-21</td>
</tr>
<tr>
<td>A4 Loads</td>
<td>I-21</td>
</tr>
<tr>
<td>A4.1 Dead Load</td>
<td>I-21</td>
</tr>
<tr>
<td>A4.2 Live or Snow Load</td>
<td>I-21</td>
</tr>
<tr>
<td>A4.3 Impact Load</td>
<td>I-21</td>
</tr>
<tr>
<td>A4.4 Wind or Earthquake Loads</td>
<td>I-21</td>
</tr>
<tr>
<td>A4.5 Ponding</td>
<td>I-21</td>
</tr>
<tr>
<td>A5 Structural Analysis and Design</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1 Design Basis</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1.1 Limit State – Strength</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1.2 Limit State – Serviceability</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1.3 Nominal Loads</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1.4 Load Factors and Load Combinations</td>
<td>I-22</td>
</tr>
<tr>
<td>A5.1.5 Resistance Factors</td>
<td>I-23</td>
</tr>
<tr>
<td>A5.2 Yield Point and Strength Increase from Cold Work of Forming</td>
<td>I-25</td>
</tr>
<tr>
<td>A5.2.1 Yield Point</td>
<td>I-25</td>
</tr>
<tr>
<td>A5.2.2 Strength Increase from Cold Work of Forming</td>
<td>I-25</td>
</tr>
<tr>
<td>A5.3 Durability</td>
<td>I-26</td>
</tr>
<tr>
<td>A6 Reference Documents</td>
<td>I-26</td>
</tr>
<tr>
<td>B. ELEMENTS</td>
<td>I-28</td>
</tr>
<tr>
<td>B1 Dimensional Limits and Considerations</td>
<td>I-28</td>
</tr>
<tr>
<td>B1.1 Flange Flat-Width-to-Thickness Considerations</td>
<td>I-28</td>
</tr>
<tr>
<td>B1.2 Maximum Web Depth-to-Thickness Ratio</td>
<td>I-29</td>
</tr>
<tr>
<td>B2 Effective Widths of Stiffened Elements</td>
<td>I-30</td>
</tr>
<tr>
<td>B2.1 Uniformly Compressed Stiffened Elements</td>
<td>I-30</td>
</tr>
<tr>
<td>B2.2 Uniformly Compressed Stiffened Elements with Circular Holes</td>
<td>I-31</td>
</tr>
<tr>
<td>B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient</td>
<td>I-32</td>
</tr>
<tr>
<td>B3 Effective Widths of Unstiffened Elements</td>
<td>I-32</td>
</tr>
<tr>
<td>B3.1 Uniformly Compressed Unstiffened Elements</td>
<td>I-32</td>
</tr>
<tr>
<td>B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient</td>
<td>I-33</td>
</tr>
<tr>
<td>B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener</td>
<td>I-34</td>
</tr>
</tbody>
</table>
B4.1 Uniformly Compressed Elements with an Intermediate Stiffener .......................... I-34  
B4.2 Uniformly Compressed Elements with an Edge Stiffener .................................. I-35  
B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or  
Stiffened Elements with More Than One Intermediate Stiffener ................................ I-36  
B6 Stiffeners .......................................................... I-37  
B6.1 Transverse Stiffeners ............................................. I-37  
B6.2 Shear Stiffeners .................................................. I-38  
B6.3 Non-Conforming Stiffeners ..................................... I-39  
C. MEMBERS .................................................................. I-40  
C1 Properties of Sections ................................................. I-40  
C2 Tension Members ....................................................... I-40  
C3 Flexural Members ........................................................ I-40  
C3.1 Strength for Bending Only .......................................... I-40  
C3.1.1 Nominal Section Strength ....................................... I-40  
C3.1.2 Lateral Buckling Strength ...................................... I-41  
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing .................... I-44  
C3.2 Strength for Shear Only ............................................ I-45  
C3.3 Strength for Combined Bending and Shear .......................... I-46  
C3.4 Web Crippling Strength .......................................... I-46  
C3.5 Combined Bending and Web Crippling Strength ....................... I-48  
C4 Concentrically Loaded Compression Members ................................................. I-49  
C4.1 Sections Not Subject to Torsional or Torsional–Flexural Buckling ......................... I-50  
C4.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or  
Torsional–Flexural Buckling ........................................ I-50  
C4.3 Nonsymmetric Sections ............................................ I-50  
C5 Combined Axial Load and Bending ..................................... I-50  
C6 Cylindrical Tubular Members ........................................ I-52  
C6.1 Bending ............................................................... I-52  
C6.2 Compression ......................................................... I-52  
C6.3 Combined Bending and Compression .................................. I-52  
D. STRUCTURAL ASSEMBLIES ........................................... I-53  
D1 Built-Up Sections ..................................................... I-53  
D1.1 I – Sections Composed of Two Channels ................................................. I-53  
D1.2 Spacing of Connections in Compression Elements ............................................. I-54  
D2 Mixed Systems ........................................................ I-54  
D3 Lateral Bracing ........................................................ I-54  
D3.1 Symmetrical Beams and Columns ..................................... I-54  
D3.2 Channel–Section and Z–Section Beams ............................... I-54  
D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load  
With Top Flange Connected to Sheathing ............................................. I-55  
D3.2.2 Neither Flange Connected to Sheathing .................................. I-56  
D3.3 Laterally Unbraced Box Beams ...................................... I-57  
D4 Wall Studs and Wall Stud Assemblies .................................... I-57  
D4.1 Wall Studs in Compression .......................................... I-58  
D4.2 Wall Studs in Bending ............................................... I-60  
D4.3 Wall Studs with Combined Axial Load and Bending ....................... I-60  
E. CONNECTIONS AND JOINTS ........................................ I-61  
E1 General Provisions ..................................................... I-61  
E2 Welded Connections ................................................... I-61  
E2.1 Groove Welds in Butt Joints ...................................... I-61
E2.2 Arc Spot Welds ................................................... I-62
E2.3 Arc Seam Welds ................................................... I-65
E2.4 Fillet Welds ...................................................... I-67
E2.5 Flare Groove Welds ................................................ I-68
E2.6 Resistance Welds .................................................. I-70
E3 Bolted Connections ...................................................... I-70
E3.1 Spacing and Edge Distance .......................................... I-71
E3.2 Tension in Connected Part ........................................... I-72
E3.3 Bearing .......................................................... I-72
E3.4 Shear and Tension in Bolts ........................................ I-73
E4 Shear Rupture .......................................................... I-75
E5 Connections to Other Materials ......................................... I-75
E5.1 Bearing .......................................................... I-75
E5.2 Tension .......................................................... I-76
E5.3 Shear ............................................................ I-76

F. TESTS FOR SPECIAL CASES ............................................. I-77
F1 Tests for Determining Structural Performance ......................... I-77
F2 Tests for Confirming Structural Performance ........................... I-80
F3 Tests for Determining Mechanical Properties ............................ I-80
F3.1 Full Section ...................................................... I-80
F3.2 Flat Elements of Formed Sections .................................... I-81
F3.3 Virgin Steel ...................................................... I-81
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Full unreduced cross-sectional area of the member</td>
<td>C3.1.1, C3.1.2, C4, C6.2, D4.1</td>
</tr>
<tr>
<td>(A_b)</td>
<td>(b_1t + A_s), for transverse stiffeners at interior support and under concentrated load, and (b_2t + A_s), for transverse stiffeners at end support</td>
<td>B6.1</td>
</tr>
<tr>
<td>(A_b)</td>
<td>Gross cross-sectional area of bolt</td>
<td>E3.4</td>
</tr>
<tr>
<td>(A_c)</td>
<td>(18t^2 + A_s), for transverse stiffeners at interior support and under concentrated load, and (10t^2 + A_s), for transverse stiffeners at end support</td>
<td>B6.1</td>
</tr>
<tr>
<td>(A_e)</td>
<td>Effective area at the stress (F_n)</td>
<td>C4, C6.2, D4.1</td>
</tr>
<tr>
<td>(A_n)</td>
<td>Net area of cross section</td>
<td>C2, E3.2</td>
</tr>
<tr>
<td>(A_s)</td>
<td>Cross-sectional area of transverse stiffeners</td>
<td>B4, B4.1, B4.2, B6.1</td>
</tr>
<tr>
<td>(A'_s)</td>
<td>Effective area of stiffener</td>
<td>B4, B4.1, B4.2</td>
</tr>
<tr>
<td>(A_{st})</td>
<td>Gross area of shear stiffener</td>
<td>B6.2</td>
</tr>
<tr>
<td>(A_{wn})</td>
<td>Net web area</td>
<td>E4</td>
</tr>
<tr>
<td>(A_t)</td>
<td>Bearing area</td>
<td>E5.1</td>
</tr>
<tr>
<td>(A_2)</td>
<td>Full cross-sectional area of concrete support</td>
<td>E5.1</td>
</tr>
<tr>
<td>(a)</td>
<td>Shear panel length of the unreinforced web element. For a reinforced web element, the distance between transverse stiffeners</td>
<td>B6.2, C3.2</td>
</tr>
<tr>
<td>(a)</td>
<td>Length of bracing interval</td>
<td>D3.2</td>
</tr>
<tr>
<td>B</td>
<td>Stud spacing</td>
<td>D4, D4.1</td>
</tr>
<tr>
<td>(B_c)</td>
<td>Term for determining the tensile yield point of corners</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>b</td>
<td>Effective design width of compression element</td>
<td>B2.1, B2.2, B2.3, B3.1, B3.2, B4.1, B4.2, B5</td>
</tr>
<tr>
<td>b</td>
<td>Flange width, Z-section</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>(b_d)</td>
<td>Effective width for deflection calculation</td>
<td>B2.1, B2.2</td>
</tr>
<tr>
<td>(b_e)</td>
<td>Effective design width of sub-element or element</td>
<td>A1.2, B2.3, B5</td>
</tr>
<tr>
<td>(b_o)</td>
<td>See Figure B4.1</td>
<td>B4, B4.1, B5</td>
</tr>
<tr>
<td>C</td>
<td>For flexural members, ratio of the total corner cross-sectional area of the controlling flange to the full cross-sectional area of the controlling flange</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>(C_b)</td>
<td>Bending coefficient dependent on moment gradient</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>(C_m)</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>C&lt;sub&gt;ms&lt;/sub&gt;</td>
<td>Coefficient for lateral bracing of Z-section</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C&lt;sub&gt;mx&lt;/sub&gt;</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
<tr>
<td>C&lt;sub&gt;my&lt;/sub&gt;</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
<tr>
<td>C&lt;sub&gt;p&lt;/sub&gt;</td>
<td>Correction Factor</td>
<td>F1</td>
</tr>
<tr>
<td>C&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Coefficient for lateral torsional buckling</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C&lt;sub&gt;TF&lt;/sub&gt;</td>
<td>End moment coefficient in interaction formula</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C&lt;sub&gt;th&lt;/sub&gt;</td>
<td>Coefficient for lateral bracing of Z-sections</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C&lt;sub&gt;g&lt;/sub&gt;</td>
<td>Coefficient for lateral bracing of Z-sections</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C&lt;sub&gt;v&lt;/sub&gt;</td>
<td>Shear stiffener coefficient</td>
<td>B6.2</td>
</tr>
<tr>
<td>C&lt;sub&gt;w&lt;/sub&gt;</td>
<td>Torsional warping constant of the cross-section</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C&lt;sub&gt;y&lt;/sub&gt;</td>
<td>Compression strain factor</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>C&lt;sub&gt;0&lt;/sub&gt;</td>
<td>Initial column imperfection</td>
<td>D4.1</td>
</tr>
<tr>
<td>C&lt;sub&gt;1&lt;/sub&gt;</td>
<td>Term used to compute shear strain in wall board</td>
<td>B4, B4.1, D4.2</td>
</tr>
<tr>
<td>C&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Coefficient as defined in Figure B4–2</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>c&lt;sub&gt;r&lt;/sub&gt;</td>
<td>Amount of curling</td>
<td>B1.1b</td>
</tr>
<tr>
<td>D</td>
<td>Outside diameter of cylindrical tube</td>
<td>C6, C6.1, C6.2, D4.2</td>
</tr>
<tr>
<td>D</td>
<td>Overall depth of lip</td>
<td>B1.1, B4, D1.1</td>
</tr>
<tr>
<td>D</td>
<td>Shear stiffener coefficient</td>
<td>B6.2</td>
</tr>
<tr>
<td>D</td>
<td>Nominal dead load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>D&lt;sub&gt;0&lt;/sub&gt;</td>
<td>Initial column imperfection</td>
<td>D4.1</td>
</tr>
<tr>
<td>d</td>
<td>Depth of section</td>
<td>B1.1b, B4, C3.1.1, C3.1.3, D1.1, D3.2.1, D4, D4.1</td>
</tr>
<tr>
<td>d</td>
<td>Width of arc seam weld</td>
<td>E2.3</td>
</tr>
<tr>
<td>d</td>
<td>Visible diameter of outer surface of arc spot weld</td>
<td>E2.2</td>
</tr>
<tr>
<td>d</td>
<td>Diameter of bolt</td>
<td>E3, E3.1, E3.2, E3.4</td>
</tr>
<tr>
<td>d&lt;sub&gt;a&lt;/sub&gt;</td>
<td>Average diameter of the arc spot weld at mid–thickness of t</td>
<td>E2.2</td>
</tr>
<tr>
<td>d&lt;sub&gt;s&lt;/sub&gt;</td>
<td>Average width of seam weld</td>
<td>E2.3</td>
</tr>
<tr>
<td>d&lt;sub&gt;e&lt;/sub&gt;</td>
<td>Effective diameter of fused area</td>
<td>E2.2</td>
</tr>
<tr>
<td>d&lt;sub&gt;e&lt;/sub&gt;</td>
<td>Effective width of arc seam weld at fused surfaces</td>
<td>E2, E2.3</td>
</tr>
<tr>
<td>d&lt;sub&gt;h&lt;/sub&gt;</td>
<td>Diameter of standard hole</td>
<td>B2.2, E3.1, E4</td>
</tr>
<tr>
<td>d&lt;sub&gt;s&lt;/sub&gt;</td>
<td>Reduced effective width of stiffener</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>d&lt;sub&gt;'s&lt;/sub&gt;</td>
<td>Actual effective width of stiffener</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>d&lt;sub&gt;we&lt;/sub&gt;</td>
<td>Coped web depth</td>
<td>E4</td>
</tr>
<tr>
<td>E</td>
<td>Modulus of elasticity of steel (29.5x10&lt;sup&gt;3&lt;/sup&gt; ksi)</td>
<td>B1.1b, B2.1, B6.1, C3.1.1, C3.2, C3.5, C4, C4.1, C5, C6.1, D1.2, D4.1, D4.2, E2.2</td>
</tr>
</tbody>
</table>
## SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>Nominal earthquake load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$E_o$</td>
<td>Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location</td>
<td>D4.1</td>
</tr>
<tr>
<td>$E_1$</td>
<td>Term used to compute shear strain in wallboard</td>
<td>D4.1</td>
</tr>
<tr>
<td>$E'$</td>
<td>Inelastic modulus of elasticity</td>
<td>D4.1</td>
</tr>
<tr>
<td>e</td>
<td>The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed</td>
<td>E3.1</td>
</tr>
<tr>
<td>$e_{min}$</td>
<td>Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed</td>
<td>E2.2</td>
</tr>
<tr>
<td>$e_y$</td>
<td>Yield strain = $F_y/E$</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>F</td>
<td>Loads due to fluids</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$F_e$</td>
<td>Elastic buckling stress</td>
<td>C4, C4.1, C4.2, C4.3, C6.2, D4.1</td>
</tr>
<tr>
<td>$F_m$</td>
<td>Mean value of the fabrication factor</td>
<td>F1</td>
</tr>
<tr>
<td>$F_n$</td>
<td>Nominal buckling stress</td>
<td>C4, C6.2, D4.1</td>
</tr>
<tr>
<td>$F_{nt}$</td>
<td>Nominal tensile strength of bolts</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F_{nv}$</td>
<td>Nominal shear strength of bolts</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F'_{nt}$</td>
<td>Nominal tensile strength for bolts subject to combination of shear and tension</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F_{sy}$</td>
<td>Yield point as specified in Sections A3.1 or A3.2</td>
<td>A3.1, A3.2, A3.3.2, E2.2, E3.1, E3.2</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Tensile strength as specified in Sections A3.1 or A3.2, or as reduced for low ductility steel</td>
<td>A3.1, A3.2, A3.3, A3.3.2, E2.2, E2.3, E2.4, E2.5, E3.1, E3.2, E3.3, E4</td>
</tr>
<tr>
<td>$F_{uv}$</td>
<td>Tensile strength of virgin steel specified by Section A3 or established in accordance with Section F3.3</td>
<td>A3, A5.2.2, E2.2, F3.3</td>
</tr>
<tr>
<td>$F_{wy}$</td>
<td>Yield point for design of transverse stiffeners</td>
<td>B6.1</td>
</tr>
<tr>
<td>$F_{xx}$</td>
<td>Strength level designation in AWS electrode classification</td>
<td>E2.2, E2.3, E2.4, E2.5</td>
</tr>
</tbody>
</table>
## SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>(F_y)</td>
<td>Yield point used for design, not to exceed the specified yield point or established in accordance with Section F3, or as increased for cold work of forming in Section A5.2.2 or as reduced for low ductility steels in Section A3.3.2</td>
<td>A1.2, A3.3, A5.2.1, A5.2.2, B2.1, B5, B6.1, C2, C3.1, C3.1.1, C3.1.3, C3.2, C3.5, C4, C6.1, C6.2, D1.2, D4, D4.2, E2</td>
</tr>
<tr>
<td>(F_{y\text{av}})</td>
<td>Average yield point of section</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>(F_{yc})</td>
<td>Tensile yield point of corners</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>(F_{yi})</td>
<td>Weighted average tensile yield point of the flat portions</td>
<td>A5.2.2, F3.2</td>
</tr>
<tr>
<td>(F_{yf})</td>
<td>Yield point of stiffener steel</td>
<td>B6.1</td>
</tr>
<tr>
<td>(F_{yv})</td>
<td>Tensile yield point of virgin steel specified by Section A3 or established in accordance with Section F3.3</td>
<td>A3, A5.2.2, F3.3</td>
</tr>
<tr>
<td>(f)</td>
<td>Stress in the compression element computed on the basis of the effective design width</td>
<td>B2.1, B2.2, B3.2, B4, B4.1</td>
</tr>
<tr>
<td>(f_{av\text{w}})</td>
<td>Average computed stress in the full, unreduced flange width</td>
<td>B1.1b</td>
</tr>
<tr>
<td>(f_c)</td>
<td>Stress at service load in the cover plate or sheet</td>
<td>D1.2</td>
</tr>
<tr>
<td>(f'_c)</td>
<td>Specified compression stress of concrete</td>
<td>E5.1</td>
</tr>
<tr>
<td>(f_d)</td>
<td>Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined.</td>
<td>B2.1, B2.2, B3.1, B4.1, B4.2</td>
</tr>
<tr>
<td>(f_{d1}, f_{d2})</td>
<td>Computed stresses (f_1) and (f_2) as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.</td>
<td>B2.3</td>
</tr>
<tr>
<td>(f_{d3})</td>
<td>Computed stress (f_3) in edge stiffener, as shown in Figure B4–2. Calculations are based on the effective section at the load for which deflections are determined</td>
<td>B3.2</td>
</tr>
<tr>
<td>(f_v)</td>
<td>Computed shear stress on a bolt</td>
<td>E4</td>
</tr>
<tr>
<td>(f_1, f_2)</td>
<td>Web stresses defined by Figure B2.3–1</td>
<td>B2.3</td>
</tr>
<tr>
<td>(f_3)</td>
<td>Edge stiffener stress defined by Figure B4–2</td>
<td>B3.2</td>
</tr>
<tr>
<td>(G)</td>
<td>Shear modulus of steel (11,300 ksi)</td>
<td>C3.1.1, D4.1</td>
</tr>
<tr>
<td>(G')</td>
<td>Inelastic shear modulus</td>
<td>D4.1</td>
</tr>
<tr>
<td>(g)</td>
<td>Vertical distance between two rows of connections nearest to the top and bottom flanges</td>
<td>D1.1</td>
</tr>
</tbody>
</table>
# SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H$</td>
<td>Loads due to the weight and lateral pressure of soil and water in soil</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$h$</td>
<td>Depth of flat portion of web measured along the plane of web</td>
<td>B1.2, B6.2, C3.2, C3.4</td>
</tr>
<tr>
<td>$I_a$</td>
<td>Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element</td>
<td>B1.1, B4, B4.1, B4.2</td>
</tr>
<tr>
<td>$I_b$</td>
<td>Moment of inertia of the full unreduced section about the bending axis</td>
<td>C5</td>
</tr>
<tr>
<td>$I_s$</td>
<td>Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened</td>
<td>B1.1, B4, B4.1, B4.2, B5</td>
</tr>
<tr>
<td>$I_{sf}$</td>
<td>Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened</td>
<td>B5</td>
</tr>
<tr>
<td>$I_x, I_y$</td>
<td>Moment of inertia of full section about principal axis</td>
<td>D1.1, D3.2.2</td>
</tr>
<tr>
<td>$I_{xy}$</td>
<td>Product of inertia of full section about major and minor centroidal axes</td>
<td>D3.2.2, D4.1</td>
</tr>
<tr>
<td>$I_{ye}$</td>
<td>Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$J$</td>
<td>St. Venant torsion constant</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$j$</td>
<td>Section property for torsional–flexural buckling</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$K$</td>
<td>Effective length factor</td>
<td>C3.1.2, C4, C4.1, C5</td>
</tr>
<tr>
<td>$K'$</td>
<td>A constant</td>
<td>D3.2.2</td>
</tr>
<tr>
<td>$K_b$</td>
<td>Effective length factor in the plane of bending</td>
<td>C5</td>
</tr>
<tr>
<td>$K_t$</td>
<td>Effective length factor for torsion</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$K_x$</td>
<td>Effective length factor for bending about $x$–axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$K_y$</td>
<td>Effective length factor for bending about $y$–axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$k$</td>
<td>Plate buckling coefficient</td>
<td>B2.1, B2.3, B3.1, B3.2, B4</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Shear buckling coefficient</td>
<td>B6.2, C3.2</td>
</tr>
<tr>
<td>$L$</td>
<td>Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams</td>
<td>B1.1, D3.2.1</td>
</tr>
</tbody>
</table>
# SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbols</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Length of seam weld not including the circular ends</td>
<td>E2.3</td>
</tr>
<tr>
<td>L</td>
<td>Length of fillet weld</td>
<td>E2.4, E2.5</td>
</tr>
<tr>
<td>L</td>
<td>Unbraced length of member</td>
<td>C3.1.2, C4.1, D1.1, D4, D4.1</td>
</tr>
<tr>
<td>L</td>
<td>Nominal live load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>Lr</td>
<td>Nominal roof live load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>Lst</td>
<td>Length of transverse stiffener</td>
<td>B6.1</td>
</tr>
<tr>
<td>Lt</td>
<td>Unbraced length of compression member for torsion</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Lx</td>
<td>Unbraced length of compression member for bending about x-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Ly</td>
<td>Unbraced length of compression member for bending about y-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Me</td>
<td>Critical moment</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Me</td>
<td>Elastic critical moment</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Mm</td>
<td>Mean value of the material factor</td>
<td>F1</td>
</tr>
<tr>
<td>Mn</td>
<td>Nominal flexural strength</td>
<td>C3.1, C3.1.1, C3.1.2, C3.1.3, C6.1</td>
</tr>
<tr>
<td>Mnx,My</td>
<td>Nominal flexural strengths about the centroidal axes</td>
<td>C5</td>
</tr>
<tr>
<td>Mnxo,Myo</td>
<td>Nominal flexural strengths about the centroidal axes</td>
<td>C3.3, C3.5, D4.2, D4.3</td>
</tr>
<tr>
<td></td>
<td>determined in accordance with Section C3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>determined in accordance with Section C3.1 excluding</td>
<td></td>
</tr>
<tr>
<td></td>
<td>the provisions of Section C3.1.2</td>
<td></td>
</tr>
<tr>
<td>Mu</td>
<td>Required flexural strength</td>
<td>C3.3, C3.5</td>
</tr>
<tr>
<td>Mux</td>
<td>Required flexural strength about x-axis</td>
<td>C5</td>
</tr>
<tr>
<td>Muy</td>
<td>Required flexural strength about y-axis</td>
<td>C5</td>
</tr>
<tr>
<td>My</td>
<td>Moment causing a maximum strain ey</td>
<td>B2.1, C3.1</td>
</tr>
<tr>
<td>M1</td>
<td>Smaller end moment</td>
<td>C3.1.2, C5</td>
</tr>
<tr>
<td>M2</td>
<td>Larger end moment</td>
<td>C3.1.2, C5</td>
</tr>
<tr>
<td>m</td>
<td>Distance from the shear center of one channel to the</td>
<td>D1.1, D3.2.2</td>
</tr>
<tr>
<td></td>
<td>mid-plane of its web</td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>Actual length of bearing</td>
<td>D3.6</td>
</tr>
<tr>
<td>n</td>
<td>Number of holes</td>
<td>E4</td>
</tr>
<tr>
<td>n</td>
<td>Number of tests</td>
<td>F1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n_p )</td>
<td>Number of parallel purlin lines</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>( P )</td>
<td>Loads, forces, and effects due to ponding</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>( P_{e} )</td>
<td>( \pi^2 E b^2 / (K_b L_b)^2 )</td>
<td>C5</td>
</tr>
<tr>
<td>( P_L )</td>
<td>Force to be resisted by intermediate beam brace</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>( P_m )</td>
<td>Mean value of the tested-to-predicted load ratios</td>
<td>F1</td>
</tr>
<tr>
<td>( P_n )</td>
<td>Nominal axial strength of member</td>
<td>C4, C6.2</td>
</tr>
<tr>
<td>( P_{nc} )</td>
<td>Nominal strength of connection component</td>
<td>E2, E2.2, E2.3, E2.4, E2.5</td>
</tr>
<tr>
<td>( P_{so} )</td>
<td>Nominal axial strength of member determined in accordance with Section C4 for ( L = 0 )</td>
<td>C5</td>
</tr>
<tr>
<td>( P_p )</td>
<td>Nominal bearing capacity on concrete</td>
<td>E5.1</td>
</tr>
<tr>
<td>( P_t )</td>
<td>Concentrated load or reaction based on factored loads</td>
<td>D1.1</td>
</tr>
<tr>
<td>( P_u )</td>
<td>Required axial strength</td>
<td>C5</td>
</tr>
<tr>
<td>( Q )</td>
<td>Design shear rigidity for sheathing on both sides of the wall assembly</td>
<td>D4.1</td>
</tr>
<tr>
<td>( Q_i )</td>
<td>Load effect</td>
<td>F1</td>
</tr>
<tr>
<td>( q )</td>
<td>Uniformly distributed factored load in the plane of the web</td>
<td>D1.1</td>
</tr>
<tr>
<td>( q )</td>
<td>Design shear rigidity for sheathing per inch of stud spacing</td>
<td>D4.1</td>
</tr>
<tr>
<td>( q_0 )</td>
<td>Factor used to determine design shear rigidity</td>
<td>D4.1</td>
</tr>
<tr>
<td>( R )</td>
<td>Reduction Factor</td>
<td>C3.1.3</td>
</tr>
<tr>
<td>( R )</td>
<td>Coefficient</td>
<td>C4, C6.2</td>
</tr>
<tr>
<td>( R )</td>
<td>Inside bend radius</td>
<td>A5.2.2, C3.4</td>
</tr>
<tr>
<td>( R_b )</td>
<td>Nominal resistance</td>
<td>A1.2, F1</td>
</tr>
<tr>
<td>( R_{av} )</td>
<td>Average value of all test results</td>
<td>F1</td>
</tr>
<tr>
<td>( R_{f} )</td>
<td>Nominal roof rain load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>( r )</td>
<td>Radius of gyration of full unreduced cross section</td>
<td>C3.1.1, C4, C4.1</td>
</tr>
<tr>
<td>( r )</td>
<td>Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section</td>
<td>E3.2</td>
</tr>
<tr>
<td>( r_{cy} )</td>
<td>Radius of gyration of one channel about its centroidal axis parallel to web</td>
<td>D1.1</td>
</tr>
<tr>
<td>( r_{f} )</td>
<td>Radius of gyration of I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing</td>
<td>D1.1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r_o )</td>
<td>Polar radius of gyration of cross section about the shear center</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>( r_x, r_y )</td>
<td>Radius of gyration of cross section about centroidal principal axis</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>( S )</td>
<td>( 1.28\sqrt{E/\sigma} )</td>
<td>B4, B4.1</td>
</tr>
<tr>
<td>( S )</td>
<td>Nominal snow load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>( S_c )</td>
<td>Elastic section modulus of the effective section calculated at a stress ( \frac{M_c}{S_i} ) in the extreme compression fiber</td>
<td>C3.1.1, C3.1.2, C4</td>
</tr>
<tr>
<td>( S_e )</td>
<td>Elastic section modulus of the effective section calculated with extreme compression or tension fiber at ( F_y )</td>
<td>C3.1.1, C3.1.3</td>
</tr>
<tr>
<td>( S_r )</td>
<td>Elastic section modulus of full, unreduced section for the extreme compression fiber</td>
<td>C3.1.1, C3.1.2, C6.1</td>
</tr>
<tr>
<td>( S_{\text{max}} )</td>
<td>Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I-section</td>
<td>D1.1</td>
</tr>
<tr>
<td>( s )</td>
<td>Fastener spacing</td>
<td>D1.2, D4.1</td>
</tr>
<tr>
<td>( s )</td>
<td>Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element</td>
<td>E3.2</td>
</tr>
<tr>
<td>( s )</td>
<td>Weld spacing</td>
<td>D1.1</td>
</tr>
<tr>
<td>( T_n )</td>
<td>Nominal tensile strength</td>
<td>C2</td>
</tr>
<tr>
<td>( T_s )</td>
<td>Design strength of connection in tension</td>
<td>D1.1</td>
</tr>
<tr>
<td>( t )</td>
<td>Base steel thickness of any element or section</td>
<td>A1.2, A3.4, A5.2.1, B1.1, B1.1b, B1.2, B2.1, B4, B4.1, B4.2, B5, B6.1, C3.1.1, C3.2, C3.4, C3.5, C4, C6.1, C6.2, D1.2, D4, E2.4, E2.5</td>
</tr>
<tr>
<td>( t )</td>
<td>Total thickness of the two welded sheets</td>
<td>E2.2</td>
</tr>
<tr>
<td>( t )</td>
<td>Thickness of thinnest connected part</td>
<td>E2.2, E3.1, E4</td>
</tr>
<tr>
<td>( t_s )</td>
<td>Equivalent thickness of a multiple-stiffened element</td>
<td>B5, B6.1</td>
</tr>
<tr>
<td>( t_w )</td>
<td>Effective throat of weld</td>
<td>E2.4, E2.5</td>
</tr>
<tr>
<td>( V_F )</td>
<td>Coefficient of variation of the fabrication factor</td>
<td>F1</td>
</tr>
<tr>
<td>( V_M )</td>
<td>Coefficient of variation of the material factor</td>
<td>F1</td>
</tr>
</tbody>
</table>
## SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_v$</td>
<td>Nominal shear strength</td>
<td>B6.2, C3.2, C3.3</td>
</tr>
<tr>
<td>$V_p$</td>
<td>Coefficient of variation of the tested-to-predicted load ratios</td>
<td>F1</td>
</tr>
<tr>
<td>$V_Q$</td>
<td>Coefficient of variation of the load effect</td>
<td>F1</td>
</tr>
<tr>
<td>$V_u$</td>
<td>Required shear strength</td>
<td>C3.3</td>
</tr>
<tr>
<td>$W$</td>
<td>Factored load supported by all purlin lines being restrained</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>$W$</td>
<td>Nominal wind load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$w$</td>
<td>Flat width of element exclusive of radii</td>
<td>A1.2, B1.1, B2.1, B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1, C4, D1.2</td>
</tr>
<tr>
<td>$w$</td>
<td>Flat width of the beam flange which contacts the bearing plate</td>
<td>C3.5</td>
</tr>
<tr>
<td>$w_f$</td>
<td>Width of flange projection beyond the web or half the distance between webs</td>
<td>B1.1c</td>
</tr>
<tr>
<td>$w_f$</td>
<td>projection for box- or U-type sections</td>
<td></td>
</tr>
<tr>
<td>$w_l$</td>
<td>Projection of flanges from inside face of web</td>
<td>B1.1b, D1.1</td>
</tr>
<tr>
<td>$w_2$</td>
<td>Leg on weld</td>
<td>E2.4</td>
</tr>
<tr>
<td>$x$</td>
<td>Distance from concentrated load to brace</td>
<td>D3.2</td>
</tr>
<tr>
<td>$x_0$</td>
<td>Distance from shear center to centroid along the principal x-axis</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>$Y$</td>
<td>Yield point of web steel divided by yield point of stiffener steel</td>
<td>B6.2</td>
</tr>
<tr>
<td>$1/\alpha_{xx}$</td>
<td>Magnification factors</td>
<td>C5</td>
</tr>
<tr>
<td>$1/\alpha_{yy}$</td>
<td>Magnification factors</td>
<td></td>
</tr>
<tr>
<td>$\beta$</td>
<td>Coefficient</td>
<td>C4.2, D4.1</td>
</tr>
<tr>
<td>$\beta_0$</td>
<td>Target reliability index</td>
<td>F1</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Actual shear strain in the sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Permissible shear strain of the sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Load factor</td>
<td>F1</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle between web and bearing surface &gt;45° but no more than 90°</td>
<td>C3.4</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle between the vertical and the plane of the web of the Z-section,</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Stress related to shear strain in sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_{CR}$</td>
<td>Theoretical elastic buckling stress</td>
<td>D4.1</td>
</tr>
</tbody>
</table>
# SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{ex}$</td>
<td>$(\pi^2E)/(K_xL_x/r_x)^2$</td>
<td>C3.1.2, C4.2</td>
</tr>
<tr>
<td></td>
<td>$(\pi^2E)/(L/r_x)^2$</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_{exy}$</td>
<td>$(\pi^2EI_{exy})/(AL^2)$</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_{ey}$</td>
<td>$(\pi^2E)/(K_yL_y/r_y)^2$</td>
<td>C3.1.2</td>
</tr>
<tr>
<td></td>
<td>$(\pi^2E)/(L/r_y)^2$</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Torsional buckling stress</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Reduction factor</td>
<td>B2.1</td>
</tr>
<tr>
<td>$\lambda$, $\lambda_c$</td>
<td>Slenderness factors</td>
<td>B2.1, C3.5</td>
</tr>
<tr>
<td>$\psi$</td>
<td>$f_2/f_1$</td>
<td>B2.3</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Resistance factor</td>
<td>A5.1.5, E2, E2.1, E2.2, E2.3, E2.4, E2.5, E2.6, E3.1, E3.2, E3.3, E3.4, E4, F1</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>Resistance factor for bending strength</td>
<td>A5.1.5, C3, C3.1.1, C3.1.2, C3.1.3, C3.3, C3.5, C5, C6.1, C6.3, D4.2, D4.3</td>
</tr>
<tr>
<td>$\phi_c$</td>
<td>Resistance factor for concentrically loaded compression member</td>
<td>A3.3.1, A5.1.5, B6.1, C4, C5, C6.2, C6.3, D4.1, D4.3</td>
</tr>
<tr>
<td>$\phi_r$</td>
<td>Resistance factor for bearing strength</td>
<td>E5.1</td>
</tr>
<tr>
<td>$\phi_t$</td>
<td>Resistance factor for tension member</td>
<td>C2</td>
</tr>
<tr>
<td>$\phi_v$</td>
<td>Resistance factor for shear strength</td>
<td>B6.2, C3.2, C3.3</td>
</tr>
<tr>
<td>$\phi_w$</td>
<td>Resistance factor for web crippling strength</td>
<td>C3.4, C3.5</td>
</tr>
</tbody>
</table>
LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This Load and Resistance Factor Design Specification is an alternate to the Specification for the Design of Cold-Formed Steel Structural Members of the American Iron and Steel Institute.

This Specification shall apply to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load-carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects.

A1.2 Terms

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) Stiffened or Partially Stiffened Compression Elements. A stiffened or partially stiffened compression element is a flat compression element (i.e., a plane compression flange of a flexural member or a plane web or flange of a compression member) of which both edges parallel to the direction of stress are stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

(b) Unstiffened Compression Elements. An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) Multiple-Stiffened Elements. A multiple-stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A sub-element is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

(d) Flat-Width-to-Thickness Ratio. The flat width of an element measured along its plane, divided by its thickness.

(e) Effective Design Width. Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

(f) Thickness. The thickness, t, of any element or section shall be the base steel thickness, exclusive of coatings.

(g) Torsional–Flexural Buckling. Torsional–flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

(h) Point-Symmetric Section. A point-symmetric section is a section symmetrical about a point (centroid) such as a Z-section having equal flanges.
Yield Point. Yield point, $F_y$ or $F_{sy}$, as used in this Specification shall mean yield point or yield strength.

Stress. Stress as used in this Specification means force per unit area.

Confirmatory Test. A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

Performance Test. A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.

Virgin Steel. Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

Virgin Steel Properties. Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.

Specified Minimum Yield Point. The specified minimum yield point is the lower limit of yield point which must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold-formed steel structural member designed at that yield point.

Cold-Formed Steel Structural Members. Cold-formed steel structural members are shapes which are manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

LRFD (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

Design Strength. Factored resistance or strength (force, moment, as appropriate), $\phi R_a$, provided by the structural component.

Required Strength. Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads (using most appropriate critical load combinations).

A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

A2 Non-Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

A3 Material

A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:
ASTM A36/A36M, Structural Steel  
ASTM A242/A242M, High-Strength Low-Alloy Structural Steel  
ASTM A441M, High-Strength Low-Alloy Structural Manganese Vanadium Steel  
ASTM A446/A446M (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality  
ASTM A500, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes  
ASTM A529/A529M, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)  
ASTM A570/A570M Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality  
ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality  
ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick  
ASTM A606 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance  
ASTM A607 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled  
ASTM A611 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality  
ASTM A715 (Grades 50 and 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability  
ASTM A792 (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

Steels not listed in Section A3.1 and used for structural members and connections shall comply with one of the following ductility requirements:

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an eight-inch gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 inch gage length across the fracture shall not be less than 20%, (2) uniform elongation outside the fracture shall not be less than 3%*. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts** in

* Further information on the test procedures should be obtained from the Commentary.
** Horizontal structural members which support roof deck or panel covering and applied loads principally by bending.
accordance with Sections C3.1.1(a), C3.1.2, and C3.1.3. For purlins and girts subject to combined axial load and bending moment (Section C5), \( P_u/\phi_cP_u \) shall not exceed 0.15.

**A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels which do not meet the provisions of Section A3.3.1 may be used for particular configurations provided (1) the yield strength, \( F_y \), used for design in Chapters B, C and D is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less and (2) the tensile strength, \( F_u \), used for design in Chapter E is taken as 75 percent of the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the suitability of such steels for the configuration shall be demonstrated by load tests in accordance with Section F1. Design strengths based on these tests shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, \( F_{sy} \), for \( F_y \) and the specified minimum tensile strength, \( F_u \).

Design strengths based on existing use shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, \( F_{sy} \), for \( F_y \) and the specified minimum tensile strength, \( F_u \).

### A3.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold-formed product as delivered to the job site shall not at any location be less than 95 percent of the thickness, \( t \), used in its design; however, lesser thicknesses shall be permitted at bends, such as corners, due to cold-forming effects.

### A4 Loads

#### A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

#### A4.2 Live or Snow Load

The live or snow load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

#### A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall be increased sufficiently to provide for impact.

#### A4.4 Wind or Earthquake Loads

Wind or earthquake load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

#### A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof sys-
tem shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based on the Load and Resistance Factor Design concept. Load and Resistance Factor Design is a method of proportioning cold-formed steel structural components (i.e., members, connectors and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combination.

Two types of limit states are to be considered: 1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and 2) the limit state of the ability of the structure to perform its intended function during its life. These limit states will be called the Limit State of Strength and the Limit State of Serviceability, respectively, in these criteria.

A5.1.1 Limit State - Strength

The design meets this Specification when the required strengths, as determined from the assigned nominal loads which are multiplied by appropriate load factors, are smaller than or equal to the design strength of each structural component.

The design strength is equal to \( \phi R_o \), where \( \phi \) is a resistance factor and \( R_o \) is the nominal strength determined according to the formulas given in Chapter C for members, in Chapter D for structural assemblies and in Chapter E for connections. Values of resistance factors \( \phi \) are given in Section A5.1.5 for the appropriate limit states governing member and connection strength.

A5.1.2 Limit State - Serviceability

Serviceability is satisfactory if a nominal structural response (e.g. live load deflection) due to the applicable nominal loads is less than or equal to the appropriate acceptable or allowable value of this response.

A5.1.3 Nominal Loads

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American Society of Civil Engineers Standard, ANSI/ASCE 7-88, Minimum Design Loads for Buildings and Other Structures. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads.

A5.1.4 Load Factors and Load Combinations*

The structure and its components must be designed for the appropriate most critical load combination. The following load combinations of the factored nominal loads shall be used in the computation of the required strengths:

1. \( 1.4 D + L \)
2. \( 1.2 D + 1.6 L + 0.5(L_r \text{ or } S \text{ or } R_r) \)
3. \( 1.2 D + (1.4 L_r \text{ or } 1.6 S \text{ or } 1.6 R_r) + (0.5 L \text{ or } 0.8 W) \)

* For roof and floor construction, recommended load combinations for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork are given in Section A5.1 of the Commentary.
4. $1.2 \, D + 1.3 \, W + 0.5 \, L + 0.5(L_r \text{ or } S \text{ or } R_r)$
5. $1.2 \, D + 1.5 \, E + (0.5 \, L \text{ or } 0.2 \, S)$
6. $0.9 \, D - (1.3 \, W \text{ or } 1.5 \, E)$

where
- $D =$ nominal dead load
- $E =$ nominal earthquake load
- $L =$ nominal live load
- $L_r =$ nominal roof live load
- $S =$ nominal snow load
- $W =$ nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for $W$ by 0.9)

Exception: The load factor for $L$ in combinations (3), (4), and (5) shall be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

When the structural effects of $F$, $H$, $P$ or $T$ are significant, they shall be considered in design as the following factored loads: $1.3 \, F$, $1.6 \, H$, $1.2 \, P$, and $1.2 \, T$, where
- $F =$ loads due to fluids with well-defined pressures and maximum heights
- $H =$ loads due to the weight and lateral pressure of soil and water in soil
- $P =$ loads, forces, and effects due to ponding
- $T =$ self-straining forces and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.

### A5.1.5 Resistance Factors

The resistance factors to be used for determining the design strengths, $\phi R_n$, of structural members and connections shall be taken as follows:

<table>
<thead>
<tr>
<th>Type of Strength</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Stiffeners</td>
<td></td>
</tr>
<tr>
<td>Transverse stiffeners</td>
<td>0.85</td>
</tr>
<tr>
<td>Shear stiffeners*</td>
<td>0.90</td>
</tr>
<tr>
<td>(b) Tension members</td>
<td>0.95</td>
</tr>
<tr>
<td>(c) Flexural members</td>
<td></td>
</tr>
<tr>
<td>Bending strength</td>
<td></td>
</tr>
<tr>
<td>For sections with stiffened or partially stiffened compression flanges</td>
<td>0.95</td>
</tr>
<tr>
<td>For sections with unstiffened compression flanges</td>
<td>0.90</td>
</tr>
<tr>
<td>Laterally unbraced beams</td>
<td>0.90</td>
</tr>
<tr>
<td>Beams having one flange through–fastened to deck or sheathing (C– or Z–sections)</td>
<td>0.90</td>
</tr>
<tr>
<td>Web design</td>
<td></td>
</tr>
<tr>
<td>Shear strength*</td>
<td>0.90</td>
</tr>
<tr>
<td>Web Crippling</td>
<td></td>
</tr>
<tr>
<td>For single unreinforced webs</td>
<td>0.75</td>
</tr>
<tr>
<td>For I–sections</td>
<td>0.80</td>
</tr>
</tbody>
</table>

*When $h \leq \sqrt{E k_v / F_y}$, $\phi = 1.0$
<table>
<thead>
<tr>
<th>Type of Strength</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d) Concentrically loaded compression members</td>
<td>0.85</td>
</tr>
<tr>
<td>(e) Combined axial load and bending</td>
<td></td>
</tr>
<tr>
<td>$\phi_c$ for compression</td>
<td>0.85</td>
</tr>
<tr>
<td>$\phi_b$ for bending</td>
<td></td>
</tr>
<tr>
<td>Using Section C3.1.1</td>
<td>0.90 - 0.95</td>
</tr>
<tr>
<td>Using Section C3.1.2</td>
<td>0.90</td>
</tr>
<tr>
<td>(f) Cylindrical tubular members</td>
<td></td>
</tr>
<tr>
<td>Bending strength</td>
<td>0.95</td>
</tr>
<tr>
<td>Axial compression</td>
<td>0.85</td>
</tr>
<tr>
<td>(g) Wall studs and wall stud assemblies</td>
<td></td>
</tr>
<tr>
<td>Wall studs in compression</td>
<td>0.85</td>
</tr>
<tr>
<td>Wall studs in bending</td>
<td></td>
</tr>
<tr>
<td>For sections with stiffened or partially stiffened compression flanges</td>
<td>0.95</td>
</tr>
<tr>
<td>For sections with unstiffened compression flanges</td>
<td>0.90</td>
</tr>
<tr>
<td>(h) Welded connections</td>
<td></td>
</tr>
<tr>
<td>Groove welds</td>
<td></td>
</tr>
<tr>
<td>Tension or compression</td>
<td>0.90</td>
</tr>
<tr>
<td>Shear (welds)</td>
<td>0.80</td>
</tr>
<tr>
<td>Shear (base metal)</td>
<td>0.90</td>
</tr>
<tr>
<td>Arc spot welds</td>
<td></td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Connected part</td>
<td>0.50 - 0.65</td>
</tr>
<tr>
<td>Minimum edge distance</td>
<td>0.60 - 0.70</td>
</tr>
<tr>
<td>Arc seam welds</td>
<td></td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Connected part</td>
<td>0.60</td>
</tr>
<tr>
<td>Fillet welds</td>
<td></td>
</tr>
<tr>
<td>Longitudinal loading (connected part)</td>
<td>0.55 - 0.60</td>
</tr>
<tr>
<td>Transverse loading (connected part)</td>
<td>0.60</td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Flare groove welds</td>
<td></td>
</tr>
<tr>
<td>Transverse loading (connected part)</td>
<td>0.55</td>
</tr>
<tr>
<td>Longitudinal loading (connected part)</td>
<td>0.55</td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Resistance Welds</td>
<td>0.65</td>
</tr>
<tr>
<td>(i) Bolted connections</td>
<td></td>
</tr>
<tr>
<td>Minimum spacing and edge distance</td>
<td>0.60 - 0.70</td>
</tr>
<tr>
<td>Tension strength on net section</td>
<td></td>
</tr>
<tr>
<td>With washers</td>
<td></td>
</tr>
<tr>
<td>Double shear connection</td>
<td>0.65</td>
</tr>
<tr>
<td>Single shear connection</td>
<td>0.55</td>
</tr>
<tr>
<td>Without washers</td>
<td>0.65</td>
</tr>
<tr>
<td>Bearing strength</td>
<td></td>
</tr>
<tr>
<td>See Tables E3.3-1 and E3.3-2</td>
<td>0.55 - 0.70</td>
</tr>
<tr>
<td>Shear strength of bolts</td>
<td>0.65</td>
</tr>
<tr>
<td>Tensile strength of bolts</td>
<td>0.75</td>
</tr>
<tr>
<td>(j) Shear rupture</td>
<td>0.75</td>
</tr>
<tr>
<td>(k) Connections to other materials (Bearing)</td>
<td>0.60</td>
</tr>
</tbody>
</table>
A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, $F_y$, shall not exceed the specified minimum yield point of steels as listed in Section A3.1 or A3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

Strength increase from cold work of forming shall be permitted by substituting $F_{ya}$ for $F_y$, where $F_{ya}$ is the average yield point of the full section. Such increase shall be limited to Sections C3.1 (excluding Section C3.1.1(b)), C4, C5, C6 and D4. The limitations and methods for determining $F_{ya}$ are as follows:

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity $p$ for load capacity is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, $F_{ya}$, of the steel shall be determined on the basis of one of the following methods:

1. full section tensile tests [see paragraph (a) of Section F3.1]
2. stub column tests [see paragraph (b) of Section F3.1]
3. computed as follows:

$$F_{ya} = CF_{yc} + (1 - C) F_{yt} \quad (Eq. \text{ A5.2.2-1})$$

where

- $F_{ya}$ = Average yield point of the steel in the full section of compression members or full flange sections of flexural members
- $C$ = For compression members, ratio of the total corner cross-sectional area to the total cross-sectional area of the full section; for flexural members, ratio of the total corner cross-sectional area of the controlling flange to the full cross-sectional area of the controlling flange
- $F_{yt}$ = Weighted average tensile yield point of the flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made

$$F_{yc} = B_{c} F_{yv} / (R/t)^m, \text{ tensile yield point of corners. This formula is applicable only when } F_{uv}/F_{yv} \geq 1.2, R/t \leq 7, \text{ and minimum included angle } \geq 120^\circ \quad (Eq. \text{ A5.2.2-2})$$

$$B_{c} = 3.69 \left( F_{uv}/F_{yv} \right) - 0.819 \left( F_{uv}/F_{yv} \right)^2 - 1.79 \quad (Eq. \text{ A5.2.2-3})$$

$$m = 0.192 \left( F_{uv}/F_{yv} \right) - 0.068 \quad (Eq. \text{ A5.2.2-4})$$

$R$ = Inside bend radius.

$$F_{yv} = \text{Tensile yield point of virgin steel* specified by Section A3 or established in accordance with Section F3.3}$$

$$F_{uv} = \text{Ultimate tensile strength of virgin steel* specified by Section A3 or established in accordance with Section F3.3}$$

(b) For axially loaded tension members the yield point of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this Section.

* Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold-forming operation.
(c) The effect of any welding on mechanical properties of a member shall be determined on the basis of tests of full section specimens containing within the gage length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

A5.3 Durability

A structure shall be designed to perform its required functions during its expected life for durability considerations.

A6 Reference Documents

The following documents are referenced in this Specification:


   ASTM A36/A36M-84a, Structural Steel
   ASTM A194-88, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
   ASTM A242/A242M-85, High-Strength Low-Alloy Structural Steel
   ASTM A307-84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners
   ASTM A325-84, High Strength Bolts for Structural Steel Joints
   ASTM A354-84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 inch)
   ASTM A370-77 Mechanical Testing of Steel Products
   ASTM A441M-85, High-Strength Low-Alloy Structural Manganese Vanadium Steel
   ASTM A446/A446M-85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality
   ASTM A449-84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 1/2 inch)
ASTM A490–84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.
ASTM A500–84, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A529/A529M–85, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)
ASTM A563–88a, Carbon and Alloy Steel Nuts
ASTM A570/A570M–85 Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality
ASTM A572/A572M–85, High-Strength Low-Alloy Columbium–Vanadium Steels of Structural Quality
ASTM A588/A588M–85, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick
ASTM A606–85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A607–85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled
ASTM A611–85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality
ASTM A715–85 (Grades 50 & 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability
ASTM A792–85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum–Zinc Alloy–Coated by the Hot-Dip Process, General Requirements
ASTM F436–86, Hardened Steel Washers
ASTM F844–83(1988), Washers, Steel, Plain (Flat), Unhardened for General Use
ASTM F959–85, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners
B. ELEMENTS

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) Maximum Flat-Width-to-Thickness Ratios
   Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding interme-
   diate stiffeners and taking as t the actual thickness of the element, shall be as fol-
   lows:

   (1) Stiffened compression element having one longitudinal edge connected to a
       web or flange element, the other stiffened by:

   Simple lip 60
   Any other kind of stiffener
   having Is > Ia and D/w < 0.8 90
   according to Section B4.2

   (2) Stiffened compression element
       with both longitudinal
       edges connected to other
       stiffened elements 500

   (3) Unstiffened compression element
       and elements with an edge stiffener having
       Is < Ia and D/w ≤ 0.8 according
       to Section B4.2 60

   Note: Unstiffened compression elements that have w/t ratios exceeding approximately
   30 and stiffened compression elements that have w/t ratios exceeding approximately
   250 are likely to develop noticeable deformation at the full design strength, without affecting
   the ability of the member to develop required strength.

   Stiffened elements having w/t ratios larger than 500 can be used with adequate
   design strength to sustain the required loads; however, substantial deformations
   of such elements usually will invalidate the design formulas of this Specifica-
   tion.

   (b) Flange Curling
   Where the flange of a flexural member is unusually wide and it is desired to limit the
   maximum amount of curling or movement of the flange toward the neutral axis, the
   following formula applies to compression and tension flanges, either stiffened or
   unstiffened:

   \[ w_f = \sqrt{0.061d E/f_w^4(100c_t/d)} \]  \hspace{1cm} (Eq. B1.1-1)

   where
   \[ w_f = \text{Width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams} \]
   \[ t = \text{Flange thickness} \]
   \[ d = \text{Depth of beam} \]
\( f = \text{Amount of curling}\)

\( f_{av} = \text{Average stress in the full, unreduced flange width. (Where members are designed by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)}\)

(c) **Shear Lag Effects – Short Spans Supporting Concentrated Loads**

Where the span of the beam is less than \(30w_f\) (\(w_f\) as defined below) and it carries one concentrated load, or several loads spaced farther apart than \(2w_f\), the effective design width of any flange, whether in tension or compression, shall be limited to the following:

### TABLE B1.1(c)

**SHORT, WIDE FLANGES**

MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

<table>
<thead>
<tr>
<th>(L/w_f)</th>
<th>Ratio</th>
<th>(L/w_f)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>1.00</td>
<td>14</td>
<td>0.82</td>
</tr>
<tr>
<td>25</td>
<td>0.96</td>
<td>12</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>0.91</td>
<td>10</td>
<td>0.73</td>
</tr>
<tr>
<td>18</td>
<td>0.89</td>
<td>8</td>
<td>0.67</td>
</tr>
<tr>
<td>16</td>
<td>0.86</td>
<td>6</td>
<td>0.55</td>
</tr>
</tbody>
</table>

where

\( L = \) Full span for simple beams; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

\( w_f = \) Width of flange projection beyond the web for I-beam and similar sections or half the distance between webs of box or U-type sections.

For flanges of I-beams and similar sections stiffened by lips at the outer edges, \(w_f\) shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

### B1.2 Maximum Web Depth-to-Thickness Ratio

The ratio, \(h/t\), of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: \((h/t)_{max} = 200\)

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:

(1) When using bearing stiffeners only, \((h/t)_{max} = 260\)

(2) When using bearing stiffeners and intermediate stiffeners, \((h/t)_{max} = 300\)

In the above,

\( h = \text{Depth of flat portion of web measured along the plane of web}\)

*The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 percent of the depth of the section is usually not considered excessive.*
t = Web thickness
Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) Load Capacity Determination

The effective widths, b, of uniformly compressed elements shall be determined from the following formulas:

\[ b = w \quad \text{when } \lambda \leq 0.673 \]  \hspace{1cm} (Eq. B2.1-1)

\[ b = \rho w \quad \text{when } \lambda > 0.673 \]  \hspace{1cm} (Eq. B2.1-2)

where

\[ w = \text{Flat width as shown in Figure B2.1-1} \]

\[ \rho = (1 - 0.22/\lambda )/\lambda \]  \hspace{1cm} (Eq. B2.1-3)

\[ \lambda \text{ is a slenderness factor determined as follows:} \]

\[ \lambda = \frac{1.052 (w) \sqrt{\frac{f}{E}}}{k (1 + \frac{1}{E})} \]  \hspace{1cm} (Eq. B2.1-4)

where

\[ t = \text{Thickness of the uniformly compressed stiffened elements, and} \]

\[ f \text{ for load capacity determination is as follows:} \]

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, \( f = F_y \) if the initial yielding is in compression in the element considered.

If the initial yielding is in tension, the compressive stress, \( f \), in the element considered shall be determined on the basis of the effective section at \( M_y \) (moment causing initial yield).

(2) If Procedure II of Section C3.1.1 is used then \( f \) is the stress in the element considered at \( M_n \) determined on the basis of the effective section.

(3) If Section C3.1.2 is used, then \( f \) is the stress \( \frac{M_{xe}}{S_t} \) as described in that Section in determining \( S_e \).

For compression members \( f \) is taken equal to \( F_n \) as determined in Section C4 or D4.1 as applicable.

\[ E = \text{Modulus of elasticity} \]

\[ k = \text{Plate buckling coefficient} \]

\[ = 4 \text{ for stiffened elements supported by a web on each longitudinal edge.} \]

Values for different types of elements are given in the applicable sections.

(b) Deflection Determination

The effective widths, \( b_d \), used in computing deflection shall be determined from the following formulas:

\[ b_d = w \quad \text{when } \lambda \leq 0.673 \]  \hspace{1cm} (Eq. B2.1-5)

\[ b_d = \rho w \quad \text{when } \lambda > 0.673 \]  \hspace{1cm} (Eq. B2.1-6)

where
Figure B2.1–1 Stiffened Elements

\( w = \) Flat width
\( \rho = \) Reduction factor determined by either of the following two procedures:

1. **Procedure I.**
   A low estimate of the effective width may be obtained from Eqs. B2.1–3 and B2.1–4 where \( f_d \) is substituted for \( f \) where \( f_d \) is the computed compressive stress in the element being considered.

2. **Procedure II.**
   For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating \( \rho \) as follows:

   \[
   \rho = 1 \quad \text{when} \quad \lambda \leq 0.673 \\
   \rho = (1.358 - 0.461/\lambda)/\lambda \quad \text{when} \quad 0.673 < \lambda < \lambda_c \\
   \rho = (0.41 + 0.59 \sqrt{F_y / f_d} - 0.22/\lambda)/\lambda \quad \text{when} \quad \lambda \geq \lambda_c \\
   \rho \text{ shall not exceed } 1.0 \text{ for all cases.}
   \]

   where

   \[
   \lambda_c = 0.256 + 0.328 (w/t) \sqrt{F_y / E}
   \]

   and \( \lambda \) is as defined by Eq. B2.1–4 except that \( f_d \) is substituted for \( f \).

**B2.2 Uniformly Compressed Stiffened Elements with Circular Holes**

(a) **Load Capacity Determination**

The effective width, \( b \), of stiffened elements with uniform compression having circular holes shall be determined as follows:

for \( 0.50 \geq \frac{d_h}{w} \geq 0, \) and \( \frac{w}{t} \leq 70 \)

center-to-center spacing of holes > \( 0.50w \), and \( 3d_h \),

\[
\begin{align*}
   b &= w - d_h \quad \text{when} \quad \lambda \leq 0.673 \\
   b &= \frac{w \left[ 1 - (0.22/\lambda) - (0.8d_h/w) \right]}{\lambda} \quad \text{when} \quad \lambda > 0.673
\end{align*}
\]

\( b \) shall not exceed \( w - d_h \)

where
\( w = \text{Flat width} \)
\( d_h = \text{Diameter of holes} \)
\( \lambda \) is as defined in Section B2.1.

(b) **Deflection Determination**

The effective width, \( b_e \), used in deflection calculations shall be equal to \( b \) determined in accordance with Procedure I of Section B2.2a except that \( f_d \) is substituted for \( f \), where \( f_d \) is the computed compressive stress in the element being considered.

### B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient

(a) **Load Capacity Determination**

The effective widths, \( b_1 \) and \( b_2 \), as shown in Figure B2.3–1 shall be determined from the following formulas:

\[
\begin{align*}
\text{For } \psi & \leq -0.236 \\
\quad b_1 & = b_e/(3 - \psi) \\
\quad b_2 & = b_e/2 \\
\text{For } \psi & > -0.236 \\
\quad b_2 & = b_e - b_1
\end{align*}
\]

where

\[
\begin{align*}
\psi & = f_2/f_1 \\
f_1, f_2 & = \text{Stresses shown in Figure B2.3–1 calculated on the basis of effective section.} \\
f_1 & = \text{Compression (+)} \text{ and } f_2 \text{ can be either tension (-) or compression. In case } f_1 \\
& \text{and } f_2 \text{ are both compression, } f_1 \geq f_2
\end{align*}
\]

(b) **Deflection Determination**

The effective widths in computing deflections at a given load shall be determined in accordance with Procedure I of Section B2.3a except that \( f_{d1} \) and \( f_{d2} \) are substituted for \( f_1 \) and \( f_2 \), where \( f_{d1}, f_{d2} = \text{Computed stresses } f_1 \text{ and } f_2 \) as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.

### B3 Effective Widths of Unstiffened Elements

#### B3.1 Uniformly Compressed Unstiffened Elements

(a) **Load Capacity Determination**

Effective widths, \( b \), of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that \( k \) shall be taken as 0.43 and \( w \) as defined in Figure B3.1–1.

(b) **Deflection Determination**

The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that \( f_d \) is substituted for \( f \) and \( k = 0.43 \).
B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) Load Capacity Determination
Effective widths, \( b \), of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with \( f = f_3 \) as in Figure B4-2 in the element and \( k = 0.43 \).

(b) Deflection Determination
Effective widths, \( b \), of unstiffened compression elements and edge stiffeners with
stress gradient shall be determined in accordance with Procedure I of Section B2.1b except that $f_d$ is substituted for $f$ and $k = 0.43$.

**B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener**

The following notation is used in this section.

- $S = 1.28 \sqrt{E/f}$ (Eq. B4-1)
- $k = $ Buckling coefficient
- $b_o = $ Dimension defined in Figure B4-1
- $d, w, D = $ Dimensions defined in Figure B4-2
- $d_s = $ Reduced effective width of the stiffener as specified in this section. $d_s$, calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4-2)
- $d'_s = $ Effective width of the stiffener calculated according to Section B3.1 (see Figure B4-2)
- $C_1, C_2 = $ Coefficients defined in Figure B4-2
- $A_s = $ Reduced area of the stiffener as specified in this section. $A_s$ is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.
- $I_s = $ Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.
- $I_s, A'_s = $ Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure B4-2:

- $I_s = (d^2 t \sin \theta)/12$ (Eq. B4-2)
- $A'_s = d'_s t$ (Eq. B4-3)

### B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) **Load Capacity Determination**

**Case I:** $b_o/t \leq S$

- $I_s = 0$ (no intermediate stiffener needed)
- $b = w$
- $A_s = A'_s$

**Case II:** $S < b_o/t < 3S$

- $l_o/t^4 = [50(b_o/t)/S] - 50$
- $b$ and $A_s$ are calculated according to Section B2.1a where $k = 3(l_o/I_s)^{1/3} + 1 \leq 4$
- $A_s/A'_s(l_o/I_s) \leq A'_s$

**Case III:** $b_o/t \geq 3S$

- $l_o/t^4 = [128(b_o/t)/S] - 285$
- $b$ and $A_s$ are calculated according to Section B2.1a where $k = 3(l_o/I_s)^{1/3} + 1 \leq 4$
- $A_s/A'_s(l_o/I_s) \leq A'_s$

(b) **Deflection Determination**

Effective widths shall be determined as in Section B4.1a except that $f_d$ is substituted for $f$. 

**Case I:** $b_o/t \leq S$

- $I_s = 0$ (no intermediate stiffener needed)
- $b = w$
- $A_s = A'_s$

**Case II:** $S < b_o/t < 3S$

- $l_o/t^4 = [50(b_o/t)/S] - 50$
- $b$ and $A_s$ are calculated according to Section B2.1a where $k = 3(l_o/I_s)^{1/3} + 1 \leq 4$
- $A_s/A'_s(l_o/I_s) \leq A'_s$

**Case III:** $b_o/t \geq 3S$

- $l_o/t^4 = [128(b_o/t)/S] - 285$
- $b$ and $A_s$ are calculated according to Section B2.1a where $k = 3(l_o/I_s)^{1/3} + 1 \leq 4$
- $A_s/A'_s(l_o/I_s) \leq A'_s$
B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) Load Capacity Determination

Case I: $w/t < S/3$

Case II: $S/3 < w/t < S$

Case III: $w/t > S$

(b) Deflection Determination

Effective widths shall be determined as in Section B4.2a except that $f_o$ is substituted for $f$. 
Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

For the determination of the effective width, the intermediate stiffener of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener shall be disregarded unless each intermediate stiffener has the minimum $I_s$ as follows:

$$I_{\text{min}} = \left[ 3.66 \sqrt{\left( \frac{w}{t} \right)^2 - \left( \frac{0.136E}{F_y} \right)^2} \right] t^4$$

but not less than $18.4 \ t^4$  \hfill (Eq. B5-1)

where

- $w/t$ = Width–thickness ratio of the larger stiffened sub-element
- $I_s$ = Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat–width to thickness ratio of the entire multiple–stiffened element, such element shall be considered as replaced by an “equivalent element” without intermediate stiffeners whose width, $b_0$, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, $t_s$, is determined as fol-
where

\[ t_s = \frac{3}{12I_{sf}} / b_o \]  

(Eq. B5–2)

For computing the effective structural properties of a member having compression sub-elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For 60 < \( w/t < 90 \):

\[ A_{ef} = \alpha A_{st} \]  

(Eq. B5–4)

where

\[ \alpha = (3 - 2b_e / w) - \frac{1}{30} \left[ 1 - \frac{b_e}{w} \right] \left[ \frac{w}{t} \right] \]  

(Eq. B5–5)

For \( w/t \geq 90 \):

\[ A_{ef} = \left( b_e / w \right) A_{st} \]  

(Eq. B5–6)

In the above expressions, \( A_{ef} \) and \( A_{st} \) refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

**B6 Stiffeners**

**B6.1 Transverse Stiffeners**

Transverse stiffeners attached to beam webs at points of concentrated loads or reactions, shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E. Required strengths for the concentrated loads or reactions shall not exceed the design strength, \( \phi_e P_n \), where \( \phi_e = 0.85 \) and \( P_n \) is the smaller value given by (a) and (b) as follows:
(a) \( P_n = F_{w}A_c \) \hspace{1cm} (Eq. B6.1-1)

(b) \( P_n \) = Nominal axial strength evaluated according to Section C4(a) with \( A_c \) replaced by \( A_b \)

where

\[ A_c = 18t^2 + A_s, \] \hspace{1cm} (Eq. B6.1-2)

\[ A_c = 10t^2 + A_s, \] \hspace{1cm} (Eq. B6.1-3)

\[ F_{w} = \text{Lower value of beam web, } F_y \text{ or stiffener section, } F_{ys} \]

\[ A_b = b_1t + A_s, \] \hspace{1cm} (Eq. B6.1-4)

\[ A_b = b_2t + A_s, \] \hspace{1cm} (Eq. B6.1-5)

\[ A_s = \text{Cross sectional area of transverse stiffeners} \]

\[ b_1 = 25t \left[ 0.0024(L_s/t) + 0.72 \right] \leq 25t \] \hspace{1cm} (Eq. B6.1-6)

\[ b_2 = 12t \left[ 0.0044(L_s/t) + 0.83 \right] \leq 12t \] \hspace{1cm} (Eq. B6.1-7)

\[ L_s = \text{Length of transverse stiffener} \]

\[ t = \text{Base thickness of beam web} \]

The width-to-thickness ratio for the stiffened and unstiffened elements of cold-formed steel transverse stiffeners shall not exceed 1.28 \( \sqrt{\left( E/F_{ys} \right)} \) and 0.37 \( \sqrt{\left( E/F_{ys} \right)} \), respectively, where \( F_{ys} \) is the yield stress, \( F_y \), and \( t \) is the thickness of the stiffener steel.

### B6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the required shear strength shall not exceed the design shear strength, \( \phi V_n \), permitted by Section C3.2, and the ratio \( a/h \) shall not exceed \( \left[ 260/(h/t) \right]^2 \) nor 3.0.

The actual moment of inertia, \( I_s \), of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value of

\[ I_{s,\text{min}} = 5ht^3 \left[ h/a - 0.7(a/h) \right] \geq (h/50)^4 \] \hspace{1cm} (Eq. B6.2-1)

The gross area of shear stiffeners shall be not less than

\[ A_s = \frac{1 - C_v}{2} \left[ \frac{a}{h} - \frac{(a/h)^2}{(a/h) + \sqrt{1 + (a/h)^2}} \right] YDht \] \hspace{1cm} (Eq. B6.2-2)

where

\[ C_v = \frac{45,000k_v}{F_y(h/t)^2} \text{ when } C_v \leq 0.8 \] \hspace{1cm} (Eq. B6.2-3)

\[ C_v = \frac{190}{h/t} \left( \frac{k_v}{F_y} \right) \text{ when } C_v > 0.8 \] \hspace{1cm} (Eq. B6.2-4)

\[ k_v = 4.00 + \frac{5.34}{(a/h)^2} \text{ when } a/h \leq 1.0 \] \hspace{1cm} (Eq. B6.2-5)

\[ k_v = 5.34 + \frac{4.00}{(a/h)^2} \text{ when } a/h > 1.0 \] \hspace{1cm} (Eq. B6.2-6)

\[ a = \text{Distance between transverse stiffeners} \]

\[ Y = \frac{\text{Yield point of web steel}}{\text{Yield point of stiffener steel}} \]

\[ D = 1.0 \text{ for stiffeners furnished in pairs} \]
$D = 1.8$ for single-angle stiffeners
$D = 2.4$ for single-plate stiffeners
$t$ and $h$ are as defined in Section B1.2

### B6.3 Non-Conforming Stiffeners

The design strength of members with transverse stiffeners that do not meet the requirements of Section B6.1 or B6.2, such as stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Chapter F of this Specification.
C. MEMBERS

C1 Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the design tensile strength, $\phi T_n$, shall be determined as follows:

$$\phi T_n = 0.95 T_n = A_n F_y$$

where
- $T_n$ = Nominal strength of member when loaded in tension
- $\phi$ = Resistance factor for tension
- $A_n$ = Net area of the cross section
- $F_y$ = Design yield stress as determined in Section A5.2.1

For tension members using bolted connections, the design tensile strength shall also be limited by Section E3.2.

C3 Flexural Members

C3.1 Strength for Bending Only

The design flexural strength, $\phi_b M_n$, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, and C3.1.3.

C3.1.1 Nominal Section Strength

The design flexural strength, $\phi_b M_n$, shall be determined with $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and the nominal section strength, $M_n$, calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

(a) Procedure I – Based on Initiation of Yielding

Effective yield moment based on section strength, $M_n$, shall be determined as follows:

$$M_n = S_e F_y$$

where
- $F_y$ = Design yield stress as determined in Section A5.2.1
- $S_e$ = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at $F_y$

(b) Procedure II – Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity may be used when the following conditions are met:
(1) The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.
(2) The effect of cold forming is not included in determining the yield point \( F_y \).
(3) The ratio of the depth of the compressed portion of the web to its thickness does not exceed \( \lambda_1 \).
(4) The shear force does not exceed 0.35\( F_y \) times the web area, \( h_t \).
(5) The angle between any web and the vertical does not exceed 30 degrees.

The nominal flexural strength, \( M_n \), shall not exceed either 1.25 \( S_e F_y \) determined according to Procedure I or that causing a maximum compression strain of \( C_y \varepsilon_y \) (no limit is placed on the maximum tensile strain).

\[
\text{where}
\]
\[ e_y = \text{Yield strain} = \frac{F_y}{E}
\]
\[ E = \text{Modulus of elasticity}
\]
\[ C_y = \text{Compression strain factor determined as follows:}
\]
(a) Stiffened compression elements without intermediate stiffeners
\[ C_y = 3 \text{ for } w/t \leq \lambda_1
\]
\[ C_y = 3 - 2 \left( \frac{w/t - \lambda_1}{\lambda_2 - \lambda_1} \right) \text{ for } \lambda_1 < \frac{w}{t} < \lambda_2
\]
\[ C_y = 1 \text{ for } w/t \geq \lambda_2
\]

where
\[
\lambda_1 = \frac{1.11}{\sqrt{\frac{F_y}{E}}}
\]
\[
\lambda_2 = \frac{1.28}{\sqrt{\frac{F_y}{E}}}
\]

(b) Unstiffened compression elements
\[ C_y = 1
\]

(c) Multiple-stiffened compression elements and compression elements with edge stiffeners
\[ C_y = 1
\]

When applicable, effective design widths shall be used in calculating section properties. \( M_n \) shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

The design strength of the laterally unbraced segments of singly-, doubly-, and point-symmetric sections* subject to lateral buckling, \( \phi_b M_n \) shall be determined with \( \phi_b = 0.90 \) and \( M_n \) calculated as follows:

\[
M_n = S_e \frac{M_c}{S_t}
\]

where

* The provisions of this Section apply to I-, Z-, C- and other singly-symmetric section flexural members (not including multiple-web deck, U- and closed box-type members, and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections. Refer to C3.1.3 for C- and Z-purlins in which the tension flange is attached to sheathing.
Sf = Elastic section modulus of the full unreduced section for the extreme compression fiber
Se = Elastic section modulus of the effective section calculated at a stress Me / Sf in the extreme compression fiber
Me = Critical moment calculated according to (a) or (b) below:

(a) For singly-, doubly-, and point-symmetric sections:

For Me > 0.5My

\[ Me = My \left(1 - \frac{My}{4Me}\right) \]  
\[ (Eq. C3.1.2-2) \]

For Me ≤ 0.5My

\[ Me = Me \]  
\[ (Eq. C3.1.2-3) \]

where

My = Moment causing initial yield at the extreme compression fiber of the full section
= Sf Fy  
\[ (Eq. C3.1.2-4) \]

Me = Elastic critical moment computed by the following equations:

\[ Me = C_s A \sqrt{\sigma_{ex}} \sigma_i \] for bending about the symmetry axis. For singly-symmetric sections, x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate.

For point-symmetric sections, use 0.5 Me.

Alternatively, Me can be calculated using the formula for doubly-symmetric I-sections or point-symmetric sections given in (b)

\[ Me = C_s A \sigma_{ex} \left[ j + C_s \sqrt{j^2 + r_s^2 (\sigma_i / \sigma_{ex})} \right] / C_T \] for bending about the centroidal axis perpendicular to the symmetry axis for singly-symmetric sections only

C_s = +1 for moment causing compression on the shear center side of the centroid
C_s = −1 for moment causing tension on the shear center side of the centroid

\[ \sigma_{ex} = \frac{\pi^2 E}{(K_s L_s / r_s)^2} \]  
\[ (Eq. C3.1.2-7) \]

\[ \sigma_{ey} = \frac{\pi^2 E}{(K_s L_y / r_y)^2} \]  
\[ (Eq. C3.1.2-8) \]

\[ \sigma_i = \frac{1}{A r_o^2} \left[ GJ + \frac{\pi^2 E C_w}{(K_i L_i)^2} \right] \]  
\[ (Eq. C3.1.2-9) \]

A = Full cross-sectional area
C_b = Bending coefficient which can conservatively be taken as unity, or calculated from

\[ C_b = 1.75 + 1.05(M_1/M_2) + 0.3 (M_1/M_2)^2 \leq 2.3 \]

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive.
when $M_1$ and $M_2$ have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), $C_b$ shall be taken as unity.

$$E = \text{Modulus of elasticity}$$

$$C_{TF} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right)$$

where $M_1$ is the smaller and $M_2$ the larger bending moment at the ends of the unbraced length, and where $M_1/M_2$, the ratio of end moments, is positive when $M_1$ and $M_2$ have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), $C_{TF}$ shall be taken as unity.

$$r_0 = \text{Polar radius of gyration of the cross section about the shear center}$$

$$r_x, r_y = \text{Radii of gyration of the cross section about the centroidal principal axes}$$

$$G = \text{Shear modulus}$$

$$K_x, K_y, K_t = \text{Effective length factors for bending about the x- and y-axes, and for twisting}$$

$$L_x, L_y, L_t = \text{Unbraced length of compression member for bending about the x- and y-axes, and for twisting}$$

$$x_0 = \text{Distance from the shear center to the centroid along the principal x-axis, taken as negative}$$

$$J = \text{St. Venant torsion constant of the cross section}$$

$$C_w = \text{Torsional warping constant of the cross section}$$

$$j = \frac{1}{21_y} \left[ \int_A x^2 dA + \int_A xy^2 dA \right] - x_0$$

(b) For I- or Z-sections bent about the centroidal axis perpendicular to the web (x-axis):

In lieu of (a), the following equations may be used to evaluate $M_e$:

For $M_e \geq 2.78M_y$

$$M_e = M_y$$

(Eq. C3.1.2-12)

For $2.78M_y > M_e > 0.56M_y$

$$M_e = \frac{10}{9} M_y \left( 1 - \frac{10M_y}{36M_e} \right)$$

(Eq. C3.1.2-13)

For $M_e \leq 0.56M_y$

$$M_e = M_e$$

(Eq. C3.1.2-14)
Me = Elastic critical moment determined either as defined in (a) above or as follows:

\[
\frac{\pi^2 E c_d I_{yc}}{L^2} \quad \text{for doubly-symmetric I-sections}
\]

\[
\frac{\pi^2 E c_d I_{yc}}{2L^2} \quad \text{for point-symmetric Z-sections}
\]

where:
- d = Depth of section
- L = Unbraced length of the member
- I_{yc} = Moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, using the full unreduced section

Other terms are defined in (a).

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

This section does not apply to a continuous beam for the region between inflection points adjacent to a support, or to a cantilever beam.

The design flexural strength, \( \phi_b M_{n} \), of a Channel or Z-section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be determined with \( \phi_b = 0.90 \) and the nominal flexural strength, \( M_n \), calculated as follows:

\[
M_n = R S_e F_y
\]

(Eq. C3.1.3-1)

where

- \( R \) = 0.40 for simple span C sections
- = 0.50 for simple span Z sections
- = 0.60 for continuous span C sections
- = 0.70 for continuous span Z sections

\( S_e \) and \( F_y \) are defined in Section C3.1.1

The reduction factor, \( R \), shall be limited to roof and wall systems meeting the following conditions:

1. Member depth less than 11.5 inches
2. The flanges are edge stiffened compression elements
3. \( 60 \leq \text{depth/thickness} \leq 170 \)
4. \( 2.8 \leq \text{depth/flange width} \leq 4.5 \)
5. \( 16 \leq \text{flat width/thickness of flange} \leq 43 \)
6. For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) shall not be less than:
   - 1.5d for Zee sections
   - 3.0d for Channel sections
7. Member span length no greater than 33 feet
8. For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
9. Both flanges are prevented from moving laterally at the supports
10. Roof or wall panels shall be steel sheets, minimum of 0.019 in. coated thickness, having a minimum rib depth of 1 in., spaced a maximum of 12 in. on
centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange

(11) Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used

(12) Fastener type: minimum No. 12 self-drilling or self-tapping sheet metal screws or 3/16 - in. rivets, washers 1/2 in. diameter

(13) Fasteners shall not be standoff type screws

(14) Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange

If variables fall outside any of the above stated limits, the user must perform full scale tests in accordance with Section F1 of the Specification, or apply another rational analysis procedure. In any case, the user is permitted to perform tests, in accordance with Section F1, as an alternate to the procedure described in this section.

C3.2 Strength for Shear Only

The design shear strength, $V_n$, at any section shall be calculated as follows:

(a) For $h/t \leq \sqrt{E_kv / F_y}$
   
   $\phi_v = 1.0$
   
   $V_n = 0.577F_yht$  
   \(\text{(Eq. C3.2-1)}\)

(b) For $\sqrt{E_kv / F_y} < h/t \leq 1.415\sqrt{E_kv / F_y}$
   
   $\phi_v = 0.90$
   
   $V_n = 0.64t^2 \sqrt{k_vF_yE}$  
   \(\text{(Eq. C3.2-2)}\)

(c) For $h/t > 1.415\sqrt{E_kv / F_y}$
   
   $\phi_v = 0.90$
   
   $V_n = 0.905E_kvt^3/h$  
   \(\text{(Eq. C3-2.3)}\)

where

$\phi_v$ = Resistance factor for shear

$V_n$ = Nominal shear strength of beam

$t$ = Web thickness

$h$ = Depth of the flat portion of the web measured along the plane of the web

$k_v$ = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \leq 1.0$

$k_v = 4.00 + \frac{5.34}{(a/h)^2}$  
\(\text{(Eq. C3.2-4)}\)

when $a/h > 1.0$

$k_v = 5.34 + \frac{4.00}{(a/h)^2}$  
\(\text{(Eq. C3.2-5)}\)

where

$a$ = the shear panel length for unreinforced web element

$= distance between transverse stiffeners for reinforced web elements.
For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the required flexural strength, \( M_u \), and the required shear strength, \( V_u \), shall satisfy the following interaction equation:

\[
\left( \frac{M_u}{\phi_b M_{n xo}} \right)^2 + \left( \frac{V_u}{\phi_s V_n} \right)^2 \leq 1.0
\]  

(Eq. C3.3-1)

For beams with transverse web stiffeners, the required flexural strength, \( M_u \), and the required shear strength, \( V_u \), shall not exceed \( \phi_b M_n \) and \( \phi_s V_n \), respectively. When \( M_u/(\phi_b M_{n xo}) > 0.5 \) and \( V_u/(\phi_s V_n) > 0.7 \), then \( M_u \) and \( V_u \) shall satisfy the following interaction equation:

\[
0.6 \left( \frac{M_u}{\phi_b M_{n xo}} \right) + \left( \frac{V_u}{\phi_s V_n} \right) \leq 1.3
\]  

(Eq. C3.3-2)

In the above:
- \( \phi_b \) = Resistance factor for bending (See Section C3.1)
- \( \phi_s \) = Resistance factor for shear (See Section C3.2)
- \( M_n \) = Nominal flexural strength when bending alone exists
- \( M_{n xo} \) = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2
- \( V_n \) = Nominal shear strength when shear alone exists

C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, and in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, \( h/t \), equal to or less than 200, the required strength for the concentrated loads and reactions shall not exceed the values of \( \phi_w P_n \), with \( \phi_w = 0.75 \) and 0.80 for single unreinforced webs and I-sections, respectively, and \( P_n \) given in Table C3.4-1. Webs of flexural members for which \( h/t \) is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4-1 apply to beams when \( R/t \leq 6 \) and to deck when \( R/t \leq 7 \), \( N/t \leq 210 \) and \( N/h \leq 3.5 \).

\( P_n \) represents the nominal strength for concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, \( P_n \) shall be computed for each individual web and the results added to obtain the nominal load or reaction for the multiple web.

For built-up I-sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practical.
<table>
<thead>
<tr>
<th>Opposing Loads</th>
<th>Shapes Having Single Webs</th>
<th>I-Sections or Similar Sections&lt;sup&gt;(1)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spaced &gt; 1.5h&lt;sub&gt;(2)&lt;/sub&gt;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>End Reaction&lt;sup&gt;(3)&lt;/sup&gt;</td>
<td>Eq. C3.4–1</td>
</tr>
<tr>
<td></td>
<td>Interior Reaction&lt;sup&gt;(4)&lt;/sup&gt;</td>
<td>Eq. C3.4–4</td>
</tr>
<tr>
<td>Spaced ≤ 1.5h&lt;sub&gt;(5)&lt;/sub&gt;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>End Reaction&lt;sup&gt;(3)&lt;/sup&gt;</td>
<td>Eq. C3.4–6</td>
</tr>
<tr>
<td></td>
<td>Interior Reaction&lt;sup&gt;(4)&lt;/sup&gt;</td>
<td>Eq. C3.4–8</td>
</tr>
</tbody>
</table>

Footnotes and Equation References to Table C3.4–1:

1. I-sections made of two channels connected back to back or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel).

2. At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

3. For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

4. For reactions and concentrated loads when the distance from the edge of the bearing to the end of the beam is equal to or greater than 1.5h.

5. At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

Equations for Table C3.4–1:

\[
t^2 \phi_\omega C_3 C_4 C_6 \left[ 331 - 0.61 \left( \frac{h}{t} \right) \right] \left[ 1 + 0.01 \left( \frac{N}{t} \right) \right]
\]

\[
t^2 \phi_\omega C_3 C_4 C_6 \left[ 217 - 0.28 \left( \frac{h}{t} \right) \right] \left[ 1 + 0.01 \left( \frac{N}{t} \right) \right]
\]

\[
\text{When } N/t > 60, \text{ the factor } \left[ 1 + 0.01 \left( \frac{N}{t} \right) \right] \text{ may be increased to } \left[ 0.71 + 0.015 \left( \frac{N}{t} \right) \right]
\]

\[
t^2 F_y C_6 \left[ 10.0 + 1.25 \sqrt{\frac{N}{t}} \right]
\]

\[
t^2 F_y C_6 \left[ 538 - 0.74 \left( \frac{h}{t} \right) \right] \left[ 1 + 0.007 \left( \frac{N}{t} \right) \right]
\]

\[
\text{When } N/t > 60, \text{ the factor } \left[ 1 + 0.007 \left( \frac{N}{t} \right) \right] \text{ may be increased to } \left[ 0.75 + 0.011 \left( \frac{N}{t} \right) \right]
\]

\[
t^2 F_y C_6 (0.88 + 0.12m) \left[ 15.0 + 3.25 \sqrt{\frac{N}{t}} \right]
\]

\[
t^2 k C_3 C_4 C_6 \left[ 244 - 0.57 \left( \frac{h}{t} \right) \right] \left[ 1 + 0.01 \left( \frac{N}{t} \right) \right]
\]

\[
t^2 F_y C_6 (0.64 + 0.31m) \left[ 10.0 + 1.25 \sqrt{\frac{N}{t}} \right]
\]

\[
t^2 k C_3 C_4 C_6 \left[ 771 - 2.26 \left( \frac{h}{t} \right) \right] \left[ 1 + 0.0013 \left( \frac{N}{t} \right) \right]
\]

\[
t^2 F_y C_6 (0.82 + 0.15m) \left[ 15.0 + 3.25 \sqrt{\frac{N}{t}} \right]
\]

In the above-referenced formulas:

\[ \phi_\omega = \text{Resistance factor for web crippling} \]

\[ P_n = \text{Nominal strength for concentrated load or reaction per web} \]

\[ C_1 = (1.22 - 0.22k) \]

\[ C_2 = (1.06 - 0.06R/t) \leq 1.0 \]
$C_3 = (1.33 - 0.33k)$  \hspace{1cm} (Eq. C3.4-12)

$C_4 = (1.15 - 0.15R/t) \leq 1.0$ but not less than 0.50 \hspace{1cm} (Eq. C3.4-13)

$C_5 = (1.49 - 0.53k) \geq 0.6$ \hspace{1cm} (Eq. C3.4-14)

$C_6 = 1 + \left( \frac{h}{t} \right) \text{ when } h/t \leq 150$ \hspace{1cm} (Eq. C3.4-15)

$= 1.20$, when $h/t > 150$ \hspace{1cm} (Eq. C3.4-16)

$C_7 = 1/k$, when $h/t \leq 66.5$ \hspace{1cm} (Eq. C3.4-17)

$= \left[ 1.10 - \frac{h}{665} \frac{1}{k} \right] \text{ when } h/t > 66.5$ \hspace{1cm} (Eq. C3.4-18)

$C_8 = 0.98 - \frac{h}{865} \frac{1}{k}$ \hspace{1cm} (Eq. C3.4-19)

$C_9 = 0.7 + 0.3 (\theta/90)^2$ \hspace{1cm} (Eq. C3.4-20)

$F_y = \text{Design yield stress of the web, see Section A5.2.1}$

$h = \text{Depth of the flat portion of the web measured along the plane of the web}$

$k = \frac{F_y}{33}$

$m = \frac{t}{0.075}$ \hspace{1cm} (Eq. C3.4-21)

$t = \text{Web thickness, inches}$

$N = \text{Actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of } N \text{ shall be taken}$

$R = \text{Inside bend radius}$

$\theta = \text{Angle between the plane of the web and the plane of the bearing surface} \geq 45^\circ$, but not more than $90^\circ$

### C3.5 Combined Bending and Web Crippling Strength

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

(a) For shapes having single unreinforced webs:

\[ 1.07 \left( \frac{P_u}{P_n} \right) + \left( \frac{M_u}{M_{n,x}} \right) \leq 1.42 \]  \hspace{1cm} (Eq. C3.5-1)

**Exception:** At the interior supports of continuous spans, the above formula is not applicable to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 inches.

(b) For shapes having multiple unreinforced webs such as I-sections made of two channels connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel):

\[ 0.82 \left( \frac{P_u}{P_n} \right) + \left( \frac{M_u}{M_{n,xo}} \right) \leq 1.32 \]  \hspace{1cm} (Eq. C3.5-2)

**Exception:** When $h/t \leq 2.33/\sqrt{(F_y/E)}$ and $\lambda \leq 0.673$, the nominal concentrated load or reaction strength may be determined by Section C3.4.
In the above formulas:

- $G_b$ = Resistance factor for bending (See Section 3.1)
- $G_w$ = Resistance factor for web crippling (See Section C3.4)
- $P_u$ = Required strength for the concentrated load or reaction in the presence of bending moment
- $P_n$ = Nominal strength for concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4
- $M_u$ = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction $P_u$
- $M_{max}$ = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2
- $w$ = Flat width of the beam flange which contacts the bearing plate
- $t$ = Thickness of the web or flange
- $\lambda$ = Slenderness factor given by Section B2.1

### C4 Concentrically Loaded Compression Members

This section applies to members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the stress, $F_n$, defined in this section.

(a) The design axial strength, $\phi P_n$, shall be calculated as follows:

$$\phi P_n = A_e F_n$$

(Eq. C4-1)

where

$A_e$ = Effective area at the stress $F_n$. For sections with circular holes, $A_e$ shall be determined according to Section B2.2a, subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, $A_e$ can be determined ignoring the holes.

$F_n$ is determined as follows:

- For $F_e > F_y/2$  
  $$F_n = F_y (1 - F_y/4F_e)$$  
  (Eq. C4-2)
- For $F_e \leq F_y/2$  
  $$F_n = F_e$$  
  (Eq. C4-3)

$F_e$ is the least of the elastic flexural, torsional and torsional-flexural buckling stress determined according to Sections C4.1 through C4.3.

(b) For C- and Z-shapes, and single-angle sections with unstiffened flanges, $P_n$ shall be taken as the smaller of $P_n$ calculated above and $P_n$ calculated as follows:

$$P_n = \frac{A\pi^2E}{25.7(w/t)^2}$$

(Eq. C4-4)

where

- $A$ = Area of the full, unreduced cross section
- $w$ = Flat width of the unstiffened element
- $t$ = Thickness of the unstiffened element

(c) Angle sections shall be designed for the required axial strength, $P_u$, acting simultaneously with a moment equal to $P_u L/1000$ applied about the minor principal axis causing compression in the tips of the angle legs.

(d) The slenderness ratio, $KL/r$, of all compression members preferably should not exceed 200, except that during construction only, $KL/r$ preferably should not exceed 300.
C4.1 Sections Not Subject to Torsional or Torsional-Flexural Buckling

For doubly-symmetric sections, closed cross sections and any other sections which can be shown not to be subject to torsional or torsional-flexural buckling, the elastic flexural buckling stress, \( F_e \), shall be determined as follows:

\[
F_e = \pi^2 \frac{E}{(KL / r)^2}
\]

(Eq. C4.1-1)

where
- \( E \) = Modulus of elasticity
- \( K \) = Effective length factor*
- \( L \) = Unbraced length of member
- \( r \) = Radius of gyration of the full, unreduced cross section

C4.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Torsional-Flexural Buckling

For sections subject to torsional or torsional-flexural buckling, \( F_e \) shall be taken as the smaller of \( F_e \) calculated according to Section C4.1 and \( F_e \) calculated as follows:

\[
F_e = \frac{1}{2\beta} \left[ (\sigma_{ex} + \sigma_{t}) - \sqrt{(\sigma_{ex} + \sigma_{t})^2 - 4\beta \sigma_{ex} \sigma_{t}} \right]
\]

(Eq. C4.2-1)

Alternatively, a conservative estimate of \( F_e \) can be obtained using the following equation:

\[
F_e = \frac{\sigma_{t} \sigma_{ex}}{\sigma_{t} + \sigma_{ex}}
\]

(Eq. C4.2-2)

where \( \sigma_t \) and \( \sigma_{ex} \) are as defined in C3.1.2(a):

\[
\beta = 1 - (\lambda / \omega)^2
\]

(Eq. C4.2-3)

For singly-symmetric sections, the \( x \)-axis is assumed to be the axis of symmetry.

C4.3 Nonsymmetric Sections

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, \( F_e \) shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Chapter F.

C5 Combined Axial Load and Bending

The required strengths \( P_u \), \( M_{ux} \), and \( M_{uy} \) shall satisfy the following interaction equations:

\[
\frac{P_u}{\phi_x P_{ne}} + \frac{C_{mx} M_{ux}}{\phi_b m_x \alpha_{ax}} + \frac{C_{my} M_{uy}}{\phi_b M_{ny} \alpha_{ny}} \leq 1.0
\]

(Eq. C5-1)

\[
\frac{P_u}{\phi_x P_{ne}} + \frac{M_{ux}}{\phi_b m_x} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0
\]

(Eq. C5-2)

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, \( K \), for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, shall be taken as unity, unless analysis shows that a smaller value may be used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length, \( KL \), of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.
When $\frac{P_u}{\phi_c P_n} \leq 0.15$, the following formula may be used in lieu of the above two formulas:

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$$

(Eq. C5-3)

where

- $P_u$ = Required axial strength
- $M_{ux}$ and $M_{uy}$ = Required flexural strengths with respect to the centroidal axes of the effective section determined for the required axial strength alone. For angle sections, $M_{uy}$ shall be taken either as the required flexural strength or the required flexural strength plus $P_u L/1000$, whichever results in a lower value of $P_n$.
- $P_n$ = Nominal axial strength determined in accordance with Section C4
- $P_{no}$ = Nominal axial strength determined in accordance with Section C4, with $F_n = F_y$
- $M_{nx}$ and $M_{ny}$ = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3
- $I/\alpha_{nx}, I/\alpha_{ny}$ = Magnification factors

$$= 1 \left[ 1 - \frac{P_u}{\phi_c P_n} \right]$$

(Eq. C5-4)

- $\phi_b$ = 0.95 and 0.90 for bending strength (Section C3.1.1) or 0.90 for laterally unbraced beam (Section C3.1.2)
- $\phi_c$ = 0.85
- $P_E = \frac{\pi^2 E I_b}{(K_b L_b)^2}$

(Eq. C5-5)

- $I_b$ = Moment of inertia of the full, unreduced cross section about the axis of bending
- $L_b$ = Actual unbraced length in the plane of bending
- $K_b$ = Effective length factor in the plane of bending
- $C_{mx}, C_{my}$ = Coefficients whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway)
   $$C_m = 0.85$$

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending
   $$C_m = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right)$$

(Eq. C5-6)

where

$M_1/M_2$ is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. $M_1/M_2$ is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of $C_m$ may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

   (a) for members whose ends are restrained, $C_m = 0.85$,
   (b) for members whose ends are unrestrained, $C_m = 1.0$. 
C6 Cylindrical Tubular Members

The requirements of this Section apply to cylindrical tubular members having a ratio of outside diameter to wall thickness, \( \frac{D}{t} \), not greater than 0.441 \( \frac{E}{F_y} \).

C6.1 Bending

For flexural members, the required flexural strength uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed \( \phi_b M_n \), where \( \phi_b = 0.95 \) and \( M_n \) is calculated as follows:

For \( D/t \leq 0.070 \frac{E}{F_y} \)

\[
M_n = 1.25 F_y S_f \quad (Eq. C6.1-1)
\]

For \( 0.070 \frac{E}{F_y} < D/t \leq 0.319 \frac{E}{F_y} \)

\[
M_n = \left[ 0.970 + 0.020 \left( \frac{E}{F_y} \right) \right] F_y S_f \quad (Eq. C6.1-2)
\]

For \( 0.319 \frac{E}{F_y} < D/t \leq 0.441 \frac{E}{F_y} \)

\[
M_n = \left[ 0.328 \frac{E}{D/t} \right] S_f \quad (Eq. C6.1-3)
\]

where

\( S_f \) = Elastic section modulus of the full, unreduced cross section

C6.2 Compression

The requirements of this Section apply to members in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section.

The design axial strength, \( \phi_c P_n \), shall be calculated as follows:

\[ \phi_c = 0.85 \]

\[ P_n = F_n A_e \quad (Eq. C6.2-1) \]

In the above equation,

For \( F_e > \frac{F_y}{2} \)

\[ F_n = \text{Flexural buckling stress} = F_y \left[ 1 - \frac{F_y}{4F_e} \right] \quad (Eq. C6.2-2) \]

\( F_e \) = The elastic flexural buckling stress determined according to Section C4.1

\[ A_e = \left[ 1 - (1 - R^2)(1 - A_e/A) \right] A \quad (Eq. C6.2-3) \]

\[ R = \sqrt{F_y / 2F_e} \quad (Eq. C6.2-4) \]

\[ A_e = \left[ \frac{0.037}{DF_y} + 0.667 \right] \frac{A}{tE} \quad \text{for} \quad \frac{D}{t} \leq 0.441 \frac{E}{F_y} \quad (Eq. C6.2-5) \]

\( A \) = Area of the unreduced cross section

For \( F_e \leq \frac{F_y}{2} \)

\[ F_n = F_e \]

\[ A_e = A \]

C6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section C5.
D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I - Sections Composed of Two Channels

The maximum permissible longitudinal spacing of welds or other connectors, $s_{\text{max}}$, joining two channels to form an I-section shall be

(a) For compression members:

$$s_{\text{max}} = \frac{L_{r_{cy}}}{2r_1}$$

where:

- $L$ = Unbraced length of compression member
- $r_1$ = Radius of gyration of the I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing
- $r_{cy}$ = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

$$s_{\text{max}} = \frac{L}{6}$$

In no case shall the spacing exceed the value

$$s_{\text{max}} = \frac{2gT_s}{mq}$$

where:

- $L$ = Span of beam
- $T_s$ = Design strength of connection in tension (Section E)
- $g$ = Vertical distance between the two rows of connections nearest to the top and bottom flanges
- $q$ = Intensity of factored load on the beam (For methods of determination, see below)
- $m$ = Distance from the shear center of one channel to the mid-plane of its web. For simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3}$$

For channels with stiffening lips at the outer edges,

$$m = \frac{w_f d t}{4I_s} \left[ w_f d + 2D \left( d - \frac{4D^2}{3d} \right) \right]$$

where:

- $w_f$ = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, $w_f$ shall be taken as the width of the wider flange)
- $d$ = Depth of channel or beam
- $D$ = Overall depth of lip
- $I_s$ = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of factored load, $q$, is obtained by dividing the magnitude of factored concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, $q$ shall be taken equal to three times the intensity of the uniformly distributed factored load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, $s$, the required design strength of the welds or connections closest to the load or reaction is

$$T_s = \frac{P_{r}m}{2g}$$

where $P_{r}$ is a concentrated load or reaction based on factored loads.
The required maximum spacing of connections, \( s_{max} \), depends upon the intensity of the factored load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The design shear strength of the connections joining these plates to the flanges shall then be used for \( T_s \), and \( g \) shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

The spacing, \( s \), in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) \( 1.16t \sqrt{\left( \frac{E}{f_c} \right)} \), where \( t \) is the thickness of the cover plate or sheet, and \( f_c \) is the stress at service load in the cover plate or sheet; nor

(c) three times the flat width, \( w \), of the narrowest unstiffened compression element tributary to the connections, but need not be less than \( 1.11t \sqrt{\left( \frac{E}{F_y} \right)} \) if \( w/t < 0.50 \sqrt{\left( \frac{E}{F_y} \right)} \), or \( 1.33t \sqrt{\left( \frac{E}{F_y} \right)} \) if \( w/t \geq 0.50 \sqrt{\left( \frac{E}{F_y} \right)} \), unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one-half inch. In all other cases, the spacing shall be taken as the center-to-center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

D2 Mixed Systems

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this Specification and the applicable Specification of the other material.

D3 Lateral Bracing

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

D3.2 Channel-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of channels and Z-sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange*, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

* Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.
D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing

For channels and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

Anchored braces need to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof deck and its fastening system. Anchored braces shall be as close as possible to the flange which is connected to the deck or sheathing. Anchored braces shall be provided for each purlin bay.

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z-section assembly is equal to or greater than its nominal flexural strength, instead of that required by Chapter F.

(a) Channel Sections

For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the factored load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z-Sections

For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z-purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one-third points, each brace shall be designed to resist a force determined as follows:

(1) Single-Span System with Restraints at the Supports:

\[ P_L = 0.5 \left[ \frac{0.220b^{1.50}}{n_p^{0.72}d^{0.90}t^{0.60}} - \sin \theta \right] W \]  \hspace{1cm} (Eq. D3.2.1-1)

(2) Single-Span System with Third-Point Restraints:

\[ P_L = 0.5 \left[ \frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin \theta \right] W \]  \hspace{1cm} (Eq. D3.2.1-2)

(3) Single-Span System with Midspan Restraint:

\[ P_L = \left[ \frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}} - \sin \theta \right] W \]  \hspace{1cm} (Eq. D3.2.1-3)

(4) Multiple-Span System with Restraints at the Supports:
with
\[ C_n = \begin{cases} 
0.63 & \text{for braces at end supports of multiple-span systems} \\
0.87 & \text{for braces at the first interior supports} \\
0.81 & \text{for all other braces}
\end{cases} \]

(5) Multiple-Span System with Third-Point Restraints:
\[ P_L = C_{th} \left[ \frac{0.181b^{1.15}L^{0.25}}{n_p^{0.34}d^{1.11}t^{0.29}} - \sin\theta \right] W \]  \hspace{1cm} (Eq. D3.2.1-5)

with
\[ C_{th} = \begin{cases} 
0.57 & \text{for outer braces in exterior spans} \\
0.48 & \text{for all other braces}
\end{cases} \]

(6) Multiple-Span System with Midspan Restraints:
\[ P_L = C_{ms} \left[ \frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}d^{1.00}t^{0.50}} - \sin\theta \right] W \]  \hspace{1cm} (Eq. D3.2.1-6)

with
\[ C_{ms} = \begin{cases} 
1.05 & \text{for braces in exterior spans} \\
0.90 & \text{for all other braces}
\end{cases} \]

where
\[ b = \text{Flange width, in.} \]
\[ d = \text{Depth of section, in.} \]
\[ t = \text{Thickness, in.} \]
\[ L = \text{Span length, in.} \]
\[ \theta = \text{Angle between the vertical and the plane of the web of the Z-section, degrees} \]
\[ n_p = \text{Number of parallel purlin lines} \]
\[ W = \text{Total factored load supported by the purlin lines between adjacent supports, pounds} \]

The force, \( P_L \), is positive when restraint is required to prevent movement of the purlin flanges in the upward roof slope direction.

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1–1 through D3.2.1–6, with \( n_p = 4 \). For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1–1 through D3.2.1–6, with \( n_p = 20 \).

**D3.2.2 Neither Flange Connected to Sheathing**

Each intermediate brace, at the top and bottom flange, shall be designed to resist a required lateral force, \( P_L \), determined as follows:

(a) For uniform loads, \( P_L = 1.5K' \) times the factored load within a distance 0.5a each side of the brace.

(b) For concentrated loads, \( P_L = 1.0K' \) times each concentrated load within a distance 0.3a each side of the brace, plus 1.4K' \( (1-x/a) \) times each factored concentrated load located farther than 0.3a but not farther than 1.0a from the brace.

In the above formulas:
For channels and Z-sections:
- \( x \) = Distance from the concentrated load to the brace
- \( a \) = Distance between center line of braces

For channels:
\[
K' = \frac{m}{d}
\]
where
- \( m \) = Distance from the shear center to the mid-plane of the web, as specified in Section D1.1
- \( d \) = Depth of channel

For Z-sections:
\[
K' = \frac{I_{xy}}{I_x}
\]
where
- \( I_{xy} \) = Product of inertia of the full section about centroidal axes parallel and perpendicular to the web
- \( I_x \) = Moment of inertia of the full section about the centroidal axis perpendicular to the web

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total factored load on the beam is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: When all loads and reactions on a beam are transmitted through members which frame into the section in such a manner as to effectively restrain the section against rotation and lateral displacement, no other braces will be required.

**D3.3 Laterally Unbraced Box Beams**

For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed 0.086 \( E/F_y \).

**D4 Wall Studs and Wall Stud Assemblies**

The design strength of a stud may be computed on the basis of Section C (neglecting sheathing and using steel only) or on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

When sheathing is utilized for stability of the wall studs, the sheathing shall retain adequate strength and stiffness for the expected service life of the wall and additional bracing shall be provided as required for adequate structural integrity during construction and in the completed structure.
The equations given are based on solid-web steel studs and are applicable within the following limits:

- Yield point, $F_y \leq 50$ ksi
- Section depth, $d \leq 6.0$ in.
- Thickness, $t \leq 0.075$ in.
- Overall length, $L \leq 16$ ft.
- Stud spacing, $B$, not less than 12 in. nor greater than 24 in.

Studs with perforations shall be designed using the results of stub column tests and/or rational analysis.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing*, the design axial strength, $\phi_c P_n$, shall be calculated as follows:

$$\phi_c = 0.85$$
$$P_n = A_e F_n$$

where

- $\phi_c$ = Resistance factor for axial compression
- $A_e$ = Effective area determined at $F_n$
- $F_n$ = The lowest value determined by the following three conditions:

(a) To prevent column buckling between fasteners in the plane of the wall, $F_n$ shall be calculated according to Section C4 with $K_L$ equal to two times the distance between fasteners.

(b) To prevent flexural and/or torsional overall column buckling, $F_n$ shall be calculated in accordance with Section C4 with $F_c$ taken as the smaller of the two $\sigma_{CR}$ values specified for the following section types, where $\sigma_{CR}$ is the theoretical elastic buckling stress under concentric loading.

1. Singly-symmetric channels and C-Sections

$$\sigma_{CR} = \sigma_{cy} + \overline{Q}_s$$

$$\sigma_{CR} = \frac{1}{2\beta} \left[ \left( \sigma_{ex} + \sigma_{IQ} \right) - \sqrt{\left( \sigma_{ex} + \sigma_{IQ} \right)^2 - 4\beta \sigma_{ex} \sigma_{IQ}} \right]$$

2. Z-Sections

$$\sigma_{CR} = \sigma_c + \overline{Q}_t$$

$$\sigma_{CR} = \frac{1}{2} \left[ \left( \sigma_{ex} + \sigma_{cy} + \overline{Q}_s \right) - \sqrt{\left( \left( \sigma_{ex} + \sigma_{cy} + \overline{Q}_s \right)^2 - 4\left( \sigma_{ex} \sigma_{cy} + \sigma_{ex} \overline{Q}_s - \sigma_{exy} \right) \right)} \right]$$

3. I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_s$$

$$\sigma_{CR} = \sigma_{ex}$$

In the above formulas:

$$\sigma_{ex} = \frac{\pi^2 E}{(L / \tau_x)^2}$$

$$\sigma_{exy} = (\pi^2 E_s I_{xy}) / (AL^2)$$

*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.
\[ \sigma_{sy} = \frac{\pi^2E}{(L/\eta_y)^2} \]  
\[ \sigma_1 = \frac{1}{A_{t0}} \left[ GJ + \frac{\pi^2EC_w}{L^2} \right] \]  
\[ \sigma_Q = \sigma_1 + \overline{Q} \]  
\[ \overline{Q} = \overline{q}B = \text{Design shear rigidity for sheathing on both sides of the wall assembly} \]  
\[ \overline{q} = \text{Design shear rigidity for sheathing per inch of stud spacing (see Table D4)} \]  
\[ B = \text{Stud spacing} \]  
\[ \overline{Q}_a = \overline{Q}/A \]  
\[ A = \text{Area of full unreduced cross section} \]  
\[ L = \text{Length of stud} \]  
\[ \overline{Q}_t = \frac{(\overline{Q}d^2)}{4A_{t0}^2} \]  
\[ d = \text{Depth of section} \]  
\[ I_{xy} = \text{Product of inertia} \]  

(c) To prevent shear failure of the sheathing, a value of \( F_n \) shall be used in the following equations so that the shear strain of the sheathing, \( \gamma \), does not exceed the permissible shear strain, \( \gamma \). The shear strain, \( \gamma \), shall be determined as follows:

\[ \gamma = \left( \frac{\pi}{L} \right) \left[ C_1 + \left( E_1 \frac{d}{2} \right) \right] \]  

where

\( C_1 \) and \( E_1 \) are the absolute values of \( C_1 \) and \( E_1 \) specified below for each section type:

(1) Singly-Symmetric Channels and C-Sections

\[ C_1 = \frac{F_n}{C_0} \left( \sigma_{sy} - F_n + \overline{Q}_a \right) \]  
\[ E_1 = \frac{F_n \left[ (\sigma_{st} - F_n)(\sigma_{ex} - F_n + D_0) - F_n x_0 (D_0 - x_0 E_0) \right]}{\left( \sigma_{ex} - F_n \right)^2 (\sigma_{QQ} - F_n - (F_n x_0)^2} \]  

(2) Z-Sections

\[ C_1 = \frac{F_n}{C_0} \left( \sigma_{sy} - F_n + \overline{Q}_a \right) \]  
\[ E_1 = \frac{F_n E_0}{(\sigma_{QQ} - F_n)} \]  

(3) I-Sections

\[ C_1 = \frac{F_n}{C_0} \left( \sigma_{sy} - F_n + \overline{Q}_a \right) \]  
\[ E_1 = 0 \]  

where

\( x_0 \) = distance from shear center to centroid along principal x-axis, in. (absolute value)

\( C_0, E_0, \) and \( D_0 \) are initial column imperfections which shall be assumed to be at least

\[ C_0 = L/350 \text{ in a direction parallel to the wall} \]  
\[ D_0 = L/700 \text{ in a direction perpendicular to the wall} \]  
\[ E_0 = L/(d \times 10,000), \text{ rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape} \]  

If \( F_n > 0.5 F_y \), then in the definitions for \( \sigma_{sy}, \sigma_{ex}, \sigma_{exy} \) and \( \sigma_{QQ} \), the parameters \( E \) and \( G \) shall be replaced by \( E' \) and \( G' \), respectively, as defined below

\[ E' = 4EF_n(F_y - F_n)/F_y^2 \]  
\[ G' = G (E'/E) \]
Sheathing parameters \( \tilde{q}_0 \) and \( \tilde{\gamma} \) may be determined from representative full-scale tests, conducted and evaluated as described by published documented methods (see Commentary), or from the small-scale-test values given in Table D4.

### TABLE D4

<table>
<thead>
<tr>
<th>Sheathing(2)</th>
<th>( \tilde{q}_0 )</th>
<th>( \tilde{\gamma} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 to 5/8 in. thick gypsum</td>
<td>2.0</td>
<td>0.008</td>
</tr>
<tr>
<td>Lignocellulosic board</td>
<td>1.0</td>
<td>0.009</td>
</tr>
<tr>
<td>Fiberboard (regular or impregnated)</td>
<td>0.6</td>
<td>0.007</td>
</tr>
<tr>
<td>Fiberboard (heavy impregnated)</td>
<td>1.2</td>
<td>0.010</td>
</tr>
</tbody>
</table>

(1) The values given are subject to the following limitations:

- All values are for sheathing on both sides of the wall assembly.
- All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 6- to 12-inch spacing.

(2) All sheathing is 1/2-inch thick except as noted.

(3) \( \tilde{q} = \tilde{q}_0 \left(2 - \frac{s}{12}\right) \)  \((Eq. D4.1-26)\)

where \( s \) = fastener spacing, in.

For other types of sheathing, \( \tilde{q}_0 \) and \( \tilde{\gamma} \) may be determined conservatively from representative small-specimen tests as described by published documented methods (see Commentary).

**D4.2 Wall Studs in Bending**

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the design flexural strengths are \( \phi_b M_{\text{Max}} \) and \( \phi_b M_{\text{Max}} \).

where

\[ \phi_b = 0.95 \text{ for sections with stiffened or partially stiffened compression flanges} \]
\[ = 0.90 \text{ for sections with unstiffened compression flanges} \]

\( M_{\text{Max}} \) and \( M_{\text{Max}} \) = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling).

**D4.3 Wall Studs with Combined Axial Load and Bending**

The required axial strength and flexural strength shall satisfy the interaction equations of Section C5 with the following redefined terms:

\( P_n = \) Nominal axial strength determined according to Section D4.1

\( M_{\text{Max}} \) and \( M_{\text{Max}} \) in Equations C5-1, C5-2 and C5-3 shall be replaced by nominal flexural strengths, \( M_{\text{Max}} \) and \( M_{\text{Max}} \), respectively.

---

* Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.
E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following LRFD design criteria govern welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings".

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D-1.3 (Reference 3 of Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1, "Recommended Practices for Resistance Welding" or AWS C1.3, "Recommended Practice for Resistance Welding Coated Low Carbon Steels."

| TABLE E2 |
|------------------|------------------|------------------|------------------|------------------|
| Connection       | Welding Position |                  |                  |                  |
|                  | Square Groove    | Arc Spot         | Arc Seam         | Fillet           | Flare–Bevel      | Flare–V          |
|                  | Butt Weld        | Weld            | Weld             | Weld, Lap or T   | Groove           | Groove          |
| Sheet to Sheet   | F                | —               | F                | F                | F                | F               |
| H                | —                | —               | —                | —                | —                | —               |
| V                | —                | —               | —                | —                | —                | —               |
| OH               | —                | —               | —                | —                | —                | —               |
| Sheet to         |                  | F               | F                | F                | —                | —               |
| Supporting       |                  | —               | —                | —                | —                | —               |
| Member           |                  | —               | —                | —                | —                | —               |
|                  |                  |                  |                  |                  |                  |                  |

(F = flat, H = horizontal, V = vertical, OH = overhead)

The required strength on each weld shall not exceed the design strength, \( \phi P_n \),

where

\( \phi = \) Resistance factor for arc welded connections defined in Sections E2.1 through E2.5.

\( P_n = \) Nominal strength of welds determined according to Sections E2.1 through E2.5.

E2.1 Groove Welds in Butt Joints

The design strength, \( \phi P_n \), of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

(a) Tension or compression normal to the effective area or parallel to the axis of the weld

\[ \phi = 0.90 \]
\[ P_n = L t e F_y \]  
\[ \text{where} \]
\[ \phi = \text{Resistance factor for welded connections} \]
\[ P_n = \text{Nominal strength of a groove weld} \]
\[ F_{xx} = \text{Strength level designation in AWS electrode classification} \]
\[ F_y = \text{Specified minimum yield point of the lower strength base steel} \]
\[ L = \text{Length of weld} \]
\[ t_e = \text{Effective throat dimension for groove weld} \]

**(E2.1-1)**

**(E2.1-2)**

**(E2.1-3)**

### E2.2 Arc Spot Welds

Arc spot welds permitted by this Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers, Figures E2.2(A) and E2.2(B), shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of \( \frac{3}{16} \) inch diameter.

![Figure E2.2A Typical Weld Washer](image-url)
Arc spot welds shall be specified by minimum effective diameter of fused area, $d_e$. Minimum allowable effective diameter is $\frac{3}{8}$ inch.

The design shear strength, $\phi P_n$, of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

(a) $\phi = 0.60$

$$P_n = 0.589 d_e^2 F_{xx}; \text{ or}$$

(b) For $(d/t) \leq 0.815 \sqrt{(E/F_u)}$:

$$\phi = 0.60$$

$$P_n = 2.20 t d_a F_u$$

For $0.815 \sqrt{(E/F_u)} < (d/t) < 1.397 \sqrt{(E/F_u)}$:

$$\phi = 0.50$$

$$P_n = 0.280 \left[ 1 + 5.59 \sqrt{E/F_u} \right] t d_a F_u$$

For $(d/t) \geq 1.397 \sqrt{(E/F_u)}$:

$$\phi = 0.50$$

$$P_n = 1.40 t d_a F_u$$

where

- $\phi$ = Resistance factor for welded connections
- $P_n$ = Nominal shear strength of an arc spot weld
- $d$ = Visible diameter of outer surface of arc spot weld
- $d_a$ = Average diameter of the arc spot weld at mid-thickness of $t$ where $d_a = (d - t)$ for a single sheet, and $(d - 2t)$ for multiple sheets (not more than four lapped sheets over a supporting member)
- $d_e$ = Effective diameter of fused area
- $d_e = 0.7d - 1.5t$ but $\leq 0.55d$
- $t$ = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer
- $F_{xx}$ = Stress level designation in AWS electrode classification
- $F_u$ = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low ductility steel.

**Note:** See Figures E2.2(C) and E2.2(D) for diameter definitions

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of $e_{\text{min}}$ as given below:
Figure E2.2 C, D  Arc Spot Welds

\[ e_{\text{min}} = \frac{P_u}{\phi F_{u} t} \]  

(Eq. E2.2–6)

where

\[ \phi = \text{Resistance factor for welded connections} \]
\[ = 0.70 \text{ when } F_u/F_{sy} \geq 1.15 \]
\[ = 0.60 \text{ when } F_u/F_{sy} < 1.15 \]

\[ P_u = \text{Required strength transmitted by weld} \]
\[ t = \text{Thickness of thinnest connected sheet} \]
\[ F_{sy} = \text{Yield point as specified in Sections A3.1 or A3.2} \]

Note: See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.
The design tensile strength, $q$, on each arc spot weld between sheet and supporting member, shall be determined as follows:

$$ q = 0.65 \frac{P_n}{0.7 \times d_a \times F_u} $$

The following additional limitations for use in Eq. 2.2-7 shall apply:

- $e_{min} \geq d$
- $F_{xx} \geq 60$ ksi
- $F_u \leq 60$ ksi
- $t \geq 0.028$ in.

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, $d_e$, or average diameter, $d_a$, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

**E2.3 Arc Seam Welds**

Arc seam welds [Figure E2.3(A)] covered by this Specification apply only to the following joints:
The design shear strength, $\phi P_n$, of arc seam welds shall be determined by using the smaller of either

(a) $\phi = 0.60$

$$P_n = \left[ \frac{\pi d_e^2}{4} + L d_e \right] 0.75F_{xx} \text{; or}$$

(Eq. E2.3-1)

(b) $\phi = 0.60$

$$P_n = 2.5 t F_u (0.25L + 0.96 d_e)$$

(Eq. E2.3-2)

where

- $\phi$ = Resistance factor for welded connections
- $P_n$ = Nominal shear strength of an arc seam weld
- $d_e$ = Width of arc seam weld
- $L$ = Length of seam weld not including the circular ends
  (For computation purposes, $L$ shall not exceed 3d)
- $d_e$ = Average width of seam weld

where

$$d_e = \begin{cases} 
(d - t) & \text{for a single sheet, and} \\
(d - 2t) & \text{for a double sheet} 
\end{cases}$$

(Eq. E2.3-3)

(Eq. E2.3-4)

- $d_e$ = Effective width of arc seam weld at fused surfaces
- $d_e = 0.7d - 1.5t$ (Eq. E2.3-5)

and $F_u$ and $F_{xx}$ are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2. See Figure E2.3(B).
E2.4 Fillet Welds

Fillet welds covered by this Specification apply to the welding of joints in any position, either

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

The design shear strength, $\phi P_n$, of a fillet weld shall be determined as follows:

(a) For longitudinal loading:
- For $L/t < 25$:
  \[ \phi = 0.60 \]
  \[ P_n = \left( 1 - \frac{0.01L}{t} \right) tLF_u \]  
  \( (Eq. \ E2.4-1) \)
- For $L/t \geq 25$:
  \[ \phi = 0.55 \]
  \[ P_n = 0.75 tLF_u \]
  \( (Eq. \ E2.4-2) \)

(b) For transverse loading:
  \[ \phi = 0.60 \]
  \[ P_n = tLF_u \]
  \( (Eq. \ E2.4-3) \)

where $t =$ Least value of $t_1$ or $t_2$, Figure E2.4

In addition, for $t > 0.150$ inch the design strength determined above shall not exceed the following value of $\phi P_n$:

\[ \phi = 0.60 \]
\[ P_n = 0.75 t_w LF_{xx} \]  
\( (Eq. \ E2.4-4) \)

where
- $\phi =$ Resistance factor for welded connections
- $P_n =$ Nominal strength of a fillet weld
- $L =$ Length of fillet weld
- $t_w =$ Effective throat $= 0.707 w_1$ or $0.707 w_2$, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.
- $w_1$ and $w_2 =$ leg on weld (see Figure E2.4).
- $F_u$ and $F_{xx}$ are defined in Section E2.2.
E2.5 Flare Groove Welds

Flare groove welds covered by this Specification apply to welding of joints in any position, either:
(a) Sheet to sheet for flare–V groove welds, or
(b) Sheet to sheet for flare–bevel groove welds, or
(c) Sheet to thicker steel member for flare–bevel groove welds.

The design shear strength, $\phi P_n$, of a flare groove weld shall be determined as follows:
(a) For flare–bevel groove welds, transverse loading [see Figure E2.5(A)]:
   \[ \phi = 0.55 \]
   \[ P_n = 0.833tLF_u \]  
   (Eq. E2.5-1)
(b) For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:
   (1) For $t \leq t_w < 2t$ or if the lip height is less than weld length, $L$:
      \[ \phi = 0.55 \]
      \[ P_n = 0.75tLF_u \]  
      (Eq. E2.5-2)
   (2) For $t_w \geq 2t$ and the lip height is equal to or greater than $L$:
      \[ \phi = 0.55 \]
      \[ P_n = 1.50tLF_u \]  
      (Eq. E2.5-3)
In addition, if $t > 0.15$ inch, the design strength determined above shall not exceed the following value of $\phi P_n$:
   \[ \phi = 0.60 \]
   \[ P_n = 0.75t_wLF_{xx} \]  
   (Eq. E2.5-4)
Figure E2.5A Flare-Bevel Groove Weld

(B) Flare Bevel Groove

(C) Flare V-Groove

(D) Throat

Figure E2.5 B, C, D Shear in Flare Groove Welds
E2.6 Resistance Welds

The design shear strength, $\phi P_n$, of spot welding shall be determined as follows:

$\phi = 0.65$

$P_n = \text{Tabulated value given in Table E2.6}$

<table>
<thead>
<tr>
<th>Thickness of Thinnest Outside Sheet, in.</th>
<th>Nominal Shear Strength per Spot, kips</th>
<th>Thickness of Thinnest Outside Sheet, in.</th>
<th>Nominal Shear Strength per Spot, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.010</td>
<td>0.13</td>
<td>0.080</td>
<td>3.33</td>
</tr>
<tr>
<td>0.020</td>
<td>0.48</td>
<td>0.090</td>
<td>4.00</td>
</tr>
<tr>
<td>0.030</td>
<td>1.00</td>
<td>0.100</td>
<td>4.99</td>
</tr>
<tr>
<td>0.040</td>
<td>1.42</td>
<td>0.110</td>
<td>6.07</td>
</tr>
<tr>
<td>0.050</td>
<td>1.65</td>
<td>0.125</td>
<td>7.29</td>
</tr>
<tr>
<td>0.060</td>
<td>2.28</td>
<td>0.190</td>
<td>10.16</td>
</tr>
<tr>
<td>0.070</td>
<td>2.83</td>
<td>0.250</td>
<td>15.00</td>
</tr>
</tbody>
</table>

E3 Bolted Connections

The following LRFD design criteria govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than $\frac{3}{16}$ inch. For bolted connections in which the thickness of the thinnest connected part is equal to or greater than $\frac{3}{16}$ inch, refer to AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings", September 1, 1986.

Bolts, nuts, and washers shall generally conform to one of the following specifications:

- ASTM A194 Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
- ASTM A307 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners
- ASTM A325 High Strength Bolts for Structural Steel Joints
- ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than $\frac{1}{2}$ inch)
- ASTM A449 Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $\frac{1}{2}$ inch)
- ASTM A490 Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints
- ASTM A563 Carbon and Alloy Steel Nuts
- ASTM F436 Hardened Steel Washers
- ASTM F844 Washers, Steel, Plain (Flat), Unhardened for General Use
- ASTM F959 Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.
The holes for bolts shall not exceed the sizes specified in Table E3, except that larger holes may be used in column base details or structural systems connected to concrete walls.

### TABLE E3
**Maximum Size of Bolt Holes, Inches**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1/2</td>
<td>d + 1/32</td>
<td>d + 1/16</td>
<td>(d + 1/32) by (d + 1/4)</td>
<td>(d + 1/32) by (2 1/2 d)</td>
</tr>
<tr>
<td>≥ 1/2</td>
<td>d + 1/16</td>
<td>d + 1/8</td>
<td>(d + 1/16) by (d + 1/4)</td>
<td>(d + 1/16) by (2 1/2 d)</td>
</tr>
</tbody>
</table>

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

#### E3.1 Spacing and Edge Distance

The design shear strength, \( \phi P_n \), of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

\[
P_n = e F_u
\]

(Eq. E3.1-1)

(a) When \( F_u / F_y \geq 1.15 \):

\[
\phi = 0.70
\]

(b) When \( F_u / F_y < 1.15 \):

\[
\phi = 0.60
\]

where

- \( \phi \) = Resistance factor
- \( P_n \) = Nominal resistance per bolt
- \( e \) = The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part
- \( t \) = Thickness of thinnest connected part
- \( F_u \) = Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel
- \( F_y \) = Yield point of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, \( d \). Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than 1 1/2 \( d \).

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of \( e - (d_h/2) \), in which \( e \) is the required distance computed from the applicable equation given above, and \( d_h \) is the diameter of a standard hole defined in Table E3. In no case shall the clear dis-
tance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

### E3.2 Tension in Connected Part

The design tensile strength, \( \phi P_n \), on the net section of the connected part shall be determined as follows:

(a) Washers are provided under both the bolt head and the nut

\[
P_n = (1.0 - 0.9r + 3rd/s) FuAn \leq FuA_n
\]

\[\phi = 0.65 \text{ for double shear connection} \]

\[\phi = 0.55 \text{ for single shear connection} \]

(b) Either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

\[
P_n = (1.0 - r + 2.5rd/s) FuAn \leq FuA_n
\]

In addition, the design tensile strength shall not exceed the following values:

\[\phi = 0.95 \]

\[P_n = F_yA_n\]

where

\(A_n = \text{Net area of the connected part}\)

\(r = \text{Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If } r \text{ is less than 0.2, it may be taken equal to zero.}\)

\(s = \text{Spacing of bolts perpendicular to line of stress.}\)

In the case of a single bolt, \(s = \text{Width of sheet}\)

\(F_u = \text{Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel}\)

\(F_y = \text{Yield point of the connected part}\)

\(d \text{ and } t \text{ are defined in Section E3.1}\)

### E3.3 Bearing

The design bearing strength, \( \phi P_n \), shall be determined by the values of \( \phi \) and \( P_n \) given in Tables E3.3–1 and E3.3–2 for the applicable thickness and \( F_u/F_y \) ratio of the connected part and the type of joint used in the connection.

In Tables E3.3–1 and E3.3–2, the symbols \( \phi, P_n, d, F_u \) and \( t \) were previously defined. For conditions not shown, the design bearing strength of bolted connections shall be determined by tests.
TABLE E3.3-1
Nominal Bearing Strength for Bolted Connections
with Washers under Both Bolt Head and Nut

<table>
<thead>
<tr>
<th>Thickness of Connected Part in.</th>
<th>Type of joint</th>
<th>( F_u/F_{sy} ) ratio of Connected Part</th>
<th>Resistance Factor ( \phi )</th>
<th>Nominal Resistance ( P_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \geq 0.024 ) but (&lt; \frac{3}{16} )</td>
<td>Inside sheet of double shear connection</td>
<td>( \geq 1.15 )</td>
<td>0.55</td>
<td>3.33 ( F_{udt} )</td>
</tr>
<tr>
<td></td>
<td>Single shear and outside sheets of double shear connection</td>
<td>(&lt; 1.15 )</td>
<td>0.65</td>
<td>3.00 ( F_{udt} )</td>
</tr>
<tr>
<td>( \geq \frac{3}{16} )</td>
<td>See AISC LRFD Specification</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TABLE E3.3-2
Nominal Bearing Strength for Bolted Connections
Without Washers Under Both Bolt Head and Nut,
or With Only One Washer

<table>
<thead>
<tr>
<th>Thickness of Connected Part in.</th>
<th>Type of joint</th>
<th>( F_u/F_{sy} ) ratio of Connected Part</th>
<th>Resistance Factor ( \phi )</th>
<th>Nominal Resistance ( P_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \geq 0.036 ) but (&lt; \frac{3}{16} )</td>
<td>Inside sheet of double shear connection</td>
<td>( \geq 1.15 )</td>
<td>0.65</td>
<td>3.00 ( F_{udt} )</td>
</tr>
<tr>
<td></td>
<td>Single shear and outside sheets of double shear connection</td>
<td>( \geq 1.15 )</td>
<td>0.70</td>
<td>2.22 ( F_{udt} )</td>
</tr>
<tr>
<td>( \geq \frac{3}{16} )</td>
<td>See AISC LRFD Specification</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

E3.4 Shear and Tension in Bolts

The required bolt strength in shear or tension shall not exceed the design strength, \( \phi P_n \), determined as follows:

\[
\phi = \text{Resistance factor given in Table E3.4-1} \\
P_n = A_b F_n 
\]

(Eq. E3.4-1)

where

- \( A_b = \) Gross cross-sectional area of bolt
- \( F_n \) is given by \( F_{nv} \) or \( F_n \) in Table E3.4-1.
<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Tensile Strength</th>
<th>Shear Strength*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resistance Factor $\phi$</td>
<td>Nominal Stress $F_{ut}$</td>
</tr>
<tr>
<td>A307 Bolts, Grade A ((\frac{1}{4}) in. $\leq d &lt; \frac{1}{2}$ in.)</td>
<td>0.75</td>
<td>40.5</td>
</tr>
<tr>
<td>A307 Bolts, Grade A (d $\geq \frac{1}{2}$ in.)</td>
<td>0.75</td>
<td>45.0</td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td>90.0</td>
<td>59.0</td>
</tr>
<tr>
<td>A325 bolts, when threads are excluded from shear planes</td>
<td>90.0</td>
<td>90.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts ((\frac{1}{4}) in. $\leq d &lt; \frac{1}{2}$ in.), when threads are not excluded from shear planes</td>
<td>101.0</td>
<td>59.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts ((\frac{1}{4}) in. $\leq d &lt; \frac{1}{2}$ in.), when threads are excluded from shear planes</td>
<td>101.0</td>
<td>90.0</td>
</tr>
<tr>
<td>A449 Bolts ((\frac{1}{4}) in. $\leq d &lt; \frac{1}{2}$ in.), when threads are not excluded from shear planes</td>
<td>81.0</td>
<td>47.0</td>
</tr>
<tr>
<td>A449 Bolts ((\frac{1}{4}) in. $\leq d &lt; \frac{1}{2}$ in.), when threads are excluded from shear planes</td>
<td>81.0</td>
<td>72.0</td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td>112.5</td>
<td>67.5</td>
</tr>
<tr>
<td>A490 Bolts, when threads are excluded from shear planes</td>
<td>112.5</td>
<td>90.0</td>
</tr>
</tbody>
</table>

* Applies to bolts in holes as limited by Table E3. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Section F.
The pullover strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, see Section E5.2.

When bolts are subject to a combination of shear and tension produced by factored loads, the required tension strength shall not exceed the design strength, \( P_n \), based on \( \phi = 0.75 \) and \( P_n = A_1 F'_{nt} \), where \( F'_{nt} \) is given in Table E3.4–2, in which \( f_v \) is the shear stress produced by the same factored loads. The required shear strength shall not exceed the design shear strength, \( \phi A_3 F_{nv} \), determined in accordance with Table E3.4–1.

### Table E3.4–2
Nominal Tension Stress, \( F'_{nt} \), for Bolts Subject to the Combination of Shear and Tension

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Threads Not Excluded from Shear Planes</th>
<th>Threads Excluded from Shear Planes</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>113 - 2.4f_v ≤ 90</td>
<td>113 - 1.9f_v ≤ 90</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>127 - 2.4f_v ≤ 101</td>
<td>127 - 1.9f_v ≤ 101</td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>101 - 2.4f_v ≤ 81</td>
<td>101 - 1.9f_v ≤ 81</td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>141 - 2.4f_v ≤ 112.5</td>
<td>141 - 1.9f_v ≤ 112.5</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>When 1/4 in. ≤ d &lt; 1/2 in.</td>
<td>47 - 2.4f_v ≤ 40.5</td>
<td></td>
</tr>
<tr>
<td>When d ≥ 1/2 in.</td>
<td></td>
<td>52 - 2.4f_v ≤ 45</td>
</tr>
</tbody>
</table>

### E4 Shear Rupture
At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the required shear strength shall not exceed the design shear strength, \( \phi V_n \).

where

\[ \phi = 0.75 \]

\[ V_n = 0.6 F_u A_{wn} \]  
\[ A_{wn} = (d_{wc} - nh) t \]  
\[ d_{wc} = \text{Coped web depth} \]  
\[ n = \text{Number of holes in the critical plane} \]  
\[ d_h = \text{Hole diameter} \]  
\[ F_u = \text{Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel} \]  
\[ t = \text{Thickness of coped web} \]  

### E5 Connections to Other Materials

#### E5.1 Bearing
Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The required bearing strength in the contact area shall not exceed the design strength, \( \phi_e P_p \).

In the absence of code regulations, the design bearing strength on concrete may be taken as \( \phi_e P_p \).
On the full area of a concrete support \[ P_p = 0.85f'_c A_1 \]
On less than the full area of a concrete support \[ P_p = 0.85f'_c A_1 \sqrt{A_2 / A_1} \]
where
\[ \phi_c = 0.60 \]
\[ f'_c = \text{Specified compression strength of concrete} \]
\[ A_1 = \text{Bearing area} \]
\[ A_2 = \text{Full cross-sectional area of concrete support} \]
The value of \[ \sqrt{A_2 / A_1} \] shall not exceed 2.

E5.2 Tension

The pull-over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull-out force resulting from factored axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The nominal tensile strength of the fastener and the nominal imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

E5.3 Shear

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The required shear and/or bearing strength on the steel components shall not exceed that allowed by this Specification. The design shear strength on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.
F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

(b) The provisions of Chapter F do not apply to cold-formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold-formed steel structural members are such that calculation of their load-carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than four identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±10 percent. If such deviation from the average value exceeds 10 percent, at least three more tests of the same kind shall be made. The average value of all tests made shall then be regarded as the predicted capacity, $R_p$, for the series of the tests. The mean value and the coefficient of variation of the tested-to-predicted load ratios for all tests, $P_m$ and $V_p$, shall be determined for statistical analysis.

(b) The load-carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. F1-1.

$$\phi R_p \geq \sum \gamma Q_i$$  \hspace{1cm} (Eq. F1-1)

where

- $\sum \gamma Q_i$ = Required resistance based on the most critical load combination determined in accordance with Section A5.1.4. $\gamma$ and $Q_i$ are load factors and load effects, respectively.
- $R_p$ = Average value of all test results
- $\phi$ = Resistance factor
- $1.5(M_m F_m P_m) \exp\left(-\beta_0 \sqrt{V_m^2 + V_f^2 + V_p^2 + V_Q^2}\right)$  \hspace{1cm} (Eq. F1-2)
- $M_m$ = Mean value of the material factor listed in Table F1 for the type of component involved
- $F_m$ = Mean value of the fabrication factor listed in Table F1 for the type of component involved
- $P_m$ = Mean value of the tested-to-predicted load ratios determined in Section F1(a)
- $\beta_0$ = Target reliability index
- 2.5 for structural members and 3.5 for connections
- $V_m$ = Coefficient of variation of the material factor listed in Table F1 for the type of component involved
- $V_f$ = Coefficient of variation of the fabrication factor listed in Table F1 for the type of component involved
- $C_p$ = Correction factor
- $(n-1)/(n-3)$  \hspace{1cm} (Eq. F1-3)
- $V_p$ = Coefficient of variation of the tested-to-predicted load ratios determined in Section F1(a)
- $n$ = Number of tests
- $V_Q$ = Coefficient of variation of the load effect
- 0.21

* For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced, $\phi$ shall be determined with a coefficient of 1.6 in lieu of 1.5, $\beta_0 = 1.5$, and $V_Q = 0.43$. 
<table>
<thead>
<tr>
<th>Type of Component</th>
<th>$M_m$</th>
<th>$V_m$</th>
<th>$F_m$</th>
<th>$V_F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Stiffeners</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Shear Stiffeners</td>
<td>1.00</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Tension Members</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Flexural Members</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Lateral Buckling Strength</td>
<td>1.00</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>One Flange Through–Fastened to Deck or Sheathing</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Bending and Shear</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Web Crippling Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Bending and Web Crippling</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Concentrically Loaded Compression Members</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Axial Load and Bending</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Cylindrical Tubular Members</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Axial Compression</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs and Wall Stud Assemblies</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Studs in Compression</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs in Bending</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs with Combined Axial Load and Bending</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
</tbody>
</table>
TABLE F1 (Continued)
Statistical Data for the Determination of Resistance Factor

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded Connections</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arc Spot Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
</tr>
<tr>
<td>Arc Seam Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Tearing</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Fillet Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
</tr>
<tr>
<td>Flare Groove Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Resistance Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Bolted Connections</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Spacing and Edge Distance</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Tension Strength on Net Section</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Bearing Strength</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
</tbody>
</table>
The listing in Table F1 does not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.*

For steels not listed in Section A3.1, the values of $M_{m}$ and $V_{M}$ shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1-1, except that the resistance factor $\phi$ is taken as unity and that the load factor for dead load may be taken as 1.0.

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

**F2 Tests for Confirming Structural Performance**

For structural members, connections, and assemblies whose capacities can be computed according to this Specification or its specific references, confirmatory tests may be made to demonstrate the load-carrying capacity not less than the nominal resistance, $R_{n}$, specified in this Specification or its specific references for the type of behavior involved.

**F3 Tests for Determining Mechanical Properties**

**F3.1 Full Section**

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370. Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

   (1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

   (2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.

   When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method.

   (c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of $\rho$ for the specimen is unity.

* See Reference 36 of the Commentary
(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

**F3.2 Flat Elements of Formed Sections**

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, $F_{yr}$, shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, $F_{yr}$, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

**F3.3 Virgin Steel**

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.
LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

MARCH 16, 1991 EDITION

LRFD Cold-Formed Steel Design Manual - Part I

AMERICAN IRON AND STEEL INSTITUTE
1133 15th STREET, NW
WASHINGTON, DC 20005–2701
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute or any other person named herein – that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.

1st Printing – August, 1991

Produced by Computerized Structural Design, Inc.
Milwaukee, Wisconsin

Copyright American Iron and Steel Institute 1991
PREFACE

The American Iron and Steel Institute allowable stress design specification has long been used for the design of cold-formed steel structural members. The Load and Resistance Factor Design (LRFD) Specification has recently been developed from a research project sponsored by AISI at the University of Missouri-Rolla under the direction of Wei-Wen Yu with consultation of T.V. Galambos and initial contribution of M.K. Ravindra. In this LRFD Specification, separate load and resistance factors are applied to specified loads and nominal resistance to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic methodology as used for the development of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings.

This Specification contains six chapters of the LRFD recommendations for cold-formed steel structural members and connections. The background information for the design criteria is discussed in the Commentary and other related references.

AISI acknowledges the devoted efforts of the members of the Committee on Specifications for the Design of Cold-Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold-formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on this LRFD Specification are:

- R. L. Brockenbrough, Chairman
- R.B. Haws, Secretary
- R.E. Albrecht
- R. Bjorhovde
- R.E. Brown
- C.R. Clauer
- D.A. Cuoco
- D.S. Ellifritt
- S.J. Errera*  
- E.R. Estes, Jr.
- J.M. Fisher
- T.V. Galambos
- M. Golovin
- W.B. Hall
- G.S. Harris
- R.B. Heagler
- N. Iwankiw
- A.L. Johnson
- D.L. Johnson
- T.J. Jones
- H. Klein

- K. H. Klippstein*
- R.A. LaBoube
- J.N. Macadam
- R.R. McCluer
- W.R. Midgley
- T.J. Morris
- J.A. Moses
- T.M. Murray
- G.G. Nichols
- J.N. Nunnery
- T.B. Pekoz
- C.W. Pinkham
- P.G. Schurter
- R.M. Schuster
- P.A. Seaburg
- F.V. Slocum
- D.L. Tarlton
- D.S. Wolford*
- W.W. Yu
- A.S. Zakrezewski

*Past Chairman
The activities of the Committee are sponsored by AISI's Light Construction Subcommittee of the Construction Marketing Committee. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
March 16, 1991
# TABLE OF CONTENTS

**LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS**  
**MARCH 16, 1991 EDITION**

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>PREFACE</td>
<td></td>
<td>1-3</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td></td>
<td>1-5</td>
</tr>
<tr>
<td>SYMBOLS AND DEFINITIONS</td>
<td></td>
<td>1-8</td>
</tr>
<tr>
<td><strong>A. GENERAL PROVISIONS</strong></td>
<td></td>
<td>1-18</td>
</tr>
<tr>
<td>A1</td>
<td>Limits of Applicability and Terms</td>
<td>1-18</td>
</tr>
<tr>
<td></td>
<td>A1.1 Scope and Limits of Applicability</td>
<td>1-18</td>
</tr>
<tr>
<td></td>
<td>A1.2 Terms</td>
<td>1-18</td>
</tr>
<tr>
<td></td>
<td>A1.3 Units of Symbols and Terms</td>
<td>1-19</td>
</tr>
<tr>
<td>A2</td>
<td>Non-Conforming Shapes and Constructions</td>
<td>1-19</td>
</tr>
<tr>
<td>A3</td>
<td>Material</td>
<td>1-19</td>
</tr>
<tr>
<td></td>
<td>A3.1 Applicable Steels</td>
<td>1-19</td>
</tr>
<tr>
<td></td>
<td>A3.2 Other Steels</td>
<td>1-20</td>
</tr>
<tr>
<td></td>
<td>A3.3 Ductility</td>
<td>1-20</td>
</tr>
<tr>
<td></td>
<td>A3.4 Delivered Minimum Thickness</td>
<td>1-21</td>
</tr>
<tr>
<td>A4</td>
<td>Loads</td>
<td>1-21</td>
</tr>
<tr>
<td></td>
<td>A4.1 Dead Load</td>
<td>1-21</td>
</tr>
<tr>
<td></td>
<td>A4.2 Live or Snow Load</td>
<td>1-21</td>
</tr>
<tr>
<td></td>
<td>A4.3 Impact Load</td>
<td>1-21</td>
</tr>
<tr>
<td></td>
<td>A4.4 Wind or Earthquake Loads</td>
<td>1-21</td>
</tr>
<tr>
<td></td>
<td>A4.5 Ponding</td>
<td>1-21</td>
</tr>
<tr>
<td>A5</td>
<td>Structural Analysis and Design</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1 Design Basis</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1.1 Limit State – Strength</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1.2 Limit State – Serviceability</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1.3 Nominal Loads</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1.4 Load Factors and Load Combinations</td>
<td>1-22</td>
</tr>
<tr>
<td></td>
<td>A5.1.5 Resistance Factors</td>
<td>1-23</td>
</tr>
<tr>
<td></td>
<td>A5.2 Yield Point and Strength Increase from Cold Work of Forming</td>
<td>1-25</td>
</tr>
<tr>
<td></td>
<td>A5.2.1 Yield Point</td>
<td>1-25</td>
</tr>
<tr>
<td></td>
<td>A5.2.2 Strength Increase from Cold Work of Forming</td>
<td>1-25</td>
</tr>
<tr>
<td></td>
<td>A5.3 Durability</td>
<td>1-26</td>
</tr>
<tr>
<td>A6</td>
<td>Reference Documents</td>
<td>1-26</td>
</tr>
<tr>
<td><strong>B. ELEMENTS</strong></td>
<td></td>
<td>1-28</td>
</tr>
<tr>
<td>B1</td>
<td>Dimensional Limits and Considerations</td>
<td>1-28</td>
</tr>
<tr>
<td></td>
<td>B1.1 Flange Flat-Width-to-Thickness Considerations</td>
<td>1-28</td>
</tr>
<tr>
<td></td>
<td>B1.2 Maximum Web Depth-to-Thickness Ratio</td>
<td>1-29</td>
</tr>
<tr>
<td>B2</td>
<td>Effective Widths of Stiffened Elements</td>
<td>1-30</td>
</tr>
<tr>
<td></td>
<td>B2.1 Uniformly Compressed Stiffened Elements</td>
<td>1-30</td>
</tr>
<tr>
<td></td>
<td>B2.2 Uniformly Compressed Stiffened Elements with Circular Holes</td>
<td>1-31</td>
</tr>
<tr>
<td></td>
<td>B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient</td>
<td>1-32</td>
</tr>
<tr>
<td>B3</td>
<td>Effective Widths of Unstiffened Elements</td>
<td>1-32</td>
</tr>
<tr>
<td></td>
<td>B3.1 Uniformly Compressed Unstiffened Elements</td>
<td>1-32</td>
</tr>
<tr>
<td></td>
<td>B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient</td>
<td>1-33</td>
</tr>
<tr>
<td>B4</td>
<td>Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener</td>
<td>1-34</td>
</tr>
</tbody>
</table>
E2.2 Arc Spot Welds ............................................. I–62
E2.3 Arc Seam Welds ............................................. I–65
E2.4 Fillet Welds ................................................ I–67
E2.5 Flare Groove Welds ....................................... I–68
E2.6 Resistance Welds ......................................... I–70
E3 Bolted Connections ......................................... I–70
E3.1 Spacing and Edge Distance ............................... I–71
E3.2 Tension in Connected Part ............................... I–72
E3.3 Bearing .................................................... I–72
E3.4 Shear and Tension in Bolts ............................... I–73
E4 Shear Rupture ............................................... I–75
E5 Connections to Other Materials ........................... I–75
E5.1 Bearing .................................................... I–75
E5.2 Tension .................................................... I–76
E5.3 Shear ..................................................... I–76
F. TESTS FOR SPECIAL CASES ............................ I–77
F1 Tests for Determining Structural Performance .......... I–77
F2 Tests for Confirming Structural Performance .......... I–80
F3 Tests for Determining Mechanical Properties .......... I–80
F3.1 Full Section .............................................. I–80
F3.2 Flat Elements of Formed Sections ..................... I–81
F3.3 Virgin Steel .............................................. I–81
# SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Full unreduced cross-sectional area of the member</td>
<td>C3.1.1, C3.1.2, C4, C6.2, D4.1</td>
</tr>
<tr>
<td>$A_b$</td>
<td>$b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load, and $b_2t + A_s$, for transverse stiffeners at end support</td>
<td>B6.1</td>
</tr>
<tr>
<td>$A_b$</td>
<td>Gross cross-sectional area of bolt</td>
<td>E3.4</td>
</tr>
<tr>
<td>$A_c$</td>
<td>$18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load, and $10t^2 + A_s$, for transverse stiffeners at end support</td>
<td>B6.1</td>
</tr>
<tr>
<td>$A_e$</td>
<td>Effective area at the stress $F_n$</td>
<td>C4, C6.2, D4.1</td>
</tr>
<tr>
<td>$A_n$</td>
<td>Net area of cross section</td>
<td>C2, E3.2</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Cross-sectional area of transverse stiffeners</td>
<td>B4, B4.1, B4.2, B6.1</td>
</tr>
<tr>
<td>$A'_s$</td>
<td>Effective area of stiffener</td>
<td>B4, B4.1, B4.2</td>
</tr>
<tr>
<td>$A_{sh}$</td>
<td>Gross area of shear stiffener</td>
<td>B6.2</td>
</tr>
<tr>
<td>$A_{wn}$</td>
<td>Net web area</td>
<td>E4</td>
</tr>
<tr>
<td>$A_1$</td>
<td>Bearing area</td>
<td>E5.1</td>
</tr>
<tr>
<td>$A_2$</td>
<td>Full cross sectional area of concrete support</td>
<td>E5.1</td>
</tr>
<tr>
<td>$a$</td>
<td>Shear panel length of the unreinforced web element. For a reinforced web element, the distance between transverse stiffeners</td>
<td>B6.2, C3.2</td>
</tr>
<tr>
<td>$a$</td>
<td>Length of bracing interval</td>
<td>D3.2</td>
</tr>
<tr>
<td>B</td>
<td>Stud spacing</td>
<td>D4, D4.1</td>
</tr>
<tr>
<td>$B_c$</td>
<td>Term for determining the tensile yield point of corners</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>$b$</td>
<td>Effective design width of compression element</td>
<td>B2.1, B2.2, B2.3, B3.1, B3.2, B4.1, B4.2, B5</td>
</tr>
<tr>
<td>$b$</td>
<td>Flange width, Z-section</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>$b_d$</td>
<td>Effective width for deflection calculation</td>
<td>B2.1, B2.2</td>
</tr>
<tr>
<td>$b_e$</td>
<td>Effective design width of sub-element or element</td>
<td>A1.2, B2.3, B5</td>
</tr>
<tr>
<td>$b_o$</td>
<td>See Figure B4.1</td>
<td>B4, B4.1, B5</td>
</tr>
<tr>
<td>C</td>
<td>For flexural members, ratio of the total corner cross-sectional area of the controlling flange to the full cross-sectional area of the controlling flange</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>$C_b$</td>
<td>Bending coefficient dependent on moment gradient</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>$C_m$</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
</tbody>
</table>
# SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_{ms}</td>
<td>Coefficient for lateral bracing of Z-section</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C_{mx}</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
<tr>
<td>C_{my}</td>
<td>End moment coefficient in interaction formula</td>
<td>C5</td>
</tr>
<tr>
<td>C_p</td>
<td>Correction Factor</td>
<td>F1</td>
</tr>
<tr>
<td>C_t</td>
<td>Coefficient for lateral torsional buckling</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C_{TF}</td>
<td>End moment coefficient in interaction formula</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C_{Th}</td>
<td>Coefficient for lateral bracing of Z-sections</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C_r</td>
<td>Coefficient for lateral bracing of Z-sections</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>C_v</td>
<td>Shear stiffener coefficient</td>
<td>B6.2</td>
</tr>
<tr>
<td>C_w</td>
<td>Torsional warping constant of the cross-section</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>C_y</td>
<td>Compression strain factor</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>C_o</td>
<td>Initial column imperfection</td>
<td>D4.1</td>
</tr>
<tr>
<td>C_i</td>
<td>Term used to compute shear strain in wall board</td>
<td>B4, B4.1, B4.2</td>
</tr>
<tr>
<td>C_i</td>
<td>Coefficient as defined in Figure B4-2</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>c_r</td>
<td>Amount of curling</td>
<td>B1.1b</td>
</tr>
<tr>
<td>D</td>
<td>Outside diameter of cylindrical tube</td>
<td>C6, C6.1, C6.2, D4.2</td>
</tr>
<tr>
<td>D</td>
<td>Overall depth of lip</td>
<td>B1.1, B4, D1.1</td>
</tr>
<tr>
<td>D</td>
<td>Shear stiffener coefficient</td>
<td>B6.2</td>
</tr>
<tr>
<td>D</td>
<td>Nominal dead load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>D_o</td>
<td>Initial column imperfection</td>
<td>D4.1</td>
</tr>
<tr>
<td>d</td>
<td>Depth of section</td>
<td>B1.1b, B4, C3.1.1, C3.1.3, D1.1, D3.2.1, D4, D4.1</td>
</tr>
<tr>
<td>d</td>
<td>Width of arc seam weld</td>
<td>E2.3</td>
</tr>
<tr>
<td>d</td>
<td>Visible diameter of outer surface of arc spot weld</td>
<td>E2.2</td>
</tr>
<tr>
<td>d</td>
<td>Diameter of bolt</td>
<td>E3, E3.1, E3.2, E3.4</td>
</tr>
<tr>
<td>d_s</td>
<td>Average diameter of the arc spot weld at mid-thickness of t</td>
<td>E2.2</td>
</tr>
<tr>
<td>d_e</td>
<td>Average width of seam weld</td>
<td>E2.3</td>
</tr>
<tr>
<td>d_e</td>
<td>Effective diameter of fused area</td>
<td>E2.2</td>
</tr>
<tr>
<td>d_e</td>
<td>Effective width of arc seam weld at fused surfaces</td>
<td>E2, E2.3</td>
</tr>
<tr>
<td>d_h</td>
<td>Diameter of standard hole</td>
<td>B2.2, E3.1, E4</td>
</tr>
<tr>
<td>d_r</td>
<td>Reduced effective width of stiffener</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>d_r</td>
<td>Actual effective width of stiffener</td>
<td>B4, B4.2</td>
</tr>
<tr>
<td>d_{wc}</td>
<td>Coped web depth</td>
<td>E4</td>
</tr>
<tr>
<td>E</td>
<td>Modulus of elasticity of steel (29.5x10^3 ksi)</td>
<td>B1.1b, B2.1, B6.1, C3.1.1, C3.2, C3.5, C4, C4.1, C5, C6.1, D1.2, D4.1, D4.2, E2.2</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
<td>Section</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>---------</td>
</tr>
<tr>
<td>E</td>
<td>Nominal earthquake load</td>
<td></td>
</tr>
<tr>
<td>$E_0$</td>
<td>Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, un buckled location</td>
<td>D4.1</td>
</tr>
<tr>
<td>$E_1$</td>
<td>Term used to compute shear strain in wallboard</td>
<td>D4.1</td>
</tr>
<tr>
<td>$E'$</td>
<td>Inelastic modulus of elasticity</td>
<td>D4.1</td>
</tr>
<tr>
<td>e</td>
<td>The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed</td>
<td>E3.1</td>
</tr>
<tr>
<td>$e_{min}$</td>
<td>Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed</td>
<td>E2.2</td>
</tr>
<tr>
<td>$e_y$</td>
<td>Yield strain = $F_y/E$</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>F</td>
<td>Loads due to fluids</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$F_e$</td>
<td>Elastic buckling stress</td>
<td>C4, C4.1, C4.2, C4.3, C6.2, D4.1</td>
</tr>
<tr>
<td>$F_m$</td>
<td>Mean value of the fabrication factor</td>
<td>F1</td>
</tr>
<tr>
<td>$F_n$</td>
<td>Nominal buckling stress</td>
<td>C4, C6.2, D4.1</td>
</tr>
<tr>
<td>$F_{nt}$</td>
<td>Nominal tensile strength of bolts</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F_{nv}$</td>
<td>Nominal shear strength of bolts</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F'_n$</td>
<td>Nominal tensile strength for bolts subject to combination of shear and tension</td>
<td>E3.4</td>
</tr>
<tr>
<td>$F_{sy}$</td>
<td>Yield point as specified in Sections A3.1 or A3.2</td>
<td>A3.1, A3.2, A3.3.2, E2.2, E3.1, E3.2</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Tensile strength as specified in Sections A3.1 or A3.2, or as reduced for low ductility steel</td>
<td>A3.1, A3.2, A3.3, A3.3.2, E2.2, E2.3, E2.4, E2.5, E3.1, E3.2, E3.3, E4</td>
</tr>
<tr>
<td>$F_{uv}$</td>
<td>Tensile strength of virgin steel specified by Section A3 or established in accordance with Section F3.3</td>
<td>A3, A5.2.2, E2.2, F3.3</td>
</tr>
<tr>
<td>$F_{wy}$</td>
<td>Yield point for design of transverse stiffeners</td>
<td>B6.1</td>
</tr>
<tr>
<td>$F_{xx}$</td>
<td>Strength level designation in AWS electrode classification</td>
<td>E2.2, E2.3, E2.4, E2.5</td>
</tr>
</tbody>
</table>
## SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y$</td>
<td>Yield point used for design, not to exceed the specified yield point or</td>
<td>A1.2, A3.3, A5.2.1, A5.2.2, B2.1, B5, B6.1, C2, C3.1, C3.1.1, C3.1.3,</td>
</tr>
<tr>
<td></td>
<td>established in accordance with Section F3, or as increased for cold work</td>
<td>C3.2, C3.5, C4, C6.1, C6.2, D1.2, D4, D4.2, E2</td>
</tr>
<tr>
<td></td>
<td>of forming in Section A5.2.2 or as reduced for low ductility steels in</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Section A3.3.2</td>
<td></td>
</tr>
<tr>
<td>$F_{ya}$</td>
<td>Average yield point of section</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>$F_{yc}$</td>
<td>Tensile yield point of corners</td>
<td>A5.2.2</td>
</tr>
<tr>
<td>$F_{yr}$</td>
<td>Weighted average tensile yield point of the flat portions</td>
<td>A5.2.2, F3.2</td>
</tr>
<tr>
<td>$F_{ys}$</td>
<td>Yield point of stiffener steel</td>
<td>B6.1</td>
</tr>
<tr>
<td>$F_{yr}$</td>
<td>Tensile yield point of virgin steel specified by Section A3 or</td>
<td>A3, A5.2.2, F3.3</td>
</tr>
<tr>
<td></td>
<td>established in accordance with Section F3.3</td>
<td></td>
</tr>
<tr>
<td>$f$</td>
<td>Stress in the compression element computed on the basis of the effective</td>
<td>B2.1, B2.2, B3.2, B4, B4.1</td>
</tr>
<tr>
<td>$f_{av}$</td>
<td>Average computed stress in the full, unreduced flange width</td>
<td>B1.1b</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Stress at service load in the cover plate or sheet</td>
<td>D1.2</td>
</tr>
<tr>
<td>$f_{cc}$</td>
<td>Specified compression stress of concrete</td>
<td>E5.1</td>
</tr>
<tr>
<td>$f_d$</td>
<td>Computed compressive stress in the element being considered. Calculations</td>
<td>B2.1, B2.2, B3.1, B4.1, B4.2</td>
</tr>
<tr>
<td></td>
<td>are based on the effective section at the load for which deflections are</td>
<td></td>
</tr>
<tr>
<td></td>
<td>determined</td>
<td></td>
</tr>
<tr>
<td>$f_{d1, f_{d2}}$</td>
<td>Computed stresses $f_1$ and $f_2$ as shown in Figure B2.3–1. Calculations</td>
<td>B2.3</td>
</tr>
<tr>
<td></td>
<td>are based on the effective section at the load for which deflections are</td>
<td></td>
</tr>
<tr>
<td></td>
<td>determined</td>
<td></td>
</tr>
<tr>
<td>$f_{d3}$</td>
<td>Computed stress $f_3$ in edge stiffener, as shown in Figure B4–2.</td>
<td>B3.2</td>
</tr>
<tr>
<td></td>
<td>Calculations are based on the effective section at the load for which</td>
<td></td>
</tr>
<tr>
<td></td>
<td>deflections are determined</td>
<td></td>
</tr>
<tr>
<td>$f_v$</td>
<td>Computed shear stress on a bolt</td>
<td>E4</td>
</tr>
<tr>
<td>$f_1$, $f_2$</td>
<td>Web stresses defined by Figure B2.3–1</td>
<td>B2.3</td>
</tr>
<tr>
<td>$f_3$</td>
<td>Edge stiffener stress defined by Figure B4–2</td>
<td>B3.2</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus of steel (11,300 ksi)</td>
<td>C3.1.1, D4.1</td>
</tr>
<tr>
<td>$G'$</td>
<td>Inelastic shear modulus</td>
<td>D4.1</td>
</tr>
<tr>
<td>$g$</td>
<td>Vertical distance between two rows of connections nearest to the top and</td>
<td>D1.1</td>
</tr>
<tr>
<td></td>
<td>bottom flanges</td>
<td></td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>Loads due to the weight and lateral pressure of soil and water in soil</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>h</td>
<td>Depth of flat portion of web measured along the plane of web</td>
<td>B1.2, B6.2, C3.2, C3.4</td>
</tr>
<tr>
<td>lₐ</td>
<td>Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element</td>
<td>B1.1, B4, B4.1, B4.2</td>
</tr>
<tr>
<td>i₀</td>
<td>Moment of inertia of the full unreduced section about the bending axis</td>
<td>C5</td>
</tr>
<tr>
<td>Iₙ</td>
<td>Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened</td>
<td>B1.1, B4, B4.1, B4.2, B5</td>
</tr>
<tr>
<td>Iₙₚ</td>
<td>Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened</td>
<td>B5</td>
</tr>
<tr>
<td>i₁, i₇</td>
<td>Moment of inertia of full section about principal axis</td>
<td>D1.1, D3.2.2</td>
</tr>
<tr>
<td>iₓᵧ</td>
<td>Product of inertia of full section about major and minor centroidal axes</td>
<td>D3.2.2, D4.1</td>
</tr>
<tr>
<td>iₓₑ</td>
<td>Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>J</td>
<td>St. Venant torsion constant</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>j</td>
<td>Section property for torsional–flexural buckling</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>K</td>
<td>Effective length factor</td>
<td>C3.1.2, C4, C4.1, C5</td>
</tr>
<tr>
<td>K'</td>
<td>A constant</td>
<td>D3.2.2</td>
</tr>
<tr>
<td>K₀</td>
<td>Effective length factor in the plane of bending</td>
<td>C5</td>
</tr>
<tr>
<td>K₀ₚ</td>
<td>Effective length factor for torsion</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Kₓ</td>
<td>Effective length factor for bending about x-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Kᵧ</td>
<td>Effective length factor for bending about y-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>k</td>
<td>Plate buckling coefficient</td>
<td>B2.1, B2.3, B3.1, B3.2, B4, B4.1, B4.2</td>
</tr>
<tr>
<td>kᵥ</td>
<td>Shear buckling coefficient</td>
<td>B6.2, C3.2</td>
</tr>
<tr>
<td>L</td>
<td>Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams</td>
<td>B1.1c, D3.2.1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbols</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>Length of seam weld not including the circular ends</td>
<td>E2.3, E2.4, E2.5</td>
</tr>
<tr>
<td>L</td>
<td>Length of fillet weld</td>
<td>C3.1.2, C4.1, D1.1, D4, D4.1</td>
</tr>
<tr>
<td>L</td>
<td>Unbraced length of member</td>
<td></td>
</tr>
<tr>
<td>Lr</td>
<td>Nominal live load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>Lt</td>
<td>Nominal roof live load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>Ltt</td>
<td>Length of transverse stiffener</td>
<td>B6.1</td>
</tr>
<tr>
<td>Lst</td>
<td>Unbraced length of compression member for torsion</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Lx</td>
<td>Unbraced length of compression member for bending about x-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Ly</td>
<td>Unbraced length of compression member for bending about y-axis</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Me</td>
<td>Critical moment</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Me</td>
<td>Elastic critical moment</td>
<td>C3.1.2</td>
</tr>
<tr>
<td>Mn</td>
<td>Mean value of the material factor</td>
<td>F1</td>
</tr>
<tr>
<td>Mn</td>
<td>Nominal flexural strength</td>
<td>C3.1, C3.1.1, C3.1.2, C3.1.3, C6.1</td>
</tr>
<tr>
<td>Mnx, Mny</td>
<td>Nominal flexural strengths about the centroidal axes determined in accordance with Section C3</td>
<td>C5</td>
</tr>
<tr>
<td>Mnxo, Mnyo</td>
<td>Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2</td>
<td>C3.3, C3.5, D4.2, D4.3</td>
</tr>
<tr>
<td>Mu</td>
<td>Required flexural strength</td>
<td>C3.3, C3.5</td>
</tr>
<tr>
<td>Mu</td>
<td>Required flexural strength about x-axis</td>
<td>C5</td>
</tr>
<tr>
<td>Muy</td>
<td>Required flexural strength about y-axis</td>
<td>C5</td>
</tr>
<tr>
<td>My</td>
<td>Moment causing a maximum strain e_y</td>
<td>B2.1, C3.1</td>
</tr>
<tr>
<td>M1</td>
<td>Smaller end moment</td>
<td>C3.1.2, C5</td>
</tr>
<tr>
<td>M2</td>
<td>Larger end moment</td>
<td>C3.1.2, C5</td>
</tr>
<tr>
<td>m</td>
<td>Distance from the shear center of one channel to the mid-plane of its web</td>
<td>D1.1, D3.2.2</td>
</tr>
<tr>
<td>N</td>
<td>Actual length of bearing</td>
<td>D3.6</td>
</tr>
<tr>
<td>n</td>
<td>Number of holes</td>
<td>E4</td>
</tr>
<tr>
<td>n</td>
<td>Number of tests</td>
<td>F1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n_p )</td>
<td>Number of parallel purlin lines</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>( P )</td>
<td>Loads, forces, and effects due to ponding</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>( P_E )</td>
<td>( \pi^2 E b/(K b L b)^2 )</td>
<td>C5</td>
</tr>
<tr>
<td>( P_L )</td>
<td>Force to be resisted by intermediate beam brace</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>( P_m )</td>
<td>Mean value of the tested-to-predicted load ratios</td>
<td>F1</td>
</tr>
<tr>
<td>( P_n )</td>
<td>Nominal axial strength of member</td>
<td>C4, C6.2</td>
</tr>
<tr>
<td>( P_{no} )</td>
<td>Nominal axial strength of member determined in accordance with Section C4 for ( L = 0 )</td>
<td></td>
</tr>
<tr>
<td>( P_p )</td>
<td>Nominal bearing capacity on concrete</td>
<td>E5.1</td>
</tr>
<tr>
<td>( P_s )</td>
<td>Concentrated load or reaction based on factored loads</td>
<td>D1.1</td>
</tr>
<tr>
<td>( P_u )</td>
<td>Required axial strength</td>
<td>C5</td>
</tr>
<tr>
<td>( \bar{Q} )</td>
<td>Design shear rigidity for sheathing on both sides of the wall assembly</td>
<td>D4.1</td>
</tr>
<tr>
<td>( Q_i )</td>
<td>Load effect</td>
<td>F1</td>
</tr>
<tr>
<td>( q )</td>
<td>Uniformly distributed factored load in the plane of the web</td>
<td>D1.1</td>
</tr>
<tr>
<td>( \bar{q} )</td>
<td>Design shear rigidity for sheathing per inch of stud spacing</td>
<td>D4.1</td>
</tr>
<tr>
<td>( q_o )</td>
<td>Factor used to determine design shear rigidity</td>
<td>D4.1</td>
</tr>
<tr>
<td>( R )</td>
<td>Reduction Factor</td>
<td>C3.1.3</td>
</tr>
<tr>
<td>( R )</td>
<td>Coefficient</td>
<td>C4, C6.2</td>
</tr>
<tr>
<td>( R )</td>
<td>Inside bend radius</td>
<td>A5.2.2, C3.4</td>
</tr>
<tr>
<td>( R_n )</td>
<td>Nominal resistance</td>
<td>A1.2, F1</td>
</tr>
<tr>
<td>( R_p )</td>
<td>Average value of all test results</td>
<td>F1</td>
</tr>
<tr>
<td>( R_f )</td>
<td>Nominal roof rain load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>( r )</td>
<td>Radius of gyration of full unreduced cross section</td>
<td>C3.1.1, C4, C4.1</td>
</tr>
<tr>
<td>( r )</td>
<td>Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section</td>
<td>E3.2</td>
</tr>
<tr>
<td>( r_{cy} )</td>
<td>Radius of gyration of one channel about its centroidal axis parallel to web</td>
<td>D1.1</td>
</tr>
<tr>
<td>( r_1 )</td>
<td>Radius of gyration of I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing</td>
<td>D1.1</td>
</tr>
</tbody>
</table>
## SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$r_0$</td>
<td>Polar radius of gyration of cross section about the shear center</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>$r_x, r_y$</td>
<td>Radius of gyration of cross section about centroidal principal axis</td>
<td>C3.1.1</td>
</tr>
<tr>
<td>$S$</td>
<td>$1.28\sqrt{E/f}$</td>
<td>B4, B4.1</td>
</tr>
<tr>
<td>$S$</td>
<td>Nominal snow load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$S_c$</td>
<td>Elastic section modulus of the effective section calculated at a stress $M_c/S_f$ in the extreme compression fiber</td>
<td>C3.1.1, C3.1.2, C4</td>
</tr>
<tr>
<td>$S_e$</td>
<td>Elastic section modulus of the effective section calculated with extreme compression or tension fiber at $F_y$</td>
<td>C3.1.1, C3.1.3</td>
</tr>
<tr>
<td>$S_f$</td>
<td>Elastic section modulus of full, unreduced section for the extreme compression fiber</td>
<td>C3.1.1, C3.1.2, C6.1</td>
</tr>
<tr>
<td>$S_{\text{max}}$</td>
<td>Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I-section</td>
<td>D1.1</td>
</tr>
<tr>
<td>$s$</td>
<td>Fastener spacing</td>
<td>D1.2, D4.1</td>
</tr>
<tr>
<td>$s$</td>
<td>Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element</td>
<td>E3.2</td>
</tr>
<tr>
<td>$s$</td>
<td>Weld spacing</td>
<td>D1.1</td>
</tr>
<tr>
<td>$T_n$</td>
<td>Nominal tensile strength</td>
<td>C2</td>
</tr>
<tr>
<td>$T_s$</td>
<td>Design strength of connection in tension</td>
<td>D1.1</td>
</tr>
<tr>
<td>$t$</td>
<td>Total thickness of the two welded sheets</td>
<td>E2.2</td>
</tr>
<tr>
<td>$t$</td>
<td>Thickness of thinnest connected part</td>
<td>E2.2, E3.1, E4</td>
</tr>
<tr>
<td>$t_s$</td>
<td>Equivalent thickness of a multiple-stiffened element</td>
<td>B5, B6.1</td>
</tr>
<tr>
<td>$t_w$</td>
<td>Effective throat of weld</td>
<td>E2.4, E2.5</td>
</tr>
<tr>
<td>$V_F$</td>
<td>Coefficient of variation of the fabrication factor</td>
<td>F1</td>
</tr>
<tr>
<td>$V_M$</td>
<td>Coefficient of variation of the material factor</td>
<td>F1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_n$</td>
<td>Nominal shear strength</td>
<td>B6.2, C3.2, C3.3</td>
</tr>
<tr>
<td>$V_P$</td>
<td>Coefficient of variation of the tested-to-predicted load ratios</td>
<td>F1</td>
</tr>
<tr>
<td>$V_Q$</td>
<td>Coefficient of variation of the load effect</td>
<td>F1</td>
</tr>
<tr>
<td>$V_u$</td>
<td>Required shear strength</td>
<td>C3.3</td>
</tr>
<tr>
<td>$W$</td>
<td>Factored load supported by all purlin lines being restrained</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>$W$</td>
<td>Nominal wind load</td>
<td>A5.1.4</td>
</tr>
<tr>
<td>$w$</td>
<td>Flat width of element exclusive of radii</td>
<td>A1.2, B1.1, B2.1, B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1, C4, D1.2</td>
</tr>
<tr>
<td>$w$</td>
<td>Flat width of the beam flange which contacts the bearing plate</td>
<td>C3.5</td>
</tr>
<tr>
<td>$w_1$</td>
<td>Width of flange projection beyond the web or half the distance between webs for box- or U-type sections</td>
<td>B1.1c</td>
</tr>
<tr>
<td>$w_1$</td>
<td>Projection of flanges from inside face of web</td>
<td>B1.1b, D1.1</td>
</tr>
<tr>
<td>$w_1$</td>
<td>Leg on weld</td>
<td>E2.4</td>
</tr>
<tr>
<td>$w_2$</td>
<td>Leg on weld</td>
<td>E2.4</td>
</tr>
<tr>
<td>$x$</td>
<td>Distance from concentrated load to brace</td>
<td>D3.2</td>
</tr>
<tr>
<td>$x_0$</td>
<td>Distance from shear center to centroid along the principal x-axis</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>$Y$</td>
<td>Yield point of web steel divided by yield point of stiffener steel</td>
<td>B6.2</td>
</tr>
<tr>
<td>$l/a_{nx}$, $l/a_{ny}$</td>
<td>Magnification factors</td>
<td>C5</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Coefficient</td>
<td>C4.2, D4.1</td>
</tr>
<tr>
<td>$\beta_0$</td>
<td>Target reliability index</td>
<td>F1</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Actual shear strain in the sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\bar{\gamma}$</td>
<td>Permissible shear strain of the sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\gamma_1$</td>
<td>Load factor</td>
<td>F1</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle between web and bearing surface $&gt;$45° but no more than 90°</td>
<td>C3.4</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle between the vertical and the plane of the web of the Z-section, degrees</td>
<td>D3.2.1</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Stress related to shear strain in sheathing</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_{cr}$</td>
<td>Theoretical elastic buckling stress</td>
<td>D4.1</td>
</tr>
</tbody>
</table>
### SYMBOLS AND DEFINITIONS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{ex}$</td>
<td>$(\pi^2E)/(K_{xx}L/r_x)^2$</td>
<td>C3.1.2, C4.2, D4.1</td>
</tr>
<tr>
<td>$\sigma_{exy}$</td>
<td>$(\pi^2E)/(L/r_x)^2$</td>
<td></td>
</tr>
<tr>
<td>$\sigma_{exy}$</td>
<td>$(\pi^2E)/(AL^2)$</td>
<td>D4.1</td>
</tr>
<tr>
<td>$\sigma_{ey}$</td>
<td>$(\pi^2E)/(K_{yy}L/r_y)^2$</td>
<td>C3.1.2, D4.1</td>
</tr>
<tr>
<td>$\sigma_{ey}$</td>
<td>$(\pi^2E)/(L/r_y)^2$</td>
<td></td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Torsional buckling stress</td>
<td>C3.1.1, C4.2, D4.1</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Reduction factor</td>
<td>B2.1</td>
</tr>
<tr>
<td>$\lambda$, $\lambda_c$</td>
<td>Slenderness factors</td>
<td>B2.1, C3.5</td>
</tr>
<tr>
<td>$\psi$</td>
<td>$f_2/f_1$</td>
<td>B2.3</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Resistance factor</td>
<td>A5.1.5, C3, C3.1.1, C3.1.2, C3.1.3, C3.3, C3.5, C5, C6.1, C6.3, D4.2, D4.3</td>
</tr>
<tr>
<td>$\phi_b$</td>
<td>Resistance factor for bending strength</td>
<td>A5.1.5, C3, C3.1.1, C3.1.2, C3.1.3, C3.3, C3.5, C5, C6.1, C6.3, D4.2, D4.3</td>
</tr>
<tr>
<td>$\phi_c$</td>
<td>Resistance factor for concentrically loaded compression member</td>
<td>A3.3.1, A5.1.5, B6.1, C4, C5, C6.2, C6.3, D4.1, D4.3</td>
</tr>
<tr>
<td>$\phi_e$</td>
<td>Resistance factor for bearing strength</td>
<td>E5.1</td>
</tr>
<tr>
<td>$\phi_t$</td>
<td>Resistance factor for tension member</td>
<td>C2</td>
</tr>
<tr>
<td>$\phi_v$</td>
<td>Resistance factor for shear strength</td>
<td>B6.2, C3.2, C3.3</td>
</tr>
<tr>
<td>$\phi_w$</td>
<td>Resistance factor for web crippling strength</td>
<td>C3.4, C3.5</td>
</tr>
</tbody>
</table>
LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This Load and Resistance Factor Design Specification is an alternate to the Specification for the Design of Cold-Formed Steel Structural Members of the American Iron and Steel Institute.

This Specification shall apply to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load-carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects.

A1.2 Terms

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) Stiffened or Partially Stiffened Compression Elements. A stiffened or partially stiffened compression element is a flat compression element (i.e., a plane compression flange of a flexural member or a plane web or flange of a compression member) of which both edges parallel to the direction of stress are stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

(b) Unstiffened Compression Elements. An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) Multiple-Stiffened Elements. A multiple-stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A sub-element is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

(d) Flat-Width-to-Thickness Ratio. The flat width of an element measured along its plane, divided by its thickness.

(e) Effective Design Width. Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

(f) Thickness. The thickness, t, of any element or section shall be the base steel thickness, exclusive of coatings.

(g) Torsional-Flexural Buckling. Torsional-flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

(h) Point-Symmetric Section. A point-symmetric section is a section symmetrical about a point (centroid) such as a Z-section having equal flanges.
(i) **Yield Point.** Yield point, $F_y$ or $F_{sy}$, as used in this Specification shall mean yield point or yield strength.

(j) **Stress.** Stress as used in this Specification means force per unit area.

(k) **Confirmatory Test.** A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) **Performance Test.** A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.

(m) **Virgin Steel.** Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

(n) **Virgin Steel Properties.** Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.

(o) **Specified Minimum Yield Point.** The specified minimum yield point is the lower limit of yield point which must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold-formed steel structural member designed at that yield point.

(p) **Cold-Formed Steel Structural Members.** Cold-formed steel structural members are shapes which are manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

(q) **LRFD (Load and Resistance Factor Design).** A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

(r) **Design Strength.** Factored resistance or strength (force, moment, as appropriate), $R_n$, provided by the structural component.

(s) **Required Strength.** Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads (using most appropriate critical load combinations).

### A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

### A2 Non-Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

### A3 Material

#### A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:
ASTM A36/A36M, Structural Steel
ASTM A242/A242M, High-Strength Low-Alloy Structural Steel
ASTM A441M, High-Strength Low-Alloy Structural Manganese Vanadium Steel
ASTM A446/A446M (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality
ASTM A500, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A529/A529M, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)
ASTM A570/A570M Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality
ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick
ASTM A606 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A607 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled
ASTM A611 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality
ASTM A715 (Grades 50 and 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability
ASTM A792 (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

Steels not listed in Section A3.1 and used for structural members and connections shall comply with one of the following ductility requirements:

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an eight-inch gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 inch gage length across the fracture shall not be less than 20\%, (2) uniform elongation outside the fracture shall not be less than 3\%.*

When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts**

* Further information on the test procedures should be obtained from the Commentary.
** Horizontal structural members which support roof deck or panel covering and applied loads principally by bending.
accordance with Sections C3.1.1(a), C3.1.2, and C3.1.3. For purlins and girts subject
to combined axial load and bending moment (Section C5), \( P_u/q_c < P_a \) shall not exceed
0.15.

A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels
which do not meet the provisions of Section A3.3.1 may be used for particular con­
figurations provided (1) the yield strength, \( F_y \), used for design in Chapters B, C and D
is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less
and (2) the tensile strength, \( F_u \), used for design in Chapter E is taken as 75 percent of
the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the
suitability of such steels for the configuration shall be demonstrated by load tests in
accordance with Section F1. Design strengths based on these tests shall not exceed
the strengths calculated according to Chapters B through E, using the specified mini­
mum yield point, \( F_{wy} \), for \( F_y \) and the specified minimum tensile strength, \( F_u \).

Design strengths based on existing use shall not exceed the strengths calcu­
lated according to Chapters B through E, using the specified minimum yield point,
\( F_{wy} \), for \( F_y \) and the specified minimum tensile strength, \( F_u \).

A3.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold-formed product as delivered to
the job site shall not at any location be less than 95 percent of the thickness, \( t \), used in its
design; however, lesser thicknesses shall be permitted at bends, such as corners, due to
cold-forming effects.

A4 Loads

A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and
all material permanently fastened thereto or supported thereby.

A4.2 Live or Snow Load

The live or snow load shall be that stipulated by the applicable code or specifica­
tion under which the structure is being designed or that dictated by the conditions in­
volved.

A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall
be increased sufficiently to provide for impact.

A4.4 Wind or Earthquake Loads

Wind or earthquake load shall be that stipulated by the applicable code or specifica­
tion under which the structure is being designed or that dictated by the conditions in­
volved.

A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drain­
age or adequate individual drains to prevent the accumulation of rainwater, the roof sys-
tem shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based on the Load and Resistance Factor Design concept. Load and Resistance Factor Design is a method of proportioning cold-formed steel structural components (i.e., members, connectors and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combination.

Two types of limit states are to be considered: 1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and 2) the limit state of the ability of the structure to perform its intended function during its life. These limit states will be called the Limit State of Strength and the Limit State of Serviceability, respectively, in these criteria.

A5.1.1 Limit State – Strength

The design meets this Specification when the required strengths, as determined from the assigned nominal loads which are multiplied by appropriate load factors, are smaller than or equal to the design strength of each structural component.

The design strength is equal to $\phi R_n$, where $\phi$ is a resistance factor and $R_n$ is the nominal strength determined according to the formulas given in Chapter C for members, in Chapter D for structural assemblies and in Chapter E for connections. Values of resistance factors $\phi$ are given in Section A5.1.5 for the appropriate limit states governing member and connection strength.

A5.1.2 Limit State – Serviceability

Serviceability is satisfactory if a nominal structural response (e.g., live load deflection) due to the applicable nominal loads is less than or equal to the appropriate acceptable or allowable value of this response.

A5.1.3 Nominal Loads

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American Society of Civil Engineers Standard, ANSI/ASCE 7-88, Minimum Design Loads for Buildings and Other Structures. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads.

A5.1.4 Load Factors and Load Combinations*

The structure and its components must be designed for the appropriate most critical load combination. The following load combinations of the factored nominal loads shall be used in the computation of the required strengths:

1. $1.4D + L$
2. $1.2D + 1.6L + 0.5(L_r or S or R_r)$
3. $1.2D + (1.4 L_r or 1.6 S or 1.6 R_r) + (0.5 L or 0.8 W)$

* For roof and floor construction, recommended load combinations for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork are given in Section A5.1 of the Commentary.
4.  \(1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R_r)\)
5.  \(1.2D + 1.5E + (0.5L \text{ or } 0.2S)\)
6.  \(0.9D - (1.3W \text{ or } 1.5E)\)

where
- \(D\) = nominal dead load
- \(E\) = nominal earthquake load
- \(L\) = nominal live load
- \(L_r\) = nominal roof live load
- \(R_r\) = nominal roof rain load
- \(S\) = nominal snow load
- \(W\) = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for \(W\) by 0.9)

**Exception:** The load factor for \(L\) in combinations (3), (4), and (5) shall be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

When the structural effects of \(F\), \(H\), \(P\) or \(T\) are significant, they shall be considered in design as the following factored loads: 1.3\(F\), 1.6\(H\), 1.2\(P\), and 1.2\(T\), where

- \(F\) = loads due to fluids with well-defined pressures and maximum heights
- \(H\) = loads due to the weight and lateral pressure of soil and water in soil
- \(P\) = loads, forces, and effects due to ponding
- \(T\) = self-straining forces and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.

### A5.1.5 Resistance Factors

The resistance factors to be used for determining the design strengths, \(\phi R_n\), of structural members and connections shall be taken as follows:

<table>
<thead>
<tr>
<th>Type of Strength</th>
<th>Resistance Factor, (\phi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Stiffeners</td>
<td></td>
</tr>
<tr>
<td>Transverse stiffeners</td>
<td>0.85</td>
</tr>
<tr>
<td>Shear stiffeners*</td>
<td>0.90</td>
</tr>
<tr>
<td>(b) Tension members</td>
<td>0.95</td>
</tr>
<tr>
<td>(c) Flexural members</td>
<td></td>
</tr>
<tr>
<td>Bending strength</td>
<td></td>
</tr>
<tr>
<td>For sections with stiffened or partially stiffened compression flanges</td>
<td>0.95</td>
</tr>
<tr>
<td>Laterally unbraced beams</td>
<td>0.90</td>
</tr>
<tr>
<td>Beams having one flange through- fastened to deck or sheathing (C- or Z-sections)</td>
<td>0.90</td>
</tr>
<tr>
<td>Web design</td>
<td></td>
</tr>
<tr>
<td>Shear strength*</td>
<td>0.90</td>
</tr>
<tr>
<td>Web Crippling</td>
<td></td>
</tr>
<tr>
<td>For single unreinforced webs</td>
<td>0.75</td>
</tr>
<tr>
<td>For I-sections</td>
<td>0.80</td>
</tr>
</tbody>
</table>

*When \(b/t \leq \sqrt{Ek_v/F_y}\) \(\phi = 1.0\)
<table>
<thead>
<tr>
<th>Type of Strength</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d) Concentrically loaded compression members</td>
<td>0.85</td>
</tr>
<tr>
<td>(e) Combined axial load and bending</td>
<td></td>
</tr>
<tr>
<td>$\phi_c$ for compression</td>
<td>0.85</td>
</tr>
<tr>
<td>$\phi_b$ for bending</td>
<td></td>
</tr>
<tr>
<td>Using Section C3.1.1</td>
<td>0.90 - 0.95</td>
</tr>
<tr>
<td>Using Section C3.1.2</td>
<td>0.90</td>
</tr>
<tr>
<td>(f) Cylindrical tubular members</td>
<td></td>
</tr>
<tr>
<td>Bending strength</td>
<td>0.95</td>
</tr>
<tr>
<td>Axial compression</td>
<td>0.85</td>
</tr>
<tr>
<td>(g) Wall studs and wall stud assemblies</td>
<td></td>
</tr>
<tr>
<td>Wall studs in compression</td>
<td>0.85</td>
</tr>
<tr>
<td>Wall studs in bending</td>
<td></td>
</tr>
<tr>
<td>For sections with stiffened or partially stiffened compression flanges</td>
<td>0.95</td>
</tr>
<tr>
<td>For sections with unstiffened compression flanges</td>
<td>0.90</td>
</tr>
<tr>
<td>(h) Welded connections</td>
<td></td>
</tr>
<tr>
<td>Groove welds</td>
<td></td>
</tr>
<tr>
<td>Tension or compression</td>
<td>0.90</td>
</tr>
<tr>
<td>Shear (welds)</td>
<td>0.80</td>
</tr>
<tr>
<td>Shear (base metal)</td>
<td>0.90</td>
</tr>
<tr>
<td>Arc spot welds</td>
<td></td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Connected part</td>
<td>0.50 - 0.65</td>
</tr>
<tr>
<td>Minimum edge distance</td>
<td>0.60 - 0.70</td>
</tr>
<tr>
<td>Arc seam welds</td>
<td></td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Connected part</td>
<td>0.60</td>
</tr>
<tr>
<td>Fillet welds</td>
<td></td>
</tr>
<tr>
<td>Longitudinal loading (connected part)</td>
<td>0.55 - 0.60</td>
</tr>
<tr>
<td>Transverse loading (connected part)</td>
<td>0.60</td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Flare groove welds</td>
<td></td>
</tr>
<tr>
<td>Transverse loading (connected part)</td>
<td>0.55</td>
</tr>
<tr>
<td>Longitudinal loading (connected part)</td>
<td>0.55</td>
</tr>
<tr>
<td>Welds</td>
<td>0.60</td>
</tr>
<tr>
<td>Resistance Welds</td>
<td>0.65</td>
</tr>
<tr>
<td>(i) Bolted connections</td>
<td></td>
</tr>
<tr>
<td>Minimum spacing and edge distance</td>
<td>0.60 - 0.70</td>
</tr>
<tr>
<td>Tension strength on net section</td>
<td></td>
</tr>
<tr>
<td>With washers</td>
<td></td>
</tr>
<tr>
<td>Double shear connection</td>
<td>0.65</td>
</tr>
<tr>
<td>Single shear connection</td>
<td>0.55</td>
</tr>
<tr>
<td>Without washers</td>
<td>0.65</td>
</tr>
<tr>
<td>Bearing strength</td>
<td></td>
</tr>
<tr>
<td>See Tables E3.3-1 and E3.3-2</td>
<td>0.55 - 0.70</td>
</tr>
<tr>
<td>Shear strength of bolts</td>
<td>0.65</td>
</tr>
<tr>
<td>Tensile strength of bolts</td>
<td>0.75</td>
</tr>
<tr>
<td>(j) Shear rupture</td>
<td>0.75</td>
</tr>
<tr>
<td>(k) Connections to other materials (Bearing)</td>
<td>0.60</td>
</tr>
</tbody>
</table>
A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, $F_y$, shall not exceed the specified minimum yield point of steels as listed in Section A3.1 or A3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

Strength increase from cold work of forming shall be permitted by substituting $F_{ya}$ for $F_y$, where $F_{ya}$ is the average yield point of the full section. Such increase shall be limited to Sections C3.1 (excluding Section C3.1.1(b)), C4, C5, C6 and D4. The limitations and methods for determining $F_{ya}$ are as follows:

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity $p$ for load capacity is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, $F_{ya}$, of the steel shall be determined on the basis of one of the following methods:

1. full section tensile tests [see paragraph (a) of Section F3.1]
2. stub column tests [see paragraph (b) of Section F3.1]
3. computed as follows:
   \[ F_{ya} = CF_{yc} + (1 - C) F_{yf} \]  
   \[ (Eq. A5.2.2-1) \]
   where
   \[ F_{ya} = \text{Average yield point of the steel in the full section of compression members or full flange sections of flexural members} \]
   \[ C = \text{For compression members, ratio of the total corner cross-sectional area to the total cross-sectional area of the full section; for flexural members, ratio of the total corner cross-sectional area of the controlling flange to the full cross-sectional area of the controlling flange} \]
   \[ F_{yf} = \text{Weighted average tensile yield point of the flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made} \]
   \[ F_{yc} = B_c F_{yv}/(R/t)^m, \text{tensile yield point of corners. This formula is applicable only when } F_{uv}/F_{yv} \geq 1.2, R/t \leq 7, \text{and minimum included angle} \leq 120^\circ \]  
   \[ (Eq. A5.2.2-2) \]
   \[ B_c = 3.69 \left( F_{uv}/F_{yv} \right) - 0.819 \left( F_{uv}/F_{yv} \right)^2 - 1.79 \]  
   \[ (Eq. A5.2.2-3) \]
   \[ m = 0.192 \left( F_{uv}/F_{yv} \right) - 0.068 \]  
   \[ (Eq. A5.2.2-4) \]
   \[ R = \text{Inside bend radius.} \]
   \[ F_{yv} = \text{Tensile yield point of virgin steel} \]  
   \[ \text{specified by Section A3 or established in accordance with Section F3.3} \]
   \[ F_{uv} = \text{Ultimate tensile strength of virgin steel} \]  
   \[ \text{specified by Section A3 or established in accordance with Section F3.3} \]

(b) For axially loaded tension members the yield point of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this Section.

* Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold-forming operation.
(c) The effect of any welding on mechanical properties of a member shall be determined on the basis of tests of full section specimens containing within the gage length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

A5.3 Durability

A structure shall be designed to perform its required functions during its expected life for durability considerations.

A6 Reference Documents

The following documents are referenced in this Specification:

   - ASTM A36/A36M–84a, Structural Steel
   - ASTM A194–88, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
   - ASTM A242/A242M–83, High-Strength Low-Alloy Structural Steel
   - ASTM A307–84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners
   - ASTM A325–84, High Strength Bolts for Structural Steel Joints
   - ASTM A354–84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 inch)
   - ASTM A370–77 Mechanical Testing of Steel Products
   - ASTM A441M–85, High-Strength Low-Alloy Structural Manganese Vanadium Steel
   - ASTM A446/A446M–85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality
   - ASTM A449–84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 1/2 inch)
ASTM A490-84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.

ASTM A500-84, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M-85, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A563-88a, Carbon and Alloy Steel Nuts

ASTM A570/A570M-85 Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality

ASTM A572/A572M-85, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality

ASTM A588/A588M-85, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606-85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607-85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled

ASTM A611-85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality

ASTM A715-85 (Grades 50 & 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability

ASTM A792-85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

ASTM F436-86, Hardened Steel Washers

ASTM F844-83(1988), Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959-85, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners
B. ELEMENTS

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) Maximum Flat-Width-to-Thickness Ratios

Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

1. Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by:
   - Simple lip: 60
   - Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section B4.2: 90

2. Stiffened compression element with both longitudinal edges connected to other stiffened elements: 500

3. Unstiffened compression element and elements with an edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section B4.2: 60

Note: Unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full design strength, without affecting the ability of the member to develop required strength.

Stiffened elements having w/t ratios larger than 500 can be used with adequate design strength to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design formulas of this Specification.

(b) Flange Curling

Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{0.061t d E/fav \sqrt{100 c_f/d}}$$

(Eq. B1.1-1)

where

- $w_f$ = Width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams
- $t$ = Flange thickness
- $d$ = Depth of beam
\( c_t = \text{Amount of curling}^* \)
\( f_{av} = \text{Average stress in the full, unreduced flange width.} \) (Where members are designed by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)

(c) **Shear Lag Effects - Short Spans Supporting Concentrated Loads**

Where the span of the beam is less than \(30w_f\) (\(w_f\) as defined below) and it carries one concentrated load, or several loads spaced farther apart than \(2w_f\), the effective design width of any flange, whether in tension or compression, shall be limited to the following:

### TABLE B1.1(c)

**SHORT, WIDE FLANGES**

**MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH**

<table>
<thead>
<tr>
<th>(L/w_f)</th>
<th>Ratio</th>
<th>(L/w_f)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>1.00</td>
<td>14</td>
<td>0.82</td>
</tr>
<tr>
<td>25</td>
<td>0.96</td>
<td>12</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>0.91</td>
<td>10</td>
<td>0.73</td>
</tr>
<tr>
<td>18</td>
<td>0.89</td>
<td>8</td>
<td>0.67</td>
</tr>
<tr>
<td>16</td>
<td>0.86</td>
<td>6</td>
<td>0.55</td>
</tr>
</tbody>
</table>

where

\( L = \) Full span for simple beams; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.
\( w_f = \) Width of flange projection beyond the web for I-beam and similar sections or half the distance between webs of box or U-type sections.

For flanges of I-beams and similar sections stiffened by lips at the outer edges, \(w_f\) shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

### B1.2 Maximum Web Depth-to-Thickness Ratio

The ratio, \(h/t\), of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: \((h/t)_{\text{max}} = 200\)
(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:

(1) When using bearing stiffeners only, \((h/t)_{\text{max}} = 260\)
(2) When using bearing stiffeners and intermediate stiffeners, \((h/t)_{\text{max}} = 300\)

In the above, \(h = \) Depth of flat portion of web measured along the plane of web

* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 percent of the depth of the section is usually not considered excessive.
1-30 Cold-Formed LRFD Specification – March 16, 1991

\( t \) = Web thickness
Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

**B2 Effective Widths of Stiffened Elements**

**B2.1 Uniformly Compressed Stiffened Elements**

(a) **Load Capacity Determination**

The effective widths, \( b \), of uniformly compressed elements shall be determined from the following formulas:

\[
\begin{align*}
\text{when } \lambda \leq 0.673 \quad b &= w \\
\text{when } \lambda > 0.673 \quad b &= \rho w
\end{align*}
\]

where

\[
\begin{align*}
w &= \text{Flat width as shown in Figure B2.1-1} \\
\rho &= (1 - 0.22/A) / A \\
\lambda &= \frac{1.052}{k} \left( \frac{w}{t} \right) \sqrt{\frac{f}{E}}
\end{align*}
\]

where

\[
\begin{align*}
t &= \text{Thickness of the uniformly compressed stiffened elements, and} \\
f &= \text{for load capacity determination is as follows:}
\end{align*}
\]

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, \( f = F_y \) if the initial yielding is in compression in the element considered.

If the initial yielding is in tension, the compressive stress, \( f \), in the element considered shall be determined on the basis of the effective section at \( M_y \) (moment causing initial yield).

(2) If Procedure II of Section C3.1.1 is used then \( f \) is the stress in the element considered at \( M_n \) determined on the basis of the effective section.

(3) If Section C3.1.2 is used, then \( f \) is the stress \( M_n / S_t \) as described in that Section in determining \( S_x \).

For compression members \( f \) is taken equal to \( F_n \) as determined in Section C4 or D4.1 as applicable.

\[
\begin{align*}
E &= \text{Modulus of elasticity} \\
k &= \text{Plate buckling coefficient} \\
&= 4 \text{ for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.}
\end{align*}
\]

(b) **Deflection Determination**

The effective widths, \( b_d \), used in computing deflection shall be determined from the following formulas:

\[
\begin{align*}
\text{when } \lambda \leq 0.673 \quad b_d &= w \\
\text{when } \lambda > 0.673 \quad b_d &= \rho w
\end{align*}
\]

where
Figure B2.1–1 Stiffened Elements

\[
\begin{align*}
w &= \text{Flat width} \\
\rho &= \text{Reduction factor determined by either of the following two procedures:} \\
(1) \text{ Procedure I.} \\
\text{A low estimate of the effective width may be obtained from Eqs. B2.1–3} \\
\text{and B2.1–4 where } f_d \text{ is substituted for } f \text{ where } f_d \text{ is the computed compressive stress in the element being considered.} \\
(2) \text{ Procedure II.} \\
\text{For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating } \rho \\
\text{as follows:} \\
\rho &= 1 \text{ when } \lambda \leq 0.673 \quad (Eq. B2.1–7) \\
\rho &= (1.358 - 0.461/\lambda)K \text{ when } 0.673 < \lambda < \lambda_c \quad (Eq. B2.1–8) \\
\rho &= (0.41 + 0.59 \sqrt{F_y/f_d} - 0.22/\lambda)K \text{ when } \lambda \geq \lambda_c \quad (Eq. B2.1–9) \\
\rho \text{ shall not exceed 1.0 for all cases.} \\
\text{where} \\
\lambda_c &= 0.256 + 0.328 (w/t) \sqrt{F_y/E} \quad (Eq. B2.1–10) \\
\text{and } \lambda \text{ is as defined by Eq. B2.1–4 except that } f_d \text{ is substituted for } f. \\
\end{align*}
\]

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) Load Capacity Determination

The effective width, b, of stiffened elements with uniform compression having circular holes shall be determined as follows:

for \(0.50 \geq \frac{d_h}{w} \geq 0\), and \(\frac{w}{t} \leq 70\)

center-to-center spacing of holes > 0.50w, and 3d_h,

\[
\begin{align*}
b &= w - d_h \text{ when } \lambda \leq 0.673 \quad (Eq. B2.2–1) \\
b &= \frac{w \left[1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w}\right]}{\lambda} \text{ when } \lambda > 0.673 \quad (Eq. B2.2–2) \\
b \text{ shall not exceed } w - d_h
\end{align*}
\]

where
w = Flat width
\( d_h \) = Diameter of holes
\( \lambda \) is as defined in Section B2.1.

(b) *Deflection Determination*

The effective width, \( b_e \), used in deflection calculations shall be equal to \( b \) determined in accordance with Procedure I of Section B2.2a except that \( f_d \) is substituted for \( f \), where \( f_d \) is the computed compressive stress in the element being considered.

### B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient

(a) *Load Capacity Determination*

The effective widths, \( b_1 \) and \( b_2 \), as shown in Figure B2.3–1 shall be determined from the following formulas:

\[
\begin{align*}
\text{For } \psi \leq -0.236 & \\
\ b_1 &= b/3 - \psi \\
\text{For } \psi > -0.236 & \\
\ b_2 &= b_1/2 \\
\end{align*}
\]

\( b_1 + b_2 \) shall not exceed the compression portion of the web calculated on the basis of effective section.

For \( \psi > -0.236 \)

\[
\begin{align*}
\ b_2 &= b_e - b_1 \\
\end{align*}
\]

where

\[
\begin{align*}
\ b_e &= \text{Effective width } b \text{ determined in accordance with Section B2.1 with } f_1 \text{ substituted for } f \text{ and with } k \text{ determined as follows:} \\
\ k &= 4 + 2(1 - \psi)^3 + 2(1 - \psi) \\
\end{align*}
\]

\[
\begin{align*}
\ \psi &= f_2/f_1 \\
\end{align*}
\]

\( f_1, f_2 = \text{Stresses shown in Figure B2.3–1 calculated on the basis of effective section.} \)

\( f_1 \) is compression (+) and \( f_2 \) can be either tension (−) or compression. In case \( f_1 \) and \( f_2 \) are both compression, \( f_1 \geq f_2 \)

(b) *Deflection Determination*

The effective widths in computing deflections at a given load shall be determined in accordance with Section B2.3a except that \( f_d \) and \( f_d \) are substituted for \( f_1 \) and \( f_2 \), where \( f_d, f_d = \text{Computed stresses } f_1 \text{ and } f_2 \text{ as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.} \)

### B3 Effective Widths of Unstiffened Elements

#### B3.1 Uniformly Compressed Unstiffened Elements

(a) *Load Capacity Determination*

Effective widths, \( b \), of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that \( f_0 \) shall be taken as 0.43 and \( w \) as defined in Figure B3.1–1.

(b) *Deflection Determination*

The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that \( f_0 \) is substituted for \( f \) and \( k = 0.43 \).
B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) *Load Capacity Determination*

Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with \( f = f_3 \) as in Figure B4−2 in the element and \( k = 0.43 \).

(b) *Deflection Determination*

Effective widths, b, of unstiffened compression elements and edge stiffeners with...
stress gradient shall be determined in accordance with Procedure I of Section B2.1b except that \( f_d \) is substituted for \( f \) and \( k = 0.43 \).

**B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener**

The following notation is used in this section.

\[
S = 1.28\sqrt{E/f} \quad \text{(Eq. B4-1)}
\]

\( k \) = Buckling coefficient

\( b_0 \) = Dimension defined in Figure B4-1

\( d, w, D \) = Dimensions defined in Figure B4-2

\( d_s \) = Reduced effective width of the stiffener as specified in this section. \( d_s \), calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4-2)

\( d_s' \) = Effective width of the stiffener calculated according to Section B3.1 (see Figure B4-2)

\( C_1, C_2 \) = Coefficients defined in Figure B4-2

\( A_s \) = Reduced area of the stiffener as specified in this section. \( A_s \) is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

\( I_s \) = Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.

\( I_s, A_s' \) = Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure B4-2:

\[
I_s = \frac{(d_s t \sin^2 \theta)}{12} \quad \text{(Eq. B4-2)}
\]

\[
A_s' = d_s' t \quad \text{(Eq. B4-3)}
\]

**B4.1 Uniformly Compressed Elements with an Intermediate Stiffener**

(a) **Load Capacity Determination**

Case I: \( b_0 t \leq S \)

\[
I_s = 0 \quad \text{(no intermediate stiffener needed)} \quad \text{(Eq. B4.1-2)}
\]

\( b = w \)

\( A_s = A_s' \) (Eq. B4.1-4)

Case II: \( S < b_0 t < 3S \)

\[
I_s/t^4 = [50(b_0 t)/S] - 50 \quad \text{(Eq. B4.1-5)}
\]

\( b \) and \( A_s \) are calculated according to Section B2.1a where

\[
k = 3(I_s/I_s) \quad 1 \leq 4 \quad \text{(Eq. B4.1-6)}
\]

\( A_s = A_s'(I_s/I_s) \leq A_s' \) (Eq. B4.1-7)

Case III: \( b_0 t \geq 3S \)

\[
I_s/t^4 = [128(b_0 t)/S] - 285 \quad \text{(Eq. B4.1-8)}
\]

\( b \) and \( A_s \) are calculated according to Section B2.1a where

\[
k = 3(I_s/I_s) + 1 \quad 4 \quad \text{(Eq. B4.1-9)}
\]

\( A_s = A_s'(I_s/I_s) \leq A_s' \) (Eq. B4.1-10)

(b) **Deflection Determination**

Effective widths shall be determined as in Section B4.1a except that \( f_d \) is substituted for \( f \).
B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) Load Capacity Determination

Case I: \( w/t \leq S/3 \)
- \( I_a = 0 \) (no edge stiffener needed) \( (Eq. \ B4.2-1) \)
- \( b = w \) \( (Eq. \ B4.2-2) \)
- \( d_s = d_s' \) for simple lip stiffener \( (Eq. \ B4.2-3) \)
- \( A_s = A_s' \) for other stiffener shapes \( (Eq. \ B4.2-4) \)

Case II: \( S/3 < w/t < S \)
- \( I_w/t^4 = 3991 [(w/t)/S] - 0.33 \) \( (Eq. \ B4.2-5) \)
- \( n = 1/2 \) \( (Eq. \ B4.2-6) \)
- \( C_2 = I_w/I_s \leq 1 \) \( (Eq. \ B4.2-7) \)
- \( C_1 = 2 - C_2 \) \( (Eq. \ B4.2-8) \)

- \( b \) shall be calculated according to Section B2.1 where
  - \( k = [4.82 - 5(D/w)](I_w/I_s)^n + 0.43 \leq 5.25 - 5(D/w) \) for \( 0.8 \geq D/w > 0.25 \) \( (Eq. \ B4.2-9) \)
  - \( k = 3.57(I_w/I_s)^n + 0.43 \leq 4.0 \) for \( (D/w) \leq 0.25 \) \( (Eq. \ B4.2-10) \)
  - \( d_s = d_s' \) \( (I_w/I_s) \leq d_s' \) \( (Eq. \ B4.2-11) \)
  - \( A_s = A_s' \) \( (I_w/I_s) \leq A_s' \) \( (Eq. \ B4.2-12) \)

Case III: \( w/t \geq S \)
- \( I_w/t^4 = [115 (w/t)/S] + 5 \) \( (Eq. \ B4.2-13) \)
- \( C_1, C_2, b, k, d_s, A_s \) are calculated per Case II with \( n = 1/3 \).

(b) Deflection Determination

Effective widths shall be determined as in Section B4.2a except that \( f_s \) is substituted for \( f \).
B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

For the determination of the effective width, the intermediate stiffener of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener shall be disregarded unless each intermediate stiffener has the minimum $I_s$ as follows:

$$I_{min} = \left[ 3.66 \sqrt{\left( \frac{w}{t} \right)^2 - \left( \frac{0.136E}{F_y} \right)} \right] t^4$$

(Eq. B5-1)

where

- $w/t =$ Width-thickness ratio of the larger stiffened sub-element
- $I_s =$ Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened
- $W =$ Width
- $t =$ Thickness

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub-element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat-width to thickness ratio of the entire multiple-stiffened element, such element shall be considered as replaced by an "equivalent element" without intermediate stiffeners whose width, $b_e$, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, $t_e$, is determined as fol-
where
\[ I_{sf} = \text{Moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis.} \]

The moment of inertia of the entire section shall be calculated assuming the "equivalent element" to be located at the centroidal axis of the multiple stiffened element, including the intermediate stiffener. The actual extreme fiber distance shall be used in computing the section modulus.

(d) If \( w/t > 60 \), the effective width, \( b_e \), of the sub-element or element shall be determined from the following formula:

\[
\frac{b_e}{t} = \frac{b}{t} - 0.10 \left[ \frac{w}{t} - 60 \right]
\]

where
\[ \frac{w}{t} = \text{flat-width ratio of sub-element or element} \]
\[ b = \text{effective design width determined in accordance with the provisions of Section B2.1} \]
\[ b_e = \text{effective design width of sub-element or element to be used in design computations} \]

For computing the effective structural properties of a member having compression sub-elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For \( 60 < \frac{w}{t} < 90 \):

\[ A_{ef} = \alpha A_{st} \]  
(Eq. B5-4)

where
\[ \alpha = \left( 3 - \frac{2b_e}{w} \right) - \frac{1}{30} \left[ 1 - \frac{b_e}{w} \right] \frac{w}{t} \]  
(Eq. B5-5)

For \( \frac{w}{t} \geq 90 \):

\[ A_{ef} = \left( \frac{b_e}{w} \right) A_{st} \]  
(Eq. B5-6)

In the above expressions, \( A_{ef} \) and \( A_{st} \) refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

B6 Stiffeners

B6.1 Transverse Stiffeners

Transverse stiffeners attached to beam webs at points of concentrated loads or reactions, shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E. Required strengths for the concentrated loads or reactions shall not exceed the design strength, \( \phi_e P_n \), where \( \phi_e = 0.85 \) and \( P_n \) is the smaller value given by (a) and (b) as follows:
(a) \( P_n = F_{wy}A_c \)  
\( (Eq. \ B6.1-1) \)

(b) \( P_n = \text{Nominal axial strength evaluated according to Section C4(a) with } A_c \text{ replaced by } A_b \)

where

\[
A_c = 18t^2 + A_s, \text{ for transverse stiffeners at interior support and under concentrated load} \quad (Eq. \ B6.1-2)
\]

\[
F_{wy} = \text{Lower value of beam web, } F_y \text{ or stiffener section, } F_{ys} \quad (Eq. \ B6.1-3)
\]

\[
A_b = b_1t + A_s, \text{ for transverse stiffeners at interior support and under concentrated load} \quad (Eq. \ B6.1-4)
\]

\[
A_b = b_2t + A_s, \text{ for transverse stiffeners at end support} \quad (Eq. \ B6.1-5)
\]

\[
A_s = \text{Cross sectional area of transverse stiffeners} \quad (Eq. \ B6.1-6)
\]

\[
b_1 = 25t \left[ 0.0024(L_w/t) + 0.72 \right] \leq 25t \quad (Eq. \ B6.1-7)
\]

\[
b_2 = 12t \left[ 0.0044(L_w/t) + 0.83 \right] \leq 12t \quad (Eq. \ B6.1-7)
\]

\[
L_w = \text{Length of transverse stiffener} \quad (Eq. \ B6.1-7)
\]

\[
t = \text{Base thickness of beam web} \quad (Eq. \ B6.1-7)
\]

The \( w/t_b \) ratio for the stiffened and unstiffened elements of cold-formed steel transverse stiffeners shall not exceed \( 1.28 \sqrt{(E/F_{ys})} \) and \( 0.37 \sqrt{(E/F_{ys})} \), respectively, where \( F_{ys} \) is the yield stress, \( F_y \), and \( t \) the thickness of the stiffener steel.

**B6.2 Shear Stiffeners**

Where shear stiffeners are required, the spacing shall be such that the required shear strength shall not exceed the design shear strength, \( \phi \cdot V_n \), permitted by Section C3.2, and the ratio \( a/h \) shall not exceed \( [260/(h/t)]^2 \) nor 3.0.

The actual moment of inertia, \( I_s \), of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value of

\[
I_{smin} = 5ht \left[ h/a - 0.7(a/h) \right] \geq (h/50)^4 \quad (Eq. \ B6.2-1)
\]

The gross area of shear stiffeners shall be not less than

\[
A_{ss} = \frac{1-C_v}{2} \left[ a - \frac{a}{h} \left( \frac{a}{h} \right)^2 \sqrt{1+\left( \frac{a}{h} \right)^2} \right] YDht \quad (Eq. \ B6.2-2)
\]

where

\[
C_v = \frac{45,000k_v}{F_y(h/t)^2} \text{ when } C_v \leq 0.8 \quad (Eq. \ B6.2-3)
\]

\[
C_v = \frac{190}{h/t} \left( \sqrt{\frac{k_v}{F_y}} \right) \text{ when } C_v > 0.8 \quad (Eq. \ B6.2-4)
\]

\[
k_v = 4.00 + \frac{5.34}{(a/h)^2} \text{ when } a/h \leq 1.0 \quad (Eq. \ B6.2-5)
\]

\[
k_v = 5.34 + \frac{4.00}{(a/h)^2} \text{ when } a/h > 1.0 \quad (Eq. \ B6.2-6)
\]

\[
a = \text{Distance between transverse stiffeners}
\]

\[
Y = \frac{\text{Yield point of web steel}}{\text{Yield point of stiffener steel}}
\]

\[
D = 1.0 \text{ for stiffeners furnished in pairs}
\]
\[ D = 1.8 \text{ for single-angle stiffeners} \]
\[ D = 2.4 \text{ for single-plate stiffeners} \]
\[ t \text{ and } h \text{ are as defined in Section B1.2} \]

**B6.3 Non-Conforming Stiffeners**

The design strength of members with transverse stiffeners that do not meet the requirements of Section B6.1 or B6.2, such as stamped or rolled-in transverse stiffeners, shall be determined by tests in accordance with Chapter F of this Specification.
C. MEMBERS

C1 Properties of Sections

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the design tensile strength, $\phi T_n$, shall be determined as follows:

$$\phi T_n = 0.95 T_n = \frac{A_n F_y}{\text{Eq. C2-1}}$$

where

- $T_n$ = Nominal strength of member when loaded in tension
- $\phi$ = Resistance factor for tension
- $A_n$ = Net area of the cross section
- $F_y$ = Design yield stress as determined in Section A5.2.1

For tension members using bolted connections, the design tensile strength shall also be limited by Section E3.2.

C3 Flexural Members

C3.1 Strength for Bending Only

The design flexural strength, $\phi b M_n$, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, and C3.1.3.

C3.1.1 Nominal Section Strength

The design flexural strength, $\phi b M_n$, shall be determined with $\phi b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and the nominal section strength, $M_n$, calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

(a) Procedure I – Based on Initiation of Yielding

Effective yield moment based on section strength, $M_n$, shall be determined as follows:

$$M_n = S_e F_y$$  \hspace{1cm} (Eq. C3.1.1-1)

where

- $F_y$ = Design yield stress as determined in Section A5.2.1
- $S_e$ = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at $F_y$

(b) Procedure II – Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity may be used when the following conditions are met:
(1) The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.

(2) The effect of cold forming is not included in determining the yield point \( F_y \).

(3) The ratio of the depth of the compressed portion of the web to its thickness does not exceed \( \lambda_1 \).

(4) The shear force does not exceed 0.35\( F_y \) times the web area, \( b t \).

(5) The angle between any web and the vertical does not exceed 30 degrees.

The nominal flexural strength, \( M_n \), shall not exceed either 1.25 \( S_e F_y \) determined according to Procedure I or that causing a maximum compression strain of \( C_y e_y \) (no limit is placed on the maximum tensile strain).

where

\[ e_y = \text{Yield strain} = \frac{F_y}{E} \]

\[ E = \text{Modulus of elasticity} \]

\[ C_y = \text{Compression strain factor determined as follows:} \]

(a) Stiffened compression elements without intermediate stiffeners

\[ C_y = 3 \text{ for } w/t \leq \lambda_1 \]

\[ C_y = 3 - 2 \left( \frac{w/t - \lambda_1}{\lambda_2 - \lambda_1} \right) \text{ for } \lambda_1 < \frac{w}{t} < \lambda_2 \]

\[ C_y = 1 \text{ for } w/t \geq \lambda_2 \]

where

\[ \lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \]  

\[ \lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \]

(b) Unstiffened compression elements

\[ C_y = 1 \]

(c) Multiple-stiffened compression elements and compression elements with edge stiffeners

\[ C_y = 1 \]

When applicable, effective design widths shall be used in calculating section properties. \( M_n \) shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

The design strength of the laterally unbraced segments of singly-, doubly-, and point-symmetric sections* subject to lateral buckling, \( \phi_b M_n \) shall be determined with \( \phi_b = 0.90 \) and \( M_n \) calculated as follows:

\[ M_n = S_c \frac{M_c}{S_f} \]  

\[ \text{(Eq. C3.1.2-1)} \]

where

* The provisions of this Section apply to I-, Z-, C- and other singly-symmetric section flexural members (not including multiple-web deck, U- and closed box-type members, and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections. Refer to C3.1.3 for C- and Z-purlins in which the tension flange is attached to sheathing.
$S_f = \text{Elastic section modulus of the full unreduced section for the extreme compression fiber}$

$S_e = \text{Elastic section modulus of the effective section calculated at a stress } M_e / S_f$ in the extreme compression fiber

$M_e = \text{Critical moment calculated according to (a) or (b) below:}$

(a) For singly-, doubly-, and point-symmetric sections:

For $M_e > 0.5 M_y$

$$M_e = M_y \left(1 - \frac{M_y}{4 M_e}\right) \quad (\text{Eq. C3.1.2-2})$$

For $M_e \leq 0.5 M_y$

$$M_e = M_e \quad (\text{Eq. C3.1.2-3})$$

where

$M_y = \text{Moment causing initial yield at the extreme compression fiber of the full section}$

$= S_f F_y \quad (\text{Eq. C3.1.2-4})$

$M_e = \text{Elastic critical moment computed by the following equations:}$

$M_e = C_b A \sqrt{\sigma_{ey} \sigma_{ey}}$ for bending about the symmetry axis. For singly-symmetric sections, $x$-axis is the axis of symmetry oriented such that the shear center has a negative $x$-coordinate.

For point-symmetric sections, use $0.5 M_e$.

Alternatively, $M_e$ can be calculated using the formula for doubly-symmetric I-sections or point-symmetric sections given in (b)

$$M_e = C_b A \sigma_{ex} \left[ j + C_s \sqrt{j^2 + r_0^2 (\sigma_1 / \sigma_{ex})} \right] / C_{TF} \quad (\text{Eq. C3.1.2-6})$$

for bending about the centroidal axis perpendicular to the symmetry axis for singly-symmetric sections only

$C_s = +1$ for moment causing compression on the shear center side of the centroid

$C_s = -1$ for moment causing tension on the shear center side of the centroid

$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / \tau_x)^2} \quad (\text{Eq. C3.1.2-7})$

$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / \tau_y)^2} \quad (\text{Eq. C3.1.2-8})$

$\sigma_1 = \frac{1}{A r_s^2} \left[G I + \frac{\pi^2 E C_w}{(K L_w)^2} \right] \quad (\text{Eq. C3.1.2-9})$

$A = \text{Full cross-sectional area}$

$C_b = \text{Bending coefficient which can conservatively be taken as unity, or calculated from}$

$C_b = 1.75 + 1.05 (M_1 / M_2) + 0.3 (M_1 / M_2)^2 \leq 2.3$

where

$M_1$ is the smaller and $M_2$ the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where $M_1 / M_2$, the ratio of end moments, is positive
when $M_1$ and $M_2$ have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), $C_b$ shall be taken as unity.

$$E = \text{Modulus of elasticity}$$

$$C_{TF} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right)$$

where

$M_1$ is the smaller and $M_2$ the larger bending moment at the ends of the unbraced length, and where $M_1/M_2$, the ratio of end moments, is positive when $M_1$ and $M_2$ have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), $C_{TF}$ shall be taken as unity.

$r_o = \text{Polar radius of gyration of the cross section about the shear center}$

$$= \sqrt{r_x^2 + r_y^2 + x_0^2}$$  \hspace{1cm} (Eq. C3.1.2-10)

$r_x, r_y = \text{Radii of gyration of the cross section about the centroidal principal axes}$

$G = \text{Shear modulus}$

$K_x, K_y, K_t = \text{Effective length factors for bending about the } x \text{- and } y \text{-axes, and for twisting}$

$L_x, L_y, L_t = \text{Unbraced length of compression member for bending about the } x \text{- and } y \text{-axes, and for twisting}$

$x_0 = \text{Distance from the shear center to the centroid along the principal } x \text{-axis, taken as negative}$

$J = \text{St. Venant torsion constant of the cross section}$

$C_w = \text{Torsional warping constant of the cross section}$

$$j = \frac{1}{2y} \left[ \int_A x^3 dA + \int_A xy^2 dA \right] - x_0$$  \hspace{1cm} (Eq. C3.1.2-11)

(b) For I- or Z-sections bent about the centroidal axis perpendicular to the web ($x$-axis):

In lieu of (a), the following equations may be used to evaluate $M_c$:

For $M_c \geq 2.78M_y$

$$M_c = M_y$$  \hspace{1cm} (Eq. C3.1.2-12)

For $2.78M_y > M_c > 0.56M_y$

$$M_c = \frac{10}{9} M_y \left( 1 - \frac{10M_y}{36M_c} \right)$$  \hspace{1cm} (Eq. C3.1.2-13)

For $M_c \leq 0.56M_y$

$$M_c = M_c$$  \hspace{1cm} (Eq. C3.1.2-14)

where


Me = Elastic critical moment determined either as defined in (a) above or as follows:
\[
\begin{align*}
\frac{\pi^2 E I_{yc}}{L^2} & \text{ for doubly-symmetric I-sections} & \text{(Eq. C3.1.2-15)} \\
\frac{\pi^2 E I_{yc}}{2L^2} & \text{ for point-symmetric Z-sections} & \text{(Eq. C3.1.2-16)}
\end{align*}
\]

\(d\) = Depth of section
\(L\) = Unbraced length of the member
\(I_{yc}\) = Moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, using the full unreduced section

Other terms are defined in (a).

**C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing**

This section does not apply to a continuous beam for the region between inflection points adjacent to a support, or to a cantilever beam.

The design flexural strength, \(\phi_b M_n\), of a Channel or Z-section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be determined with \(\phi_b = 0.90\) and the nominal flexural strength, \(M_n\), calculated as follows:

\[M_n = RS_Fy\]  

where

\[R = \begin{align*}
0.40 & \text{ for simple span C sections} \\
0.50 & \text{ for simple span Z sections} \\
0.60 & \text{ for continuous span C sections} \\
0.70 & \text{ for continuous span Z sections}
\end{align*}\]

\(S_F\) and \(F_y\) are defined in Section C3.1.1

The reduction factor, \(R\), shall be limited to roof and wall systems meeting the following conditions:

1. Member depth less than 11.5 inches
2. The flanges are edge stiffened compression elements
3. \(60 \leq \text{depth/thickness} \leq 170\)
4. \(2.8 \leq \text{depth/flange width} \leq 4.5\)
5. \(16 \leq \text{flat width/thickness of flange} \leq 43\)
6. For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) shall not be less than:
   - \(1.5d\) for Zee sections
   - \(3.0d\) for Channel sections
7. Member span length no greater than 33 feet
8. For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
9. Both flanges are prevented from moving laterally at the supports
10. Roof or wall panels shall be steel sheets, minimum of 0.019 in. coated thickness, having a minimum rib depth of 1 in., spaced a maximum of 12 in. on
centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange.

(11) Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used.

(12) Fastener type: minimum No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. rivets, washers 1/2 in. diameter.

(13) Fasteners shall not be standoff type screws.

(14) Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange.

If variables fall outside any of the above stated limits, the user must perform full scale tests in accordance with Section F1 of the Specification, or apply another rational analysis procedure. In any case, the user is permitted to perform tests, in accordance with Section F1, as an alternate to the procedure described in this section.

C3.2 Strength for Shear Only

The design shear strength, \( \phi V_n \), at any section shall be calculated as follows:

(a) For \( h/t \leq \sqrt{E_{kv}/F_y} \)

\[ \phi_v = 1.0 \]

\[ V_n = 0.577F_yht \]

\( (Eq. C3.2-1) \)

(b) For \( \sqrt{E_{kv}/F_y} < h/t \leq 1.415\sqrt{E_{kv}/F_y} \)

\[ \phi_v = 0.90 \]

\[ V_n = 0.64t^2\sqrt{k_vF_yE} \]

\( (Eq. C3.2-2) \)

(c) For \( h/t > 1.415\sqrt{E_{kv}/F_y} \)

\[ \phi_v = 0.90 \]

\[ V_n = 0.905E_{kv}t^3/h \]

\( (Eq. C3.2-3) \)

where

\( \phi_v = \) Resistance factor for shear

\( V_n = \) Nominal shear strength of beam

\( t = \) Web thickness

\( h = \) Depth of the flat portion of the web measured along the plane of the web

\( k_v = \) Shear buckling coefficient determined as follows:

1. For unreinforced webs, \( k_v = 5.34 \)

2. For beam webs with transverse stiffeners satisfying the requirements of Section B6 when \( a/h \leq 1.0 \)

\[ k_v = 4.00 + \frac{5.34}{(a/h)^2} \]

\( (Eq. C3.2-4) \)

when \( a/h > 1.0 \)

\[ k_v = 5.34 + \frac{4.00}{(a/h)^2} \]

\( (Eq. C3.2-5) \)

where

\( a = \) the shear panel length for unreinforced web element

\( = \) distance between transverse stiffeners for reinforced web elements.
For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

### C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the required flexural strength, $M_u$, and the required shear strength, $V_u$, shall satisfy the following interaction equation:

$$\left(\frac{M_u}{\phi_b M_{n\alpha}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \leq 1.0$$  \hspace{1cm} (Eq. C3.3-1)

For beams with transverse web stiffeners, the required flexural strength, $M_u$, and the required shear strength, $V_u$, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively. When $M_u/\phi_b M_{n\alpha}) > 0.5$ and $V_u/\phi_v V_n > 0.7$, then $M_u$ and $V_u$ shall satisfy the following interaction equation:

$$0.6 \left(\frac{M_u}{\phi_b M_{n\alpha}}\right) + \left(\frac{V_u}{\phi_v V_n}\right) \leq 1.3$$ \hspace{1cm} (Eq. C3.3-2)

In the above:

- $\phi_b = \text{Resistance factor for bending (See Section C3.1)}$
- $\phi_v = \text{Resistance factor for shear (See Section C3.2)}$
- $M_n = \text{Nominal flexural strength when bending alone exists}$
- $M_{n\alpha} = \text{Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2}$
- $V_n = \text{Nominal shear strength when shear alone exists}$

### C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, and in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, $h/t$, equal to or less than 200, the required strength for the concentrated loads and reactions shall not exceed the values of $\phi_w P_n$, with $\phi_w = 0.75$ and 0.80 for single unreinforced webs and I-sections, respectively, and $P_n$ given in Table C3.4-1. Webs of flexural members for which $h/t$ is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4-1 apply to beams when $R/t \leq 6$ and to deck when $R/t \leq 7, N/t \leq 210$ and $N/h \leq 3.5$.

$P_n$ represents the nominal strength for concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, $P_n$ shall be computed for each individual web and the results added to obtain the nominal load or reaction for the multiple web.

For built-up I-sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practical.
TABLE C3.4-1

<table>
<thead>
<tr>
<th>Shapes Having Single Webs</th>
<th>I-Sections or Similar Sections(1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>Unstiffened Flanges</td>
</tr>
<tr>
<td>Opposing Loads Spaced &gt; 1.5h(3)</td>
<td>End Reaction(3)</td>
</tr>
<tr>
<td></td>
<td>Interior Reaction(4)</td>
</tr>
<tr>
<td>Opposing Loads Spaced ≤ 1.5h(3)</td>
<td>End Reaction(3)</td>
</tr>
<tr>
<td></td>
<td>Interior Reaction(4)</td>
</tr>
</tbody>
</table>

Footnotes and Equation References to Table C3.4-1:

(1) I-sections made of two channels connected back to back or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel).

(2) At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

(3) For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

(4) For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or greater than 1.5h.

(5) At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

Equations for Table C3.4-1:

\[ t^2 \cdot kC_3C_8[331 - 0.61(h/t)] [1 + 0.01(N/t)] \]  
\[ t^2 \cdot kC_3C_8[217 - 0.28(h/t)] [1 + 0.01(N/t)] \]

When \( N/t > 60 \), the factor \([1 + 0.01(N/t)]\) may be increased to \([0.71 + 0.015(N/t)]\)

\[ t^2 \cdot FC_6(10.0 + 1.25\sqrt{N/t}) \]  
\[ t^2 \cdot kC_3C_8[538 - 0.74(h/t)] [1 + 0.007(N/t)] \]

When \( N/t > 60 \), the factor \([1 + 0.007(N/t)]\) may be increased to \([0.75 + 0.011(N/t)]\)

\[ t^2 \cdot FC_7(0.88+0.12m)(15.0 + 3.25\sqrt{N/t}) \]  
\[ t^2 \cdot kC_3C_8[244 - 0.57(h/t)] [1 + 0.01(N/t)] \]

\[ t^2 \cdot FC_8(0.64+0.31m)(10.0 + 1.25\sqrt{N/t}) \]

\[ t^2 \cdot kC_3C_8[771 - 2.26(h/t)] [1 + 0.0013(N/t)] \]

\[ t^2 \cdot FC_9(0.82+0.15m)(15.0 + 3.25\sqrt{N/t}) \]

In the above-referenced formulas:

\[ \phi_w = \text{Resistance factor for web crippling} \]

\[ P_n = \text{Nominal strength for concentrated load or reaction per web} \]

\[ C_1 = (1.22 - 0.22k) \]  
\[ C_2 = (1.06 - 0.06R/t) \leq 1.0 \]
C3 = (1.33 - 0.33k)
C4 = (1.15 - 0.15R/t) ≤ 1.0 but not less than 0.50
C5 = (1.49 - 0.53k) ≥ 0.6

\[ C_6 = 1 + \left( \frac{h}{t} \right) \left( \frac{h}{750} \right) \text{ when } h/t ≤ 150 \]
\[ C_6 = 1.20, \text{ when } h/t > 150 \]

C7 = 1/k, when h/t ≤ 66.5
\[ C_7 = \left[ 1.10 - \frac{h}{t} \right] \left( \frac{1}{665} \right) k, \text{ when } h/t > 66.5 \]

C8 = \[ 0.98 - \left( \frac{h}{t} \right) \left( \frac{1}{865} \right) k \]

C9 = 0.7 + 0.3 \( \left( \frac{t}{0.075} \right)^2 \)

F_y = Design yield stress of the web, see Section A5.2.1
h = Depth of the flat portion of the web measured along the plane of the web
k = F_y/33
m = t/0.075

N = Actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken
R = Inside bend radius
θ = Angle between the plane of the web and the plane of the bearing surface ≥ 45°, but not more than 90°

C3.5 Combined Bending and Web Crippling Strength

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

(a) For shapes having single unreinforced webs:

\[ 1.07 \left( \frac{P_w}{\phi_w P_n} \right) + \left( \frac{M_y}{\phi_b M_{net}} \right) \leq 1.42 \]  

Exception: At the interior supports of continuous spans, the above formula is not applicable to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 inches.

(b) For shapes having multiple unreinforced webs such as I-sections made of two channels connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel):

\[ 0.82 \left( \frac{P_w}{\phi_w P_n} \right) + \left( \frac{M_y}{\phi_b M_{net}} \right) \leq 1.32 \]

Exception: When h/t ≤ 2.33/√(F_y/E) and λ ≤ 0.673, the nominal concentrated load or reaction strength may be determined by Section C3.4.
In the above formulas:

$\phi_b =$ Resistance factor for bending (See Section 3.1)

$\phi_w =$ Resistance factor for web crippling (See Section C3.4)

$P_u =$ Required strength for the concentrated load or reaction in the presence of bending moment

$P_n =$ Nominal strength for concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4

$M_u =$ Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction $P_u$

$M_{nxx} =$ Nominal flexural strength about the centroidal $x$-axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2

$w =$ Flat width of the beam flange which contacts the bearing plate

$t =$ Thickness of the web or flange

$\lambda =$Slenderness factor given by Section B2.1

C4 Concentrically Loaded Compression Members

This section applies to members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the stress, $F_n$, defined in this section.

(a) The design axial strength, $\phi_e P_n$, shall be calculated as follows:

$\phi_e = 0.85$

$P_n = A_e F_n$

(Eq. C4-1)

where

$A_e =$ Effective area at the stress $F_n$. For sections with circular holes, $A_e$ shall be determined according to Section B2.2a, subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, $A_e$ can be determined ignoring the holes.

$F_n$ is determined as follows:

For $F_e > F_y / 2$

$F_n = F_y (1 - F_y / 4 F_e)$

(Eq. C4-2)

For $F_e \leq F_y / 2$

$F_n = F_e$

(Eq. C4-3)

$F_e$ is the least of the elastic flexural, torsional and torsional–flexural buckling stress determined according to Sections C4.1 through C4.3.

(b) For C- and Z-shapes, and single-angle sections with unstiffened flanges, $P_n$ shall be taken as the smaller of $P_n$ calculated above and $P_n$ calculated as follows:

$P_n = \frac{A \pi^2 E}{25.7 (w/t)^2}$

(Eq. C4-4)

where

$A =$ Area of the full, unreduced cross section

$w =$ Flat width of the unstiffened element

$t =$ Thickness of the unstiffened element

(c) Angle sections shall be designed for the required axial strength, $P_u$, acting simultaneously with a moment equal to $P_u L/1000$ applied about the minor principal axis causing compression in the tips of the angle legs.

(d) The slenderness ratio, $K L/r$, of all compression members preferably should not exceed 200, except that during construction only, $K L/r$ preferably should not exceed 300.
C4.1 Sections Not Subject to Torsional or Torsional–Flexural Buckling

For doubly-symmetric sections, closed cross sections and any other sections which can be shown not to be subject to torsional or torsional–flexural buckling, the elastic flexural buckling stress, $F_e$, shall be determined as follows:

$$F_e = \frac{\pi^2 E}{(KL / r)^2}$$

where

- $E$ = Modulus of elasticity
- $K$ = Effective length factor
- $L$ = Unbraced length of member
- $r$ = Radius of gyration of the full, unreduced cross section

C4.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Torsional–Flexural Buckling

For sections subject to torsional or torsional–flexural buckling, $F_e$ shall be taken as the smaller of $F_e$ calculated according to Section C4.1 and $F_e$ calculated as follows:

$$F_e = \frac{1}{2\beta} \left[ (\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta \sigma_{ex} \sigma_t} \right]$$

Alternatively, a conservative estimate of $F_e$ can be obtained using the following equation:

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}}$$

where $\sigma_t$ and $\sigma_{ex}$ are as defined in C3.1.2(a):

$$\beta = 1 - (x_{ax}/r)^2$$

For singly-symmetric sections, the $x$-axis is assumed to be the axis of symmetry.

C4.3 Nonsymmetric Sections

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, $F_e$ shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Chapter F.

C5 Combined Axial Load and Bending

The required strengths $P_u$, $M_{ux}$, and $M_{uy}$ shall satisfy the following interaction equations:

$$\frac{P_u}{\phi_c P_n} + \frac{C_m M_{ux}}{\phi_b M_{nx} \alpha_{nx}} + \frac{C_m M_{uy}}{\phi_b M_{ny} \alpha_{ny}} \leq 1.0$$

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0$$

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, $K$, for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, shall be taken as unity, unless analysis shows that a smaller value may be used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length, KL, of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.
When \( P_u/\phi_c P_n \leq 0.15 \), the following formula may be used in lieu of the above two formulas:

\[
\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0
\]

(Eq. C5-3)

where

- \( P_u = \) Required axial strength
- \( M_{ux} \) and \( M_{uy} = \) Required flexural strengths with respect to the centroidal axes of the effective section determined for the required axial strength alone. For angle sections, \( M_{uy} \) shall be taken either as the required flexural strength or the required flexural strength plus \( P_u \times 1000 \), whichever results in a lower value of \( P_n \).
- \( P_n = \) Nominal axial strength determined in accordance with Section C4
- \( P_{ne} = \) Nominal axial strength determined in accordance with Section C4, with \( F_n = F_y \)
- \( M_{nx} \) and \( M_{ny} = \) Nominal flexural strengths about the centroidal axes determined in accordance with Section C3
- \( 1/ \alpha_{cx}, 1/ \alpha_{cy} = \) Magnification factors

\[
= 1/ \left[ \frac{1 - \phi_c P_u}{\phi_c P_{ne}} \right]
\]

(Eq. C5-4)

- \( \phi_b = 0.95 \) and \( 0.90 \) for bending strength (Section C3.1.1) or \( 0.90 \) for laterally unbraced beam (Section C3.1.2)
- \( \phi_c = 0.85 \)
- \( P_{ne} = \frac{\pi^2 E I_b}{(K_b L_b)^2} \)

(Eq. C5-5)

- \( I_b = \) Moment of inertia of the full, unreduced cross section about the axis of bending
- \( L_b = \) Actual unbraced length in the plane of bending
- \( K_b = \) Effective length factor in the plane of bending
- \( C_{nx}, C_{ny} = \) Coefficients whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sideways)
   \[ C_m = 0.85 \]

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending
   \[ C_m = 0.6 - 0.4 (M_1/M_2) \]

   (Eq. C5-6)

where

\( M_1/M_2 \) is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. \( M_1/M_2 \) is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of \( C_m \) may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

   (a) for members whose ends are restrained, \( C_m = 0.85 \).
   (b) for members whose ends are unrestrained, \( C_m = 1.0 \).
C6 Cylindrical Tubular Members

The requirements of this Section apply to cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t, not greater than 0.441 E/Fy.

C6.1 Bending

For flexural members, the required flexural strength uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed $\phi_b M_n$, where $\phi_b = 0.95$ and $M_n$ is calculated as follows:

For $D/t \leq 0.070 E/Fy$

$$M_n = 1.25 F_y S_f$$  \hspace{1cm} (Eq. C6.1-1)

For $0.070 E/Fy < D/t \leq 0.319 E/Fy$

$$M_n = \left[ 0.970 + 0.020 \left( \frac{E}{F_y} \right) \right] F_y S_f$$  \hspace{1cm} (Eq. C6.1-2)

For $0.319 E/Fy < D/t \leq 0.441 E/Fy$

$$M_n = \left[ 0.328 E/(D/t) \right] S_f$$  \hspace{1cm} (Eq. C6.1-3)

where

$S_f$ = Elastic section modulus of the full, unreduced cross section

C6.2 Compression

The requirements of this Section apply to members in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section.

The design axial strength, $\phi_c P_n$, shall be calculated as follows:

$$\phi_c = 0.85$$

$$P_n = F_n A_e$$  \hspace{1cm} (Eq. C6.2-1)

In the above equation,

For $F_e > F_y/2$

$$F_n = \text{Flexural buckling stress} = F_y [1 - F_y/4F_e]$$  \hspace{1cm} (Eq. C6.2-2)

$$F_e = \text{The elastic flexural buckling stress determined according to Section C4.1}$$

$$A_e = [1 - (1 - R^2)(1 - A_o/A)]A$$  \hspace{1cm} (Eq. C6.2-3)

$$R = \sqrt{F_y / 2F_e}$$  \hspace{1cm} (Eq. C6.2-4)

$$A_o = \left[ \frac{0.037}{DF_y} + 0.667 \right] A \leq A \text{ for } \frac{D}{t} \leq 0.441 \frac{E}{F_y}$$  \hspace{1cm} (Eq. C6.2-5)

$A = \text{Area of the unreduced cross section}$

For $F_e \leq F_y/2$

$$F_n = F_e$$

$$A_e = A$$

C6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section C5.
D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

D1.1 I - Sections Composed of Two Channels

The maximum permissible longitudinal spacing of welds or other connectors, \( s_{\text{max}} \), joining two channels to form an I-section shall be

(a) For compression members:

\[
s_{\text{max}} = \frac{L_r e_y}{2r_I} \quad (Eq. D1.1-1)
\]

where

- \( L_r \) = Unbraced length of compression member
- \( r_I \) = Radius of gyration of the I-section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing
- \( e_y \) = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

\[
s_{\text{max}} = \frac{L}{6} \quad (Eq. D1.1-2)
\]

In no case shall the spacing exceed the value

\[
s_{\text{max}} = \frac{2gT_s}{mq} \quad (Eq. D1.1-3)
\]

where

- \( L \) = Span of beam
- \( T_s \) = Design strength of connection in tension (Section E)
- \( g \) = Vertical distance between the two rows of connections nearest to the top and bottom flanges
- \( q \) = Intensity of factored load on the beam (For methods of determination, see below)
- \( m \) = Distance from the shear center of one channel to the mid-plane of its web.

For simple channels without stiffening lips at the outer edges,

\[
m = \frac{w_f^2}{2w_f + d / 3} \quad (Eq. D1.1-4)
\]

For channels with stiffening lips at the outer edges,

\[
m = \frac{w_f d t}{4I_t} \left[ w_f d + 2D \left( d - \frac{4D^2}{3d} \right) \right] \quad (Eq. D1.1-5)
\]

\( w_f \) = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, \( w_f \) shall be taken as the width of the wider flange)

- \( d \) = Depth of channel or beam
- \( D \) = Overall depth of lip
- \( I_t \) = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of factored load, \( q \), is obtained by dividing the magnitude of factored concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, \( q \) shall be taken equal to three times the intensity of the uniformly distributed factored load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, \( s \), the required design strength of the welds or connections closest to the load or reaction is

\[
T_s = \frac{P_s m}{2g} \quad (Eq. D1.1-6)
\]

where \( P_s \) is a concentrated load or reaction based on factored loads.
The required maximum spacing of connections, \( s_{\text{max}} \), depends upon the intensity of the factored load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The design shear strength of the connections joining these plates to the flanges shall then be used for \( f_s \), and \( g \) shall be taken as the depth of the beam.

**D1.2 Spacing of Connections in Compression Elements**

The spacing, \( s \), in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non-integral stiffener in compression to another element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) \( 1.16t \sqrt{\left( \frac{E}{f_c} \right)} \), where \( t \) is the thickness of the cover plate or sheet, and \( f_c \) is the stress at service load in the cover plate or sheet; nor

(c) three times the flat width, \( w \), of the narrowest unstiffened compression element tributary to the connections, but need not be less than \( 1.11t \sqrt{\left( \frac{E}{F_y} \right)} \) if \( w/t < 0.50 \sqrt{\left( \frac{E}{F_y} \right)} \), or \( 1.33t \sqrt{\left( \frac{E}{F_y} \right)} \) if \( w/t \geq 0.50 \sqrt{\left( \frac{E}{F_y} \right)} \), unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one-half inch. In all other cases, the spacing shall be taken as the center-to-center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load-carrying elements.

**D2 Mixed Systems**

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this Specification and the applicable Specification of the other material.

**D3 Lateral Bracing**

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

**D3.1 Symmetrical Beams and Columns**

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

**D3.2 Channel-Section and Z-Section Beams**

The following provisions for bracing to restrain twisting of channels and Z-sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange*, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

* Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.
D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing

For channels and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

Anchored braces need to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof deck and its fastening system. Anchored braces shall be as close as possible to the flange which is connected to the deck or sheathing. Anchored braces shall be provided for each purlin bay.

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z-section assembly is equal to or greater than its nominal flexural strength, instead of that required by Chapter F.

(a) Channel Sections
For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the factored load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z-Sections
For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z-purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one-third points, each brace shall be designed to resist a force determined as follows:

1. Single-Span System with Restraints at the Supports:
   \[ P_L = 0.5 \left[ \frac{0.220 b^{1.50}}{n_p^{0.72} d^{0.90} t^{0.60}} - \sin \theta \right] W \]  
   (Eq. D3.2.1-1)

2. Single-Span System with Third-Point Restraints:
   \[ P_L = 0.5 \left[ \frac{0.474 b^{1.22}}{n_p^{0.57} d^{0.89} t^{0.33}} - \sin \theta \right] W \]  
   (Eq. D3.2.1-2)

3. Single-Span System with Midspan Restraint:
   \[ P_L = \left[ \frac{0.224 b^{1.32}}{n_p^{0.65} d^{0.83} t^{0.50}} - \sin \theta \right] W \]  
   (Eq. D3.2.1-3)

4. Multiple-Span System with Restraints at the Supports:
The force, $P_L$, is positive when restraint is required to prevent movement of the purlin flanges in the upward roof slope direction.

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 4$. For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 20$.

### D3.2.2 Neither Flange Connected to Sheathing

Each intermediate brace, at the top and bottom flange, shall be designed to resist a required lateral force, $P_L$, determined as follows:

(a) For uniform loads, $P_L = 1.5K'$ times the factored load within a distance 0.5$a$ each side of the brace.

(b) For concentrated loads, $P_L = 1.0K'$ times each concentrated load within a distance 0.3$a$ each side of the brace, plus $1.4K'(1-x/a)$ times each factored concentrated load located farther than 0.3$a$ but not farther than 1.0$a$ from the brace.

In the above formulas:
For channels and Z-sections:
- \( x \) = Distance from the concentrated load to the brace
- \( a \) = Distance between center line of braces

For channels:
\[ K' = \frac{m}{d} \]  
\((Eq. \ D3.2.2-1)\)

where
- \( m \) = Distance from the shear center to the mid-plane of the web, as specified in Section D1.1
- \( d \) = Depth of channel

For Z-sections:
\[ K' = \frac{I_{xy}}{I_x} \]  
\((Eq. \ D3.2.2-2)\)

where
- \( I_{xy} \) = Product of inertia of the full section about centroidal axes parallel and perpendicular to the web
- \( I_x \) = Moment of inertia of the full section about the centroidal axis perpendicular to the web

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one-quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one-third or more of the total factored load on the beam is concentrated over a length of one-twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: When all loads and reactions on a beam are transmitted through members which frame into the section in such a manner as to effectively restrain the section against rotation and lateral displacement, no other braces will be required.

D3.3 Laterally Unbraced Box Beams

For closed box-type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed 0.086 \( E/F_y \).

D4 Wall Studs and Wall Stud Assemblies

The design strength of a stud may be computed on the basis of Section C (neglecting sheathing and using steel only) or on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

When sheathing is utilized for stability of the wall studs, the sheathing shall retain adequate strength and stiffness for the expected service life of the wall and additional bracing shall be provided as required for adequate structural integrity during construction and in the completed structure.
The equations given are based on solid-web steel studs and are applicable within the following limits:

- Yield point, \( F_y \leq 50 \text{ ksi} \)
- Section depth, \( d \leq 6.0 \text{ in.} \)
- Thickness, \( t \leq 0.075 \text{ in.} \)
- Overall length, \( L \leq 16 \text{ ft.} \)
- Stud spacing, \( B \), not less than 12 in. nor greater than 24 in.
- Studs with perforations shall be designed using the results of stub column tests and/or rational analysis.

### D4.1 Wall Studs In Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing\(^*\), the design axial strength, \( \phi_cP_n \), shall be calculated as follows:

\[
\phi_c = 0.85
\]

\[
P_n = A_eF_n \tag{Eq. D4.1-1}
\]

where

- \( \phi_c \) = Resistance factor for axial compression
- \( A_e \) = Effective area determined at \( F_n \)
- \( F_n \) = The lowest value determined by the following three conditions:
  1. To prevent column buckling between fasteners in the plane of the wall, \( F_n \) shall be calculated according to Section C4 with \( KL \) equal to two times the distance between fasteners.
  2. To prevent flexural and/or torsional overall column buckling, \( F_n \) shall be calculated in accordance with Section C4 with \( F_x \) taken as the smaller of the two \( \sigma_{CR} \) values specified for the following section types, where \( \sigma_{CR} \) is the theoretical elastic buckling stress under concentric loading.

   1. Singly-symmetric channels and C-Sections
   \[
   \sigma_{CR} = \sigma_{ey} + \bar{Q}_a \tag{Eq. D4.1-2}
   \]
   \[
   \sigma_{CR} = \frac{1}{2\beta}\left[ (\sigma_{ex} + \sigma_{iq}) - \sqrt{(\sigma_{ex} + \sigma_{iq})^2 - 4\sigma_{ex}\sigma_{iq}} \right] \tag{Eq. D4.1-3}
   \]

   2. Z-Sections
   \[
   \sigma_{CR} = \sigma_{z} + \bar{Q}_t \tag{Eq. D4.1-4}
   \]
   \[
   \sigma_{CR} = \frac{1}{2}\left[ (\sigma_{ex} + \sigma_{sy} + \bar{Q}_a) - \sqrt{(\sigma_{ex} + \sigma_{sy} + \bar{Q}_a)^2 - 4(\sigma_{ex}\sigma_{sy} + \sigma_{ex}\bar{Q}_a - \sigma^2_{esy})} \right] \tag{Eq. D4.1-5}
   \]

   3. I-Sections (doubly-symmetric)
   \[
   \sigma_{CR} = \sigma_{ey} + \bar{Q}_a \tag{Eq. D4.1-6}
   \]
   \[
   \sigma_{CR} = \sigma_{ex} \tag{Eq. D4.1-7}
   \]

In the above formulas:

\[
\sigma_{ex} = \frac{\pi^2E}{(L / t_x)^2} \tag{Eq. D4.1-8}
\]

\[
\sigma_{esy} = (\pi^2EI_{sy}) / (AL^2) \tag{Eq. D4.1-9}
\]

\(^*\)Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.
\[
\sigma_{xy} = \frac{\pi^2E}{(L/r)^2} \quad (Eq. \text{D4.1-10})
\]

\[
\sigma_l = \frac{1}{Ar_o} \left[ GJ + \frac{\pi^2EC_w}{L^3} \right] \quad (Eq. \text{D4.1-11})
\]

\[
\sigma_0 = \sigma_l + \overline{Q_l} \quad (Eq. \text{D4.1-12})
\]

\[
\overline{Q} = \overline{q}B = \text{Design shear rigidity for sheathing on both sides of the wall assembly}
\]

\[
\overline{q} = \text{Design shear rigidity for sheathing per inch of stud spacing (see Table D4)}
\]

\[
B = \text{Stud spacing}
\]

\[
\overline{Q_a} = \overline{Q} / A \quad (Eq. \text{D4.1-13})
\]

\[
A = \text{Area of full unreduced cross section}
\]

\[
L = \text{Length of stud}
\]

\[
\overline{Q}_t = \left( \overline{Q}d^3 \right) / \left( 4Ar_o^2 \right) \quad (Eq. \text{D4.1-14})
\]

\[
d = \text{Depth of section}
\]

\[
I_{xy} = \text{Product of inertia}
\]

(c) To prevent shear failure of the sheathing, a value of \(F_n\) shall be used in the following equations so that the shear strain of the sheathing, \(\gamma\), does not exceed the permissible shear strain, \(\overline{\gamma}\). The shear strain, \(\gamma\), shall be determined as follows:

\[
\gamma = (\pi/L) \left[ C_1 + (E_1 d/2) \right] \quad (Eq. \text{D4.1-15})
\]

where

\[
C_1 \text{ and } E_1 \text{ are the absolute values of } C_1 \text{ and } E_1 \text{ specified below for each section type:}
\]

(1) Singly-Symmetric Channels and C-Sections

\[
C_1 = \frac{(F_n C_0)}{(\sigma_{xy} - F_n + \overline{Q}_a)} \quad (Eq. \text{D4.1-16})
\]

\[
E_1 = \frac{F_n \left[ (\sigma_{xy} - F_n)(\overline{Q} - x_oD_o) - F_n x_o(D_o - x_oE_o) \right]}{(\sigma_{xy} - F_n, E_0)(\sigma_{xy} - F_n)^2} \quad (Eq. \text{D4.1-17})
\]

(2) Z-Sections

\[
C_1 = \frac{F_n \left[ C_0(\sigma_{xy} - F_n) - D_o \sigma_{xy} \right]}{(\sigma_{xy} - F_n + \overline{Q}_a), (\sigma_{xy} - F_n)^2} \quad (Eq. \text{D4.1-18})
\]

\[
E_1 = \frac{(F_n E_o)}{(\sigma_{xy} - F_n)} \quad (Eq. \text{D4.1-19})
\]

(3) I-Sections

\[
C_1 = \frac{(F_n C_0)}{(\sigma_{xy} - F_n + \overline{Q}_a)} \quad (Eq. \text{D4.1-20})
\]

\[
E_1 = 0
\]

where

\[
x_o = \text{distance from shear center to centroid along principal x-axis, in. (absolute value)}
\]

\[
C_0, E_0, \text{ and } D_o \text{ are initial column imperfections which shall be assumed to be at least}
\]

\[
C_0 = L/350 \text{ in a direction parallel to the wall} \quad (Eq. \text{D4.1-21})
\]

\[
D_o = L/700 \text{ in a direction perpendicular to the wall} \quad (Eq. \text{D4.1-22})
\]

\[
E_0 = L/(d x 10,000), \text{ rad, a measure of the initial twist of the stud from the initial, ideal, unbuckled shape} \quad (Eq. \text{D4.1-23})
\]

If \(F_n > 0.5 F_y\), then in the definitions for \(\sigma_{xy}, \sigma_{ex}, \sigma_{esy}\) and \(\sigma_0\), the parameters \(E\) and \(G\) shall be replaced by \(E'\) and \(G'\), respectively, as defined below

\[
E' = 4EF_n(F_y - F_n)/F_y^2 \quad (Eq. \text{D4.1-24})
\]

\[
G' = G(E/E) \quad (Eq. \text{D4.1-25})
\]
Sheathing parameters $\tilde{q}_o$ and $\tilde{\gamma}$ may be determined from representative full-scale tests, conducted and evaluated as described by published documented methods (see Commentary), or from the small-scale-test values given in Table D4.

**TABLE D4**  
Sheathing Parameters$^{(1)}$

<table>
<thead>
<tr>
<th>Sheathing$^{(2)}$</th>
<th>$\tilde{q}_o$</th>
<th>$\tilde{\gamma}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 to 5/8 in. thick gypsum</td>
<td>2.0</td>
<td>0.008</td>
</tr>
<tr>
<td>Lignocellulosic board</td>
<td>1.0</td>
<td>0.009</td>
</tr>
<tr>
<td>Fiberboard (regular or impregnated)</td>
<td>0.6</td>
<td>0.007</td>
</tr>
<tr>
<td>Fiberboard (heavy impregnated)</td>
<td>1.2</td>
<td>0.010</td>
</tr>
</tbody>
</table>

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.
All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 6- to 12-inch spacing.

(2) All sheathing is 1/2-inch thick except as noted.

(3) $\tilde{q} = \tilde{q}_o (2 - s/12)$  \hspace{1cm} (Eq. D4.1-26)

where $s =$ fastener spacing, in.
For other types of sheathing, $\tilde{q}_o$ and $\tilde{\gamma}$ may be determined conservatively from representative small-specimen tests as described by published documented methods (see Commentary).

**D4.2 Wall Studs in Bending**

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the design flexural strengths are $\phi_b M_{n0}$ and $\phi_b M_{n0y}$

where

$\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges
$\phi_b = 0.90$ for sections with unstiffened compression flanges

$M_{n0}$ and $M_{n0y}$ = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

**D4.3 Wall Studs with Combined Axial Load and Bending**

The required axial strength and flexural strength shall satisfy the interaction equations of Section C5 with the following redefined terms:

$P_n$ = Nominal axial strength determined according to Section D4.1

$M_{nx}$ and $M_{nxy}$ in Equations C5-1, C5-2 and C5-3 shall be replaced by nominal flexural strengths, $M_{n0}$ and $M_{n0y}$, respectively.

* Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.
E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following LRFD design criteria govern welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings".

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D1.3 (Reference 3 of Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1, "Recommended Practices for Resistance Welding" or AWS C1.3, "Recommended Practice for Resistance Welding Coated Low Carbon Steels."

<table>
<thead>
<tr>
<th>Connection</th>
<th>Square Groove Butt Weld</th>
<th>Arc Spot Weld</th>
<th>Arc Seam Weld</th>
<th>Fillet Weld, Lap or T</th>
<th>Flare-Bevel Groove</th>
<th>Flare-V Groove Weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet to</td>
<td>F</td>
<td>—</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
</tr>
<tr>
<td>Sheet</td>
<td>H</td>
<td>—</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>V</td>
<td>—</td>
<td>—</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
</tr>
<tr>
<td>OH</td>
<td>—</td>
<td>—</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
</tr>
<tr>
<td>Sheet to</td>
<td>—</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>—</td>
</tr>
<tr>
<td>Supporting</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>H</td>
<td>H</td>
<td>—</td>
</tr>
<tr>
<td>Member</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>V</td>
<td>V</td>
<td>—</td>
</tr>
</tbody>
</table>

(F = flat, H = horizontal, V = vertical, OH = overhead)

The required strength on each weld shall not exceed the design strength, \( \phi P_n \), where

\[ \phi = \text{Resistance factor for arc welded connections defined in Sections E2.1 through E2.5.} \]
\[ P_n = \text{Nominal strength of welds determined according to Sections E2.1 through E2.5.} \]

E2.1 Groove Welds in Butt Joints

The design strength, \( \phi P_n \), of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

(a) Tension or compression normal to the effective area or parallel to the axis of the weld

\[ \phi = 0.90 \]
P_n = \text{Lte}F_y 
\text{(Eq. E2.1–1)}

(b) Shear on the effective area
\[ \phi = 0.80 \]
\[ P_n = \text{Lte}(0.6F_{xx}); \text{ and} \]
\[ \phi = 0.90 \]
\[ P_n = \text{Lte}\left(\frac{F_y}{\sqrt{3}}\right) \]
\text{(Eq. E2.1–3)}

where:
- \( \phi \) = Resistance factor for welded connections
- \( P_n \) = Nominal strength of a groove weld
- \( F_{xx} \) = Strength level designation in AWS electrode classification
- \( F_y \) = Specified minimum yield point of the lower strength base steel
- \( L \) = Length of weld
- \( \text{te} \) = Effective throat dimension for groove weld

### E2.2 Arc Spot Welds

Arc spot welds permitted by this Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers, Figures E2.2(A) and E2.2(B), shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of \( \frac{3}{8} \)–inch diameter.

![Figure E2.2A Typical Weld Washer](image-url)
Arc spot welds shall be specified by minimum effective diameter of fused area, \( d_e \). Minimum allowable effective diameter is \( \frac{3}{8} \) inch.

The design shear strength, \( \phi P_n \), of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

(a) \( \phi = 0.60 \)

\[
P_n = 0.589 \, d_e^2 \, F_{xx} \quad \text{or} \quad \]

(b) For \( (d_e/t) \leq 0.815 \sqrt{(E/F_u)} \):

\[
\phi = 0.60 \\
P_n = 2.20 \, t \, d_a \, F_u 
\]

For \( 0.815 \sqrt{(E/F_u)} < (d_e/t) < 1.397 \sqrt{(E/F_u)} \):

\[
\phi = 0.50 \\
P_n = 0.280 \left[ 1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] d_a \, F_u 
\]

For \( (d_e/t) \geq 1.397 \sqrt{(E/F_u)} \):

\[
\phi = 0.50 \\
P_n = 1.40 \, t \, d_a \, F_u 
\]

where

\( \phi \) = Resistance factor for welded connections

\( P_n \) = Nominal shear strength of an arc spot weld

\( d \) = Visible diameter of outer surface of arc spot weld

\( d_a \) = Average diameter of the arc spot weld at mid-thickness of \( t \) where \( d_a = (d - t) \) for a single sheet, and \( (d - 2t) \) for multiple sheets (not more than four lapped sheets over a supporting member)

\( d_e \) = Effective diameter of fused area

\( d_e = 0.7d - 1.5t \) but \( \leq 0.55d \)  

(\( Eq. \ E2.2-5 \))

\( t \) = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer

\( F_{xx} \) = Stress level designation in AWS electrode classification

\( F_u \) = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low ductility steel.

Note: See Figures E2.2(C) and E2.2(D) for diameter definitions

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of \( e_{min} \) as given below:
where

\[
\phi = 0.70 \text{ when } F_u / F_{sy} \geq 1.15 \\
\phi = 0.60 \text{ when } F_u / F_{sy} < 1.15
\]

\[P_u = \text{Required strength transmitted by weld} \]

\[t = \text{Thickness of thinnest connected sheet} \]

\[F_{sy} = \text{Yield point as specified in Sections A3.1 or A3.2} \]

**Note:** See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.
The design tensile strength, $\phi P_n$, on each arc spot weld between sheet and supporting member, shall be determined as follows:

$$\phi = 0.65$$

$$P_n = 0.7 t d_a F_u$$

(Eq. E2.2-7)

The following additional limitations for use in Eq. 2.2-7 shall apply:

$$e_{\text{min}} \geq d$$

$$F_{xx} \geq 60 \text{ ksi}$$

$$F_u \leq 60 \text{ ksi}$$

$$t \geq 0.028 \text{ in.}$$

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, $d_e$, or average diameter, $d_a$, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

**E2.3 Arc Seam Welds**

Arc seam welds [Figure E2.3(A)] covered by this Specification apply only to the following joints:
The design shear strength, $\phi P_n$, of arc seam welds shall be determined by using the smaller of either

(a) $\phi = 0.60$

$$P_n = \left[ \frac{\pi d^2}{4} + L d_e \right] 0.75F_{xx}; \text{ or}$$  \hspace{1cm} (Eq. E2.3-1)

(b) $\phi = 0.60$

$$P_n = 2.5 t F_u (0.25L + 0.96 d_a)$$  \hspace{1cm} (Eq. E2.3-2)

where

$\phi$ = Resistance factor for welded connections
$P_n$ = Nominal shear strength of an arc seam weld
$d$ = Width of arc seam weld
$L$ = Length of seam weld not including the circular ends
   (For computation purposes, L shall not exceed 3d)
$d_a$ = Average width of seam weld

where

$$d_a = (d - t) \text{ for a single sheet, and} \hspace{1cm} (Eq. E2.3-3)$$
$$d_a = (d - 2t) \text{ for a double sheet} \hspace{1cm} (Eq. E2.3-4)$$

$d_e$ = Effective width of arc seam weld at fused surfaces
$$d_e = 0.7d - 1.5t$$  \hspace{1cm} (Eq. E2.3-5)

and $F_u$ and $F_{xx}$ are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2. See Figure E2.3(B).
E2.4 Fillet Welds

Fillet welds covered by this Specification apply to the welding of joints in any position, either
(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

The design shear strength, $\phi P_n$, of a fillet weld shall be determined as follows:
(a) For longitudinal loading:
For $L/t < 25$:
$\phi = 0.60$
$P_n = \left(1 - \frac{0.01L}{t}\right) t F_u$  \hspace{1cm} (Eq. E2.4-1)
For $L/t \geq 25$:
$\phi = 0.55$
$P_n = 0.75 t F_u$  \hspace{1cm} (Eq. E2.4-2)
(b) For transverse loading:
$\phi = 0.60$
$P_n = t F_u$  \hspace{1cm} (Eq. E2.4-3)

where $t =$ Least value of $t_1$ or $t_2$, Figure E2.4

In addition, for $t > 0.150$ inch the design strength determined above shall not exceed the following value of $\phi P_n$:
$\phi = 0.60$
$P_n = 0.75 \ t_w F_{xx}$  \hspace{1cm} (Eq. E2.4-4)

where
$\phi =$ Resistance factor for welded connections
$P_n =$ Nominal strength of a fillet weld
$L =$ Length of fillet weld
$t_w =$ Effective throat $= 0.707 w_1$ or $0.707 w_2$, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

$w_1$ and $w_2 =$ leg on weld (see Figure E2.4).
$F_u$ and $F_{xx}$ are defined in Section E2.2.
E2.5 Flare Groove Welds

Flare groove welds covered by this Specification apply to welding of joints in any position, either:
(a) Sheet to sheet for flare–V groove welds, or
(b) Sheet to sheet for flare–bevel groove welds, or
(c) Sheet to thicker steel member for flare–bevel groove welds.

The design shear strength, $P_n$, of a flare groove weld shall be determined as follows:
(a) For flare–bevel groove welds, transverse loading [see Figure E2.5(A)]:
$$
\phi = 0.55 \\
P_n = 0.833tLF_u 
$$
(Eq. E2.5–1)
(b) For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:
(1) For $t \leq t_w < 2t$ or if the lip height is less than weld length, $L$:
$$
\phi = 0.55 \\
P_n = 0.75tLF_u 
$$
(Eq. E2.5–2)
(2) For $t_w \geq 2t$ and the lip height is equal to or greater than $L$:
$$
\phi = 0.55 \\
P_n = 1.50tLF_u 
$$
(Eq. E2.5–3)
In addition, if $t > 0.15$ inch, the design strength determined above shall not exceed the following value of $\phi P_n$:
$$
\phi = 0.60 \\
P_n = 0.75t_wLF_{uL}
$$
(Eq. E2.5–4)
Figure E2.5A  Flare–Bevel Groove Weld

Figure E2.5 B, C, D  Shear in Flare Groove Welds
E2.6 Resistance Welds

The design shear strength, \( \phi P_n \), of spot welding shall be determined as follows:

\[
\phi = 0.65 \\
P_n = \text{Tabulated value given in Table E2.6}
\]

### TABLE E2.6
Nominal Shear Strength Of Spot Welding

<table>
<thead>
<tr>
<th>Thickness of Thinnest Outside Sheet, in.</th>
<th>Nominal Shear Strength per Spot, kips</th>
<th>Thickness of Thinnest Outside Sheet, in.</th>
<th>Nominal Shear Strength per Spot, kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.010</td>
<td>0.13</td>
<td>0.080</td>
<td>3.33</td>
</tr>
<tr>
<td>0.020</td>
<td>0.48</td>
<td>0.090</td>
<td>4.00</td>
</tr>
<tr>
<td>0.030</td>
<td>1.00</td>
<td>0.100</td>
<td>4.99</td>
</tr>
<tr>
<td>0.040</td>
<td>1.42</td>
<td>0.110</td>
<td>6.07</td>
</tr>
<tr>
<td>0.050</td>
<td>1.65</td>
<td>0.125</td>
<td>7.29</td>
</tr>
<tr>
<td>0.060</td>
<td>2.28</td>
<td>0.190</td>
<td>10.16</td>
</tr>
<tr>
<td>0.070</td>
<td>2.83</td>
<td>0.250</td>
<td>15.00</td>
</tr>
</tbody>
</table>

E3 Bolted Connections

The following LRFD design criteria govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than \( \frac{3}{16} \) inch. For bolted connections in which the thickness of the thinnest connected part is equal to or greater than \( \frac{3}{16} \) inch, refer to AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings", September 1, 1986.

Bolts, nuts, and washers shall generally conform to one of the following specifications:

- ASTM A194 Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
- ASTM A307(Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners
- ASTM A325 High Strength Bolts for Structural Steel Joints
- ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than \( \frac{1}{2} \) inch)
- ASTM A449 Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than \( \frac{1}{2} \) inch)
- ASTM A490 Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints
- ASTM A563 Carbon and Alloy Steel Nuts
- ASTM F436 Hardened Steel Washers
- ASTM F844 Washers, Steel, Plain (Flat), Unhardened for General Use
- ASTM F959 Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.
The holes for bolts shall not exceed the sizes specified in Table E3, except that larger holes may be used in column base details or structural systems connected to concrete walls.

### TABLE E3
**Maximum Size of Bolt Holes, Inches**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1/2</td>
<td>d + 1/32</td>
<td>d + 1/16</td>
<td>(d + 1/32) by (d + 1/4)</td>
<td>(d + 1/32) by (2 1/2 d)</td>
</tr>
<tr>
<td>≥ 1/2</td>
<td>d + 1/16</td>
<td>d + 1/8</td>
<td>(d + 1/16) by (d + 1/4)</td>
<td>(d + 1/16) by (2 1/2 d)</td>
</tr>
</tbody>
</table>

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

#### E3.1 Spacing and Edge Distance

The design shear strength, $\phi P_n$, of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

$$P_n = \phi F_u$$  \hspace{1cm} (Eq. E3.1-1)

(a) When $F_u/F_{ys} \geq 1.15$:

$\phi = 0.70$

(b) When $F_u/F_{ys} < 1.15$:

$\phi = 0.60$

where

- $\phi$ = Resistance factor
- $P_n$ = Nominal resistance per bolt
- $e$ = The distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part
- $t$ = Thickness of thinnest connected part
- $F_u$ = Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel
- $F_{ys} = $ Yield point of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1 \frac{1}{2} d$.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $e-(d_h/2)$, in which $e$ is the required distance computed from the applicable equation given above, and $d_h$ is the diameter of a standard hole defined in Table E3. In no case shall the clear dis-
tance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

**E3.2 Tension in Connected Part**

The design tensile strength, \( \phi P_n \), on the net section of the connected part shall be determined as follows:

(a) Washers are provided under both the bolt head and the nut

\[
P_n = (1.0 - 0.9r + 3rd/s) F_u A_n \leq F_u A_n
\]

\( \phi = 0.65 \) for double shear connection

\( \phi = 0.55 \) for single shear connection

(b) Either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

\[
P_n = (1.0 - r + 2.5rd/s) F_u A_n \leq F_u A_n
\]

In addition, the design tensile strength shall not exceed the following values:

\( \phi = 0.95 \)

\[
P_n = F_y A_n
\]

where

\( A_n = \) Net area of the connected part

\( r = \) Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If \( r \) is less than 0.2, it may be taken equal to zero.

\( s = \) Spacing of bolts perpendicular to line of stress.

In the case of a single bolt, \( s = \) Width of sheet

\( F_u = \) Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel

\( F_y = \) Yield point of the connected part

\( d \) and \( t \) are defined in Section E3.1

**E3.3 Bearing**

The design bearing strength, \( \phi P_n \), shall be determined by the values of \( \phi \) and \( P_n \) given in Tables E3.3-1 and E3.3-2 for the applicable thickness and \( F_u/F_y \) ratio of the connected part and the type of joint used in the connection.

In Tables E3.3-1 and E3.3-2, the symbols \( \phi \), \( P_n \), \( d \), \( F_u \) and \( t \) were previously defined. For conditions not shown, the design bearing strength of bolted connections shall be determined by tests.
TABLE E3.3–1
Nominal Bearing Strength for Bolted Connections
with Washers under Both Bolt Head and Nut

<table>
<thead>
<tr>
<th>Thickness of Connected Part in.</th>
<th>Type of joint</th>
<th>$F_u/F_y$ ratio of Connected Part</th>
<th>Resistance Factor $\phi$</th>
<th>Nominal Resistance $P_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\geq 0.024$ but $&lt;\frac{3}{16}$</td>
<td>Inside sheet of double shear connection</td>
<td>$\geq 1.15$</td>
<td>0.55</td>
<td>3.33 $F_{u dt}$</td>
</tr>
<tr>
<td></td>
<td>Single shear and outside sheets of double shear connection</td>
<td>$&lt; 1.15$</td>
<td>0.65</td>
<td>3.00 $F_{u dt}$</td>
</tr>
<tr>
<td>$\geq \frac{3}{16}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

See AISC LRFD Specification

TABLE E3.3–2
Nominal Bearing Strength for Bolted Connections
Without Washers Under Both Bolt Head and Nut, or With Only One Washer

<table>
<thead>
<tr>
<th>Thickness of Connected Part in.</th>
<th>Type of joint</th>
<th>$F_u/F_y$ ratio of Connected Part</th>
<th>Resistance Factor $\phi$</th>
<th>Nominal Resistance $P_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\geq 0.036$ but $&lt;\frac{3}{16}$</td>
<td>Inside sheet of double shear connection</td>
<td>$\geq 1.15$</td>
<td>0.65</td>
<td>3.00 $F_{u dt}$</td>
</tr>
<tr>
<td></td>
<td>Single shear and outside sheets of double shear connection</td>
<td>$\geq 1.15$</td>
<td>0.70</td>
<td>2.22 $F_{u dt}$</td>
</tr>
<tr>
<td>$\geq \frac{3}{16}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

See AISC LRFD Specification

E3.4 Shear and Tension in Bolts

The required bolt strength in shear or tension shall not exceed the design strength, $\phi P_n$, determined as follows:

$\phi = \text{Resistance factor given in Table E3.4–1}\)

$P_n = A_b F_n$  \hspace{1cm} (Eq. E3.4–1)

where

$A_b = \text{Gross cross-sectional area of bolt}

P_n \text{ is given by } F_{nv} \text{ or } F_{nt} \text{ in Table E3.4–1.}$
### TABLE E3.4–1
Nominal Tensile and Shear Strength for Bolts

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Tensile Strength</th>
<th>Shear Strength*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resistance Factor $\phi$</td>
<td>Nominal Stress $F_{nt}$</td>
</tr>
<tr>
<td>A307 Bolts, Grade A ((\frac{1}{4}) in. (\leq d &lt; \frac{1}{2}) in.)</td>
<td>0.75</td>
<td>40.5</td>
</tr>
<tr>
<td>A307 Bolts, Grade A (d (\geq \frac{1}{2}) in.)</td>
<td></td>
<td>45.0</td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td></td>
<td>90.0</td>
</tr>
<tr>
<td>A325 bolts, when threads are excluded from shear planes</td>
<td></td>
<td>90.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts ((\frac{1}{4}) in. (\leq d &lt; \frac{1}{2}) in.), when threads are not excluded from shear planes</td>
<td>101.0</td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts ((\frac{1}{4}) in. (\leq d &lt; \frac{1}{2}) in.), when threads are excluded from shear planes</td>
<td>101.0</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts ((\frac{1}{4}) in. (\leq d &lt; \frac{1}{2}) in.), when threads are not excluded from shear planes</td>
<td>81.0</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts ((\frac{1}{4}) in. (\leq d &lt; \frac{1}{2}) in.), when threads are excluded from shear planes</td>
<td>81.0</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td>112.5</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are excluded from shear planes</td>
<td>112.5</td>
<td></td>
</tr>
</tbody>
</table>

* Applies to bolts in holes as limited by Table E3. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Section F.
The pull-over strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, see Section E5.2.

When bolts are subject to a combination of shear and tension produced by factored loads, the required tension strength shall not exceed the design strength, $P_n$, based on $\phi = 0.75$ and $P_n = A_bF'_nt$, where $F'_nt$ is given in Table E3.4–2, in which $f_v$ is the shear stress produced by the same factored loads. The required shear strength shall not exceed the design shear strength, $\phi A_bF_nv$, determined in accordance with Table E3.4–1.

### TABLE E3.4–2
Nominal Tension Stress, $F'_nt$, for Bolts Subject to the Combination of Shear and Tension

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Threads Not Excluded from Shear Planes</th>
<th>Threads Excluded from Shear Planes</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>113 - 2.4$f_v$ ≤ 90</td>
<td>113 - 1.9$f_v$ ≤ 90</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>127 - 2.4$f_v$ ≤ 101</td>
<td>127 - 1.9$f_v$ ≤ 101</td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>101 - 2.4$f_v$ ≤ 81</td>
<td>101 - 1.9$f_v$ ≤ 81</td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>141 - 2.4$f_v$ ≤ 112.5</td>
<td>141 - 1.9$f_v$ ≤ 112.5</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td>47 - 2.4$f_v$ ≤ 40.5</td>
<td></td>
</tr>
<tr>
<td>When $\frac{1}{4}$ in. ≤ $d$ &lt; $\frac{1}{2}$ in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>When $d$ ≥ $\frac{1}{2}$ in.</td>
<td>52 - 2.4$f_v$ ≤ 45</td>
<td></td>
</tr>
</tbody>
</table>

### E4 Shear Rupture

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the required shear strength shall not exceed the design shear strength, $\phi V_n$.

where

$$\phi = 0.75$$

$$V_n = 0.6 F_v A_{wn} \quad (Eq. E4-1)$$

$$A_{wn} = (d_{wc} - n_d h) t \quad (Eq. E4-2)$$

$d_{wc}$ = Coped web depth

$n$ = Number of holes in the critical plane

$d_h$ = Hole diameter

$F_v$ = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low-ductility steel

$t$ = Thickness of coped web

### E5 Connections to Other Materials

#### E5.1 Bearing

Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The required bearing strength in the contact area shall not exceed the design strength, $\phi P_p$.

In the absence of code regulations, the design bearing strength on concrete may be taken as $\phi P_p$.
On the full area of a concrete support \( \ldots \ldots \ldots \ldots \ldots P_p = 0.85 f'_e A_1 \)
On less than the full area of a concrete support \( \ldots \ldots \ldots \ldots \ldots P_p = 0.85 f'_e A_1 \sqrt{A_2 / A_1} \)

where

\( \phi_e = 0.60 \)
\( f'_e = \text{Specified compression strength of concrete} \)
\( A_1 = \text{Bearing area} \)
\( A_2 = \text{Full cross-sectional area of concrete support} \)

The value of \( \sqrt{A_2 / A_1} \) shall not exceed 2.

**E5.2 Tension**

The pull-over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull-out force resulting from factored axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The nominal tensile strength of the fastener and the nominal imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

**E5.3 Shear**

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The required shear and/or bearing strength on the steel components shall not exceed that allowed by this Specification. The design shear strength on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.
F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

(b) The provisions of Chapter F do not apply to cold-formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold-formed steel structural members are such that calculation of their load-carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than four identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±10 percent. If such deviation from the average value exceeds 10 percent, at least three more tests of the same kind shall be made. The average value of all tests made shall then be regarded as the predicted capacity, $R_p$, for the series of the tests. The mean value and the coefficient of variation of the tested-to-predicted load ratios for all tests, $P_m$ and $V_p$, shall be determined for statistical analysis.

(b) The load-carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. F1-1.

$$ \phi R_p \geq \Sigma \gamma_i Q_i $$  \hspace{1cm} (Eq. F1-1)

where

$\Sigma \gamma_i Q_i$ = Required resistance based on the most critical load combination determined in accordance with Section A5.1.4. $\gamma_i$ and $Q_i$ are load factors and load effects, respectively.

$R_p$ = Average value of all test results

$\phi$ = Resistance factor

$$ \phi = 1.5(M_m F_m P_m) \exp\left(-\beta_o \sqrt{V_M^2 + V_F^2 + C_p V_P^2 + V_Q^2}\right) $$ \hspace{1cm} (Eq. F1-2)

$M_m$ = Mean value of the material factor listed in Table F1 for the type of component involved

$F_m$ = Mean value of the fabrication factor listed in Table F1 for the type of component involved

$P_m$ = Mean value of the tested-to-predicted load ratios determined in Section F1(a)

$\beta_o$ = Target reliability index

= 2.5 for structural members and 3.5 for connections

$V_M$ = Coefficient of variation of the material factor listed in Table F1 for the type of component involved

$V_F$ = Coefficient of variation of the fabrication factor listed in Table F1 for the type of component involved

$C_p$ = Correction factor

$$ C_p = \frac{(n-1)}{(n-3)} $$ \hspace{1cm} (Eq. F1-3)

$V_P$ = Coefficient of variation of the tested-to-predicted load ratios determined in Section F1(a)

$n$ = Number of tests

$V_Q$ = Coefficient of variation of the load effect

= 0.21

* For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced, $\phi$ shall be determined with a coefficient of 1.6 in lieu of 1.5, $\beta_o = 1.5$, and $V_Q = 0.43$. 


### TABLE F1

**Statistical Data for the Determination of Resistance Factor**

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse Stiffeners</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Shear Stiffeners</td>
<td>1.00</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Tension Members</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Flexural Members</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Lateral Buckling Strength</td>
<td>1.00</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>One Flange Through–Fastened to Deck or Sheathing</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Bending and Shear</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Web Crippling Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Bending and Web Crippling</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Concentrically Loaded Compression Members</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Combined Axial Load and Bending</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Cylindrical Tubular Members</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Axial Compression</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs and Wall Stud Assemblies</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Studs in Compression</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs in Bending</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Wall Studs with Combined Axial Load and Bending</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Type of Component</td>
<td>$M_m$</td>
<td>$V_m$</td>
<td>$F_m$</td>
<td>$V_f$</td>
</tr>
<tr>
<td>-------------------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
<td>-------</td>
</tr>
<tr>
<td><strong>Welded Connections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arc Spot Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
</tr>
<tr>
<td>Arc Seam Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Tearing</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Fillet Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
</tr>
<tr>
<td>Flare Groove Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Plate Failure</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Resistance Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolted Connections</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Spacing and Edge Distance</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Tension Strength on Net Section</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Bearing Strength</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
</tbody>
</table>
The listing in Table F1 does not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.*

For steels not listed in Section A3.1, the values of $M_m$ and $V_M$ shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1–1, except that the resistance factor $\phi$ is taken as unity and that the load factor for dead load may be taken as 1.0.

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies whose capacities can be computed according to this Specification or its specific references, confirmatory tests may be made to demonstrate the load–carrying capacity not less than the nominal resistance, $R_n$, specified in this Specification or its specific references for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370. Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

(1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

(2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.

When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of $\rho$ for the specimen is unity.

* See Reference 36 of the Commentary
(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

**F3.2 Flat Elements of Formed Sections**

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, \( F_yf \), shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold-formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, \( F_yf \), shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

**F3.3 Virgin Steel**

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.
COMMENTARY ON THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

MARCH 16, 1991 EDITION

LRFD Cold-Formed Steel Design Manual - Part II

AMERICAN IRON AND STEEL INSTITUTE
1133 15th STREET, NW
WASHINGTON, DC 20005-2701
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute – or any other person named herein – that the information is suitable for any general or particular use or of freedom from infringement of liability arising from such use.

1st Printing – August 1991

Prepared by University of Missouri-Rolla
Rolla, Missouri

Copyright American Iron and Steel Institute, 1991
Preface

This document provides a commentary on the background for the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members.

# TABLE OF CONTENTS

**PREFACE** ................................................................. II-3

**TABLE OF CONTENTS** .................................................. II-4

**INTRODUCTION** ......................................................... II-7

**A. GENERAL PROVISIONS** ............................................... II-7

A1 Limits of Applicability and Terms ................................ II-7

A2 Non-Conforming Shapes and Constructions ...................... II-7

A3 Material ................................................................. II-7

A4 Loads ................................................................. II-8

A5 Structural Analysis and Design .................................. II-8

A5.1 Design Basis ....................................................... II-8

A5.2 Yield Point and Strength Increase from Cold Work of Forming ................ II-15

A6 Reference Documents ............................................... II-15

**B. ELEMENTS** ............................................................. II-17

B1 Dimensional Limits and Considerations .......................... II-17

B2 Effective Widths of Stiffened Elements ......................... II-17

B3 Effective Widths of Unstiffened Elements ...................... II-17

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener ........................ II-17

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener ................................ II-17

B6 Stiffeners ........................................................... II-17

B6.1 Transverse Stiffeners ............................................ II-17

B6.2 Shear Stiffeners ................................................ II-18

B6.3 Non-Conforming Stiffeners ..................................... II-18
C. MEMBERS .............................................................. II-19
C1 Properties of Sections ........................................ II-19
C2 Tension Members ................................................ II-19
C3 Flexural Members ............................................... II-19
C3.1 Strength for Bending Only .................................. II-19
C3.1.1 Nominal Section Strength ............................... II-19
C3.1.2 Lateral Buckling Strength .............................. II-20
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing .................. II-21
C3.2 Strength for Shear Only .................................. II-23
C3.3 Strength for Combined Bending and Shear ............. II-24
C3.4 Web Crippling Strength .................................. II-24
C3.5 Combined Bending and Web Crippling Strength ....... II-26
C4 Concentrically Loaded Compression Members .......... II-26
C5 Combined Axial Load and Bending ......................... II-29
C6 Cylindrical Tubular Members ............................... II-30

D. STRUCTURAL ASSEMBLIES ...................................... II-31
D1 Built-Up Sections ............................................. II-31
D2 Mixed Systems ................................................ II-31
D3 Lateral Bracing ............................................... II-31
D4 Wall Studs and Wall Stud Assemblies ..................... II-31
D4.1 Wall Studs in Compression ............................... II-31
D4.2 Wall Studs in Bending .................................. II-32
D4.3 Wall Studs with Combined Axial Load and Bending .... II-32

E. CONNECTIONS AND JOINTS ..................................... II-33
E1 General Provisions ............................................ II-33
E2 Welded Connections ......................................... II-33
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>E3</td>
<td>Bolted Connections</td>
<td>II-35</td>
</tr>
<tr>
<td>E4</td>
<td>Shear Rupture</td>
<td>II-38</td>
</tr>
<tr>
<td>E5</td>
<td>Connections to Other Materials</td>
<td>II-38</td>
</tr>
<tr>
<td>F1</td>
<td>Tests for Determining Structural Performance</td>
<td>II-39</td>
</tr>
<tr>
<td>F2</td>
<td>Tests for Confirming Structural Performance</td>
<td>II-39</td>
</tr>
<tr>
<td>F3</td>
<td>Tests for Determining Mechanical Properties</td>
<td>II-39</td>
</tr>
<tr>
<td>REFERENCES</td>
<td></td>
<td>II-40</td>
</tr>
</tbody>
</table>
INTRODUCTION

In the design of steel buildings, the "Allowable Stress Design Criteria" have long been used for the design of cold-formed steel structural members in the United States and other countries. Even though the theoretical concept of reliability analysis has been available for some time and the significance of such a concept in structural safety and design is well recognized, the probabilistic method has not yet been explicitly adopted as a basis for the American design standard for cold-formed steel structures.

Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. It became evident that the development of a new specification for load and resistance factor design of cold-formed steel is highly desirable because the design criteria for heavy hot-rolled steel construction cannot possibly cover the design features of thin-walled, cold-formed steel construction completely.

Since 1976, a joint project has been conducted at University of Missouri-Rolla and Washington University to develop the new design criteria for cold-formed steel structural members and connections based on the probabilistic approach.

The Load and Resistance Factor Design criteria developed on the basis of the 1986 Edition of the AISI Specification with 1989 Addendum for allowable stress design are included in Sections A through F of this Specification.

This commentary contains a brief presentation of the methodology used for the development of the load and resistance factor design criteria. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the Specification. For detailed background information, reference is made to the research reports given in the bibliography.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

Section A1 of the LRFD Specification is essentially the same as Section A1 of the AISI Specification for allowable stress design. The definitions and various terms used for the LRFD criteria are the same as that used for the allowable stress design.

A2 Non-Conforming Shapes and Constructions

Section A2 of the LRFD Specification is essentially the same as Section A2 of the AISI Specification for allowable stress design.

A3 Material

This Section is essentially the same as Section A3 of the AISI Specification for allowable stress design.
In lieu of the tensile-to-yield strength limit of 1.08, the Specification permits the use of elongation requirements using the measurement technique as given in Ref. 1, and Part VII of the Manual. Because of limited experimental verification of the structural performance of members using material having a tensile-to-yield strength ratio less than 1.08 (Ref. 2), the Specification limits the use of this material to purlins and girts meeting the elastic design requirements of Sections C3.1.1(a), C3.1.2, and C3.1.3. Thus, the use of such steel in other applications (compression members, tension members, other flexural members including those whose strength is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins and girts, concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Section C5 are met and $P_u / P_n$ does not exceed 0.15.

**A4 Loads**

This Section is the same as Section A4 of the AISI Specification for allowable stress design.

With regard to ponding, design guidance can be found from Section K2 of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings (Ref. 3).

**A5 Structural Analysis and Design**

**A5.1 Design Basis**

The current method of designing cold-formed steel structural members, as presented in the 1986 AISI Specification (Ref. 4), is based on the Allowable Stress Design method. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the AISI Specification. The AISI allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. Usual factors of safety inherent in the AISI Specification for the Design of Cold-Formed Steel Structural Members are 5/3 for beams and 23/12 for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the Commentary on the AISI Specification (Refs. 5 and 6)(see also Refs. 7 and 8), and a continuing research effort provides further improvement in understanding them.

The factors of safety are provided to account for the uncertainties and variabilities inherent in the loads, the analysis, the limit state model, the material properties, the geometry, and the fabrication. Through experience it has been established that the present factors of safety provide satisfactory design.

The allowable stress design method employs only one factor of safety for a limit state. The use of multiple load factors provides a refinement in the design which can account for the different degrees of the uncertainties and variabilities of the design parameters. Such a design method is called Load and Resistance Factor Design, and its format is expressed by the following criterion:

$$\phi R_n \geq \Sigma \eta Q_i$$  \hspace{1cm} (CA5.1-1)
Commentary on the Cold-Formed Steel LRFD Specification – March, 1991

where

\[ R_n = \text{the nominal resistance} \]
\[ \phi = \text{resistance factor} \]
\[ \gamma_i = \text{load factors} \]
\[ Q_i = \text{load effects} \]

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor \( \phi \) accounts for the uncertainties and variabilities inherent in the \( R_n \), and it is usually less than unity. The load effects \( Q_i \) are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and \( \gamma_i \) are the corresponding load factors which account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

The advantages of LRFD are: (1) the uncertainties and the variabilities of different types of loads and resistances are different (e.g., dead load is less variable than wind load), and so these differences can be accounted for by use of multiple factors, and (2) by using probability theory designs can ideally achieve a more consistent reliability. Thus LRFD provides the basis for a more rational and refined design method than is possible with the Allowable Stress Design method.

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects \( Q \) to nominal resistances \( R \), but both \( Q \) and \( R \) are random parameters (see Fig. CA5.1-1). A limit state is violated if \( R < Q \). While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of \( Q \) and \( R \) were known, then the probability of \( R < Q \) could be exactly determined for any design. In general the distributions of \( Q \) and \( R \) are not known, and only the means, \( Q_m \) and \( R_m \), and the standard deviations, \( \sigma_Q \) and \( \sigma_R \), are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Fig. CA5.1-2. The distribution curve shown is for \( \ln(R/Q) \), and a limit state is exceeded when \( \ln(R/Q) \leq 0 \). The area under \( \ln(R/Q) \leq 0 \) is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of \( \ln(R/Q) \). For given statistical data \( R_m, Q_m, \sigma_R \) and \( \sigma_Q \), the area under \( \ln(R/Q) \leq 0 \) can be varied by changing the value of \( \beta \) (Fig. CA5.1-2), since \( \beta \sigma_{\ln(R/Q)} = \ln(R/Q)_m \), from which approximately

\[ \beta = \frac{\ln(R_m/Q_m)}{\sqrt{\frac{\sigma_R^2}{R} + \frac{\sigma_Q^2}{Q}}} \]  \hspace{1cm} (CA5.1.2)

where \( V_R = \sigma_R/R_m \) and \( V_Q = \sigma_Q/Q_m \), the coefficients of variation of \( R \) and \( Q \), respectively. The index \( \beta \) is called the “reliability index”, and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger \( \beta \) is more reliable.

The concept of the reliability index can be used in determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported, braced beams subjected to dead and live loading.
Fig. CA5.1-1 Definition of the Randomness of $Q$ and $R$

$$\ln(\frac{R}{Q})_m$$

Fig. CA5.1-2 Definition of the Reliability Index $\beta$
The design requirement of the 1986 AISI Specification for such a beam is

\[ S_e F_y / FS = (L_s^2 s/8)(D+L) \]  \hspace{1cm} (CA5.1-3)

where

- \( S_e \) = elastic section modulus based on the effective section
- \( FS = 5/3 \) = the factor of safety for bending
- \( F_y \) = specified yield point
- \( L_s \) = span length, and \( s \) = beam spacing
- \( D \) and \( L \) are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ref. 9)

\[ R_m = R_n (P_m M_m F_m) \]  \hspace{1cm} (CA5.1-4)

In this equation \( R_n \) is the nominal resistance, which in this case is

\[ R_n = S_e F_y \]  \hspace{1cm} (CA5.1-5)

that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange. The mean values \( P_m, M_m, \) and \( F_m \), and the corresponding coefficients of variation \( V_{P}, V_{M}, \) and \( V_{F} \), are the statistical parameters which define the variability of the resistance:

- \( P_m \) = the mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens
- \( M_m \) = mean ratio of the yield point to the minimum specified value
- \( F_m \) = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of \( R \) equals

\[ V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \]  \hspace{1cm} (CA5.1 - 6)

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield point values from tests and cross-sectional dimensions from many measurements. This information was developed in Ref. 10 and is given below:

\( P_m = 1.11, V_P = 0.09; M_m = 1.10, V_M = 0.10; F_m = 1.0, V_F = 0.05 \) and thus \( R_m = 1.22 R_n \) and \( V_R = 0.14 \).

The mean load effect is equal to

\[ Q_m = (L_s^2 s/8)(D_m + L_m) \]  \hspace{1cm} (CA5.1 - 7)

and

\[ V_Q = \sqrt{(D_m V_D)^2 + (L_m V_L)^2} / D_m + L_m \]  \hspace{1cm} (CA5.1 - 8)

where \( D_m \) and \( L_m \) are the mean dead and live load intensities, respectively, and \( V_D \) and \( V_L \) are the corresponding coefficients of variation.

Load statistics have been analyzed in Ref. 11, where it was shown that \( D_m = 1.05 D, V_D = 0.1; L_m = L, V_L = 0.25 \).
The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. CA5.1-7 and CA5.1-8 gives:

\[ Q_m = \frac{L^2 s}{8} \left( \frac{1.05D}{L} + 1 \right) L \]  
\[ V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_f^2}}{(1.05D/L + 1)} \]  

\( Q_m \) and \( V_Q \) thus depend on the dead-to-live load ratio. Cold-formed beams typically have small \( D/L \), and for the purposes of checking the reliability of these LRFD criteria it will be assumed that \( D/L = 1/5 \), and so \( Q_m = 1.21L(L^2 s/8) \) and \( V_Q = 0.21 \).

From Eq. CA5.1-3 we obtain the nominal design capacity for \( D/L = 1/5 \) and \( FS = 5/3 \). Thus:

\[ R_m = \frac{1.22 \times 2.0 \times L(L^2 s/8)}{1.21L(L^2 s/8)} = 2.02 \]

and, from Eq. CA5.1-2

\[ \beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79 \]

Of itself \( \beta = 2.79 \) for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI Specification means nothing. However, when this is compared to \( \beta \) for other types of cold-formed members, and to \( \beta \) for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Ref. 12).

**Basis for LRFD of Cold-Formed Steel Structures**

A great deal of work has been performed for determining the values of the reliability index \( \beta \) inherent in traditional design as exemplified by the current structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized in Ref 9, where also many further papers are referenced which contain additional data. The determination of \( \beta \) for cold-formed steel elements or members is presented in Refs. 10 and 13 through 17., where both the basic research data as well as the \( \beta \)’s inherent in the AISI Specification are presented in great detail. The \( \beta \)’s computed in the above referenced publications were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was re-analyzed in Refs. 11, 12 and 18 by using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 11, 12 and 18 and also only the final conclusions from the analysis are summarized here:

(1) The values of the reliability index \( \beta \) vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a
given material design specification. In order to achieve more consistent reliability, it was suggested in Ref. 18 that the following values of $\beta$ would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities $\beta_o$ for use in LRFD are:

- **Basic case:** Gravity loading, $\beta_o = 3.0$
- For connections: $\beta_o = 4.5$
- For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI/ASCE 7-88 Load Code (Ref. 19).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the 1986 AISI allowable stress design specification or to any previous version of this specification, it was shown above that for the representative dead-to-live load ratio of 1/5 the reliability index $\beta = 2.8$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.8, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit for the new LRFD Specification. The resistance factors $\phi$ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual $\beta$'s for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by ANSI/ASCE 7-88 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the 1986 AISC LRFD Specification (Ref. 3).

(2) The following load factors and load combinations were developed in Refs. 11 and 18 to give essentially the same $\beta$'s as the target $\beta_o$'s, and are recommended for use with the ANSI/ASCE 7-88 Load Code (Ref. 19) for all materials, including cold-formed steel:

1. $1.4D$
2. $1.2D + 1.0L + 0.5(L_r$ or $S$ or $R_r$)
3. $1.2D + 1.6(L_r$ or $S$ or $R_r$) + ($0.5L$ or $0.8W$)
4. $1.2D + 1.3W + 0.5L + 0.5(L_r$ or $S$ or $R_r$)
5. $1.2D + 1.5E + (0.5L$ or $0.2S)$
6. $0.9D - (1.3W$ or $1.5E)$

where:

- $D$ = nominal dead load
- $E$ = nominal earthquake load
- $L$ = nominal live load due to occupancy; weight of wet concrete for composite construction
- $L_r$ = nominal roof live load
- $R_r$ = nominal roof rain load
- $S$ = nominal snow load
- $W$ = nominal wind load
In view of the fact that the dead load of cold-formed steel structures is usually smaller than that of heavy construction, the first case of load combinations included in Section A5.1.4 of the Specification is \((1.4D+L)\) instead of the ANSI/ASCE value of \(1.4D\). This AISI requirement is identical with the ANSI/ASCE Code when \(L = 0\).

Because of special circumstances inherent in cold-formed steel structures, the following additional LRFD criteria apply for roof, floor and wall construction using cold-formed steel:

(a) For roof and floor composite construction

\[1.2D_k + 1.6C_w + 1.4C\]

where

\[D_k = \text{weight of steel deck}\]
\[C_w = \text{nominal weight of wet concrete during construction}\]
\[C = \text{nominal construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.}\]

This suggestion provides safe construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load reflects a general practice of 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, \(L_r\), in Section A5.1.4 of the AISI Specification is 1.4 instead of the ANSI/ASCE value of 1.6. The use of a relatively small load factor is because the roof live load is due to the presence of workmen and materials during repair operations and, therefore, can be considered as a type of construction load.

(b) For roof and wall construction, the load factor for the nominal wind load \(W\) to be used for the design of individual purlins, girts, wall panels and roof decks should be multiplied by a reduction factor of 0.9 because these elements are secondary members subjected to a short duration of wind load and thus can be designed for a smaller reliability than primary members such as beams and columns. For example, the reliability index of a wall panel under wind load alone is approximately 1.5 with this reduction factor. With this reduction factor designs comparable to current practice are obtained.

Deflection calculations for serviceability criteria should be made with the appropriate unfactored loads.

The load factors and load combinations given above are recommended for use with the LRFD criteria for cold-formed steel. The following portions of this Commentary present the background for the resistance factors \(\beta\) which are recommended in Section A5.1.5 for the various members and connections in Sections B, C, D and E. These \(\beta\) factors are determined in conformance with the load factors given above to approximately provide a target \(\beta_0\) of 2.5 for members and 3.5 for connections, respectively, for the load combination \(1.2D+1.6L\). For practical reasons it is desirable to have relatively few different resistance factors, and so the actual values of \(\beta\) will differ from the derived targets. This means that

\[\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL\]  
(CA5.1-11)

where \(c\) is the deterministic influence coefficient translating load intensities to load effects.
By assuming $D/L = 1/5$, Eqs. CA5.1-11 and CA5.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi)$$  \hspace{1cm} (CA5.1-12)

$$Q_m = (1.05D/L+1)cL = 1.21cL$$  \hspace{1cm} (CA5.1-13)

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n)$$  \hspace{1cm} (CA5.1-14)

The $\phi$ factors can be computed from Eq. CA5.1-14 and the following equation by using $V_Q = 0.21$:

$$\ln\left(\frac{R_m}{Q_m}\right) = \frac{1}{\sqrt{V_R^2 + V_Q^2}}$$  \hspace{1cm} (CA5.1-15)

### A5.2 Yield Point and Strength Increase from Cold Work of Forming

This section is the same as Section A5.2 of the 1986 AISI Specification.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were developed in Refs. 13 and 14 for use in the derivation of the resistance factors $\phi$:

$$(F_y)_m = 1.10F_y; \ M_m = 1.10; \ V_{F_y} = V_M = 0.10$$

$$(F_{ya})_m = 1.10F_{ya}; \ M_m = 1.10; \ V_{F_{ya}} = V_M = 0.11$$

$$(F_u)_m = 1.10F_u; \ M_m = 1.10; \ V_{F_u} = V_M = 0.08$$

$F_m = 1.00; \ V_F = 0.05$

The subscript $m$ refers to mean values. The symbol $V$ stands for coefficient of variation. The symbols $M$ and $F$ are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and $F_y$, $F_{ya}$, and $F_u$ are, respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the specified minimum tensile strength.

These data are based on the analysis of many samples, and they are representative properties of materials and cross sections used in the industrial application of cold-formed steel structures.

### A6 Reference Documents

The specifications and standards to which this Specification makes reference in various provisions are listed in Section A6 to provide the effective dates of these standards at the time of approval of this Specification.

Additional references which the designer may use for related information are:


4. Steel Deck Institute, "Design Manual for Composite Decks, Formed Decks, and Roof Decks," Steel Deck Institute, Inc. (SDI), P. O. Box 9506, Canton, Ohio 44711, 1984

5. Steel Joist Institute, "Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders," Steel Joist Institute (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986


B. ELEMENTS

B1 Dimensional Limits and Considerations

This section is the same as Section B1 of the AISI Specification for allowable stress design.

B2 Effective Widths of Stiffened Elements

This section is the same as Section B2 of the AISI Specification for allowable stress design.

B3 Effective Widths of Unstiffened Elements

This section is the same as Section B3 of the AISI Specification for allowable stress design.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

This section is the same as Section B4 of the AISI Specification for allowable stress design.

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in Section B4.2. However, proprietary testing conducted in 1989 revealed that lip lengths with a d/t ratio of greater than 14 gave unconservative results.

A review of the original research data showed a lack of data for simple stiffening lips with d/t ratios greater than 14. Therefore, an upper limit of 14 is recommended pending further research.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

This section is the same as Section B5 of the AISI Specification for allowable stress design.

B6 Stiffeners

B6.1 Transverse Stiffeners

The available experimental data on cold-formed steel transverse stiffeners were evaluated in Ref. 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Load capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 61 tests were examined. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 3.32 to 3.41. A summary of the information is given in Table CB6.1.
Table CB6.1
Computed Safety Index $\beta$ for Transverse Stiffeners
($\phi_c = 0.85$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1762</td>
<td>0.08658</td>
<td>3.32</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2099</td>
<td>0.09073</td>
<td>3.41</td>
</tr>
<tr>
<td>3</td>
<td>61</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1916</td>
<td>0.08897</td>
<td>3.36</td>
</tr>
</tbody>
</table>

Note: Case 1 = Transverse stiffeners at interior support and under concentrated load
Case 2 = Transverse stiffeners at end support
Case 3 = Sum of Cases 1 and 2

B6.2 Shear Stiffeners
The available experimental data on shear strength of the beam webs with shear stiffeners were calibrated in Ref. 10. The $\phi_v$ factors were taken as the same as those for shear strength of beams (Section C3.2). The statistical data used for determining the $\phi_v$ factor are given in Ref. 10 as follows:

\[
P_m = 1.60; \quad V_P = 0.09
\]
\[
M_m = 1.00; \quad V_M = 0.06
\]
\[
F_m = 1.00; \quad V_F = 0.05
\]

Based on all these data, the value of $\beta$ was found to be $4.10$ for $\phi_v = 0.90$.

It should be noted that the equations for determining $I_{smin}$ and $A_{si}$ of attached shear stiffeners are based on the studies summarized in Ref. 43.

B6.3 Non-Conforming Stiffeners
This Section is the same as Section B6.3 of the AISI Specification for allowable stress design.
C. MEMBERS

C1 Properties of Sections

This section is the same as Section C1 of the AISI Specification for allowable stress design.

C2 Tension Members

Section C2 of the LRFD criteria was developed on the basis of Section C2 of the AISI Specification for allowable stress design, in which the design of tension members is based only on the yield point of steel.

The resistance factor of $f_t = 0.95$ used for tension member design was derived from the procedure described in Section A5.1 of this Commentary and a selected $\beta_0$ value of 2.5. In the determination of the resistance factor, the following formulas were used for $R_m$ and $R_n$:

\begin{align*}
R_m &= A_n (F_y)_m \\
R_n &= A_n F_y \\
i.e. \quad R_m/R_n &= (F_y)_m/F_y
\end{align*}

in which $A_n$ is the net area of the cross section, $(F_y)_m$ is equal to 1.10$F_y$ as discussed in Section A5.2 of the Commentary. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation $V_R$ is:

$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11$

Based on $V_Q = 0.21$ and the resistance factor of 0.95, the value of $\beta$ is 2.4, which is close to the stated target value of $\beta_0 = 2.5$.

C3 Flexural Members

C3.1 Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength (Sec. C3.1.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Sec. C3.1.2). For C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (Sec. C3.1.3).

C3.1.1 Nominal Section Strength

The bending strength of beams with a compression flange having stiffened, partially stiffened, or unstiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1986 AISI Specification. References 5, 6, 7, and 8 provide an extensive treatment of the background research.

The experimental bases for the post-buckling strengths of cold-formed beams were examined in Refs. 8 and 10, where different cases were studied according to the types of compression flanges and the effectiveness of webs.

On the basis of the initiation of yielding, the nominal strength $R_n$ is based on the nominal effective cross section and on the specified minimum yield point, i.e., $R_n = S_e F_y$. 


Table CC3.1.1
Computed Safety Index $\beta$ for Section Bending Strength of Beams
Based on Initiation of Yielding

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened or Partially Stiffened Compression Flanges ($\phi_b = 0.95$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF. FW.</td>
<td>8</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10543</td>
<td>0.03928</td>
<td>2.76</td>
</tr>
<tr>
<td>PF. FW.</td>
<td>30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.11400</td>
<td>0.08889</td>
<td>2.65</td>
</tr>
<tr>
<td>PF. PW.</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.08162</td>
<td>0.09157</td>
<td>2.53</td>
</tr>
<tr>
<td>Unstiffened Compression Flanges ($\phi_b = 0.90$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF. FW.</td>
<td>3</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.43330</td>
<td>0.04337</td>
<td>4.05</td>
</tr>
<tr>
<td>PF. FW.</td>
<td>40</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.12384</td>
<td>0.13923</td>
<td>2.67</td>
</tr>
<tr>
<td>PF. PW.</td>
<td>10</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.03162</td>
<td>0.05538</td>
<td>2.66</td>
</tr>
</tbody>
</table>

Note: FF. = Fully effective flanges
      PF. = Partially effective flanges
      FW. = Fully effective webs
      PW. = Partially effective webs

The computed values of $\beta$ for the selected values of $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and for a dead-to-live load ratio of 1/5 for different cases are listed in Table CC3.1.1. It can be seen that the $\beta$ values vary from 2.53 to 4.05. In Table CC3.1.1, the values of $M_m$, $V_M$, $F_m$ and $V_F$ are the values presented in Sec. A5.2 of this Commentary for the material strength.

C3.1.2 Lateral Buckling Strength

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 10, and they are compared with predictions from AISI design formulas, theoretical formulas and SSRC formulas.

The statistical data used in Ref. 10 are listed in Table CC3.1.2. The symbol $P$ is the ratio of the tested capacity to the predicted value, $M$ is the ratio of the actual to the specified value of the modulus of elasticity, and $F$ is the ratio of the actual to the nominal sectional properties.

Using the recommended resistance factor $\phi_b = 0.90$, the values of $\beta$ vary from 2.35 to 3.8. See Table CC3.1.2. It should be noted that the recommended design criteria use some simplified and conservative formulas, which are the same as the allowable stress design rules included in the 1986 AISI Specification.
Table CC3.1.2
Computed Safety Index $\beta$ for Lateral Buckling Strength of Bending
($\phi_b = 0.90$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>2.5213</td>
<td>0.30955</td>
<td>3.79</td>
</tr>
<tr>
<td>2</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2359</td>
<td>0.19494</td>
<td>2.48</td>
</tr>
<tr>
<td>3</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1800</td>
<td>0.19000</td>
<td>2.35</td>
</tr>
<tr>
<td>4</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.7951</td>
<td>0.21994</td>
<td>3.53</td>
</tr>
<tr>
<td>5</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.8782</td>
<td>0.20534</td>
<td>3.80</td>
</tr>
</tbody>
</table>

Note: Case 1 = AISI approach
Case 2 = Theoretical approach with $J = 0.0026$ in.$^4$
Case 3 = SSRC approach with $J = 0.0026$ in.$^4$
Case 4 = Theoretical approach with $J = 0.0008213$ in.$^4$
Case 5 = SSRC approach with $J = 0.0008213$ in.$^4$

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The Specification contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (Refs. 20 to 24).

As indicated in Ref. 25, the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. For compressed glass fiber blanket insulation of initial thicknesses of zero to six inches, the rotational stiffness was not measurably affected (Ref. 25). To ensure adequate rotational stiffness of the roof and wall systems designed using the Specification provision, Section C3.1.3 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the $R$ values were calculated from the failure loads, using as a maximum positive moment, $M = 0.08wL^2$.

The provisions of Section C3.1.3 apply to beams on which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Refs. 20, 23, 37, 38, and 39) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression flange and braces to the compression flange located at third points or more frequently, member capacities may be increased over those without discrete braces.
In this section, the $\phi$ factor is determined for the load combination of $1.17W - 0.90D$ to approximately provide a target $\beta_0$ of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target $\beta_0$ are discussed in Section A5.1 of this Commentary. Based on this type of load combination, the following equations can be established:

\[
\begin{align*}
\phi R_n &= c(1.17W - 0.90D) = (1.17 - 0.90D/W)cW \\
Q_m &= c(W_m - D_m) \\
V_Q &= \frac{\sqrt{(W_mV_w)^2 + (D_mV_D)^2}}{W_m - D_m}
\end{align*}
\]

where $W_m$ is the mean wind load intensity and $V_w$ is the corresponding coefficient of variation.

Load statistics have been analyzed in Ref. 11, where it was shown that

\[
D_m = 1.05D, \quad V_D = 0.1; \quad W_m = 0.78W, \quad V_W = 0.37
\]

The substitution of the load statistics into Eqs. CC3.1.3-2 and CC3.1.3-3 gives

\[
\begin{align*}
Q_m &= c(0.78W - 1.05D) = (0.78 - 1.05D/W)cW \\
V_Q &= \frac{\sqrt{(0.78x0.37)^2 + (1.05D/Wx0.1)^2}}{0.78 - 1.05D/W}
\end{align*}
\]

By assuming $D/W = 0.1$, Eqs. CC3.1.3-1, CC3.1.3-4, and CC3.1.3-5 can be rewritten as follows:

\[
\begin{align*}
\phi R_n &= 1.08cW \\
Q_m &= 0.675cW = 0.675(\phi R_n/1.08) = 0.625\phi R_n \\
V_Q &= 0.43
\end{align*}
\]

The application of Eqs. CA5.1-2, CA5.1-4, CC3.1.3-6, and CA5.1-6 gives

\[
\beta = -\ln\left(\frac{1.6(M_mF_mP_m/\phi)}{V^2_M + V^2_F + V^2_P + V^2_Q}\right)
\]

or

\[
\phi = 1.6(M_mF_mP_m)\exp(-\beta\sqrt{\frac{V^2_M}{M} + \frac{V^2_F}{F} + \frac{V^2_P}{P} + \frac{V^2_Q}{Q}})
\]

The computed values of $\beta$ for the selected value of $\phi = 0.90$ for different cases are listed in Table CC3.1.3. It can be seen that the $\beta$ values vary from 1.50 to 1.60 which are satisfactory for the target value of 1.5. In Table CC3.1.3, the values of $M_m$, $V_M$, $F_m$, and $V_F$ are the values presented in Section A5.2 of this Commentary for the material strength and fabrication.
Table CC3.1.3
Computed Safety Index $\beta$ for Beams Having One Flange
Through-Fastened to Deck or Sheathing
($\phi_b = 0.90$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1995</td>
<td>0.2991</td>
<td>1.60</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0128</td>
<td>0.1112</td>
<td>1.50</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0466</td>
<td>0.1010</td>
<td>1.58</td>
</tr>
<tr>
<td>4</td>
<td>14</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0034</td>
<td>0.0689</td>
<td>1.51</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = Simple span C-sections  
Case 2 = Simple span Z-sections  
Case 3 = Continuous span C-sections  
Case 4 = Continuous span Z-sections

C3.2 Strength for Shear Only

The shear strength of beam webs is governed by either yielding or buckling, depending on the $h/t$ ratio and the mechanical properties of steel. For beam webs having small $h/t$ ratios, the shear strength is governed by shear yielding, i.e.:

$$V_n = A_w \tau_y = A_w F_y / \sqrt[3]{3} = 0.577 F_y h t$$  \hspace{1cm} (CC3.2-1)

in which $A_w$ is the area of the beam web computed by $(ht)$, and $\tau_y$ is the yield point of steel in shear, which can be computed by $F_y / \sqrt{3}$.

For beam webs having large $h/t$ ratios, the shear strength is governed by elastic shear buckling, i.e.:

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1 - \mu^2)(h/t)^2}$$  \hspace{1cm} (CC3.2-2)

in which $\tau_{cr}$ is the critical shear buckling stress in the elastic range, $k_v$ is the shear buckling coefficient, $E$ is the modulus of elasticity, $\mu$ is the Poisson's ratio, $h$ is the web depth, and $t$ is the web thickness. By using $\mu = 0.3$, the shear strength, $V_n$, can be determined as follows:

$$V_n = 0.905 E k_v t^2 / h$$  \hspace{1cm} (CC3.2-3)

For beam webs having moderate $h/t$ ratios, the shear strength is based on the inelastic buckling, i.e.:

$$V_n = 0.64 t^3 / k_v F_y E$$  \hspace{1cm} (CC3.2-4)

In view of the fact that the appropriate test data on shear are not available, the $\phi_v$ factors used in Section C3.2 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method. Thus,

$$(R_n)_{LRFD} = (R_n)_{ASD}$$  \hspace{1cm} (CC3.2-5)
Since

\[
(R_n)_{LRFD} \geq c(1.2D + 1.6l) / \phi_v \\
(R_n)_{ASD} \geq c(F.S.)(D + L)
\]

the resistance factors can be computed from the following formula:

\[
\phi_v = \frac{1.2D + 1.6l}{(F.S.)(D + L)} = \frac{1.2(D/L) + 1.6}{(F.S.)(D/L + 1)}
\]

By using a dead-to-live load ratio of D/L = 1/5, the \(\phi_v\) factors computed from the above equation are listed in Table CC3.2 for three different ranges of h/t ratios. The factors of safety are adopted from the AISI Specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

<table>
<thead>
<tr>
<th>Range of h/t Ratio</th>
<th>F.S. for Allowable Load Design</th>
<th>(\phi_v) Factor computed by Eq. CC3.2-8</th>
<th>Recommended (\phi_v) Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>(h/t \leq \sqrt{Ek_v/F_y})</td>
<td>1.44</td>
<td>1.06</td>
<td>1.00</td>
</tr>
<tr>
<td>(\sqrt{Ek_v/F_y} \leq h/t \leq 1.415 \sqrt{Ek_v/F_y})</td>
<td>1.67</td>
<td>0.92</td>
<td>0.90</td>
</tr>
<tr>
<td>(h/t &gt; 1.415 \sqrt{Ek_v/F_y})</td>
<td>1.71</td>
<td>0.90</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**C3.3 Strength for Combined Bending and Shear**

This section is based on the interaction formulas included in Section C3.3 of the AISI Specification for allowable stress design.

**C3.4 Web Crippling Strength**

The nominal concentrated load or reaction, \(P_n\), is determined by the allowable load given in Section C3.4 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. C3.4-1, C3.4-2, C3.4-4, C3.4-6 and C3.4-8, and a factor of safety of 2.0 is used for Eqs. C3.4-3, C3.4-5, C3.4-7 and C3.4-9.

On the basis of the statistical analysis of the available test data on web crippling, the values of \(P_m, M_m, F_m, V_p, V_m\) and \(V_F\) were computed and selected. These values are presented in Table CC3.4 (see Table 76 of Ref. 10). By using \(\beta = 2.5\), the resistance factors \(\phi_w = 0.75\) and 0.80 were selected for single unreinforced webs and I-sections, respectively, and is used in Sections A5.1.5 and C3.4. The values of \(\beta\) corresponding to these values of \(\phi_w\) are also given in Table CC3.4.
### Table CC3.4
Computed Safety Index $\beta$ for Web Crippling Strength of Beams

<table>
<thead>
<tr>
<th>Case No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Single, Unreinforced Webs ($\phi_w = 0.75$)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1(SF) 68</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.00</td>
<td>0.12</td>
<td>3.01</td>
</tr>
<tr>
<td>1(UF) 30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.00</td>
<td>0.16</td>
<td>2.80</td>
</tr>
<tr>
<td>2(UMR) 54</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.99</td>
<td>0.11</td>
<td>3.02</td>
</tr>
<tr>
<td>2(CA) 38</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.86</td>
<td>0.14</td>
<td>2.36</td>
</tr>
<tr>
<td>2(SUM) 92</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.94</td>
<td>0.14</td>
<td>2.67</td>
</tr>
<tr>
<td>3(UMR) 26</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.99</td>
<td>0.09</td>
<td>3.11</td>
</tr>
<tr>
<td>3(CA) 63</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.72</td>
<td>0.26</td>
<td>3.80</td>
</tr>
<tr>
<td>3(SUM) 89</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.51</td>
<td>0.34</td>
<td>2.95</td>
</tr>
<tr>
<td>4(UMR) 26</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.98</td>
<td>0.10</td>
<td>3.03</td>
</tr>
<tr>
<td>4(CA) 70</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.04</td>
<td>0.26</td>
<td>2.39</td>
</tr>
<tr>
<td>4(SUM) 96</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02</td>
<td>0.23</td>
<td>2.49</td>
</tr>
<tr>
<td><strong>I-Sections ($\phi_w = 0.80$)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 72</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10</td>
<td>0.19</td>
<td>2.74</td>
</tr>
<tr>
<td>2 27</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.96</td>
<td>0.13</td>
<td>2.57</td>
</tr>
<tr>
<td>3 53</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.01</td>
<td>0.13</td>
<td>2.76</td>
</tr>
<tr>
<td>4 62</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02</td>
<td>0.11</td>
<td>2.89</td>
</tr>
</tbody>
</table>

**Note:**
- Case 1 = End one-flange loading
- Case 2 = Interior one-flange loading
- Case 3 = End two-flange loading
- Case 4 = Interior two-flange loading
- SF = Stiffened flanges
- UF = Unstiffened flanges
- UMR = UMR and Cornell tests only
- CA = Canadian tests only
- SUM = Combine UMR and Canadian tests together
C3.5 Combined Bending and Web Crippling Strength

This section is based on the interaction formulas included in Section C3.5 of the AISI Specification for allowable stress design.

A total of 551 tests were calibrated for combined bending and web crippling strength. Six different cases were studied. Based on \( \phi_w = 0.75 \) for single unreinforced webs and \( \phi_w = 0.80 \) for I-sections, the values of safety indices vary from 2.45 to 3.27 as given in Table CC3.5.

### Table CC3.5
Computed Safety Index \( \beta \) for Combined Bending and Web Crippling

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>( M_m )</th>
<th>( V_m )</th>
<th>( F_m )</th>
<th>( V_F )</th>
<th>( P_m )</th>
<th>( V_p )</th>
<th>( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>74</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.01</td>
<td>0.07</td>
<td>3.27</td>
</tr>
<tr>
<td>2</td>
<td>202</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.87</td>
<td>0.13</td>
<td>2.45</td>
</tr>
<tr>
<td>3</td>
<td>103</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.95</td>
<td>0.10</td>
<td>2.91</td>
</tr>
<tr>
<td>4</td>
<td>66</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.03</td>
<td>0.18</td>
<td>2.79</td>
</tr>
<tr>
<td>5</td>
<td>445</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.94</td>
<td>0.14</td>
<td>2.68</td>
</tr>
</tbody>
</table>

Single, Unreinforced Webs (Interior one-flange loading)
(Based on \( \phi_w = 0.75 \))

I-Sections (Interior one-flange loading)
(Based on \( \phi_w = 0.80 \))

Note:
- Case 1 = UMR and Cornell tests only
- Case 2 = Canadian brake-formed section tests only
- Case 3 = Canadian roll-formed section tests only
- Case 4 = Hoglund's tests only
- Case 5 = Combine all tests together

C4 Concentrically Loaded Compression Members

The available experimental data on cold-formed steel concentrically loaded compression members were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Column capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 264 tests were examined; 14 different cases were studied according to the types of the column, the types of the compression flanges and the failure modes. The resistance factor \( \phi_c = 0.85 \) was selected on
the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 2.39 to 3.34. A summary of the information is given in Table CC4.

The safety indices were determined from Eq. CA5.1-2 for a D/L ratio of 1/5. Different $f_c$ factors could have been used for different cases.
### Table CC4
Computed Safety Index $\beta$ for Concentrically Loaded Compression Member
($\phi_c = 0.85$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_m$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.14610</td>
<td>0.10452</td>
<td>3.13</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.05053</td>
<td>0.07971</td>
<td>2.89</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.05523</td>
<td>0.07488</td>
<td>2.93</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10550</td>
<td>0.07601</td>
<td>3.11</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.04750</td>
<td>0.11072</td>
<td>2.76</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.22391</td>
<td>0.21814</td>
<td>2.72</td>
</tr>
<tr>
<td>7</td>
<td>9</td>
<td>1.00</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>0.96330</td>
<td>0.04424</td>
<td>2.39</td>
</tr>
<tr>
<td>8</td>
<td>41</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.19620</td>
<td>0.09608</td>
<td>3.34</td>
</tr>
<tr>
<td>9</td>
<td>18</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02900</td>
<td>0.08131</td>
<td>2.81</td>
</tr>
<tr>
<td>10</td>
<td>12</td>
<td>1.10</td>
<td>0.11</td>
<td>1.0</td>
<td>0.05</td>
<td>1.06180</td>
<td>0.11062</td>
<td>2.77</td>
</tr>
<tr>
<td>11</td>
<td>8</td>
<td>1.00</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.15290</td>
<td>0.10544</td>
<td>2.92</td>
</tr>
<tr>
<td>12</td>
<td>30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.07960</td>
<td>0.15061</td>
<td>2.68</td>
</tr>
<tr>
<td>13</td>
<td>14</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.07930</td>
<td>0.08042</td>
<td>3.00</td>
</tr>
<tr>
<td>14</td>
<td>32</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.08050</td>
<td>0.10772</td>
<td>2.89</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = Stub columns having unstiffened flanges with fully effective widths  
Case 2 = Stub columns having unstiffened flanges with partially effective widths  
Case 3 = Thin plates with partially effective widths  
Case 4 = Stub columns having stiffened compression flanges with fully effective flanges and webs  
Case 5 = Stub columns having stiffened compression flanges with partially effective flanges and fully effective webs  
Case 6 = Stub columns having stiffened compression flanges with partially effective flanges and partially effective webs  
Case 7 = Long columns having unstiffened compression flanges subjected to elastic flexural buckling  
Case 8 = Long columns having unstiffened compression flanges subjected to inelastic flexural buckling  
Case 9 = Long columns having stiffened compression flanges subjected to inelastic flexural buckling  
Case 10 = Long columns subjected to inelastic flexural buckling (include cold-work)  
Case 11 = Long columns subjected to elastic torsional-flexural buckling  
Case 12 = Long columns subjected to inelastic torsional-flexural buckling  
Case 13 = Stub columns with circular perforations  
Case 14 = Long columns with circular perforations
C5 Combined Axial Load and Bending

The LRFD Specification provide the similar interaction equations as the 1986 Edition of the AISI Specification with 1989 Addendum.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, the stable conditions and the loading conditions. Based on \( \phi_c = 0.85 \), \( \phi_h = 0.95 \) or 0.90 for nominal section strength (see Section C3.1.1), and \( \phi_h = 0.90 \) for lateral buckling strength, the values of safety indices vary from 2.7 to 3.34 as given in Table CC5.

### Table CC5
Computed Safety Index \( \beta \) for Combined Axial Load and Bending
(based on \( \phi_c = 0.85 \))

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>( M_m )</th>
<th>( V_M )</th>
<th>( F_m )</th>
<th>( V_P )</th>
<th>( P_m )</th>
<th>( V_p )</th>
<th>( \beta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0367</td>
<td>0.06619</td>
<td>2.70</td>
</tr>
<tr>
<td>2</td>
<td>13</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0509</td>
<td>0.07792</td>
<td>2.72</td>
</tr>
<tr>
<td>3</td>
<td>33</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1028</td>
<td>0.09182</td>
<td>2.86</td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1489</td>
<td>0.10478</td>
<td>2.96</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1600</td>
<td>0.13000</td>
<td>2.87</td>
</tr>
<tr>
<td>6</td>
<td>17</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1200</td>
<td>0.09000</td>
<td>2.92</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2300</td>
<td>0.08000</td>
<td>3.34</td>
</tr>
<tr>
<td>8</td>
<td>17</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0910</td>
<td>0.07950</td>
<td>2.86</td>
</tr>
<tr>
<td>9</td>
<td>12</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1110</td>
<td>0.11450</td>
<td>2.79</td>
</tr>
</tbody>
</table>

Note: Case 1 = Locally stable beam-columns, hat sections of Pekoz and Winter (1967)\(^{26}\)
Case 2 = Locally unstable beam-columns, lipped channel sections of Thomasson (1978)\(^{27}\)
Case 3 = Locally unstable beam-columns, lipped channel sections of Loughlan (1979)\(^{28}\)
Case 4 = Locally unstable beam-columns, lipped channel sections of Mulligan and Pekoz (1983)\(^{29}\)
Case 5 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985)\(^{30}\) with \( e_x \neq 0 \) and \( e_y = 0 \)
Case 6 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with \( e_x = 0 \) and \( e_y \neq 0 \)
Case 7 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with \( e_x = 0 \) and \( e_y \neq 0 \)
Case 8 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with \( e_x = 0 \) and \( e_y \neq 0 \)
Case 9 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with \( e_x \neq 0 \) and \( e_y \neq 0 \)
C6 Cylindrical Tubular Members

Section C6 of the LRFD criteria is based on Section C6 of the AISI Specification for allowable stress design.

The $\phi_b$ factor of 0.95 used in Section C6.1 for bending is the same as that used in Section C3.1.1, while the $\phi_c$ factor of 0.85 used in Section C6.2 for compression is the same as that used in Section C4 for concentrically loaded compression members.
D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

This section is the same as Section D1 of the AISI Specification for allowable stress design.

D2 Mixed Systems

This section is the same as Section D2 of the AISI Specification for allowable stress design.

D3 Lateral Bracing

This section is the same as Section D3 of the AISI Specification for allowable stress design.

With regard to the footnote for Section D3.2, it should be noted that in conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self-drilling or self-tapping, through-the-sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z-purlin supported roof systems are based on a first order, elastic stiffness model (Reference 31).

D4 Wall Studs and Wall Stud Assemblies

D4.1 Wall Studs in Compression

The AISI design provisions on the compression strength of wall studs were calibrated in Ref. 10. The statistical data used for determining the $\phi_c$ factor are given in Ref. 10 as follows:

\[
\begin{align*}
P_m &= 1.14; \\
V_P &= 0.10 \\
M_m &= 1.10; \\
V_M &= 0.10 \\
F_m &= 1.00; \\
V_F &= 0.05
\end{align*}
\]

Based on all these data and $\phi_c = 0.85$, the $\beta$ value was found to be 3.14.

The provisions in this Specification section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for nominal axial strengths for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material.

A table of shearing parameters and an equation for determining the shear rigidity are provided in this Specification. These values are based on the small scale tests described in References 32 and 33. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the stress and checking whether the shear strain at the load corresponding to the stress exceeds the permissible design shear strain of the sheathing material. In principle, the procedure is one of successive approximations.
However, if the smaller of $F_e$ (provision a) or $\sigma_{cr}$ (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

### D4.2 Wall Studs in Bending

The $\phi_b$ factors for bending strength of wall studs were taken as the same as those for section bending strength of beams (Section C3.1.1). The available test data on wall stud sections with stiffened or partially stiffened compression flanges were calibrated in Ref. 10. The statistical data used for determining the $\beta$ value are given in Ref. 10 as follows:

\[
\begin{align*}
    \phi_m &= 1.27; \quad V_F = 0.01 \\
    \phi_m &= 1.10; \quad V_M = 0.10 \\
    \phi_m &= 1.00; \quad V_P = 0.05 \\
\end{align*}
\]

Based on all these data and $\phi_b = 0.95$, the $\beta$ value was found to be 3.37.

### D4.3 Wall Studs with Combined Axial Load and Bending

The LRFD criteria provide the same interaction equations as the AISI Specification for allowable stress design.

The available test data on wall studs with combined axial load and bending were calibrated in Ref. 10. The statistical data used for determining the $\beta$ value are given in Ref. 10 as follows:

\[
\begin{align*}
    \phi_m &= 1.19; \quad V_F = 0.13 \\
    \phi_m &= 1.05; \quad V_M = 0.10 \\
    \phi_m &= 1.00; \quad V_P = 0.05 \\
\end{align*}
\]

Based on $\phi_c = 0.85$ and $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges or $\phi_b = 0.90$ for sections with unstiffened compression flanges, the $\beta$ value was found to be 2.94.
E. CONNECTIONS AND JOINTS

Section E of the LRFD criteria is based on Section E of the AISI Specification for allowable stress design. This section contains the design provisions for welded connections, bolted connections, shear rupture and connections to other materials.

The resistance factors used for welded and bolted connections were derived for a target reliability index $\beta_0 = 3.5$ and the statistical data are summarized in the subsequent sections.

E1 General Provisions

This section is based on Section E1 of the AISI Specification for allowable stress design.

E2 Welded Connections

Section E2 contains the design provisions for arc-welds (groove welds in butt joints, arc spot welds, arc seam welds, fillet welds, and flared groove welds) and resistance welds. The design equations for the nominal strength and the $\phi$ factors for groove welds in butt joints are the same as that used in the AISC LRFD criteria. (Ref. 3)

For arc spot welds, the $\phi$ factor of 0.60 used for determining the design shear strength of welds is based on the test data reported in Ref. 34. It gives a $\beta$ value of 3.55. The statistical data used for deriving the $\phi$ factor are given in Ref 10 as follows:

$$P_m = 1.17; \quad V_P = 0.22$$
$$M_m = 1.10; \quad V_M = 0.10$$
$$F_m = 1.00; \quad V_F = 0.10$$

Table CE2-1
Computed Safety Index $\beta$ for Plate Failure in Weld Connections

<table>
<thead>
<tr>
<th>Case</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\phi$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arc Spot Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.10</td>
<td>0.17</td>
<td>0.60</td>
<td>3.52</td>
</tr>
<tr>
<td>2</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>0.98</td>
<td>0.18</td>
<td>0.50</td>
<td>3.64</td>
</tr>
<tr>
<td>Fillet Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.01</td>
<td>0.08</td>
<td>0.60</td>
<td>3.65</td>
</tr>
<tr>
<td>4</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>0.89</td>
<td>0.09</td>
<td>0.55</td>
<td>3.59</td>
</tr>
<tr>
<td>5</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.05</td>
<td>0.11</td>
<td>0.60</td>
<td>3.72</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = For $d_a/t \leq 0.815 \sqrt{(E/F_u)}$  
Case 2 = For $d_a/t > 1.397 \sqrt{(E/F_u)}$  
Case 3 = Longitudinal Loading, $L/t < 25$  
Case 4 = Longitudinal Loading, $L/t \geq 25$  
Case 5 = Transverse Loading
Table CE2-2
Computed Safety Index $\beta$ for Tensile Strength of Arc Spot Weld
($\phi = 0.65$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>103</td>
<td>1.10</td>
<td>0.08</td>
<td>1.0</td>
<td>0.15</td>
<td>1.5405</td>
<td>0.2949</td>
<td>3.45</td>
</tr>
<tr>
<td>2</td>
<td>103</td>
<td>1.10</td>
<td>0.08</td>
<td>1.0</td>
<td>0.15</td>
<td>1.5405</td>
<td>0.2949</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Note: Case 1 is for 1.20+$1.6L$ ($\beta_o = 3.5$)
Case 2 is for 1.17W-$0.9D$ ($\beta_o = 2.5$)

With regard to the types of the plate failure governed by Eqs. E2.2-2 through Eq. E2.2-4 in the design criteria, $\phi$ factors were derived from the statistical data presented in Table CE2-1 (Ref. 35). The $\phi$ factors used for minimum edge distance were taken as the same as those used for bolted connections.

For tension loads on arc spot welds, tests were performed which included variables of steel strengths from 50 to 68.5 ksi, steel sheet thickness ranging from 0.031 to 0.072 inches, galvanized steel sheet, prime painted and galvanized steel plate, and visible diameter of welds ranging from 0.47 to 0.94 inches. Thus, the additional limitations only applicable to arc spot welds in tension were included in the Specification. For use in calculating the tension load on arc spot welds, the tensile strength of the connected sheet, $F_u$, is limited to a maximum of 60 ksi, although sheet with greater tensile strengths may be used. The development of the equation is contained in Reference 40. The tests were reported in Reference 41.

The statistical data for deriving the $\phi$ factor are presented in Table CE2-2 (Ref. 42). Two cases were considered in the determination of $\phi$ factor: (1) 1.2D+1.6L with $\beta_o = 3.5$, and (2) 1.17W-0.9D with $\beta_o = 2.5$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load). $\phi = 0.65$ was selected for both cases, and the values of $\beta$ corresponding to the selected $\phi$ factor are given in Table CE2-2. It can be seen that for both cases, the $\beta$ values compare satisfactorily to the target reliability indices.

For arc seam welds, the design shear strength of welds is determined from the same $\phi$ factor used for arc spot welds. The derivation of the $\phi$ factor for plate tearing is based on the following statistical data (Ref. 10):

$$P_m = 1.00; \quad V_P = 0.10$$
$$M_m = 1.10; \quad V_M = 0.10$$
$$F_m = 1.00; \quad V_F = 0.10$$

For the selected value of $\phi = 0.60$, the value of $\beta = 3.81$.

For fillet welds, the $\phi$ factors used for longitudinal loading (Eqs. E2.4-1 and E2.4-2) and transverse loading (Eq. E2.4-3) are based on the statistical data presented in Table CE2-1 (Ref. 35).

Similar to the arc spot welds, a $\phi$ factor of 0.60 is used for the design shear strength of welds.

For flare groove welds, the following statistical data were used to determined the $\phi$ factors:
(a) Transverse Flare Bevel Welds \((\phi = 0.55, \beta = 3.81)\)
- \(P_m = 1.04\); \(V_p = 0.17\)
- \(M_m = 1.10\); \(V_M = 0.10\)
- \(F_m = 1.00\); \(V_F = 0.10\)

(b) Longitudinal Flare Bevel Welds \((\phi = 0.55, \beta = 3.56)\)
- \(P_m = 0.97\); \(V_p = 0.17\)
- \(M_m = 1.10\); \(V_M = 0.10\)
- \(F_m = 1.00\); \(V_F = 0.10\)

For detailed information, see Ref. 10.

For resistance welds, the nominal shear strength is based on the following equation:

\[ R_n = (2.5) \times (\text{allowable shear per spot specified in Section E2.6 of the AISI Specification for allowable stress design}) \]

In the above equation, the safety factor is 2.5.

The \(\phi\) factor of 0.65 used in Section E2.6 for the design of resistance welds was determined on the basis of the following statistical data reported in Ref. 10. It gives a \(\beta\) value of 3.71.

- \(P_m = 1.00\); \(V_p = 0.03\)
- \(M_m = 1.10\); \(V_M = 0.10\)
- \(F_m = 1.00\); \(V_F = 0.10\)

**E3 Bolted Connections**

Section E3 of the LRFD criteria is based on Section E3 of the AISI Specification for allowable stress design. It deals only with the design of bolted connections used for connected parts thinner than 3/16 inch in thickness. For the design of bolted connections using materials equal to or greater than 3/16 inch in thickness, the AISC Specification should be used.

All \(\phi\) factors were computed from the statistical data given in Ref. 10 and \(\beta_0 = 3.5\). The statistical data used in the study are presented in Table CE3.

The \(\phi\) factors used for the high strength bolts for design shear and tensile strengths are adopted from Ref. 3.
### Table CE3
Computed Safety Index $\beta$ for Bolted Connections

<table>
<thead>
<tr>
<th>Case</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\phi$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Minimum Spacing and Edge Distance</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.13</td>
<td>0.12</td>
<td>0.70</td>
<td>3.75</td>
</tr>
<tr>
<td>2</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.18</td>
<td>0.14</td>
<td>0.70</td>
<td>3.84</td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.84</td>
<td>0.05</td>
<td>0.60</td>
<td>3.61</td>
</tr>
<tr>
<td>4</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.94</td>
<td>0.09</td>
<td>0.60</td>
<td>3.90</td>
</tr>
<tr>
<td>5</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.06</td>
<td>0.11</td>
<td>0.70</td>
<td>3.62</td>
</tr>
<tr>
<td>6</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.14</td>
<td>0.19</td>
<td>0.60</td>
<td>3.87</td>
</tr>
<tr>
<td><strong>Tension Stress on Net Section</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.14</td>
<td>0.20</td>
<td>0.65</td>
<td>3.53</td>
</tr>
<tr>
<td>8</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.95</td>
<td>0.21</td>
<td>0.55</td>
<td>3.41</td>
</tr>
<tr>
<td>9</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.04</td>
<td>0.14</td>
<td>0.65</td>
<td>3.63</td>
</tr>
<tr>
<td><strong>Bearing Stress on Bolted Connections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.08</td>
<td>0.23</td>
<td>0.55</td>
<td>3.65</td>
</tr>
<tr>
<td>11</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.97</td>
<td>0.07</td>
<td>0.65</td>
<td>3.80</td>
</tr>
<tr>
<td>12</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.02</td>
<td>0.20</td>
<td>0.60</td>
<td>3.43</td>
</tr>
<tr>
<td>13</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.05</td>
<td>0.13</td>
<td>0.60</td>
<td>4.06</td>
</tr>
<tr>
<td>14</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.01</td>
<td>0.04</td>
<td>0.70</td>
<td>3.71</td>
</tr>
<tr>
<td>15</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.93</td>
<td>0.05</td>
<td>0.65</td>
<td>3.70</td>
</tr>
<tr>
<td><strong>Shear Strength on A307 Bolts</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>1.28</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.68</td>
<td>0.11</td>
<td>0.65</td>
<td>4.73</td>
</tr>
<tr>
<td>17</td>
<td>1.13</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.60</td>
<td>0.10</td>
<td>0.65</td>
<td>3.85</td>
</tr>
<tr>
<td>18</td>
<td>1.28</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.75</td>
<td>0.10</td>
<td>0.65</td>
<td>5.23</td>
</tr>
<tr>
<td>19</td>
<td>1.36</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.63</td>
<td>0.06</td>
<td>0.65</td>
<td>4.49</td>
</tr>
<tr>
<td>20</td>
<td>1.13</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.76</td>
<td>0.06</td>
<td>0.65</td>
<td>5.09</td>
</tr>
</tbody>
</table>

Note: 
- **Case 1** = Single shear, with washers, $F_u/F_y \geq 1.15$
- **Case 2** = Double shear, with washers, $F_u/F_y \geq 1.15$
<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Single shear, with washers, $F_u / F_y &lt; 1.15$</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Double shear, with washers, $F_u / F_y &lt; 1.15$</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Single shear, without washers, $F_u / F_y \geq 1.15$</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Single shear, without washers, $F_u / F_y &lt; 1.15$</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>$t &lt; 3/16$ in., double shear, with washers</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>$t &lt; 3/16$ in., single shear, with washers</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>$t &lt; 3/16$ in., single shear, without washers</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>$0.024 \leq t &lt; 3/16$ in., double shear, with washers, $F_u / F_y \geq 1.15$</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>$0.024 \leq t &lt; 3/16$ in., double shear, with washers, $F_u / F_y &lt; 1.15$</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>$0.024 \leq t &lt; 3/16$ in., single shear, with washers, $F_u / F_y \geq 1.15$</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>$0.024 \leq t &lt; 3/16$ in., single shear, with washers, $F_u / F_y &lt; 1.15$</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>$0.036 \leq t &lt; 3/16$ in., single shear, without washers, $F_u / F_y \geq 1.15$</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>$0.036 \leq t &lt; 3/16$ in., double shear, without washers, $F_u / F_y \geq 1.15$</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Double shear, with washers, 3/8 in. diameter</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Double shear, with washers, 3/4 in. diameter</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>Single shear, with washers, 3/8 in. diameter</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Single shear, with washers, 1/2 in. diameter</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>Single shear, with washers, 3/4 in. diameter</td>
<td></td>
</tr>
</tbody>
</table>
E4 Shear Rupture

Section E4 of the LRFD criteria is based on Section E4 of the AISI Specification for allowable stress design. The $\phi$ factor used in this section is adopted from Ref. 3.

E5 Connections to Other Materials

Section E5 of the LRFD criteria is based on Section E5 of the AISI Specification for allowable stress design. The $\phi$ factor used for bearing is adopted from Ref. 3.
F. TESTS FOR SPECIAL CASES

F1 Tests for Determining Structural Performance

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same basis for the LRFD design criteria. The correction factor $C_p$ is used in the determination of $\phi$ factor to account for the influence due to the small number of tests (Ref. 36). It should be noted that when the number of tests is large enough, the effect of correction factor is negligible.

For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced (subject to wind uplift), the calibration is based on a load combination of $1.17W - 0.9D$ with $D/W = 0.1$ (see Section C3.1.3 of this Commentary for detailed discussion).

The statistical data needed for the determination of resistance factor are listed in Table F1.

F2 Tests for Confirming Structural Performance

This section is basically the same as Section F2 of the AISI Specification for allowable stress design.

F3 Tests for Determining Mechanical Properties

This section is the same as Section F3 of the AISI Specification for allowable stress design.
REFERENCES


35. Supornsilaphachai, B., "Load and Resistance Factor Design of Cold-Formed Steel Structural Members," Thesis presented to the University of Missouri-Rolla, Missouri, in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1980.

36. Pekoz, T., and Hall, W. B., "Probabilistic Evaluation of Test Results," Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1988.


40. Albrecht, R. E., "Developments and Future Needs in Welding Cold-Formed Steel," Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1988.


42. Hsiao, L. E., "Reliability Based Criteria for Cold-Formed Steel Members," Thesis presented to the University of Missouri-Rolla, Missouri, in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1989.

This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute – or any other person named herein – that the information is suitable for any general or particular use or of freedom from infringement of liability arising from such use.

1st Printing – August 1991

Prepared by University of Missouri-Rolla
Rolla, Missouri

Copyright American Iron and Steel Institute, 1991
Preface

This document provides a commentary on the background for the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members.

TABLE OF CONTENTS

PREFACE ......................................................... II-3

TABLE OF CONTENTS ............................................ II-4

INTRODUCTION .................................................. II-7

A. GENERAL PROVISIONS ....................................... II-7
   A1 Limits of Applicability and Terms ..................... II-7
   A2 Non-Conforming Shapes and Constructions ............... II-7
   A3 Material ................................................ II-7
   A4 Loads .................................................... II-8
   A5 Structural Analysis and Design ......................... II-8
      A5.1 Design Basis ......................................... II-8
      A5.2 Yield Point and Strength Increase from Cold Work of Forming ... II-15
   A6 Reference Documents ................................... II-15

B. ELEMENTS .................................................... II-17
   B1 Dimensional Limits and Considerations ................. II-17
   B2 Effective Widths of Stiffened Elements ................. II-17
   B3 Effective Widths of Unstiffened Elements ............... II-17
   B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener .......... II-17
   B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener .......... II-17
   B6 Stiffeners ............................................. II-17
      B6.1 Transverse Stiffeners ................................ II-17
      B6.2 Shear Stiffeners .................................... II-18
      B6.3 Non-Conforming Stiffeners ......................... II-18
C. MEMBERS

C1 Properties of Sections
C2 Tension Members
C3 Flexural Members
  C3.1 Strength for Bending Only
    C3.1.1 Nominal Section Strength
    C3.1.2 Lateral Buckling Strength
    C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing
C3.2 Strength for Shear Only
C3.3 Strength for Combined Bending and Shear
C3.4 Web Crippling Strength
C3.5 Combined Bending and Web Crippling Strength
C4 Concentrically Loaded Compression Members
C5 Combined Axial Load and Bending
C6 Cylindrical Tubular Members

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections
D2 Mixed Systems
D3 Lateral Bracing
D4 Wall Studs and Wall Stud Assemblies
  D4.1 Wall Studs in Compression
  D4.2 Wall Studs in Bending
  D4.3 Wall Studs with Combined Axial Load and Bending

E. CONNECTIONS AND JOINTS

E1 General Provisions
E2 Welded Connections
E3  Bolted Connections .......................... II-35
E4  Shear Rupture .................................. II-38
E5  Connections to Other Materials .................. II-38

F. TESTS FOR SPECIAL CASES ......................... II-39

F1  Tests for Determining Structural Performance ................ II-39
F2  Tests for Confirming Structural Performance ................. II-39
F3  Tests for Determining Mechanical Properties ................. II-39

REFERENCES ........................................ II-40
INTRODUCTION

In the design of steel buildings, the "Allowable Stress Design Criteria" have long been used for the design of cold-formed steel structural members in the United States and other countries. Even though the theoretical concept of reliability analysis has been available for some time and the significance of such a concept in structural safety and design is well recognized, the probabilistic method has not yet been explicitly adopted as a basis for the American design standard for cold-formed steel structures.

Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. It became evident that the development of a new specification for load and resistance factor design of cold-formed steel is highly desirable because the design criteria for heavy hot-rolled steel construction cannot possibly cover the design features of thin-walled, cold-formed steel construction completely.

Since 1976, a joint project has been conducted at University of Missouri-Rolla and Washington University to develop the new design criteria for cold-formed steel structural members and connections based on the probabilistic approach.

The Load and Resistance Factor Design criteria developed on the basis of the 1986 Edition of the AISI Specification with 1989 Addendum for allowable stress design are included in Sections A through F of this Specification.

This commentary contains a brief presentation of the methodology used for the development of the load and resistance factor design criteria. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the Specification. For detailed background information, reference is made to the research reports given in the bibliography.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

Section A1 of the LRFD Specification is essentially the same as Section A1 of the AISI Specification for allowable stress design. The definitions and various terms used for the LRFD criteria are the same as that used for the allowable stress design.

A2 Non-Conforming Shapes and Constructions

Section A2 of the LRFD Specification is essentially the same as Section A2 of the AISI Specification for allowable stress design.

A3 Material

This Section is essentially the same as Section A3 of the AISI Specification for allowable stress design.
In lieu of the tensile-to-yield strength limit of 1.08, the Specification permits the use of elongation requirements using the measurement technique as given in Ref. 1, and Part VII of the Manual. Because of limited experimental verification of the structural performance of members using material having a tensile-to-yield strength ratio less than 1.08 (Ref. 2), the Specification limits the use of this material to purlins and girts meeting the elastic design requirements of Sections C3.1.1(a), C3.1.2, and C3.1.3. Thus, the use of such steel in other applications (compression members, tension members, other flexural members including those whose strength is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins and girts, concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Section C5 are met and $P_u \phi_c P_n$ does not exceed 0.15.

A4 Loads

This Section is the same as Section A4 of the AISI Specification for allowable stress design.

With regard to ponding, design guidance can be found from Section K2 of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings (Ref. 3).

A5 Structural Analysis and Design

A5.1 Design Basis

The current method of designing cold-formed steel structural members, as presented in the 1986 AISI Specification (Ref. 4), is based on the Allowable Stress Design method. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the AISI Specification. The AISI allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. Usual factors of safety inherent in the AISI Specification for the Design of Cold-Formed Steel Structural Members are 5/3 for beams and 23/12 for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the Commentary on the AISI Specification (Refs. 5 and 6)(see also Refs. 7 and 8), and a continuing research effort provides further improvement in understanding them.

The factors of safety are provided to account for the uncertainties and variabilities inherent in the loads, the analysis, the limit state model, the material properties, the geometry, and the fabrication. Through experience it has been established that the present factors of safety provide satisfactory design.

The allowable stress design method employs only one factor of safety for a limit state. The use of multiple load factors provides a refinement in the design which can account for the different degrees of the uncertainties and variabilities of the design parameters. Such a design method is called Load and Resistance Factor Design, and its format is expressed by the following criterion:

$$\phi R_n \geq \Sigma \gamma_i Q_i$$  \hspace{1cm} (CA5.1-1)
Commentary on the Cold-Formed Steel LRFD Specification – March, 1991

where

\[ R_n = \text{the nominal resistance} \]
\[ \phi = \text{resistance factor} \]
\[ \gamma_i = \text{load factors} \]
\[ Q_i = \text{load effects} \]

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor \( \phi \) accounts for the uncertainties and variabilities inherent in the \( R_n \), and it is usually less than unity. The load effects \( Q_i \) are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and \( \gamma_i \) are the corresponding load factors which account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

The advantages of LRFD are: (1) the uncertainties and the variabilities of different types of loads and resistances are different (e.g., dead load is less variable than wind load), and so these differences can be accounted for by use of multiple factors, and (2) by using probability theory designs can ideally achieve a more consistent reliability. Thus LRFD provides the basis for a more rational and refined design method than is possible with the Allowable Stress Design method.

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects \( Q \) to nominal resistances \( R \), but both \( Q \) and \( R \) are random parameters (see Fig. CA5.1-1). A limit state is violated if \( R < Q \). While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of \( Q \) and \( R \) were known, then the probability of \( (R - Q) < 0 \) could be exactly determined for any design. In general the distributions of \( Q \) and \( R \) are not known, and only the means, \( Q_m \) and \( R_m \), and the standard deviations, \( \sigma_Q \) and \( \sigma_R \), are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Fig. CA5.1-2. The distribution curve shown is for \( \ln(R/Q) \), and a limit state is exceeded when \( \ln(R/Q) \leq 0 \). The area under \( \ln(R/Q) \leq 0 \) is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of \( \ln(R/Q) \). For given statistical data \( R_m, Q_m, \sigma_R \) and \( \sigma_Q \), the area under \( \ln(R/Q) \leq 0 \) can be varied by changing the value of \( \beta \) (Fig. CA5.1-2), since \( \beta \sigma_{\ln(R/Q)} = \ln(R/Q)_m \), from which approximately

\[ \beta = \frac{\ln(R_m/Q_m)}{\sqrt{V^2_R + V^2_Q}} \]  

\[ \text{CA5.1-2} \]

where \( V_R = \sigma_R/R_m \) and \( V_Q = \sigma_Q/Q_m \), the coefficients of variation of \( R \) and \( Q \), respectively. The index \( \beta \) is called the "reliability index", and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger \( \beta \) is more reliable.

The concept of the reliability index can be used in determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported, braced beams subjected to dead and live loading.
Fig. CA5.1-1 Definition of the Randomness of Q and R

\[ \ln\left(\frac{R}{Q}\right)_m \]

\[ \beta \sigma_{\ln(R/Q)} \]

Fig. CA5.1-2 Definition of the Reliability Index \( \beta \)
The design requirement of the 1986 AISI Specification for such a beam is

\[ S_e F_y / FS = (L_s s/8)(D+L) \]  

(CA5.1-3)

where

- \( S_e \) = elastic section modulus based on the effective section
- \( FS = 5/3 \) = the factor of safety for bending
- \( F_y \) = specified yield point
- \( L_s \) = span length, and \( s \) = beam spacing
- \( D \) and \( L \) are, respectively, the code-specified dead and live load intensities.

The mean resistance is defined as (Ref. 9)

\[ R_m = R_n(P_m M_m F_m) \]  

(CA5.1-4)

In this equation \( R_n \) is the nominal resistance, which in this case is

\[ R_n = S_e F_y \]  

(CA5.1-5)

that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange. The mean values \( P_m \), \( M_m \), and \( F_m \), and the corresponding coefficients of variation \( V_p \), \( V_M \) and \( V_F \), are the statistical parameters which define the variability of the resistance:

- \( P_m \) = the mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens
- \( M_m \) = mean ratio of the yield point to the minimum specified value
- \( F_m \) = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of \( R \) equals

\[ V_R = \sqrt{V_p^2 + V_M^2 + V_F^2} \]  

(CA5.1-6)

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield point values from tests and cross-sectional dimensions from many measurements. This information was developed in Ref. 10 and is given below:

\( P_m = 1.11, V_p = 0.09; M_m = 1.10, V_M = 0.10; F_m = 1.0, V_F = 0.05 \) and thus \( R_m = 1.22 R_n \)

and \( V_R = 0.14 \).

The mean load effect is equal to

\[ Q_m = (L_s s/8)(D_m + L_m) \]  

(CA5.1-7)

and

\[ V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \]  

(CA5.1-8)

where \( D_m \) and \( L_m \) are the mean dead and live load intensities, respectively, and \( V_D \) and \( V_L \) are the corresponding coefficients of variation.

Load statistics have been analyzed in Ref. 11, where it was shown that \( D_m = 1.05 D \), \( V_D = 0.1; L_m = L, V_L = 0.25 \).
The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. CA5.1-7 and CA5.1-8 gives

\[ Q_m = \frac{L^2}{8} \left( \frac{1.05D}{L} + 1 \right) \]  
(CA5.1 - 9)

\[ V_Q = \sqrt{\frac{(1.05DL)^2 + V_f^2}{(1.05DL + 1)}} \]  
(CA5.1 - 10)

Q_m and V_Q thus depend on the dead-to-live load ratio. Cold-formed beams typically have small D/L, and for the purposes of checking the reliability of these LRFD criteria it will be assumed that D/L = 1/5, and so Q_m = 1.21L(L^2/8) and V_Q = 0.21.

From Eq. CA5.1-3 we obtain the nominal design capacity for D/L = 1/5 and FS = 5/3. Thus

\[ \frac{R_m}{Q_m} = \frac{1.22 \times 2.0 L(L^2/8)}{1.21 L(L^2/8)} = 2.02 \]

and, from Eq. CA5.1-2

\[ \beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79 \]

Of itself \( \beta = 2.79 \) for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI Specification means nothing. However, when this is compared to \( \beta \) for other types of cold-formed members, and to \( \beta \) for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Ref. 12).

Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index \( \beta \) inherent in traditional design as exemplified by the current structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized in Ref 9, where also many further papers are referenced which contain additional data. The determination of \( \beta \) for cold-formed steel elements or members is presented in Refs. 10 and 13 through 17, where both the basic research data as well as the \( \beta \)'s inherent in the AISI Specification are presented in great detail. The \( \beta \)'s computed in the above referenced publications were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was re-analyzed in Refs. 11, 12 and 18 by using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 11, 12 and 18 and also only the final conclusions from the analysis are summarized here:

1. The values of the reliability index \( \beta \) vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a
given material design specification. In order to achieve more consistent reliability, it was suggested in Ref. 18 that the following values of $\beta$ would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities $\beta_o$ for use in LRFD are:

- Basic case: Gravity loading, $\beta_o = 3.0$
- For connections: $\beta_o = 4.5$
- For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI/ASCE 7-88 Load Code (Ref. 19).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the 1986 AISI allowable stress design specification or to any previous version of this specification, it was shown above that for the representative dead-to-live load ratio of 1/5 the reliability index $\beta = 2.8$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.8, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit for the new LRFD Specification. The resistance factors $\phi$ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual $\beta$'s for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by ANSI/ASCE 7-88 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the 1986 AISC LRFD Specification (Ref. 3).

(2) The following load factors and load combinations were developed in Refs. 11 and 18 to give essentially the same $\beta$'s as the target $\beta_o$'s, and are recommended for use with the ANSI/ASCE 7-88 Load Code (Ref. 19) for all materials, including cold-formed steel:

1. $1.4D$
2. $1.2D+1.6L+0.5(L_r \text{ or } S \text{ or } R_r)$
3. $1.2D+1.6(L_r \text{ or } S \text{ or } R_r)+(0.5L \text{ or } 0.8W)$
4. $1.2D+1.3W+0.5L+0.5(L_r \text{ or } S \text{ or } R_r)$
5. $1.2D+1.5E+(0.5L \text{ or } 0.2S)$
6. $0.9D-(1.3W \text{ or } 1.5E)$

where

- $D =$ nominal dead load
- $E =$ nominal earthquake load
- $L =$ nominal live load due to occupancy; weight of wet concrete for composite construction
- $L_r =$ nominal roof live load
- $R_r =$ nominal roof rain load
- $S =$ nominal snow load
- $W =$ nominal wind load
In view of the fact that the dead load of cold-formed steel structures is usually smaller than that of heavy construction, the first case of load combinations included in Section A5.1.4 of the Specification is \((1.4D+L)\) instead of the ANSI/ASCE value of \(1.4D\). This AISI requirement is identical with the ANSI/ASCE Code when \(L = 0\).

Because of special circumstances inherent in cold-formed steel structures, the following additional LRFD criteria apply for roof, floor and wall construction using cold-formed steel:

(a) For roof and floor composite construction

\[1.2D_s + 1.6C_w + 1.4C\]

where

\[D_s = \text{weight of steel deck}\]
\[C_w = \text{nominal weight of wet concrete during construction}\]
\[C = \text{nominal construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.}\]

This suggestion provides safe construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load reflects a general practice of 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, \(L_r\), in Section A5.1.4 of the AISI Specification is 1.4 instead of the ANSI/ASCE value of 1.6. The use of a relatively small load factor is because the roof live load is due to the presence of workmen and materials during repair operations and, therefore, can be considered as a type of construction load.

(b) For roof and wall construction, the load factor for the nominal wind load \(W\) to be used for the design of individual purlins, girts, wall panels and roof decks should be multiplied by a reduction factor of 0.9 because these elements are secondary members subjected to a short duration of wind load and thus can be designed for a smaller reliability than primary members such as beams and columns. For example, the reliability index of a wall panel under wind load alone is approximately 1.5 with this reduction factor. With this reduction factor designs comparable to current practice are obtained.

Deflection calculations for serviceability criteria should be made with the appropriate unfactored loads.

The load factors and load combinations given above are recommended for use with the LRFD criteria for cold-formed steel. The following portions of this Commentary present the background for the resistance factors \(\phi\) which are recommended in Section A5.1.5 for the various members and connections in Sections B, C, D and E. These \(\phi\) factors are determined in conformance with the load factors given above to approximately provide a target \(\beta_o\) of 2.5 for members and 3.5 for connections, respectively, for the load combination \(1.2D+1.6L\). For practical reasons it is desirable to have relatively few different resistance factors, and so the actual values of \(\beta\) will differ from the derived targets. This means that

\[\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL\]  \hspace{1cm} (CA5.1-11)

where \(c\) is the deterministic influence coefficient translating load intensities to load effects.
By assuming \( D/L = 1/5 \), Eqs. CA5.1-11 and CA5.1-9 can be rewritten as follows:

\[
\begin{align*}
R_n &= 1.84(c_L/\phi) \quad \text{(CA5.1-12)} \\
Q_m &= (1.05D/L+1)c_L = 1.21c_L \quad \text{(CA5.1-13)}
\end{align*}
\]

Therefore,

\[
\frac{R_n}{Q_m} = (1.521/\phi)(R_n/R_m) \quad \text{(CA5.1-14)}
\]

The \( \phi \) factors can be computed from Eq. CA5.1-14 and the following equation by using \( V_Q = 0.21 \):

\[
\text{Target } \beta_o = \frac{\ln(R_m/Q_m)}{\sqrt{\frac{V_R^2}{R} + \frac{V_Q^2}{Q}}} \quad \text{(CA5.1 - 15)}
\]

### A5.2 Yield Point and Strength Increase from Cold Work of Forming

This section is the same as Section A5.2 of the 1986 AISI Specification.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were developed in Refs. 13 and 14 for use in the derivation of the resistance factors \( \phi \):

\[
\begin{align*}
(F_y)_m &= 1.10F_y; \quad M_m = 1.10; \quad V_{F_y} = V_M = 0.10 \\
(F_{ya})_m &= 1.10F_{ya}; \quad M_m = 1.10; \quad V_{F_{ya}} = V_M = 0.11 \\
(F_u)_m &= 1.10F_u; \quad M_m = 1.10; \quad V_{F_u} = V_M = 0.08 \\
F_m &= 1.00; \quad V_F = 0.05
\end{align*}
\]

The subscript \( m \) refers to mean values. The symbol \( V \) stands for coefficient of variation. The symbols \( M \) and \( F \) are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and \( F_y, F_{ya} \) and \( F_u \) are, respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the specified minimum tensile strength.

These data are based on the analysis of many samples, and they are representative properties of materials and cross sections used in the industrial application of cold-formed steel structures.

### A6 Reference Documents

The specifications and standards to which this Specification makes reference in various provisions are listed in Section A6 to provide the effective dates of these standards at the time of approval of this Specification.

Additional references which the designer may use for related information are:

4. Steel Deck Institute, “Design Manual for Composite Decks, Formed Decks, and Roof Decks,” Steel Deck Institute, Inc. (SDI), P. O. Box 9506, Canton, Ohio 44711, 1984
5. Steel Joist Institute, “Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders,” Steel Joist Institute (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986
B. ELEMENTS

B1 Dimensional Limits and Considerations

This section is the same as Section B1 of the AISI Specification for allowable stress design.

B2 Effective Widths of Stiffened Elements

This section is the same as Section B2 of the AISI Specification for allowable stress design.

B3 Effective Widths of Unstiffened Elements

This section is the same as Section B3 of the AISI Specification for allowable stress design.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

This section is the same as Section B4 of the AISI Specification for allowable stress design.

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in Section B4.2. However, proprietary testing conducted in 1989 revealed that lip lengths with a d/t ratio of greater than 14 gave unconservative results.

A review of the original research data showed a lack of data for simple stiffening lips with d/t ratios greater than 14. Therefore, an upper limit of 14 is recommended pending further research.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

This section is the same as Section B5 of the AISI Specification for allowable stress design.

B6 Stiffeners

B6.1 Transverse Stiffeners

The available experimental data on cold-formed steel transverse stiffeners were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Load capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 61 tests were examined. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 3.32 to 3.41. A summary of the information is given in Table CB6.1.
Table CB6.1
Computed Safety Index $\beta$ for Transverse Stiffeners
($\phi_c = 0.85$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>33</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1762</td>
<td>0.08658</td>
<td>3.32</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2099</td>
<td>0.09073</td>
<td>3.41</td>
</tr>
<tr>
<td>3</td>
<td>61</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1916</td>
<td>0.08897</td>
<td>3.36</td>
</tr>
</tbody>
</table>

Note: Case 1 = Transverse stiffeners at interior support and under concentrated load
Case 2 = Transverse stiffeners at end support
Case 3 = Sum of Cases 1 and 2

B6.2 Shear Stiffeners

The available experimental data on shear strength of the beam webs with shear stiffeners were calibrated in Ref. 10. The $\phi_v$ factors were taken as the same as those for shear strength of beams (Section C3.2). The statistical data used for determining the $\phi_v$ factor are given in Ref. 10 as follows:

\[ P_m = 1.60; \quad V_P = 0.09 \]
\[ M_m = 1.00; \quad V_M = 0.06 \]
\[ F_m = 1.00; \quad V_F = 0.05 \]

Based on all these data, the value of $\beta$ was found to be 4.10 for $\phi_v = 0.90$.

It should be noted that the equations for determining $I_{\text{min}}$ and $A_u$ of attached shear stiffeners are based on the studies summarized in Ref. 43.

B6.3 Non-Conforming Stiffeners

This Section is the same as Section B6.3 of the AISI Specification for allowable stress design.
C. MEMBERS

C1 Properties of Sections

This section is the same as Section C1 of the AISI Specification for allowable stress design.

C2 Tension Members

Section C2 of the LRFD criteria was developed on the basis of Section C2 of the AISI Specification for allowable stress design, in which the design of tension members is based only on the yield point of steel.

The resistance factor of $\phi_t = 0.95$ used for tension member design was derived from the procedure described in Section A5.1 of this Commentary and a selected $\beta_o$ value of 2.5. In the determination of the resistance factor, the following formulas were used for $R_m$ and $R_n$:

$$R_m = A_n (F_y)_m$$

$$R_n = A_n F_y$$

e.g. $R_m/R_n = (F_y)_m/F_y$

in which $A_n$ is the net area of the cross section, $(F_y)_m$ is equal to $1.10F_y$ as discussed in Section A5.2 of the Commentary. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation $V_R$ is:

$$V_R = \sqrt{\frac{V^2}{M} + \frac{V^2}{F} + \frac{V^2}{P}} = 0.11$$

Based on $V_Q = 0.21$ and the resistance factor of 0.95, the value of $\beta$ is 2.4, which is close to the stated target value of $\beta_o = 2.5$.

C3 Flexural Members

C3.1 Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength (Sec. C3.1.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Sec. C3.1.2). For C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (Sec. C3.1.3).

C3.1.1 Nominal Section Strength

The bending strength of beams with a compression flange having stiffened, partially stiffened, or unstiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1986 AISI Specification. References 5, 6, 7, and 8 provide an extensive treatment of the background research.

The experimental bases for the post-buckling strengths of cold-formed beams were examined in Refs. 8 and 10, where different cases were studied according to the types of compression flanges and the effectiveness of webs.

On the basis of the initiation of yielding, the nominal strength $R_n$ is based on the nominal effective cross section and on the specified minimum yield point, i.e., $R_n = S_0F_y$. 
Table CC3.1.1
Computed Safety Index $\beta$ for Section Bending Strength of Beams Based on Initiation of Yielding

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened or Partially Stiffened Compression Flanges ($\phi_b = 0.95$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF. FW.</td>
<td>8</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10543</td>
<td>0.03928</td>
<td>2.76</td>
</tr>
<tr>
<td>PF. FW.</td>
<td>30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.11400</td>
<td>0.08889</td>
<td>2.65</td>
</tr>
<tr>
<td>PF. PW.</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.08162</td>
<td>0.09157</td>
<td>2.53</td>
</tr>
<tr>
<td>Unstiffened Compression Flanges ($\phi_b = 0.90$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FF. FW.</td>
<td>3</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.43330</td>
<td>0.04337</td>
<td>4.05</td>
</tr>
<tr>
<td>PF. FW.</td>
<td>40</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.12384</td>
<td>0.13923</td>
<td>2.67</td>
</tr>
<tr>
<td>PF. PW.</td>
<td>10</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.03162</td>
<td>0.05538</td>
<td>2.66</td>
</tr>
</tbody>
</table>

Note: FF. = Fully effective flanges  
PF. = Partially effective flanges  
FW. = Fully effective webs  
PW. = Partially effective webs  
For details, see Ref. 10.

The computed values of $\beta$ for the selected values of $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and for a dead-to-live load ratio of 1/5 for different cases are listed in Table CC3.1.1. It can be seen that the $\beta$ values vary from 2.53 to 4.05. In Table CC3.1.1, the values of $M_m$, $V_M$, $F_m$ and $V_F$ are the values presented in Sec. A5.2 of this Commentary for the material strength.

C3.1.2 Lateral Buckling Strength

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 10, and they are compared with predictions from AISI design formulas, theoretical formulas and SSRC formulas.

The statistical data used in Ref. 10 are listed in Table CC3.1.2. The symbol $P$ is the ratio of the tested capacity to the predicted value, $M$ is the ratio of the actual to the specified value of the modulus of elasticity, and $F$ is the ratio of the actual to the nominal sectional properties.

Using the recommended resistance factor $\phi_b = 0.90$, the values of $\beta$ vary from 2.35 to 3.8. See Table CC3.1.2. It should be noted that the recommended design criteria use some simplified and conservative formulas, which are the same as the allowable stress design rules included in the 1986 AISI Specification.
Table CC3.1.2
Computed Safety Index $\beta$ for Lateral Buckling Strength of Bending
($\phi_b = 0.90$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>2.5213</td>
<td>0.30955</td>
<td>3.79</td>
</tr>
<tr>
<td>2</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2359</td>
<td>0.19494</td>
<td>2.48</td>
</tr>
<tr>
<td>3</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1800</td>
<td>0.19000</td>
<td>2.35</td>
</tr>
<tr>
<td>4</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.7951</td>
<td>0.21994</td>
<td>3.53</td>
</tr>
<tr>
<td>5</td>
<td>47</td>
<td>1.0</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.8782</td>
<td>0.20534</td>
<td>3.80</td>
</tr>
</tbody>
</table>

Note: Case 1 = AISI approach
Case 2 = Theoretical approach with $J = 0.0026$ in.$^4$
Case 3 = SSRC approach with $J = 0.0026$ in.$^4$
Case 4 = Theoretical approach with $J = 0.0008213$ in.$^4$
Case 5 = SSRC approach with $J = 0.0008213$ in.$^4$

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The Specification contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (Refs. 20 to 24).

As indicated in Ref. 25, the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. For compressed glass fiber blanket insulation of initial thicknesses of zero to six inches, the rotational stiffness was not measurably affected (Ref. 25). To ensure adequate rotational stiffness of the roof and wall systems designed using the Specification provision, Section C3.1.3 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the $R$ values were calculated from the failure loads, using as a maximum positive moment, $M = 0.08wL^2$.

The provisions of Section C3.1.3 apply to beams on which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Refs. 20, 23, 37, 38, and 39) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression flange and braces to the compression flange located at third points or more frequently, member capacities may be increased over those without discrete braces.
In this section, the \( \phi_b \) factor is determined for the load combination of 1.17W - 0.9D to approximately provide a target \( \beta_0 \) of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target \( \beta_0 \) are discussed in Section A5.1 of this Commentary. Based on this type of load combination, the following equations can be established:

\[
\phi R_n = c(1.17W - 0.9D) = (1.17 - 0.9D/W)cW \quad (CC3.1.3-1)
\]

\[
Q_m = c(W_m - D_m) \quad (CC3.1.3-2)
\]

\[
V_Q = \frac{\sqrt{(W_m V_w)^2 + (D_m V_D)^2}}{W_m - D_m} \quad (CC3.1.3-3)
\]

where \( W_m \) is the mean wind load intensity and \( V_w \) is the corresponding coefficient of variation.

Load statistics have been analyzed in Ref. 11, where it was shown that

\[
D_m = 1.05D, \quad V_D = 0.1; \quad W_m = 0.78W, \quad V_w = 0.37
\]

The substitution of the load statistics into Eqs. CC3.1.3-2 and CC3.1.3-3 gives

\[
Q_m = c(0.78W - 1.05D) = (0.78 - 1.05D/W)cW \quad (CC3.1.3-4)
\]

\[
V_Q = \frac{\sqrt{(0.78x0.37)^2 + (1.05D/Wx0.1)^2}}{0.78 - 1.05D/W} \quad (CC3.1.3-5)
\]

By assuming \( D/W = 0.1 \), Eqs. CC3.1.3-1, CC3.1.3-4, and CC3.1.3-5 can be rewritten as follows:

\[
\phi R_n = 1.08cW
\]

\[
Q_m = 0.675cW = 0.675(\phi R_n/1.08) = 0.625\phi R_n \quad (CC3.1.3-6)
\]

\[
V_Q = 0.43
\]

The application of Eqs. CA5.1-2, CA5.1-4, CC3.1.3-6, and CA5.1-6 gives

\[
\beta = \frac{1n(1.6M_m F_m P_m /\phi)}{\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}} \quad (CC3.1.3-7)
\]

or

\[
\phi = 1.6(M_m F_m P_m)\exp(-\beta\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}) \quad (CC3.1.3-8)
\]

The computed values of \( \beta \) for the selected value of \( \phi_b = 0.90 \) for different cases are listed in Table CC3.1.3. It can be seen that the \( \beta \) values vary from 1.50 to 1.60 which are satisfactory for the target value of 1.5. In Table CC3.1.3, the values of \( M_m, V_M, F_m, \) and \( V_F \) are the values presented in Section A5.2 of this Commentary for the material strength and fabrication.
### Table CC3.1.3

Computed Safety Index $\beta$ for Beams Having One Flange Through-Fastened to Deck or Sheathing

$(\phi_b = 0.90)$

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1995</td>
<td>0.2991</td>
<td>1.60</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0128</td>
<td>0.1112</td>
<td>1.50</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0466</td>
<td>0.1010</td>
<td>1.58</td>
</tr>
<tr>
<td>4</td>
<td>14</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0034</td>
<td>0.0689</td>
<td>1.51</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = Simple span C-sections  
Case 2 = Simple span Z-sections  
Case 3 = Continuous span C-sections  
Case 4 = Continuous span Z-sections

### C3.2 Strength for Shear Only

The shear strength of beam webs is governed by either yielding or buckling, depending on the $h/t$ ratio and the mechanical properties of steel. For beam webs having small $h/t$ ratios, the shear strength is governed by shear yielding, i.e.:

$$V_n = A_w\tau_y = A_wF_y\sqrt{3} = 0.577F_yht$$  \hspace{1cm} (CC3.2-1)

in which $A_w$ is the area of the beam web computed by $(ht)$, and $\tau_y$ is the yield point of steel in shear, which can be computed by $F_y\sqrt{3}$.

For beam webs having large $h/t$ ratios, the shear strength is governed by elastic shear buckling, i.e.:

$$V_n = A_w\tau_{cr} = \frac{k_v\pi^2EA_w}{12(1-\mu^2)(h/t)^2}$$  \hspace{1cm} (CC3.2-2)

in which $\tau_{cr}$ is the critical shear buckling stress in the elastic range, $k_v$ is the shear buckling coefficient, $E$ is the modulus of elasticity, $\mu$ is the Poisson’s ratio, $h$ is the web depth, and $t$ is the web thickness. By using $\mu = 0.3$, the shear strength, $V_n$, can be determined as follows:

$$V_n = 0.905Ek_vt^3/h$$  \hspace{1cm} (CC3.2-3)

For beam webs having moderate $h/t$ ratios, the shear strength is based on the inelastic buckling, i.e.:

$$V_n = 0.64\sqrt{3}k_vF_yE$$  \hspace{1cm} (CC3.2-4)

In view of the fact that the appropriate test data on shear are not available, the $\phi_y$ factors used in Section C3.2 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method. Thus,

$$R_{n\text{LRFD}} = R_{n\text{ASD}}$$  \hspace{1cm} (CC3.2-5)
Since

\[
(R_n)_{LRFD} \geq c(1.2D + 1.6L) / \psi_v \tag{CC3.2-6}
\]

\[
(R_n)_{ASD} \geq c(F.S.)(D + L) \tag{CC3.2-7}
\]

the resistance factors can be computed from the following formula:

\[
\psi_v = \frac{1.2D + 1.6L}{(F.S.)(D + L)}
\]

\[
= \frac{1.2(D/L) + 1.6}{(F.S.)(D/L + 1)} \tag{CC3.2-8}
\]

By using a dead-to-live load ratio of \(D/L = 1/5\), the \(\psi_v\) factors computed from the above equation are listed in Table CC3.2 for three different ranges of \(h/t\) ratios. The factors of safety are adopted from the AISI Specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

### Table CC3.2

<table>
<thead>
<tr>
<th>Range of (h/t) Ratio</th>
<th>F.S. for Allowable Load Design</th>
<th>(\psi_v) Factor computed by Eq. CC3.2-8</th>
<th>Recommended (\psi_v) Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>(h/t \leq \sqrt{E_{kv}}/F_y)</td>
<td>1.44</td>
<td>1.06</td>
<td>1.00</td>
</tr>
<tr>
<td>(\sqrt{E_{kv}}/F_y \leq h/t \leq 1.415 \sqrt{E_{kv}}/F_y)</td>
<td>1.67</td>
<td>0.92</td>
<td>0.90</td>
</tr>
<tr>
<td>(h/t &gt; 1.415 \sqrt{E_{kv}}/F_y)</td>
<td>1.71</td>
<td>0.90</td>
<td>0.90</td>
</tr>
</tbody>
</table>

### C3.3 Strength for Combined Bending and Shear

This section is based on the interaction formulas included in Section C3.3 of the AISI Specification for allowable stress design.

### C3.4 Web Crippling Strength

The nominal concentrated load or reaction, \(P_n\), is determined by the allowable load given in Section C3.4 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. C3.4-1, C3.4-2, C3.4-4, C3.4-6 and C3.4-8, and a factor of safety of 2.0 is used for Eqs. C3.4-3, C3.4-5, C3.4-7 and C3.4-9.

On the basis of the statistical analysis of the available test data on web crippling, the values of \(P_m\), \(M_m\), \(F_m\), \(V_p\), \(V_M\) and \(V_F\) were computed and selected. These values are presented in Table CC3.4 (see Table 76 of Ref. 10). By using \(\beta_a = 2.5\), the resistance factors \(\phi_w = 0.75\) and 0.80 were selected for single unreinforced webs and I-sections, respectively, and is used in Sections A5.1.5 and C3.4. The values of \(\beta\) corresponding to these values of \(\phi_w\) are also given in Table CC3.4.
### Table CC3.4
Computed Safety Index $\beta$ for Web Crippling Strength of Beams

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Single, Unreinforced Webs ($\phi_w = 0.75$)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1(SF)</td>
<td>68</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.00</td>
<td>0.12</td>
<td>3.01</td>
</tr>
<tr>
<td>1(UF)</td>
<td>30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.00</td>
<td>0.16</td>
<td>2.80</td>
</tr>
<tr>
<td>2(UMR)</td>
<td>54</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.99</td>
<td>0.11</td>
<td>3.02</td>
</tr>
<tr>
<td>2(CA)</td>
<td>38</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.86</td>
<td>0.14</td>
<td>2.36</td>
</tr>
<tr>
<td>2(SUM)</td>
<td>92</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.94</td>
<td>0.14</td>
<td>2.67</td>
</tr>
<tr>
<td>3(UMR)</td>
<td>26</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.99</td>
<td>0.09</td>
<td>3.11</td>
</tr>
<tr>
<td>3(CA)</td>
<td>63</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.72</td>
<td>0.26</td>
<td>3.80</td>
</tr>
<tr>
<td>3(SUM)</td>
<td>89</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.51</td>
<td>0.34</td>
<td>2.95</td>
</tr>
<tr>
<td>4(UMR)</td>
<td>26</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.98</td>
<td>0.10</td>
<td>3.03</td>
</tr>
<tr>
<td>4(CA)</td>
<td>70</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.04</td>
<td>0.26</td>
<td>2.39</td>
</tr>
<tr>
<td>4(SUM)</td>
<td>96</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02</td>
<td>0.23</td>
<td>2.49</td>
</tr>
<tr>
<td><strong>I-Sections ($\phi_w = 0.80$)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>72</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10</td>
<td>0.19</td>
<td>2.74</td>
</tr>
<tr>
<td>2</td>
<td>27</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.96</td>
<td>0.13</td>
<td>2.57</td>
</tr>
<tr>
<td>3</td>
<td>53</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.01</td>
<td>0.13</td>
<td>2.76</td>
</tr>
<tr>
<td>4</td>
<td>62</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02</td>
<td>0.11</td>
<td>2.89</td>
</tr>
</tbody>
</table>

**Note:**
Case 1 = End one-flange loading  
Case 2 = Interior one-flange loading  
Case 3 = End two-flange loading  
Case 4 = Interior two-flange loading  
SF = Stiffened flanges  
UF = Unstiffened flanges  
UMR = UMR and Cornell tests only  
CA = Canadian tests only  
SUM = Combine UMR and Canadian tests together
**C3.5 Combined Bending and Web Crippling Strength**

This section is based on the interaction formulas included in Section C3.5 of the AISI Specification for allowable stress design.

A total of 551 tests were calibrated for combined bending and web crippling strength. Six different cases were studied. Based on $\phi_w = 0.75$ for single unreinforced webs and $\phi_w = 0.80$ for I-sections, the values of safety indices vary from 2.45 to 3.27 as given in Table CC3.5.

**Table CC3.5**

**Computed Safety Index $\beta$ for Combined Bending and Web Crippling**

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests M</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Single, Unreinforced Webs (Interior one-flange loading)</strong> (Based on $\phi_w = 0.75$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>74</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.01</td>
<td>0.07</td>
</tr>
<tr>
<td>2</td>
<td>202</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.87</td>
<td>0.13</td>
</tr>
<tr>
<td>3</td>
<td>103</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.95</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>66</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.03</td>
<td>0.18</td>
</tr>
<tr>
<td>5</td>
<td>445</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>0.94</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>I-Sections (Interior one-flange loading)</strong> (Based on $\phi_w = 0.80$)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>106</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.06</td>
<td>0.12</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = UMR and Cornell tests only  
Case 2 = Canadian brake-formed section tests only  
Case 3 = Canadian roll-formed section tests only  
Case 4 = Hoglund's tests only  
Case 5 = Combine all tests together

**C4 Concentrically Loaded Compression Members**

The available experimental data on cold-formed steel concentrically loaded compression members were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Column capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 264 tests were examined; 14 different cases were studied according to the types of the column, the types of the compression flanges and the failure modes. The resistance factor $\phi_c = 0.85$ was selected on
the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 2.39 to 3.34. A summary of the information is given in Table CC4.

The safety indices were determined from Eq. CA5.1-2 for a D/L ratio of 1/5. Different $\phi_c$ factors could have been used for different cases.
Table CC4
Computed Safety Index $\beta$ for Concentrically Loaded Compression Member
($\phi_c = 0.85$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.14610</td>
<td>0.10452</td>
<td>3.13</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.05053</td>
<td>0.07971</td>
<td>2.89</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.05523</td>
<td>0.07488</td>
<td>2.93</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.10550</td>
<td>0.07601</td>
<td>3.11</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.04750</td>
<td>0.10742</td>
<td>2.76</td>
</tr>
<tr>
<td>6</td>
<td>25</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.22391</td>
<td>0.21814</td>
<td>2.72</td>
</tr>
<tr>
<td>7</td>
<td>9</td>
<td>1.00</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>0.96330</td>
<td>0.04424</td>
<td>2.39</td>
</tr>
<tr>
<td>8</td>
<td>41</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.19620</td>
<td>0.09608</td>
<td>3.34</td>
</tr>
<tr>
<td>9</td>
<td>18</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.02900</td>
<td>0.08131</td>
<td>2.81</td>
</tr>
<tr>
<td>10</td>
<td>12</td>
<td>1.10</td>
<td>0.11</td>
<td>1.0</td>
<td>0.05</td>
<td>1.06180</td>
<td>0.11062</td>
<td>2.77</td>
</tr>
<tr>
<td>11</td>
<td>8</td>
<td>1.00</td>
<td>0.06</td>
<td>1.0</td>
<td>0.05</td>
<td>1.15290</td>
<td>0.10544</td>
<td>2.92</td>
</tr>
<tr>
<td>12</td>
<td>30</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.07960</td>
<td>0.15061</td>
<td>2.68</td>
</tr>
<tr>
<td>13</td>
<td>14</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.07930</td>
<td>0.08042</td>
<td>3.00</td>
</tr>
<tr>
<td>14</td>
<td>32</td>
<td>1.10</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.08050</td>
<td>0.10772</td>
<td>2.89</td>
</tr>
</tbody>
</table>

Note: Case 1 = Stub columns having unstiffened flanges with fully effective widths
Case 2 = Stub columns having unstiffened flanges with partially effective widths
Case 3 = Thin plates with partially effective widths
Case 4 = Stub columns having stiffened compression flanges with fully effective flanges and webs
Case 5 = Stub columns having stiffened compression flanges with partially effective flanges and fully effective webs
Case 6 = Stub columns having stiffened compression flanges with partially effective flanges and partially effective webs
Case 7 = Long columns having unstiffened compression flanges subjected to elastic flexural buckling
Case 8 = Long columns having unstiffened compression flanges subjected to inelastic flexural buckling
Case 9 = Long columns having stiffened compression flanges subjected to inelastic flexural buckling
Case 10 = Long columns subjected to inelastic flexural buckling (include cold-work)
Case 11 = Long columns subjected to elastic torsional-flexural buckling
Case 12 = Long columns subjected to inelastic torsional-flexural buckling
Case 13 = Stub columns with circular perforations
Case 14 = Long columns with circular perforations
C5 Combined Axial Load and Bending

The LRFD Specification provide the similar interaction equations as the 1986 Edition of the AISI Specification with 1989 Addendum.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, the stable conditions and the loading conditions. Based on $\phi_c = 0.85$, $\phi_b = 0.95$ or 0.90 for nominal section strength (see Section C3.1.1), and $\phi_b = 0.90$ for lateral buckling strength, the values of safety indices vary from 2.7 to 3.34 as given in Table CC5.

### Table CC5

Computed Safety Index $\beta$ for Combined Axial Load and Bending (based on $\phi_c = 0.85$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_p$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0367</td>
<td>0.06619</td>
<td>2.70</td>
</tr>
<tr>
<td>2</td>
<td>13</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0509</td>
<td>0.07792</td>
<td>2.72</td>
</tr>
<tr>
<td>3</td>
<td>33</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1028</td>
<td>0.09182</td>
<td>2.86</td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1489</td>
<td>0.10478</td>
<td>2.96</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1600</td>
<td>0.13000</td>
<td>2.87</td>
</tr>
<tr>
<td>6</td>
<td>17</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1200</td>
<td>0.09000</td>
<td>2.92</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.2300</td>
<td>0.08000</td>
<td>3.34</td>
</tr>
<tr>
<td>8</td>
<td>17</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.0910</td>
<td>0.07950</td>
<td>2.86</td>
</tr>
<tr>
<td>9</td>
<td>12</td>
<td>1.05</td>
<td>0.10</td>
<td>1.0</td>
<td>0.05</td>
<td>1.1110</td>
<td>0.11450</td>
<td>2.79</td>
</tr>
</tbody>
</table>

Note:  
Case 1 = Locally stable beam-columns, hat sections of Pekoz and Winter (1967)\(^{26}\)  
Case 2 = Locally unstable beam-columns, lipped channel sections of Thomason (1978)\(^{27}\)  
Case 3 = Locally unstable beam-columns, lipped channel sections of Loughlan (1979)\(^{28}\)  
Case 4 = Locally unstable beam-columns, lipped channel sections of Mulligan and Pekoz (1983)\(^{29}\)  
Case 5 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985)\(^{30}\) with $e_x \neq 0$ and $e_y = 0$  
Case 6 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$  
Case 7 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$  
Case 8 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$  
Case 9 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$
C6 Cylindrical Tubular Members

Section C6 of the LRFD criteria is based on Section C6 of the AISI Specification for allowable stress design.

The $\phi_b$ factor of 0.95 used in Section C6.1 for bending is the same as that used in Section C3.1.1, while the $\phi_c$ factor of 0.85 used in Section C6.2 for compression is the same as that used in Section C4 for concentrically loaded compression members.
D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

This section is the same as Section D1 of the AISI Specification for allowable stress design.

D2 Mixed Systems

This section is the same as Section D2 of the AISI Specification for allowable stress design.

D3 Lateral Bracing

This section is the same as Section D3 of the AISI Specification for allowable stress design.

With regard to the footnote for Section D3.2, it should be noted that in conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self-drilling or self-tapping, through-the-sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z-purlin supported roof systems are based on a first order, elastic stiffness model (Reference 31).

D4 Wall Studs and Wall Stud Assemblies

D4.1 Wall Studs in Compression

The AISI design provisions on the compression strength of wall studs were calibrated in Ref. 10. The statistical data used for determining the $\phi_c$ factor are given in Ref. 10 as follows:

\[
\begin{align*}
P_m &= 1.14; \\
M_m &= 1.10; \\
F_m &= 1.00; \\
V_p &= 0.10 \\
V_m &= 0.10 \\
V_F &= 0.05
\end{align*}
\]

Based on all these data and $\phi_c = 0.85$, the $\beta$ value was found to be 3.14.

The provisions in this Specification section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for nominal axial strengths for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material.

A table of shearing parameters and an equation for determining the shear rigidity are provided in this Specification. These values are based on the small scale tests described in References 32 and 33. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the stress and checking whether the shear strain at the load corresponding to the stress exceeds the permissible design shear strain of the sheathing material. In principle, the procedure is one of successive approximations.
However, if the smaller of $F_c$ (provision a) or $\sigma_{ef}$ (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

**D4.2 Wall Studs in Bending**

The $\phi_b$ factors for bending strength of wall studs were taken as the same as those for section bending strength of beams (Section C3.1.1). The available test data on wall stud sections with stiffened or partially stiffened compression flanges were calibrated in Ref. 10. The statistical data used for determining the $\beta$ value are given in Ref. 10 as follows:

\[
\begin{align*}
\Phi_m &= 1.27; \\
M_m &= 1.10; \\
F_m &= 1.00; \\
V_p &= 0.01 \\
V_M &= 0.10 \\
V_F &= 0.05
\end{align*}
\]

Based on all these data and $\phi_b = 0.95$, the $\beta$ value was found to be 3.37.

**D4.3 Wall Studs with Combined Axial Load and Bending**

The LRFD criteria provide the same interaction equations as the AISI Specification for allowable stress design.

The available test data on wall studs with combined axial load and bending were calibrated in Ref. 10. The statistical data used for determining the $\beta$ value are given in Ref. 10 as follows:

\[
\begin{align*}
\Phi_m &= 1.19; \\
M_m &= 1.05; \\
F_m &= 1.00; \\
V_p &= 0.13 \\
V_M &= 0.10 \\
V_F &= 0.05
\end{align*}
\]

Based on $\phi_c = 0.85$ and $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges or $\phi_b = 0.90$ for sections with unstiffened compression flanges, the $\beta$ value was found to be 2.94.
E. CONNECTIONS AND JOINTS

Section E of the LRFD criteria is based on Section E of the AISI Specification for allowable stress design. This section contains the design provisions for welded connections, bolted connections, shear rupture and connections to other materials.

The resistance factors used for welded and bolted connections were derived for a target reliability index $\beta_0 = 3.5$ and the statistical data are summarized in the subsequent sections.

E1 General Provisions

This section is based on Section E1 of the AISI Specification for allowable stress design.

E2 Welded Connections

Section E2 contains the design provisions for arc-welds (groove welds in butt joints, arc spot welds, arc seam welds, fillet welds, and flare groove welds) and resistance welds. The design equations for the nominal strength and the $\phi$ factors for groove welds in butt joints are the same as that used in the AISC LRFD criteria. (Ref. 3)

For arc spot welds, the $\phi$ factor of 0.60 used for determining the design shear strength of welds is based on the test data reported in Ref. 34. It gives a $\beta$ value of 3.55. The statistical data used for deriving the $\phi$ factor are given in Ref 10 as follows:

$$P_m = 1.17; \quad V_p = 0.22$$
$$M_m = 1.10; \quad V_M = 0.10$$
$$F_m = 1.00; \quad V_F = 0.10$$

Table CE2-1
Computed Safety Index $\beta$ for Plate Failure in Weld Connections

<table>
<thead>
<tr>
<th>Case</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\phi$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arc Spot Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.10</td>
<td>0.17</td>
<td>0.60</td>
<td>3.52</td>
</tr>
<tr>
<td>2</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>0.98</td>
<td>0.18</td>
<td>0.50</td>
<td>3.64</td>
</tr>
<tr>
<td>Fillet Welds</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.01</td>
<td>0.08</td>
<td>0.60</td>
<td>3.65</td>
</tr>
<tr>
<td>4</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>0.89</td>
<td>0.09</td>
<td>0.55</td>
<td>3.59</td>
</tr>
<tr>
<td>5</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</td>
<td>1.05</td>
<td>0.11</td>
<td>0.60</td>
<td>3.72</td>
</tr>
</tbody>
</table>

Note: Case 1 = For $d_w/L \leq 0.815 \sqrt{(E/F_u)}$
Case 2 = For $d_w/L > 1.397 \sqrt{(E/F_u)}$
Case 3 = Longitudinal Loading, $L/t < 25$
Case 4 = Longitudinal Loading, $L/t \geq 25$
Case 5 = Transverse Loading
Table CE2-2
Computed Safety Index $\beta$ for Tensile Strength of Arc Spot Weld
($\phi = 0.65$)

<table>
<thead>
<tr>
<th>Case</th>
<th>No. of Tests</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>103</td>
<td>1.10</td>
<td>0.08</td>
<td>1.0</td>
<td>0.15</td>
<td>1.5405</td>
<td>0.2949</td>
<td>3.45</td>
</tr>
<tr>
<td>2</td>
<td>103</td>
<td>1.10</td>
<td>0.08</td>
<td>1.0</td>
<td>0.15</td>
<td>1.5405</td>
<td>0.2949</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Note:  
Case 1 is for $1.2D+1.6L$ ($\beta_0 = 3.5$)  
Case 2 is for $1.17W-0.9D$ ($\beta_0 = 2.5$)

With regard to the types of the plate failure governed by Eqs. E2.2-2 through Eq. E2.2-4 in the design criteria, $\phi$ factors were derived from the statistical data presented in Table CE2-1 (Ref. 35). The $\phi$ factors used for minimum edge distance were taken as the same as those used for bolted connections.

For tension loads on arc spot welds, tests were performed which included variables of steel strengths from 50 to 68.5 ksi, steel sheet thickness ranging from 0.031 to 0.072 inches, galvanized steel sheet, prime painted and galvanized steel plate, and visible diameter of welds ranging from 0.47 to 0.94 inches. Thus, the additional limitations only applicable to arc spot welds in tension were included in the Specification. For use in calculating the tension load on arc spot welds, the tensile strength of the connected sheet, $F_u$, is limited to a maximum of 60 ksi, although sheet with greater tensile strengths may be used. The development of the equation is contained in Reference 40. The tests were reported in Reference 41.

The statistical data for deriving the $\phi$ factor are presented in Table CE2-2 (Ref. 42). Two cases were considered in the determination of $\phi$ factor: (1) $1.2D+1.6L$ with $\beta_0 = 3.5$, and (2) $1.17W-0.9D$ with $\beta_0 = 2.5$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load). $\phi = 0.65$ was selected for both cases, and the values of $\beta$ corresponding to the selected $\phi$ factor are given in Table CE2-2. It can be seen that for both cases, the $\beta$ values compare satisfactorily to the target reliability indices.

For arc seam welds, the design shear strength of welds is determined from the same $\phi$ factor used for arc spot welds. The derivation of the $\phi$ factor for plate tearing is based on the following statistical data (Ref. 10):

\[
\begin{align*}
M_m &= 1.10; & V_M &= 0.10 \\
F_m &= 1.00; & V_F &= 0.10 \\
\end{align*}
\]

For the selected value of $\phi = 0.60$, the value of $\beta = 3.81$.

For fillet welds, the $\phi$ factors used for longitudinal loading (Eqs. E2.4-1 and E2.4-2) and transverse loading (Eq. E2.4-3) are based on the statistical data presented in Table CE2-1 (Ref. 35).

Similar to the arc spot welds, a $\phi$ factor of 0.60 is used for the design shear strength of welds.

For flare groove welds, the following statistical data were used to determine the $\phi$ factors:
(a) Transverse Flare Bevel Welds ($\phi = 0.55, \beta = 3.81$)

\[
\begin{align*}
P_m &= 1.04; \quad V_P = 0.17 \\
M_m &= 1.10; \quad V_M = 0.10 \\
F_m &= 1.00; \quad V_F = 0.10
\end{align*}
\]

(b) Longitudinal Flare Bevel Welds ($\phi = 0.55, \beta = 3.56$)

\[
\begin{align*}
P_m &= 0.97; \quad V_P = 0.17 \\
M_m &= 1.10; \quad V_M = 0.10 \\
F_m &= 1.00; \quad V_F = 0.10
\end{align*}
\]

For detailed information, see Ref. 10.

For resistance welds, the nominal shear strength is based on the following equation:

\[ R_n = (2.5) \times \text{(allowable shear per spot specified in Section E2.6 of the AISI Specification for allowable stress design)} \]

In the above equation, the safety factor is 2.5.

The $\phi$ factor of 0.65 used in Section E2.6 for the design of resistance welds was determined on the basis of the following statistical data reported in Ref. 10. It gives a $\beta$ value of 3.71.

\[
\begin{align*}
P_m &= 1.00; \quad V_P = 0.03 \\
M_m &= 1.10; \quad V_M = 0.10 \\
F_m &= 1.00; \quad V_F = 0.10
\end{align*}
\]

**E3 Bolted Connections**

Section E3 of the LRFD criteria is based on Section E3 of the AISI Specification for allowable stress design. It deals only with the design of bolted connections used for connected parts thinner than 3/16 inch in thickness. For the design of bolted connections using materials equal to or greater than 3/16 inch in thickness, the AISC Specification should be used.

All $\phi$ factors were computed from the statistical data given in Ref. 10 and $\beta_0 = 3.5$. The statistical data used in the study are presented in Table CE3.

The $\phi$ factors used for the high strength bolts for design shear and tensile strengths are adopted from Ref. 3.
### Table CE3
Computed Safety Index $\beta$ for Bolted Connections

<table>
<thead>
<tr>
<th>Case</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
<th>$P_m$</th>
<th>$V_P$</th>
<th>$\phi$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.13</td>
<td>0.12</td>
<td>0.70</td>
<td>3.75</td>
</tr>
<tr>
<td>2</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.18</td>
<td>0.14</td>
<td>0.70</td>
<td>3.84</td>
</tr>
<tr>
<td>3</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.84</td>
<td>0.05</td>
<td>0.60</td>
<td>3.61</td>
</tr>
<tr>
<td>4</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.94</td>
<td>0.09</td>
<td>0.60</td>
<td>3.90</td>
</tr>
<tr>
<td>5</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.06</td>
<td>0.11</td>
<td>0.70</td>
<td>3.62</td>
</tr>
<tr>
<td>6</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.14</td>
<td>0.19</td>
<td>0.60</td>
<td>3.87</td>
</tr>
</tbody>
</table>

### Minimum Spacing and Edge Distance

<table>
<thead>
<tr>
<th>7</th>
<th>1.10</th>
<th>0.08</th>
<th>1.00</th>
<th>0.05</th>
<th>1.14</th>
<th>0.20</th>
<th>0.65</th>
<th>3.53</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>0.95</td>
<td>0.21</td>
<td>0.55</td>
<td>3.41</td>
</tr>
<tr>
<td>9</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
<td>1.04</td>
<td>0.14</td>
<td>0.65</td>
<td>3.63</td>
</tr>
</tbody>
</table>

### Tension Stress on Net Section

| 10 | 1.10 | 0.08 | 1.00 | 0.05 | 1.08 | 0.23 | 0.55 | 3.65 |
| 11 | 1.10 | 0.08 | 1.00 | 0.05 | 0.97 | 0.07 | 0.65 | 3.80 |
| 12 | 1.10 | 0.08 | 1.00 | 0.05 | 1.02 | 0.20 | 0.60 | 3.43 |
| 13 | 1.10 | 0.08 | 1.00 | 0.05 | 1.05 | 0.13 | 0.60 | 4.06 |
| 14 | 1.10 | 0.08 | 1.00 | 0.05 | 1.01 | 0.04 | 0.70 | 3.71 |
| 15 | 1.10 | 0.08 | 1.00 | 0.05 | 0.93 | 0.05 | 0.65 | 3.70 |

### Bearing Stress on Bolted Connections

| 16 | 1.28 | 0.08 | 1.00 | 0.05 | 0.68 | 0.11 | 0.65 | 4.73 |
| 17 | 1.13 | 0.08 | 1.00 | 0.05 | 0.60 | 0.10 | 0.65 | 3.85 |
| 18 | 1.28 | 0.08 | 1.00 | 0.05 | 0.75 | 0.10 | 0.65 | 5.23 |
| 19 | 1.36 | 0.08 | 1.00 | 0.05 | 0.63 | 0.06 | 0.65 | 4.49 |
| 20 | 1.13 | 0.08 | 1.00 | 0.05 | 0.76 | 0.06 | 0.65 | 5.09 |

Note:  
Case 1 = Single shear, with washers, $F_u/F_y \geq 1.15$  
Case 2 = Double shear, with washers, $F_u/F_y \geq 1.15$
Table CE3 (Continued)

Computed Safety Index $\beta$ for Bolted Connections

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Single shear, with washers, $F_u/F_y &lt; 1.15$</td>
</tr>
<tr>
<td>4</td>
<td>Double shear, with washers, $F_u/F_y &lt; 1.15$</td>
</tr>
<tr>
<td>5</td>
<td>Single shear, without washers, $F_u/F_y \geq 1.15$</td>
</tr>
<tr>
<td>6</td>
<td>Single shear, without washers, $F_u/F_y &lt; 1.15$</td>
</tr>
<tr>
<td>7</td>
<td>$t &lt; 3/16$ in., double shear, with washers</td>
</tr>
<tr>
<td>8</td>
<td>$t &lt; 3/16$ in., single shear, with washers</td>
</tr>
<tr>
<td>9</td>
<td>$t &lt; 3/16$ in., single shear, without washers</td>
</tr>
<tr>
<td>10</td>
<td>$0.024 \leq t &lt; 3/16$ in., double shear, with washers, $F_u/F_y \geq 1.15$</td>
</tr>
<tr>
<td>11</td>
<td>$0.024 \leq t &lt; 3/16$ in., double shear, with washers, $F_u/F_y &lt; 1.15$</td>
</tr>
<tr>
<td>12</td>
<td>$0.024 \leq t &lt; 3/16$ in., single shear, with washers, $F_u/F_y \geq 1.15$</td>
</tr>
<tr>
<td>13</td>
<td>$0.024 \leq t &lt; 3/16$ in., single shear, with washers, $F_u/F_y &lt; 1.15$</td>
</tr>
<tr>
<td>14</td>
<td>$0.036 \leq t &lt; 3/16$ in., single shear, without washers, $F_u/F_y \geq 1.15$</td>
</tr>
<tr>
<td>15</td>
<td>$0.036 \leq t &lt; 3/16$ in., double shear, without washers, $F_u/F_y \geq 1.15$</td>
</tr>
<tr>
<td>16</td>
<td>Double shear, with washers, 3/8 in. diameter</td>
</tr>
<tr>
<td>17</td>
<td>Double shear, with washers, 3/4 in. diameter</td>
</tr>
<tr>
<td>18</td>
<td>Single shear, with washers, 3/8 in. diameter</td>
</tr>
<tr>
<td>19</td>
<td>Single shear, with washers, 1/2 in. diameter</td>
</tr>
<tr>
<td>20</td>
<td>Single shear, with washers, 3/4 in. diameter</td>
</tr>
</tbody>
</table>
E4 Shear Rupture

Section E4 of the LRFD criteria is based on Section E4 of the AISI Specification for allowable stress design. The $\phi$ factor used in this section is adopted from Ref. 3.

E5 Connections to Other Materials

Section E5 of the LRFD criteria is based on Section E5 of the AISI Specification for allowable stress design. The $\phi$ factor used for bearing is adopted from Ref. 3.
F. TESTS FOR SPECIAL CASES

F1 Tests for Determining Structural Performance

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same basis for the LRFD design criteria. The correction factor $C_p$ is used in the determination of $\phi$ factor to account for the influence due to the small number of tests (Ref. 36). It should be noted that when the number of tests is large enough, the effect of correction factor is negligible.

For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced (subject to wind uplift), the calibration is based on a load combination of $1.17W - 0.9D$ with $D/W = 0.1$ (see Section C3.1.3 of this Commentary for detailed discussion).

The statistical data needed for the determination of resistance factor are listed in Table F1.

F2 Tests for Confirming Structural Performance

This section is basically the same as Section F2 of the AISI Specification for allowable stress design.

F3 Tests for Determining Mechanical Properties

This section is the same as Section F3 of the AISI Specification for allowable stress design.
REFERENCES


20. Pekoz, T. and Soroushian, D., "Behavior of C- and Z- Purlins Under Wind Uplift," Proceedings of the Sixth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1982.


24. Haussler, R. W., "Theory of Cold-Formed Steel Purlin/Girt Flexure," Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1988.


35. Supornsilaphachai, B., "Load and Resistance Factor Design of Cold-Formed Steel Structural Members," Thesis presented to the University of Missouri-Rolla, Missouri, in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1980.

36. Pekoz, T., and Hall, W. B., "Probabilistic Evaluation of Test Results," Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1988.


40. Albrecht, R. E., "Developments and Future Needs in Welding Cold-Formed Steel," Proceedings of the Ninth International Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla, Missouri, 1988.


42. Hsiao, L. E., "Reliability Based Criteria for Cold-Formed Steel Members," Thesis presented to the University of Missouri-Rolla, Missouri, in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1989.

SUPPLEMENTARY INFORMATION
ON THE MARCH 16, 1991 EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part III

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE

This document, Part III of the LRFD Cold-Formed Steel Design Manual supplements the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members. It contains two different types of information: (a) design procedures of specification nature which are not included in the Specification itself, either because they are infrequently used or are regarded as too complex for routine design, and (b) other information intended to assist users of cold-formed steel.

This Supplementary Information should be used in conjunction with the other parts of the Design Manual, which include Commentary (Part II), Illustrative Examples (Part IV), Charts and Tables (Part V), Computer Aids (Part VI), and Test Procedures (Part VII), in addition to the Specification (Part I).

American Iron and Steel Institute
December 1991
# TABLE OF CONTENTS

## PART III
### SUPPLEMENTARY INFORMATION
#### ON THE
##### MARCH 16, 1991, EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

### PREFACE .................................................................... 3

### SECTION 1—LINEAR METHOD FOR COMPUTING PROPERTIES OF FORMED SECTIONS ........................................................................................................... 5

1.1 Properties of Line Elements ............................................................. 6
   1.1.1 Straight Line Elements ............................................................. 6
   1.1.2 Circular Line Elements ............................................................. 7

1.2 Properties of Sections ................................................................ 8
   1.2.1 Equal Angles (Singly-Symmetric) With and Without Lips .......... 8
   1.2.2 Channels (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric) ................................................................. 9
   1.2.3 I-Sections with Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric) ................................................................. 11
   1.2.4 Z-Sections (Point-Symmetric) With and Without Lips ................ 13

### SECTION 2—GRAPHICAL DESIGN AIDS ......................................................... 15

2.1 Design Aids for Specification Section C3.1.2—Lateral Buckling Strength ... 15
2.2 Design Aids for Specification Section C4—Concentrically Loaded Compression Members ................................................................. 15
   2.2.1 Buckling Mode ......................................................................... 15
   2.2.2 Determination of Buckling Parameters ......................................... 15

2.3 Design Aids for Specification Section D4.1—Wall Studs in Compression .... 15

### SECTION 3—LATERALLY UNBRACED COMPRESSION FLANGES .................. 15

### SECTION 4—TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES ........................................................................................................ 17

### SECTION 5—SUMMARY OF SCOPE AND PRINCIPLE TENSILE PROPERTIES, ASTM SPECIFICATIONS ................... 19

### SECTION 6—SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION 22
PART III
SUPPLEMENTARY INFORMATION
ON THE MARCH 16, 1991, EDITION OF THE
LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION
FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 1—LINEAR METHOD FOR COMPUTING PROPERTIES OF
FORMED SECTIONS

Computation of properties of formed sections may be simplified by using a so-called linear
method, in which the material of the section is considered concentrated along the centerline of
the steel sheet and the area elements replaced by straight or curved “line elements.” The
thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation: Area = L x t, where L is the total
length of all line elements.

The moment of inertia of the section, I, is found from the relation: I = I' x t, where I' is the
moment of inertia of the centerline of the steel sheet. The section modulus is computed as usual
by dividing I or I' x t by the distance from the neutral axis to the extreme fiber, not to the
centerline of the extreme element.

First power dimensions, such as x, y, and r (radius of gyration) are obtained directly by the
linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes,
the effective design width, b, is used directly to compute the total effective length L_eff of the
line elements.

The elements into which most sections may be divided for application of the linear method
consist of straight lines and circular arcs. For convenient reference, the moments of inertia and
location of centroid of such elements are identified in the sketches and formulas in Section 1.1.

The formulas for line elements are exact, since the line as such has no thickness dimension;
but in computing the properties of an actual section, where the line element represents an
actual element with a thickness dimension, the results will be approximate for the following
reasons:

(1) The moment of inertia of a straight actual element about its longitudinal axis is consid­
ered negligible.

(2) The moment of inertia of a straight actual element inclined to the axes of reference is
slightly larger than that of the corresponding line element, but for elements of like length
the error involved is even less than the error involved in neglecting the moment of inertia
of the element about its longitudinal axis. Obviously, the error disappears when the
element is normal to the axis.

(3) Small errors are involved in using the properties of a linear arc to find those of an actual
corner, but with the usual small corner radii the error in the location of the centroid of the
corner is of little importance, and the moment of inertia generally negligible. When the
mean radius of a circular element is over four times its thickness, as for tubular sections
and for sheets with circular corrugations, the errors in using linear arc properties
practically disappear.

Using the computed values of I_x, I_y, and I_xy the moment of inertia about principal axes of
the section can be calculated by the following equation:

\[ I_{\text{Max}}_{\text{Min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_x - I_y}{2}\right)^2 + I_{xy}^2} \]
where $I_x$ and $I_y$ are the moment of inertia of the section about $x$- and $y$-axis, respectively, and $I_{xy}$ is the product of inertia.

The angle between the $x$-axis, and the minor axis is

$$
\theta = \frac{1}{2} \tan^{-1} \left[ \frac{2I_{xy}}{I_y - I_x} \right]
$$

Examples of Part IV illustrate the application of the linear method.

1.1 Properties of Line Elements

1.1.1 Straight Line Elements

Moments of inertia of straight line elements can be calculated using the equations given below:

$$
I_1 = \frac{b^2}{12}, \quad I_3 = 0
$$

$$
I_3 = l^2 + \frac{b^2}{12} = l \left( l^2 + \frac{b^2}{12} \right)
$$

$$
I_1 = 0, \quad I_2 = \frac{b^2}{12}
$$

$$
I_3 = l a^2
$$

$$
I_1 = \left[ \cos^2 \theta \frac{l^2}{12} \right] = \frac{l h^2}{12}
$$

$$
I_2 = \left[ \sin^2 \theta \frac{l^2}{12} \right] = \frac{l m^2}{12}
$$

$$
I_{12} = \left[ \sin \theta \cos \theta \frac{l^2}{12} \right] = \frac{l m n}{12}
$$

$$
I_3 = l a^2 + \frac{h b^2}{12} = l \left( a^2 + \frac{b^2}{12} \right)
$$
1.1.2 Circular Line Elements

Moments of inertia of circular line elements can be calculated using the equations given below:

\( \theta \) (expressed in radians) = 0.01743 \( \theta \) (expressed in degrees and decimals thereof)

\[ l = (\theta_2 - \theta_1) R \]

\[ c_1 = \frac{\sin \theta_2 - \sin \theta_1}{\theta_2 - \theta_1} R, \quad c_2 = \frac{\cos \theta_1 - \cos \theta_2}{\theta_2 - \theta_1} R \]

\[ I_1 = \left[ \frac{\theta_2 - \theta_1}{2} + \sin \theta_2 \cos \theta_1 - \sin \theta_1 \cos \theta_2 \right] \frac{(\theta_2 - \sin \theta_1)}{\theta_2 - \theta_1} R^3 \]

\[ I_2 = \left[ \frac{\theta_2 - \theta_1}{2} + \cos \theta_1 \cos \theta_2 - \sin \theta_1 \sin \theta_2 \right] \frac{(\cos \theta_1 - \cos \theta_2)}{\theta_2 - \theta_1} R^3 \]

\[ I_{12} = \left[ \frac{\sin^2 \theta_1 - \sin^2 \theta_2}{2} + \frac{\sin \theta_2 - \sin \theta_1}{\theta_2 - \theta_1} (\cos \theta_2 - \cos \theta_1) \right] \frac{(\sin \theta_2 - \sin \theta_1)}{\theta_2 - \theta_1} R^3 \]

\[ I_3 = \left[ \frac{\theta_2 - \theta_1}{2} + \sin \theta_2 \cos \theta_1 - \sin \theta_1 \cos \theta_2 \right] \frac{(\theta_2 - \sin \theta_1)}{\theta_2 - \theta_1} R^3 \]

\[ I_4 = \left[ \frac{\theta_2 - \theta_1}{2} + \cos \theta_1 \cos \theta_2 - \sin \theta_1 \sin \theta_2 \right] \frac{(\cos \theta_1 - \cos \theta_2)}{\theta_2 - \theta_1} R^3 \]

**CASE I:** \( \theta_1 = 0, \theta_2 = 90^\circ \)

\[ l = 1.57 R, \quad c = 0.637 R \]

\[ I_1 = 0.149 R^3 \]

\[ I_{12} = 0.137 R^3 \]

\[ I_3 = 0.785 R^2 \]

\[ I_{14} = 0.5 R^3 \]

**CASE II:** \( \theta_1 = 0, \theta_2 = \theta \)

\[ l = \theta R \]

\[ c_1 = \frac{R \sin \theta}{\theta} \]

\[ c_2 = \frac{R (1 - \cos \theta)}{\theta} \]

\[ I_1 = \left[ \frac{\theta + \sin \theta \cos \theta}{2} - \frac{\sin^2 \theta}{\theta} \right] R^3 \]

\[ I_{12} = \left[ \frac{\sin^2 \theta}{2} + \frac{\sin \theta (\cos \theta - 1)}{\theta} \right] R^3 \]

\[ I_3 = \left[ \frac{\theta + \sin \theta \cos \theta}{2} \right] R^2 \]

\[ I_{14} = \left[ \frac{\theta - \sin \theta \cos \theta}{2} \right] R^3 \]

\[ I_{14} = \left[ \frac{\sin^2 \theta}{2} \right] R^3 \]
1.2 Properties of Sections

Section properties of some sections can be calculated using the equations given below. The following are to be noted:

(1) Three different types of dimensions are employed: capital letters (A) for outside dimensions, lower case barred letters (\(a\)) for centerline dimensions, lower case letters (\(a\)) for flat dimensions. The flat dimensions are required to obtain properties such as \(I\) where corners are assumed to be round. The centerline dimensions are needed for torsional properties such as \(C_w\) where corners are assumed to be square. The outside dimensions are shown because they are the dimensions usually given in tables.

(2) All expressions consider the sections to contain round corners with the exception of those for some torsional properties (\(m\), \(j\) and \(C_w\)). These expressions are based on a square corner approximation with the exception that round corner values are used for quantities such as moment of inertia which appear in the torsional property expressions. However, allowable stresses calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(3) In the moment of inertia calculations, all quantities are accounted for except the moment of inertia of a flat element about its own axis when this is the weak axis. Moments of inertia of corners about their own axis are included to provide for the case of sections with large corner radii.

(4) All expressions are given for the full, unreduced sections.

1.2.1 Equal Angles (Singly-Symmetric) With and Without Lips

![Diagram of Equal Angle (Singly-Symmetric) With Lips](image)

NOTE: The \(x\)- and \(y\)-axes defined in these figures are referred to as the \(x_2\) and \(y_2\)-axes in the Tables of Section Properties, Part V of the Design Manual.

1. Basic parameters

\[a = A^\prime - \left[r + t/2 + \alpha(r + t/2)\right]\]
\[\tilde{a} = A^\prime - \left[t/2 + \alpha t/2\right]\]
\[c = \alpha[C' - (r + t/2)]\]
\[\bar{c} = \alpha[C' - t/2]\]
\[u = 1.57r\]

*For sections with lips, \(\alpha = 1.0\); for sections without lips, \(\alpha = 0\)*
2. Cross-sectional area
   \[ A = t[2a + u + \alpha(2c + 2u)] \]

3. Moment of inertia about x-axis
   \[ I_x = 2t\{a[0.0417a^2 + 0.0417c^2] + 0.143r^2 + \alpha[0.707a + 0.898r]^2 + 0.014r^2\} + u(0.707a + 0.898r)^2 + 0.014r^2 \]

4. Distance between centroid and centerline of corner
   \[ \bar{x} = \frac{2t}{A} \left\{ a(0.353a + 0.293r) + u(0.707a + 0.898r) \right\} \]

5. Moment of inertia about y-axis
   \[ I_y = 2t\{a(0.353a + 0.293r)^2 + 0.0417a^3 + 0.015r^3 + \alpha(0.707a + 0.353c + 1.707r)^2 + 0.417c^3 + 0.285r^3\} - A(\bar{x})^2 \]

6. Distance between shear center and centerline of corner
   \[ m = \frac{t\bar{a}(\bar{c})^2}{3\sqrt{2}\bar{I}_x}(3\bar{a} - 2\bar{c}) \]

7. St. Venant torsion constant
   \[ J = \frac{t^3}{3} [2a + u + \alpha(2c + 2u)] \]

8. Warping constant \( C_w = \frac{t^2(\bar{a})(\bar{c})^3}{18 I_x}(4\bar{a} + 3\bar{c}) \)

9. Distance from centroid to shear center \( x_o = -(\bar{x} + m)* \)

10. Parameter used to determine elastic critical moment
    \[ j = \frac{\sqrt{2t}}{48} I_y [(\bar{a})^4 + 4(\bar{a})^3(\bar{c}) - 6(\bar{a})^2(\bar{c})^2 + (\bar{c})^4] - x_o \]

### 1.2.2 Channels (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric)

1. Basic parameters
   \[ a = A' - (2r + t) \]
   \[ \bar{a} = A' - t \]
   \[ b = B' - [r + t/2 + \alpha(r + t/2)] \]
   \[ \bar{b} = B' - (t/2 + \alpha t/2) \]
   \[ c = \alpha(C' - r) \]
   \[ \bar{c} = \alpha(C' - t/2) \]
   \[ u = 1.57r \]

2. Cross-sectional area \( A = t[a + 2b + 2u + \alpha(2c + 2u)] \)

3. Moment of inertia about x-axis
   Channel: \[ I_x = 2t\{0.0417a^2 + b(0.5a + r)^2 + u(a/2 + 0.637r)^2 + 0.149 r^2 \]
   \[ + \alpha[0.0833c^2 + \frac{c}{4}(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\]
   Hat Section: \[ I_x = 2t\{0.0417a^2 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149 r^2 \]
   \[ + \alpha[0.0833c^2 + \frac{c}{4}(a + c + 4r)^2 + u(a/2 + 1.363r)^2 + 0.149r^3]\]

*Negative sign indicates \( x \) is measured in negative \( x \) direction.
4. Distance between centroid and web centerline

\[ \bar{x} = \frac{2t}{A} \left( b \left( \frac{b}{2} + r \right) + u(0.363r) + \alpha [u(b + 1.637r) + c(b + 2r)] \right) \]

5. Moment of inertia about y-axis

\[ I_y = 2t \left( b \left( \frac{b}{2} + r \right)^2 + 0.0833b^3 + 0.356r^3 + \alpha [c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3] \right) - A(x)^2 \]

6. Distance between shear center and web centerline

\[ m = \frac{bt}{12I_z} \left[ 6 \bar{c}(\bar{a})^2 + 3 \bar{b}(\bar{a})^2 - 8(\bar{c})^3 \right] \]

7. Distance between centroid and shear center

\[ x_o = -(\bar{x} + m)^* \]

8. St. Venant torsion constant

\[ J = \frac{t^3}{3} \left[ a + 2b + 2u + \alpha (2c + 2u) \right] \]

*Negative sign indicates \( x_o \) is measured in negative \( x \) direction.
9. Warping constant
   a) Channel with lips:
   \[ C_w = \frac{t^2 A}{A'} \left[ \frac{t^2}{3} + m^2 - mb \right] + \frac{A}{3t} \left[ (m y a)^2 + (b y b)^2 (2 \bar{c} + 3 \bar{a}) \right] - \frac{I_x m^2}{t} (2 \bar{a} + 4 \bar{c}) + \frac{m (\bar{c})^2}{3} \left[ 8 (\bar{b})^2 (\bar{c}) + 2m (2 \bar{c}(-\bar{a}) + b(2 \bar{c} - 3 \bar{a})) \right] + \frac{(\bar{b})^2 (\bar{a})^2}{6} \left[ (3 \bar{c} + b) (4 \bar{c} + \bar{a}) - 6 (\bar{c})^2 \right] - \frac{m^2 (\bar{a})^4}{4} \]

   Channel without lips:
   \[ C_w = \frac{t \bar{a} \bar{b}^3}{12} \left( \frac{3 \bar{b} + 2 \bar{a}}{6 \bar{b} + \bar{a}} \right) \]

   b) Hat section:
   \[ C_w = \frac{(\bar{a})^2}{4} \left[ I_y + (\bar{a})^2 A \left( 1 - \frac{\bar{a}^2}{4 I_y} \right) \right] + \frac{[2(\bar{b})^2 t (\bar{c})^3]}{3} - \bar{a} (\bar{b})^2 (\bar{c})^2 t + \frac{(\bar{a})^2 b t (\bar{c})^2 \bar{a}}{3 I_y} - \frac{4 (\bar{b})^2 (\bar{c})^5}{9 I_y} \]

10. Parameter \( \beta_w \)

   \[ \beta_w = \left[ 0.0833 \left( \frac{t \bar{x} \bar{a}}{y} \right) + t \left( \frac{t \bar{a}}{y} \right) \right] \]

11. Parameter \( \beta_t \)

   \[ \beta_t = \frac{t}{2} \left( (\bar{b} - \bar{x})^4 - (\bar{x})^4 \right) + \frac{t (\bar{a})^2}{4} \left( (\bar{b} - \bar{x})^2 - (\bar{x})^2 \right) \]

12. Parameter \( \beta_i \)

   a) Channel: \( \beta_i = 2CT (\bar{b} - \bar{x})^3 + \frac{2}{3} t (\bar{b} - \bar{x}) \left[ \left( \bar{a} / 2 \right)^3 - \left( \bar{a} / 2 - \bar{c} \right)^3 \right] \]

   b) Hat section: \( \beta_i = 2CT \left( \bar{b} - \bar{x} \right)^3 + \frac{2}{3} t (\bar{b} - \bar{x}) \left[ \left( \bar{a} / 2 + \bar{c} \right)^3 - \left( \bar{a} / 2 \right)^3 \right] \)

13. Parameter used in determination of elastic critical moment

   \[ j = \frac{1}{2I_y} \left( \beta_w + \beta_t + \beta_i \right) - x_o \]

1.2.3 I-Sections With Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric)

1. Basic parameters
   a = \( A' - \left[ r + t / 2 + \alpha (r + t / 2) \right] \)
   \( \bar{a} = A' - (t / 2 + \alpha t / 2) \)
   b = B' - (r + t / 2)
   \( \bar{b} = B' - t / 2 \)
   c = \( \alpha [ C' - (r + t / 2) ] \)
   \( \bar{c} = \alpha (C' - t / 2) \)
   u = 1.57r

*For I-sections \( \alpha = 1.0 \); for T-sections \( \alpha = 0 \)
2. Cross-sectional area

\[ A = t[2a + 2b + 2u + \alpha(2c + 2u)] \]

3. Moment of inertia about x-axis

\[ I_x = 2t(b(b/2 + r + t/2)^2 + 0.0833b^3 + u(0.363r + t/2)^2 + 0.149r^3 + \alpha(c/c/2 + r + t/2)^2 + 0.0633b^3 + u(0.363r + t/2)^2 + 0.149r^3) \]

4. Distance between centroid and longer flange centerline

\[ \bar{x} = \frac{2t}{A} \left\{ u(0.363r) + \alpha(a/2 + r) + \alpha[u(a + 1.637r) + c(a + 2r)] \right\} \]

5. Moment of inertia about y-axis

\[ I_y = 2t(0.358r^3 + \alpha(a/2 + r)^2 + 0.0633a^3 + \alpha[u(a + 1.637r)^2 + 0.149r^3 + c(a + 2r)^2] - A(x)^2 \]
6. Distance between shear center and longer flange centerline

\[ m = \pi \left( 1 - \frac{(b + c)^3}{(b)^3 + (c)^3} \right) \]

7. Distance between shear center and centroid

\[ x_o = -(\bar{x} - m) \]

8. St. Venant torsion constant

\[ J = \frac{2a^4}{3} \left[ a + b + u + \alpha(a + c) \right] \]

9. Warping constant

For I-Sections, the value of \( C_w \) is twice the value of each channel if fastened at the middle of the webs; however, if the two channels are continuously welded at both edges of the web to form the I-Section, the warping constants are as follows:

Unlipped I-Sections and T-Sections

\[ C_w = \frac{t^2}{12} \left( \frac{8(b)^3(c)^3}{(b)^3 + (c)^3} \right) \]

For double symmetric, lipped I-Sections

\[ C_w = \frac{t(b)^2}{3} \left[ (\bar{a}b^2 + 3(\bar{a})(\bar{c})^2 + 6\bar{a}(\bar{c})^2 + 4(\bar{c})^3) \right] \]

10. Parameter used in determination of elastic critical moment

\[ j = \frac{t}{2I_y} \left[ -2\bar{a}b((\bar{x})^2 + (\bar{b})^2/3) + 2\bar{a}(\bar{a} - \bar{x})(\bar{a} - \bar{x})^2 + (\bar{c})^2/3 \right] \]

\[ + \frac{1}{4} \left[ (\bar{a} - \bar{x})^4 - (\bar{c})^4 \right] \]

\[ - x_o \]

1.2.4 Z-Sections (Point-Symmetric) With and Without Lips

1. Basic parameters

\[ a = A' - (2r + t) \]
\[ \bar{a} = A' - t \]
\[ b = B' - [r + t/2 + \alpha(r + t/2)]^{**} \]
\[ \bar{b} = B' - (t/2 + \alpha t/2) \]
\[ c = \alpha[C' - (r + t/2)] \]
\[ \bar{c} = \alpha(C' - t/2) \]
\[ u = 1.57r \]

2. Cross-sectional area

\[ A = t[a + 2b + 2u + \alpha(2c + 2u)] \]

3. Moment of inertia about x-axis

\[ I_x = 2t(0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \]
\[ + \alpha[0.149r^3 + u(a/2 + 0.637r)^2 + 0.0833c^3 + \frac{c}{4} (a - c)^2] \]

4. Moment of inertia about y-axis

\[ I_y = 2t(b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 \]
\[ + u(b + 1.637r)^2 + 0.149r^3] \]

* Negative sign indicates \( x_o \) is measured in negative \( x \) direction.

** For sections with lips \( \alpha = 1.0 \), for sections without lips, \( \alpha = 0 \)
5. Product of inertia (See note below)

\[ I_{xy} = 2t(b(a/2 + r)(b/2 + r) + 0.5r^2 + 0.285ar^2 + \alpha [c(2r + b)
\quad (a/2 - c/2) - 0.137r^3 + u(b + 1.637r)(0.5a + 0.637r)] \]

6. Location of principal axis (See note below)

\[ 2\theta = \arctan \frac{2I_{xy}}{I_y - I_x} \]

7. Moment of inertia about \( x \) axis (See note below)

\[ I_x = I_x \cos^2 \theta + I_y \sin^2 \theta - 2I_{xy} \sin \theta \cos \theta \]

8. Moment of inertia about \( y \) axis (See note below)

\[ I_y = I_x \sin^2 \theta + I_y \cos^2 \theta + 2I_{xy} \sin \theta \cos \theta \]

Note: The algebraic signs in Formulas 5, 6, 7 and 8 are correct for the cross-section oriented with respect to the coordinate axes as shown in Figure 1.2.4-1 and Figure 1.2.4-2.

9. Radius of gyration about any axis

\[ r = \sqrt{I/\bar{A}} \]

10. Minimum radius of gyration, about \( x \) axis

\[ r_{min} = \sqrt{I_{x2}/\bar{A}} \]

11. St. Venant torsion constant

\[ J = \frac{\pi}{3} [a + 2b + 2u + \alpha(2c + 2u)] \]

12. Warping constant

\[ C_w = \frac{t(b)^2}{12} [(\bar{a})^2(\bar{b})^2 + 2\bar{a}\bar{b} + 4\bar{a}^2\bar{c} + 6\bar{a}\bar{c} + 4\bar{a}\bar{c}(\bar{e})^2 + 3\bar{a}\bar{b} + 4\bar{b}^2 + 2\bar{a}\bar{c} + (\bar{e})^2)]/(\bar{a} + 2\bar{b} + 2\bar{c}) \]
SECTION 2—GRAPHICAL DESIGN AIDS

The use of the charts given in Part V are explained in this section. The charts are given for angle, lipped angle, channel, lipped channel and hat sections.

2.1 Design Aids for Specification Section C3.1.2—Lateral Buckling Strength

Buckling parameters $\sigma_{ex}$ and $\sigma_{t}$ used in Specification Sections C3.1.2a, C4.2 and D4.1 can be determined using the Charts given in Part V.

Using the value of $C_x$ given in Charts V1.2, V2.2 and V3.2, parameter $\sigma_{ex}$ can be determined as follows:

$$\sigma_{ex} = \left( \frac{C_x}{a^2} \right) \left( \frac{h}{KL} \right)^{\frac{2}{3}}$$

Using the values of $C_T$ and $\sigma_{16}$ given in Charts VI.3, VI.4, V2.3, V2.4, V3.3 and V3.4, parameter $\sigma_{t}$ can be determined as follows:

$$\sigma_{t} = \frac{\sigma_{16} a^2}{t^2} \left( \frac{t}{a} \right)^{2} + \frac{C_T}{a^2} \left( \frac{h}{KL} \right)^{\frac{2}{3}}$$

The lateral buckling moment about the centroidal axis perpendicular to the symmetry axis causing tension on the shear center side of the centroid, $M_e$ can be calculated using the value of $W$ and $G$ determined from Charts V1.6, V1.7, V2.6, V2.7, V3.6 and V3.7 as follows:

$$M_e = A \sigma_{ex} a W_G$$

when $A$ is the full cross-sectional area.

2.2 Design Aids for Specification Section C4—Concentrically Loaded Compression Members

2.2.1 Buckling Mode

In Specification Section C4, $F_e$ is the least of the elastic flexural, torsional or torsional-flexural buckling stress determined according to Sections C4.1 through C4.3. Charts V1.1, V2.1 and V3.1 provide an easy means of determining which buckling mode governs. Based on the cross-sectional dimensions, the governing mode of buckling is determined as explained in these charts. These charts apply when $K_{a}$ and $K_{t}$ are equal to 1.0.

2.2.2 Determination of Buckling Parameters

Parameters $\sigma_{ex}$ and $\sigma_{t}$ are determined as described in Section 2.1 above. The torsional-flexural buckling stress $F_e$ needed in Specification Section C4.2 can be determined using the value of $F$ given in Charts V1.5, V2.5 and V3.5 as follows:

$$F_e = F \sigma_{ex}$$

2.3 Design Aids for Specification Section D4.1—Wall Studs in Compression

Parameters $\sigma_{x}$ and $\sigma_{t}$ are determined as described in Section 2.1 above.

SECTION 3—LATERALLY UNBRACED COMPRESSION FLANGES

There are many situations in cold-formed steel structures where a flexural member is so shaped or connected that it will not buckle laterally as a unit, but where the compression flange or flanges themselves are laterally unbraced and can buckle separately by a deflection of the compression flange relative to the tension flange, accompanied by out-of-plane bending of the web and the rest of the section. An example of such a situation is the use of a hat section as a flexural member in such a manner that the "brims" are in compression.

An accurate analysis of such situations is extremely complex and beyond the scope of routine design procedures. The method outlined below is based on considerable simplifications of an exact analysis. Its results have been checked against more than a hundred tests. It has been found that discrepancies rarely exceed 30 percent on the conservative to 20 percent on the
The following design procedure was developed based upon tests on individual roof panels or hat-shaped beams. These members were tested as simply supported members with two concentrated loads thus creating a region of uniform moment. Therefore, the design procedure is applicable only to an individual hat-shape type section having its free flange subjected to compression resulting from flexure. It does not apply to the following:

1. The compression elements of roof panels interconnected by welds, mechanical fasteners or mechanical seams.
2. A system comprised of flexural members and panels.

For ease of explanation, the design procedure is presented in the following 11 steps:

1. Determine the location of the neutral axis and define as the "equivalent column" the portion of the beam from the extreme compression fiber to a level which is
   \[
   \frac{3c_c - c_t}{12c_c} \text{ d distance from the extreme compression fiber.}
   \]
   In this expression \( c_c \) and \( c_t \) are the distances from the neutral axis to the extreme compression and tension fibers respectively; \( d \) is the depth of the section.

2. Determine the distance, \( y_0 \), measured parallel to the web, from the centroid of the equivalent column to its shear center. (If the cross section of the equivalent column is of angle or T-shape, its shear center is at the intersection of web and flange; if of channel shape, the location of the shear center is obtained from Section D1.1 of the Specification. If the flanges of the channel are of unequal width, for an approximation take \( w \) as the mean of the two flange widths, or compute the location of the shear center by rigorous methods.)

3. To determine the spring constant \( \beta \), isolate a portion of the member one inch long, apply a force of 0.001 kip perpendicular to the web at the level of the column centroid, and compute the corresponding lateral deflection \( D \) of that centroid. Then the spring constant \( \beta = 0.001/D \)

4. Calculate \( T_o = h/(h + 3.4y_0) \) where \( h \) is the distance from the tension flange to the centroid of the equivalent column in inches.

5. If the flange is laterally braced at two or more points calculate
   \[
   P_e = 290,000 I/L^2, \quad C = \beta L^2/P_e, \quad \text{and} \quad L' = 3.7 \sqrt{f(h/t)^3}
   \]
   where \( I \) = moment of inertia of equivalent column about its gravity parallel to web, in.\(^4\), \( L \) = unbraced length of equivalent column, in.
   If \( C \) is smaller than or equal to 30, compute
   \[
   P_{er} = TP_e[1 + \beta L^2/(\pi^2P_e)]
   \]
   If \( C \) is larger than 30, compute
   \[
   P_{er} = TP_e(0.60 + 0.635 \sqrt{\beta L^2/P_e})
   \]
   In both cases, \( T = T_o \) if \( L \) is equal to or greater than \( L' \)
   \[
   T = LT_o/L' \text{ if } L \text{ is less than } L'
   \]

6. If the flange is braced at less than two points, compute
   \[
   P_{er} = T_o \sqrt{4\beta EI}
   \]

7. Determine the slenderness ratio of the equivalent column
   \[
   (KL/r)_{eq} = 490/\sqrt{P_{er}/A_c} \quad \text{where} \quad A_c = \text{cross-sectional area of equivalent column.}
   \]

8. From paragraph (a) of Section C4 of the Specification, compute the stress, \( F_n \), corresponding to \( (KL/r) \)

9. The design compression bending stress is \( F_{cb} = 1.15 F_n(c/c) \) with a maximum of \( F_y \)

10. \( M_c = F_{cb}S_f \)

11. Go to Equation C3.1.2-1
where
\[ c_c = \text{distance from neutral axis of beam to extreme compression fiber, in.} \]
\[ y_c = \text{distance from neutral axis of beam to centroid of equivalent column, in.} \]

**SECTION 4—TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES**

Torsional-flexural buckling of non-symmetrical sections is not covered by the Specification. These sections can be designed by taking \( F_i \) in Section C4 equal to \( \sigma_{TFO} \).

The elastic torsional-flexural buckling stress, \( \sigma_{TFO} \), is less than the smallest of the Euler buckling stresses about the x- and y-axes and the torsional buckling stress. The value of \( \sigma_{TFO} \) can be obtained from the following equation by trial and error:

\[
\frac{\left( \alpha_{TFO} \right)^3}{\alpha_{ex} \alpha_{ey} \alpha_{I}} \rho - \left( \frac{\alpha_{TFO}^3}{\alpha_{ey} \alpha_{I}} \right) \gamma - \left( \frac{\alpha_{TFO}^3}{\alpha_{ex} \alpha_{I}} \right) \beta - \left( \frac{\alpha_{TFO}^3}{\alpha_{ex} \alpha_{ey}} \right) + \frac{\alpha_{TFO}}{\sigma_{ex}} + \frac{\alpha_{TFO}}{\sigma_{ey}} + \frac{\alpha_{TFO}}{\sigma_{I}} = 1
\]

The following equation may be used for a first approximation:

\[
\sigma_{TFO} = \frac{(\sigma_{ex} \sigma_{ey} + \sigma_{ex} \sigma_{ey} + \sigma_{ey} \sigma_{I}) - \sqrt{(\sigma_{ex} \sigma_{ey} + \sigma_{ex} \sigma_{ey} + \sigma_{ey} \sigma_{I})^2 - 4(\sigma_{ex} \sigma_{ey} \sigma_{I})(\sigma_{ex} \sigma_{ey} \beta + \sigma_{I})}}{2(\sigma_{ex} \gamma + \sigma_{ey} \beta + \sigma_{I})}
\]

where
\[
\sigma_{ex} = \frac{\pi^2 E}{(KL/r_x)^2}, \text{ksi}
\]
\[
\sigma_{ey} = \frac{\pi^2 E}{(KL/r_y)^2}, \text{ksi}
\]
\[
\sigma_{I} = \frac{1}{I_p} \left[ \frac{GJ + \frac{\pi^2 E C_w}{(KL)^2}}{1} \right], \text{ksi}
\]
\[
\rho = 1 - (x_o/r_o)^2 - (y_o/r_o)^2
\]
\[
\gamma = 1 - (y_o/r_o)^2
\]
\[
\beta = 1 - (x_o/r_o)^2
\]
\[
E = \text{modulus of elasticity} = 29,500 \text{ ksi}
\]
\[
L = \text{unbraced length of compression member, in.}
\]
\[
r_x = \text{radius of gyration of cross section about the x-axis, in.}
\]
\[
r_y = \text{radius of gyration of cross section about the y-axis, in.}
\]
\[
r_o = \text{polar radius of gyration of cross section about the shear center, in.}
\]
\[
I_p = \text{moment of inertia about shear center, in.}^4 = A_{x}^2 = I_x + I_y + A_{x}^2 + A_{y}^2
\]
\[
G = \text{shear modulus} = 11,300 \text{ ksi}
\]
\[
J = \text{St. Venant torsion constant of the cross section, in.}^4 \text{ For open sections composed of n segments of uniform thickness} = (1/3) (l_1 t_1^3 + l_2 t_2^3 + \ldots + l_n t_n^3)
\]
\[
C_w = \text{warping constant of torsion of the cross section, in.}^6
\]
\[
l_i = \text{length of cross section middle line of segment i, in.}
\]
\[
t_i = \text{wall thickness of segment i, in.}
\]
\[
x_o = \text{distance from shear center to centroid along the principal x-axis, in.}
\]
\[
y_o = \text{distance from shear center to centroid along the principal y-axis, in.}
\]

For any section, the values of \( x_o, y_o \) and \( C_w \) can be computed from the following relationships (terms are defined in Figure 4-1):

\[
x_o = \frac{1}{l_x} \int_0^l w_y t \, ds, \text{ in.}
\]
\[ y_0 = \frac{1}{I_y} \int_0^l w_xt \, ds, \text{ in.} \]

\[ C_w = \int_0^l (w_x)^2 t \, ds - \frac{1}{A} \left[ \int_0^l w_x t \, ds \right]^2, \text{ in.}^6 \]

where

- \( I_x \) and \( I_y \) = centroidal moments of inertia of the cross section about the principal \( x \)- and \( y \)-axes, in.\(^4\)
- \( A \) = total area of the cross section, in.\(^2\)
- \( t \) = wall thickness, in.
- \( w_c \) = \( \int_0^s R_c \, ds \), in.\(^2\)
- \( w_o \) = \( \int_0^s R_o \, ds \), in.\(^2\)

**Figure 4-1 Non-Symmetric Cross-Section**

- \( x \) and \( y \) = the coordinates measured from the centroid to any point \( P \) along the middle line of the cross section, in.
- \( s \) = distance measured along middle line of cross section from one end to the point \( P \), in.
- \( l \) = total length of the middle line of the cross section, in.
- \( R_c \) and \( R_o \) = perpendicular distances from the centroid (C.G.) and shear center (S.C.), respectively, to the middle line at \( P \). \( R_c \) or \( R_o \) is positive if a vector tangent to the middle line at \( P \) in the direction of increasing \( s \) has a counter-clockwise moment about C.G. or S.C. as shown in Figure 4-1, in.
### SECTION 5—SUMMARY OF SCOPE AND PRINCIPLE TENSILE PROPERTIES, ASTM SPECIFICATIONS

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>SCOPE (after ASTM)</th>
<th>PRODUCT</th>
<th>GRADE</th>
<th>( F_y ) ksi</th>
<th>( F_u ) ksi</th>
<th>Percent elongation in 2 inches</th>
<th>( \frac{F_y}{F_u} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36/A36M-84a</td>
<td>This specification covers carbon steel shapes, plates, and bars of structural quality for use in riveted, bolted, or welded construction of bridges and buildings, and for general structural purposes. Supplemental requirements are provided where improved notch toughness is important. These shall apply only when specified by the purchaser in the order. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.</td>
<td>Plates and Bars</td>
<td>—</td>
<td>35</td>
<td>58-80 (range)</td>
<td>23</td>
<td>1.61-2.22</td>
</tr>
<tr>
<td>A242/A242M-85</td>
<td>This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction intended primarily for use as structural members where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance of approximately two times that of carbon structural steels with copper (Note). This specification is limited to material up to 4 in. (100 mm), inclusive, in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.</td>
<td>Plates and Bars</td>
<td>—</td>
<td>50</td>
<td>70</td>
<td>18 (in 8 inches)</td>
<td>1.40</td>
</tr>
<tr>
<td>A441/A441M-85</td>
<td>This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where saving in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately twice that of structural carbon steel. This specification is limited to material up to 8 in. (200 mm) incl. in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.</td>
<td>Plates and Bars</td>
<td>—</td>
<td>50</td>
<td>70</td>
<td>18 (in 8 inches)</td>
<td>1.40</td>
</tr>
<tr>
<td>A446/A446M-85</td>
<td>This specification covers steel sheet of structural (physical) quality in coils and cut lengths, zinc-coated (galvanized). Material of this quality is intended primarily where mechanical or structural properties of the base metal are specified or required. Such properties or values include those indicated by tension, hardness, or other commonly accepted mechanical tests. Material of this quality can be produced in six grades, A through F, according to the base metal mechanical requirements prescribed in the adjacent table. Structural (physical) quality galvanized sheet is produced with any of the types of coating and coating designations listed in the latest revision of Specification A525 or A525M.</td>
<td>Sheet</td>
<td>A</td>
<td>33</td>
<td>45</td>
<td>20</td>
<td>1.36</td>
</tr>
<tr>
<td>A500-84</td>
<td>This specification covers cold-formed welded and seamless carbon steel round, square, rectangular, or special shape structural tubing for welded, riveted, or bolted construction of bridges and buildings, and for general structural purposes. This tubing is produced in both welded and seamless sizes with a maximum periphery of 64 in. (1626 mm) and a maximum wall of 0.625 in. (15.88 mm). Note: Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.</td>
<td>Round Tubing</td>
<td>A</td>
<td>33</td>
<td>45</td>
<td>25</td>
<td>1.36</td>
</tr>
<tr>
<td>A500-84</td>
<td></td>
<td></td>
<td>B</td>
<td>42</td>
<td>58</td>
<td>23</td>
<td>1.38</td>
</tr>
<tr>
<td>A500-84</td>
<td></td>
<td></td>
<td>C</td>
<td>46</td>
<td>62</td>
<td>21</td>
<td>1.35</td>
</tr>
<tr>
<td>A500-84</td>
<td></td>
<td>Shaped Tubing</td>
<td>A</td>
<td>39</td>
<td>45</td>
<td>25</td>
<td>1.15</td>
</tr>
<tr>
<td>A500-84</td>
<td></td>
<td></td>
<td>B</td>
<td>46</td>
<td>58</td>
<td>23</td>
<td>1.26</td>
</tr>
<tr>
<td>A500-84</td>
<td></td>
<td></td>
<td>C</td>
<td>50</td>
<td>62</td>
<td>21</td>
<td>1.24</td>
</tr>
</tbody>
</table>
## Supplementary Information on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>SCOPE (after ASTM)</th>
<th>PRODUCT</th>
<th>GRADE</th>
<th>$F_{u}$ ksi (min)</th>
<th>$F_{y}$ ksi (min)</th>
<th>Percent elongation in 2 inches (min)</th>
<th>$\frac{F_{u}}{F_{y}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A529/A539M-85</td>
<td>This specification covers carbon steel plates and bars 1/4 in. [13 mm] and under in thickness or diameter and Group 1 shapes shown in Table A of Specification A6/A6M of structural quality for use in metal building system frames, trusses, and related riveted, bolted, or welded construction. When used in welded construction, welding procedures shall be suitable for the steel and the intended service. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.</td>
<td>Plates and Bars</td>
<td>—</td>
<td>42</td>
<td>60-85 (range)</td>
<td>19 (in 8 inches)</td>
<td>1.43-2.02</td>
</tr>
<tr>
<td>A570-85</td>
<td>This specification covers hot-rolled carbon steel sheets and strip of structural quality in cut lengths or coils. This material is intended for structural purposes where mechanical test values are required, and is available in a maximum thickness of 0.229 in. (5.8 mm) except as limited by Specification A668, A668M, A749, or A749M.</td>
<td>Sheet and Strip</td>
<td>29</td>
<td>30</td>
<td>30</td>
<td>49</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>33</td>
<td>33</td>
<td>52</td>
<td>18 to 23</td>
<td>1.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>36</td>
<td>36</td>
<td>53</td>
<td>17 to 22</td>
<td>1.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td>40</td>
<td>55</td>
<td>15 to 21</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>45</td>
<td>60</td>
<td>13 to 19</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>50</td>
<td>65</td>
<td>11 to 17</td>
<td>1.30</td>
</tr>
<tr>
<td>A572/A572M-85</td>
<td>This specification covers four grades of high-strength low-alloy structural steel shapes, plates, sheet piling, and bars. Grades 42 (290) and 50 (345) are intended for riveted, bolted, or welded construction of bridges, buildings, and other structures. Grades 60 (415) and 65 (450) are intended for riveted or bolted construction of bridges, or for riveted, bolted, or welded construction in other applications. For welded bridge construction notch toughness is an important requirement. For this or other applications where notch-toughness requirements are indicated, they shall be negotiated between the purchaser and the producer. The use of columbium, vanadium, and nitrogen, or combinations thereof, within the limitations noted in Section 5, shall be at the option of the producer unless otherwise specified. Where designation of one of these elements or combination of elements is desired, reference is made to Supplementary Requirement S1 in which these elements and their common combinations are listed as to type. When such a designation is desired, both the grade and type must be specified. The maximum thicknesses available in the grades and products covered by this specification are shown in Table 1.</td>
<td>Plates and Bars</td>
<td>42</td>
<td>60</td>
<td>24</td>
<td>1.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>66</td>
<td>21</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>75</td>
<td>18</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>80</td>
<td>17</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>A588/A588M-85</td>
<td>This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately two times that of carbon structural steel with copper. This specification is limited to material up to 8 in. [200 mm] inclusive in thickness. Note: Two times carbon structural steel with copper is equivalent to four times carbon structural steel without copper (Cu 0.02 max). When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.</td>
<td>Plates and Bars</td>
<td>—</td>
<td>50</td>
<td>70</td>
<td>21</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>55</td>
<td>75</td>
<td>22</td>
<td>1.35</td>
<td></td>
</tr>
<tr>
<td>A606-85</td>
<td>This specification covers high-strength, low-alloy, hot-and-cold rolled sheet and strip in cut lengths or coils, intended for use in structural and miscellaneous purposes, where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance and are supplied in two types: Type 2 having corrosion resistance at least two times that of plain carbon steel and Type 4 having corrosion resistance at least four times that of plain carbon steel. The degree of corrosion resistance is based on data acceptable to the consumer. Note: Type 2 cold-rolled material is intended to replace ASTM Specification A374, for High-Strength Low-Alloy Cold-Rolled Steel Sheets and Strip, and Type 2 hot-rolled material is intended to replace ASTM Specification A375, for High-Strength Low-Alloy Hot-Rolled Steel Sheets and Strip, which appear in the 1971 Annual Book of ASTM Standards, Part 3.</td>
<td>Sheet and Strip</td>
<td>Hot Rolled</td>
<td>50</td>
<td>70</td>
<td>22</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>—</td>
<td>As Rolled Cut Lengths</td>
<td>70</td>
<td>22</td>
<td>1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>Hot Rolled Coils</td>
<td>65</td>
<td>22</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>Annealed or Normalized</td>
<td>65</td>
<td>22</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>Cold Rolled</td>
<td>65</td>
<td>22</td>
<td>1.44</td>
</tr>
</tbody>
</table>
### Supplementary Information on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

### ASTM Percent Designation FY ' FUt elongation

<table>
<thead>
<tr>
<th>SCOPE (after ASTM)</th>
<th>PRODUCT</th>
<th>GRADE</th>
<th>$F_{y}$ ksi (min)</th>
<th>$F_{u}$ ksi (min)</th>
<th>Percent elongation in 2 inches (min)</th>
<th>$F_{e}$ ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A607-85</strong></td>
<td></td>
<td>Class 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>This specification covers high-strength, low-alloy columbium, or vanadium hot-rolled sheet and strip, or cold-rolled sheet, or combinations thereof, in either cut lengths or coils, intended for applications where greater strength and savings in weight are important. The material is available as two classes. They are similar in strength level except that Class 2 offers improved weldability and more formability than Class 1. Atmospheric corrosion resistance of these steels is equivalent to plain carbon steels. With copper specified, the atmospheric corrosion resistance is twice that of plain carbon steel. Class 1 material was previously A607 without a class designation.</td>
<td>Sheet and Strip</td>
<td>45</td>
<td>45</td>
<td>60</td>
<td>Hot-Rolled 22-25 Cold-Rolled 22</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>50</td>
<td>65</td>
<td>Hot-Rolled 20-22 Cold-Rolled 20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>55</td>
<td>55</td>
<td>70</td>
<td>Hot-Rolled 18-20 Cold-Rolled 18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>60</td>
<td>75</td>
<td>Hot-Rolled 16-18 Cold-Rolled 16</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>65</td>
<td>80</td>
<td>Hot-Rolled 14-16 Cold-Rolled 15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>70</td>
<td>70</td>
<td>85</td>
<td>Hot-Rolled 12-14 Cold-Rolled 14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Class 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>45</td>
<td>55</td>
<td>Hot-Rolled 22-26 Cold-Rolled 26</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>50</td>
<td>60</td>
<td>Hot-Rolled 20-22 Cold-Rolled 20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>55</td>
<td>55</td>
<td>65</td>
<td>Hot-Rolled 18-20 Cold-Rolled 18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>60</td>
<td>70</td>
<td>Hot-Rolled 16-18 Cold-Rolled 16</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>65</td>
<td>65</td>
<td>75</td>
<td>Hot-Rolled 14-16 Cold-Rolled 15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>70</td>
<td>70</td>
<td>80</td>
<td>Hot-Rolled 12-14 Cold-Rolled 14</td>
</tr>
</tbody>
</table>

### A611-85

This specification covers cold-rolled carbon steel sheet, in cut lengths or coils. It includes five strength levels designated as Grade A with yield point 25 ksi (170 MPa) minimum; Grade B with 30 ksi (205 MPa) minimum; Grade C with 33 ksi (230 MPa) minimum; Grade D types 1 and 2 with 40 ksi (275 MPa) minimum; and Grade E with 50 ksi (350 MPa) minimum.

### A715-85

This specification covers high-strength low-alloy, hot-rolled steel sheet and strip having improved formability when compared with steels covered by Specifications A606 and A607. The product is furnished as either cut lengths or coils and is available in four strength levels, Grades 50, 60, 70, and 80 (corresponding to minimum yield point), in eight types (according to chemical composition). Not all grades are available in all types. The steel is killed, made to a fine grain practice, and includes microalloying elements such as columbium, titanium, vanadium, zirconium, etc. The product is intended for structural and miscellaneous applications where higher strength, savings in weight, improved formability, and weldability are important.

### A792-88a

This specification covers aluminum-zinc alloy-coated steel sheet in coils and cut lengths coated by the hot-dip process. The aluminum-zinc alloy composition by weight is nominally 55% aluminum, 1.5% silicon, and the balance zinc. The product is intended for applications requiring corrosion resistance or heat resistance or both. Aluminum-zinc alloy-coated sheet is available as Commercial Quality, Lock-Forming Quality, and Structural (Physical) Quality. The available grades of Structural Quality are shown in this Table.
SECTION 6—SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION

General
Those taking advantage of the economies in building construction afforded by cold-formed steel structural framing are warned to observe the procedures outlined herein to obtain quality construction.

Designing
Building design involving cold-formed steel structural framing for floors, roofs, load-bearing walls, or curtain walls should be performed by, or under the supervision of, registered professional structural engineers.

Detailing
Framing drawings should show size, thickness, type, and spacing of all structural members including bridging and bracing. Large-scale details should be included for all connections either welded or screwed. Details should show the method of anchorage of walls to the foundation.

Fabrication
Manufacturers of cold-formed steel structural members maintain in-house quality control programs. Assembly of components into walls, etc., may be done on the job by the "stick built" method or off the job by the "panelized" method utilizing assembly jigs. Assemblers must follow details shown on fabrication and/or erection drawings.

Erection
Erection should be performed by experienced mechanics who follow the plans and specifications under the supervision of an experienced foreman or superintendent.

Inspection of Construction
Periodic inspections during the construction phase should be made by a professional structural engineer who either is or represents the design engineer of record.

Sign Off
At the conclusion of the cold-formed steel construction process a final inspection should be made by the engineer of record who should certify that the cold-formed steel framing has been constructed in accordance with the plans and specifications and in accordance with all applicable building codes and regulatory requirements.
ILLUSTRATIVE EXAMPLES
BASED ON THE MARCH 16, 1991 EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part IV

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE

This document, Part IV of the LRFD Cold-Formed Steel Design Manual, contains examples intended to illustrate the application of various provisions of the Specification.

These Illustrative Examples should be used in conjunction with the other parts of the Design Manual, which includes Commentary (Part II), Supplementary Information (Part III), and Design Aids (Part V), in addition to the Specification (Part I).

As a general rule, section properties are computed to three significant figures, while dimensions are given to three decimal places. However, in some cases it was impractical to adhere strictly to this guideline.

The weight of the sections is calculated based on steel weighing 40.80 pounds per square foot per inch thickness.

Slight discrepancies should be expected between the calculated section properties given in the examples and the tabulated values given in Part V of the Manual which were calculated by computer.

For the design of compression members, results obtained by utilizing either the graphical or the analytical procedure as outlined in Part III of the Manual will differ somewhat. The reason for this is that in the graphical procedure, properties are computed assuming square corners for the section, while the analytical procedure is based on round corners (except for the torsional properties given by C_w, j and m which are based on square corners). In general, this will cause only small differences in the results.

The exception occurs when dealing with angle sections. The parameter x which is the distance between the centroid of the section and the centerline of the corner is sensitive to the type of corner utilized. This causes discrepancies in the order of ten percent between the two procedures.

The linear method outlined in Part III of the Manual is used for computing the properties of formed sections.

These Illustrative Examples were prepared at Cornell University by graduate students V. Sagan, M. Bou-Shahri, and T. Miller, except for the purlin examples which were prepared by graduate students at the University of Florida.

American Iron and Steel Institute
December 1991
TABLE OF CONTENTS

PART IV
ILLUSTRATIVE EXAMPLES
BASED ON THE
MARCH 16, 1991, EDITION OF THE LRFD
SPECIFICATION FOR THE DESIGN OF COLD-FORMED
STEEL STRUCTURAL MEMBERS

Preface .......................................................... 3

FLEXURAL MEMBERS
Example 1. Channel w/unstiffened flange ..................... 5
Example 2. Channel w/stiffened flange ....................... 11
Example 3. Channel w/stiffened flange and bracing ....... 18
Example 4. Z-Section w/stiffened flange ................... 24
Example 4A. Deep Z-Section w/stiffened flange .......... 30
Example 5. Hat Section ..................................... 40
Example 6. Hat Section w/intermediate stiffener ......... 49
Example 7. Hat Section w/inelastic reserve ............... 58
Example 8. Deck Section .................................... 63
Example 9. Tubular Section .................................. 85

COMPRESSION MEMBERS
Example 10. C-Section ....................................... 87
Example 11. C-Section w/holes ................................ 92
Example 12. C-Section w/wide flange ....................... 97
Example 13. Tubular Section—Square ......................... 102
Example 14. Tubular Section—Round ......................... 105

BEAM-COLUMN MEMBERS
Example 15. C-Section ....................................... 107
Example 16. C-Section—Wall Stud ......................... 124
Example 17. Tubular Section .................................. 133

TENSION MEMBERS & CONNECTIONS
Example 18. Flat Section w/bolded connections .......... 139
Example 19. Flat Section w/Arc spot welded connection .. 142
Example 20. Flat Section w/Arc seam welded connection .. 145
Example 21. Flat Section w/Lap fillet welded connection .. 150
Example 22. Flat Section w/groove welded connection .. 150

PURLINS
Example 23. C-Section Bracing under gravity loading ..... 152
Example 24. Z-Section Bracing under gravity loading ..... 154

CALCULATION OF SECTION PROPERTIES
Example 25. Wall Panel Section .............................. 156
Example 26. Build-up Section Connecting two channels .. 171
Example 27. Strength Increase from cold work of forming .. 173
Example 28. Flange Curling .................................. 175
Example 29. Slear Lag ......................................... 180
Example 30. Flat Section w/groove welded connection in butt joint .. 185
Example 31. I-Section ......................................... 187
Example 32. Channel Section Braced ......................... 192
EXAMPLE NO. 1
CHANNEL SECTION

Given:
1. Steel: \( F_y = 50 \) ksi.
2. Section: \( 6 \times 1.625 \times 0.060 \) channel with unstiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio \( D/L = 1/5 \) and \( 1.2D + 1.6L \) governs the design.

Required:
1. Design flexural strength, \( \phi_b M_n \), based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:
1. Calculation of the design flexural strength, \( \phi_b M_n \):
   Properties of 90° corners:
   \[
   r = R + t/2 = 3/32 + 0.060/2 = 0.124 \text{ in.}
   \]
Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \) in.

Distance of c.g. from center of radius,
\( c = 0.637r = 0.637 \times 0.124 = 0.079 \) in.

Computation of \( I'_x \):

For the first approximation, assume a compression stress of \( f = F_y = 50 \) ksi in the top fibers of the section and that the web is fully effective.

Compression flange: \( k = 0.43 \) (unstiffened compression element) (Section B3.1)

\[ w/t = 1.471/0.060 = 24.52 < 60 \text{ OK (Section B1.1-(a)-(3))} \]

\[ \lambda = \frac{1.052\sqrt{k}}{w/t}\sqrt{\frac{f}{E}} = \frac{1.052\sqrt{0.43}}{24.52}\sqrt{50/29500} = 1.619 > 0.673 \]  

\( \rho = \frac{[1-(0.22/\lambda)]/\lambda}{[1-(0.22/1.619)]/1.619} = 0.534 \)

\( b = \rho w \)  
\[ = 0.534 \times 1.471 \]
\[ = 0.786 \text{ in.} \]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) Effective Length (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( Ly ) (in.(^2))</th>
<th>( Ly^2 ) (in.(^3))</th>
<th>( I'_1 ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>5.692</td>
<td>3.000</td>
<td>17.076</td>
<td>51.228</td>
<td>15.368</td>
</tr>
<tr>
<td>Upper Corner</td>
<td>0.195</td>
<td>0.075</td>
<td>0.015</td>
<td>0.001</td>
<td>—</td>
</tr>
<tr>
<td>Lower Corner</td>
<td>0.195</td>
<td>5.925</td>
<td>1.155</td>
<td>6.846</td>
<td>—</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>0.786</td>
<td>0.030</td>
<td>0.024</td>
<td>0.001</td>
<td>—</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>1.471</td>
<td>5.970</td>
<td>8.782</td>
<td>52.428</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>8.339</td>
<td>27.052</td>
<td>110.504</td>
<td>15.368</td>
<td>—</td>
</tr>
</tbody>
</table>

Distance from top fiber to \( x \)-axis is
\[ y_{cg} = 27.052/8.339 = 3.244 \text{ in.} \]

Since the distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yield is in compression).

To check if web is fully effective (Section B2.3):

\[ f_I = [(3.244 - 0.154)/3.244] \times 50 = 47.63 \text{ ksi(compression)} \]
\[ f_2 = \frac{(2.756 - 0.154)}{3.244} \times 50 = -40.10 \text{ksi(tension)} \]
\[ \psi = \frac{f_2}{f_1} = \frac{-40.10}{47.63} = -0.842 \]
\[ k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \]  
\[ = 4 + 2(1 - (-0.842))^3 + 2(1 - (-0.842)) \]
\[ = 20.184 \]
\[ h = w = 5.692 \text{ in.}, \frac{h}{t} = \frac{w}{t} = \frac{5.692}{0.060} = 94.87 \]
\[ h/t = 94.87 < 200 \text{ OK (Section B1.2-(a))} \]
\[ \lambda = (1.052/\sqrt{20.184})(94.87) \sqrt{47.63/29500} = 0.893 > 0.673 \]
\[ \rho = \frac{[1-(0.22/0.893)]}{0.893} = 0.844 \]
\[ b_e = 0.844 \times 5.692 = 4.804 \text{ in.} \]
\[ b_2 = \frac{b_e}{2} \]  
\[ = 4.804/2 = 2.402 \text{ in.} \]
\[ b_1 = \frac{b_e}{(3-\psi)} \]  
\[ = 4.804/\{3 - (-0.842)\} = 1.250 \text{ in.} \]
\[ b_1 + b_2 = 1.250 + 2.402 = 3.652 \text{ in.} \]

Compression portion of the web calculated on the basis of the effective section  
\[ y_{eq} = 3.244 - 0.154 = 3.090 \text{ in.} \]  
\[ (\text{Eq. B2.3-4}) \]
\[ (\text{Eq. B2.3-2}) \]
\[ (\text{Eq. B2.3-1}) \]

Since \( b_1 + b_2 = 3.652 \text{ in.} > 3.090 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 3.090 in. This verifies the assumption that the web is fully effective.

\[ I_x' = Ly^2 + I_y' = Ly_{cg}^2 \]
\[ = 110.504 + 15.368 - 8.339(3.244)^2 \]
\[ = 38.116 \text{ in.}^3 \]

Actual \( I_x = I_{xt} \)
\[ = 38.116 \times 0.060 \]
\[ = 2.287 \text{ in.}^4 \]

\[ S_e = \frac{I_y}{y_{cg}} \]
\[ = 2.287/3.244 \]
\[ = 0.705 \text{ in.}^3 \]

\[ M_n = S_eF_y \]  
\[ = 0.705 \times 50 \]
\[ = 35.25 \text{ kip-in.} \]
\[ \phi_b = 0.90 \]  
\[ (\text{Section C3.1.1}) \]

\[ \phi_b M_n = 0.90 \times 35.25 = 31.73 \text{ kip-in. (positive bending)} \]
2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment $M_s$:

$$
\phi_0 M_n := 1.2M_{DL} + 1.6M_{LL}
= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}
= [1.2(1/5) + 1.6]M_{LL} = 1.84 M_{LL}
$$

$$
M_{LL} = \phi_0 M_n / 1.84 = 31.73 / 1.84 = 17.24 \text{ kip-in.}
$$

$$
M_s = M_{DL} + M_{LL}
= (1/5+1)M_{LL}
= 1.2(17.24) = 20.69 \text{ kip-in.}
$$

where

$M_{DL}$ = Moment determined on the basis of nominal dead load

$M_{LL}$ = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress $f$ under this service moment $M_s$. Knowing $f$, one proceeds as usual to obtain $S_e$ and checks to see if $(f \times S_e)$ is equal to $M_s$ as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of $f = F_y/2 = 25 \text{ ksi}$ in the top fibers of the section and that the web is fully effective.

Compression flange:

$$
\lambda = (1.052/\sqrt{0.43})(24.52)/29500 = 1.145 > 0.673 \quad (\text{Eq. B2.1-4})
$$

$$
\rho = [1 - (0.22/1.145)]/1.145 = -.706 \quad (\text{Eq. B2.1-3})
$$

$$
b_d = \rho w = 0.706 \times 1.471 = 1.039 \text{ in.} \quad (\text{Eq. B2.1-6})
$$

Effective section properties about x-axis:

$$
L = 8.339 - 0.786 + 1.039 = 8.592 \text{ in.}
$$

$$
L_y = 27.052 - 0.024 + 1.039(0.030) = 27.059 \text{ in.}^2
$$

$$
L_y^2 = 110.504 - 0.001 + 1.039(0.030)^2 = 110.504 \text{ in.}^3
$$

$$
i' = 15.368 \text{ in.}^3
$$

$$
y_{cg} = 27.059 / 8.592 = 3.149 \text{ in.} \text{ greater than one half beam depth. Thus top compression fiber controls in determination of } S_e.
$$

To check if web is fully effective (Section B2.3-(a),(b)):

$$
f_1 = [(3.149 - 0.154)/3.149] \times 25 = 23.78 \text{ ksi}
$$

$$
f_2 = -[(2.851 - 0.154)/3.149] \times 25 = -21.41 \text{ ksi}
$$

$$
\psi = -21.41 / 23.78 = -0.900
$$
\[ k = 4 + 2[1 - (-0.900)]^3 + 2[1 - (-0.900)] = 21.518 \]

\[ \lambda = (1.052/21.518)(94.87)/23.78/29500 = 0.611 < 0.673 \]

\[ b_c = w = 5.692 \text{ in.} \]  
\[ b_2 = 5.692/2 = 2.846 \text{ in.} \]  
\[ b_1 = 5.692/[3 - (-0.900)] = 1.459 \text{ in.} \]  

Compress portion of the web calculated on the basis of the effective section = 3.149 - 0.154 = 2.995 in.

Since \( b_1 + b_2 = 4.305 \text{ in.} > 2.995 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 2.995 in.. This verifies the assumption that the web is fully effective.

\[ I'_x = 110.504 + 15.368 - 8.592(3.149)^2 = 40.672 \text{ in.}^3 \]

Actual \( I_x = 40.672 \times 0.060 = 2.440 \text{ in.}^4 \)

\[ S_c = 2.440/3.149 = 0.775 \text{ in.}^3 \]

\[ M = f \times S_c = 25 \times 0.775 = 19.38 \text{ kip-in.} < M_s = 20.69 \text{ kip-in.} \]

Need to do another iteration and also to increase \( f \).

b. After several iterations, assume \( f = 27.01 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:

\[ \lambda = (1.052/\sqrt{0.43})(24.52) \sqrt{27.01}/29500 = 1.190 > 0.673 \]

\[ \rho = [1 - (0.22/1.190)]/1.190 = .685 \]

\[ b_d = .685 \times 1.471 = 1.008 \text{ in.} \]

Effective section properties about x-axis:

\[ L = 8.339 - 0.786 + 1.008 = 8.561 \text{ in.} \]

\[ L_y = 27.052 - 0.024 + 1.008 \times 0.030 = 27.058 \text{ in.}^2 \]

\[ L_y^2 = 110.504 - 0.001 + 1.008(0.030)^2 = 110.504 \text{ in.}^3 \]

\[ Y'_1 = 15.368 \text{ in.}^3 \]

\[ y_{cg} = 27.058/8.561 = 3.161 \text{ in.} \] greater than one half beam depth. Thus top compression fiber controls in determination of \( S_c \).

To check if web is fully effective:

\[ f_1 = [3.161 - 0.154]/3.161 \times 27.01 = 25.69 \text{ ksi} \]
\[ f_2 = -\left(\frac{(2.839 - 0.154)}{3.161}\right) \times 27.01 = -22.94 \text{ ksi} \]
\[ \psi = \frac{-22.94}{25.69} = 0.893 \]
\[ k = 4 + 2[1 - (-0.893)] \times 3 + 2[1 - (-0.893)] = 21.353 \]
\[ \lambda = (1.052/\sqrt{21.353})(94.87) \times \sqrt{25.69/29500} = 0.637 < 0.673 \]
\[ b_c = 5.692 \text{ in.} \]
\[ b_2 = 5.692/2 = 2.846 \text{ in.} \]
\[ b_1 = 5.692/[3 - (-0.893)] = 1.462 \text{ in.} \]

Compression portion of the web calculated on the basis of the effective section = 3.161 - 0.154 = 3.007 in.

Since \( b_1 + b_2 = 4.308 \text{ in.} > 3.007 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 3.004 in.. This verifies the assumption that the web is fully effective.

\[ I'_x = 110.504 + 15.368 - 8.561(3.161)^2 \]
\[ = 40.331 \text{ in.}^3 \]

Actual \( I_x = 40.331 \times 0.060 \]
\[ = 2.420 \text{ in.}^4 \]
\[ S_e = 2.420/3.161 = 0.766 \text{ in.}^3 \]
\[ M = \sigma \times S_e = 27.01 \times 0.766 \]
\[ = 20.68 \text{ kip-in.} \quad \text{M}_s \text{ OK} \]

Thus \( I_x = 2.420 \text{ in.}^4 \) using procedure I for deflection determination.
EXAMPLE NO. 2
C-SECTION

Given:
1. Steel: \( F_y = 50 \) ksi.
2. Section: 6 x 1.625 x 0.060 channel with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio \( D/L = 1/5 \) and \( 1.2D + 1.6L \) governs the design.

Required:
1. Design flexural strength, \( \phi_b M_n \), based on initiation of yielding.
2. Effective moment of inertia based on procedure 1 for deflection determination at the service moment.

Solution:
1. Calculation of the design flexural strength, \( \phi_b M_n \):
   Properties of 90° corners:
\[ r = R + \frac{t}{2} = 3/32 + 0.060/2 = 0.124 \text{ in.} \]
Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \text{ in.} \)
Distance of c.g. from center of radius, 
\( c = 0.637r = 0.637 \times 0.124 = 0.079 \text{ in.} \)

Computation of \( I_x \):
For the first approximation, assume a compression stress of \( f = F_y = 50 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:
\[
\begin{align*}
  w &= 1.317 \text{ in.} \\
  \frac{w}{t} &= 1.317/0.060 = 21.95 \\
  S &= 1.28 \sqrt{\frac{E}{f}} \quad \text{(Eq. B4-1)} \\
  &= 1.28 \sqrt{29500/50} = 31.09 \\
  \frac{S}{3} &= 10.36 < \left(\frac{w}{t}\right) = 21.95 < S = 31.09 \\
  I_a &= 399r^4 \left[\left(\frac{w}{t}/S\right)^{-0.33}\right]^3 \quad \text{(Eq. B4.2-6)} \\
  &= 399(0.060)^4 \left[\left(21.95/31.09\right)^{-0.33}\right]^3 \\
  &= 0.000275 \text{ in.}^4 \\
  D &= 0.450 \text{ in.} \\
  d &= 0.296 \text{ in., } \frac{d}{t} = 0.296/0.060 = 4.93 < 14 \text{ OK (Section B4 of the Commentary)} \\
  I_s &= \frac{d^3}{12} \quad \text{(Eq. B4-2)} \\
  I_s &= (0.296)^3(0.060)/12 = 0.000130 \text{ in.}^4 \\
  \frac{D}{w} &= 0.450/1.317 = 0.342, 0.25 < \frac{D}{w} = 0.342 < 0.80 \quad \text{(Eq. B4-2)} \\
  k &= [4.82-5(\frac{D}{w})](\frac{I_s}{I_a})^{n/2}+0.43 \leq 5.25-5(\frac{D}{w}) \quad \text{(Eq. B4.2-9)} \\
  n &= 1/2 \\
  [4.82-5(0.342)](0.000130/0.000275)^{1/2} + 0.43 = 2.568 \\
  5.25-5(0.342) = 3.540 > 2.568 \\
  k &= 2.568 \\
\end{align*}
\]
Since \( I_s < I_a \), the stiffener is considered a simple lip.
\[
\begin{align*}
  \frac{w}{t} &= 21.95 < 60 \text{ OK (Section B1.1-(a)-(1))} \\
  \lambda &= (1.052/\sqrt{k})(\frac{w}{t}) \sqrt{\frac{E}{f}} \quad \text{(Eq. B2.1-4)} \\
  &= (1.052/\sqrt{2.568})(21.95) \sqrt{50/29500} = 0.59 < 0.673 \\
  b &= w \quad \text{(Eq. B2.1-1)} \\
  &= 1.317 \text{ in. (i.e. compression flange fully effective)} \\
\end{align*}
\]
Compression (upper) stiffener:
\[
\begin{align*}
  k &= 0.43 \text{ (unstiffened compression element)} \\
  \frac{d}{t} &= 4.93 \\
\end{align*}
\]
Also conservatively assume \( f = 50 \text{ ksi} \) as for top compression fiber.

\[
\lambda = \left( \frac{1.052}{\sqrt{0.43}} \right) (4.93) \sqrt{50/29500} = 0.32 < 0.673
\]

therefore,

\[
d_s' = d = 0.296 \text{ in.}
\]

\[
d_s = d_s' I_{s/a} \leq d_s'
\]  
(Eq. B4.2-11)

\[
= 0.296\left(0.000130/0.000275\right)
\]

\[
= 0.140 \text{ in.} < 0.296 \text{ in.}
\]

\[
d_s = 0.140 \text{ in.} \quad \text{(i.e. compression stiffener is not fully effective)}
\]

Effective section properties about \( x \)-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) Effective Length (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L y ) (in.^2)</th>
<th>( L y^2 ) (in.^3)</th>
<th>( \gamma' ) About Own Axis (in.^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>5.692</td>
<td>3.000</td>
<td>17.076</td>
<td>51.228</td>
<td>15.368</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>2\times0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
<td>-</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>2\times0.195 = 0.390</td>
<td>5.925</td>
<td>2.311</td>
<td>13.691</td>
<td>-</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>1.317</td>
<td>0.030</td>
<td>0.040</td>
<td>0.001</td>
<td>-</td>
</tr>
<tr>
<td>Upper Stiffener</td>
<td>0.140</td>
<td>0.224</td>
<td>0.031</td>
<td>0.007</td>
<td>-</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>1.317</td>
<td>5.970</td>
<td>7.862</td>
<td>46.939</td>
<td>-</td>
</tr>
<tr>
<td>Lower Stiffener</td>
<td>0.296</td>
<td>5.698</td>
<td>1.687</td>
<td>9.610</td>
<td>0.002</td>
</tr>
<tr>
<td>Sum</td>
<td>9.542</td>
<td>29.036</td>
<td>121.478</td>
<td>15.370</td>
<td>-</td>
</tr>
</tbody>
</table>

Distance from top fiber to \( x \)-axis is

\[
y_{eq} = 29.036/9.542 = 3.043 \text{ in.}
\]

Since the distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yield is in compression).

To check if web is fully effective (Section B2.3):

\[
f_1 = [(3.043-0.154)/3.043] \times 50 = 47.47 \text{ ksi (compression)}
\]

\[
f_2 = -[(2.957-0.154)/3.043] \times 50 = -46.06 \text{ ksi (tension)}
\]

\[
\psi = f_2/f_1 = -46.06/47.47 = -0.970
\]

\[
k = 4 + 2(1 - \psi)^3 + 2(1 - \psi)
\]

\[
= 4 + 2 [1 - (-0.970)]^3 + 2 [1 - (-0.970)]
\]

\[
= 23.231
\]

\[
h = w = 5.692 \text{ in.}, \quad h/t = w/t = 5.692/0.060 = 94.87
\]

\[
h/t = 94.87 < 200 \text{ OK (Section B1.2-(a))}
\]
\[ \lambda = \frac{(1.052/\sqrt{23.231})(94.87)\sqrt{47.47/29500}}{0.831} > 0.673 \]
\[ \rho = \frac{[1 - (0.22/\lambda)]/\lambda}{[1 - (0.22/0.831)]/0.831} = 0.885 \quad \text{(Eq. B2.1-3)} \]
\[ b_e = \rho w \quad \text{(Eq. B2.1-2)} \]
\[ b_2 = \frac{b_e}{2} \quad \text{(Eq. B2.3-2)} \]
\[ b_1 = \frac{b_e(3 - \psi)}{5.037/\left[3-(0.970)\right]} = 1.269 \text{ in.} \]
\[ b_1 + b_2 = 1.269 + 2.519 = 3.788 \text{ in.} \]
Compression portion of the web calculated on the basis of the effective section = \( y_{cg} - 0.154 = 3.043 - 0.154 = 2.889 \text{ in.} \)

Since \( b_1 + b_2 = 3.788 \text{ in.} > 2.889 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 2.889 in. This verifies the assumption that the web is fully effective.

\[ \Gamma_x = Ly^2 + \Gamma_{1x} - Ly_{cg}^2 \]
\[ = 121.478 + 15.370 - 9.542(3.043)^2 \]
\[ = 48.491 \text{ in.}^3 \]

Actual \( I_x = \Gamma_{xt} \)
\[ = 48.491 \times 0.060 \]
\[ = 2.909 \text{ in.}^4 \]

\[ S_e = I_x/y_{cg} = 2.909/3.043 \]
\[ = 0.956 \text{ in.}^3 \]

\[ M_n = S_eF_y \quad \text{(Eq. C3.1.1-1)} \]
\[ = 0.956 \times 50 \]
\[ = 47.80 \text{ kip-in.} \]

\[ \phi_b = 0.95 \quad \text{(Section C3.1.1)} \]

\[ \phi_b M_n = 0.95 \times 47.80 = 45.41 \text{ kip-in.} \]

2. Calculation of the effective moment of inertia based on procedure 1 for deflection determination at the service moment \( M_s \):
\[ \phi_b M_n = 1.2M_{DL} + 1.6M_{LL} \]
\[ = [1.2(M_{DL}/M_{LL}) + 1.6]M_{LL} \]
\[ = [1.2(1/5) + 1.6]M_{LL} \]
\[ = 1.84M_{LL} \]

\[ M_{LL} = \phi_b M_n / 1.84 = 45.41 / 1.84 = 24.68 \text{ kip-in.} \]

\[ M_s = M_{DL} + M_{LL} \]
\[ = (1/5 + 1)M_{LL} \]
= 1.2(24.68) = 29.62 kip-in.

where

\[ M_{DL} = \text{Moment determined on the basis of nominal dead load} \]

\[ M_{LL} = \text{Moment determined on the basis of nominal live load} \]

The procedure is iterative: one assumes the actual compressive stress \( f \) under this service moment \( M_s \). Knowing \( f \), one proceeds as usual to obtain \( S_e \) and checks to see if \( f \times S_e \) is equal to \( M_s \) as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of \( f = F_y/2 = 25 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:

\[ S = 1.28 \sqrt{29500/25} = 43.97 \]  \hspace{1cm} (Eq. B4-1)

\[ S/3 = 14.66 < (w/t) = 21.95 < S = 43.97 \]

\[ I_a = 399(0.060)^4[(21.95/43.97)-0.33]^3 \]

\[ = 0.000025 \text{ in.}^4 \]

\[ I_a/I_a = 0.000130/0.000025 = 5.20 \]

\[ k = [4.82-5(0.342)](5.20)^{1/2}+0.43 = 7.522 > 3.540 \]  \hspace{1cm} (Eq. B4.2-9)

\[ k = 3.540 \]

\[ \lambda = (1.052/\sqrt{3.540})(21.95) \sqrt{25/29500} = 0.357 < 0.673 \]  \hspace{1cm} (Eq. B2.1-4)

\[ b_d = 1.317 \text{ in. (i.e. compression flange fully effective)} \]

Compression (upper) stiffener:

Again assume conservatively \( f = 25 \text{ ksi} \) as in top compression fiber

\[ \lambda = (1.052/\sqrt{0.43})(4.93) \sqrt{25/29500} = 0.230 < 0.673 \]

therefore \( d'_s = 0.296 \text{ in.} \)

Since \( I_y/I_a = 5.20 > 1.0 \), it follows that \( d_s = d'_s = 0.296 \text{ in. (i.e. compression stiffener fully effective)} \).

Thus, one concludes that the section is fully effective.

\[ y_{cg} = 6/2 = 3.000 \text{ in. (from symmetry)} \]
Full section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L (in.)</th>
<th>$y$ Distance from Centerline of Section (in.)</th>
<th>$Ly^2$ (in.$^3$)</th>
<th>$I'_{1}$ About Own Axis (in.$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>5.692</td>
<td>—</td>
<td>—</td>
<td>15.368</td>
</tr>
<tr>
<td>Stiffeners</td>
<td>2 x 0.296 = 0.592</td>
<td>2.698</td>
<td>4.309</td>
<td>0.004</td>
</tr>
<tr>
<td>Corners</td>
<td>4 x 0.195 = 0.780</td>
<td>2.925</td>
<td>6.673</td>
<td>—</td>
</tr>
<tr>
<td>Flanges</td>
<td>2 x 1.317 = 2.634</td>
<td>2.970</td>
<td>23.234</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td>34.216</td>
<td>15.372</td>
<td></td>
</tr>
</tbody>
</table>

Since section is singly symmetric about x-axis and fully effective, top compression fiber (and also bottom tension fiber) may be used in computing $S_c$.

To check if web is fully effective:

\[
f_1 = \frac{[(3.000-0.154)/3.000] \times 25}{23.72 \text{ ksi (compression)}}
\]

\[
f_2 = -23.72 \text{ ksi (tension)}
\]

\[
\psi = \frac{23.72}{23.72} = -1.000
\]

\[
k = 4 + 2 \left[\frac{1}{2}ight] = 24.000
\]

\[
\lambda = \left(1.052/\sqrt{24}\right)(94.87) \frac{23.72}{29500} = 0.578 < 0.673
\]

\[
b_c = w = 5.692 \text{ in.}
\]

\[
b_2 = \frac{5.692}{2} = 2.846 \text{ in.}
\]

\[
b_1 = \frac{5.692}{3} = 1.423 \text{ in.}
\]

\[
b_1 + b_2 = 4.269 \text{ in.}
\]

Compression portion of the web = 3.000 - 0.154 = 2.846 in.

Since $b_1 + b_2 = 4.269 \text{ in.} > 2.846 \text{ in.}$, $b_1 + b_2$ shall be taken as 2.846 in. This verifies the assumption that the web is fully effective.

\[
I'_{1x} = 34.216 + 15.372 = 49.588 \text{ in.}^2
\]

Actual $I_x = 49.588 \times 0.060 = 2.975 \text{ in.}^4$

\[
S_c = 2.975/3.000 = 0.992 \text{ in.}^3
\]

\[
M = f \times S_c = 25 \times 0.992
\]

\[
= 24.80 \text{ kip-in.} < M_x = 29.62 \text{ kip-in.}
\]

Need to do another iteration and also to increase $f$.

b. After several iterations, assume a compression stress of $f = 29.86 \text{ ksi}$ in the top fibers of the section and that the web is fully effective.
Compression flange:

\[ S = 1.28 \cdot \sqrt{29500/29.86} = 40.23 \]
\[ S/3 = 13.41 < (w/t) = 21.95 < S = 40.23 \]
\[ I_a = 399(0.060)^4 \left[ (21.95/40.23) - 0.33 \right]^3 \]
\[ = 0.000052 \text{ in.}^4 \]
\[ I_y/I_a = 0.000130/0.000052 = 2.50 \]
\[ k = [4.82 - 5(0.342)](2.50)^{1/2} + 0.43 = 5.347 > 3.540 \]
\[ k = 3.540 \]
\[ \lambda = (1.052/\sqrt[4]{3.540})(21.95) \sqrt{29.86/29500} = 0.390 < 0.673 \]
\[ b_d = 1.317 \text{ in.} \text{ (i.e. compression flange fully effective)} \]

Compression (upper) stiffener:

\( f \) conservatively taken as for top compression fiber
\[ \lambda = (1.052/\sqrt[4]{0.43})(4.93) \sqrt{29.86/29500} = 0.252 < 0.673 \]
\[ d'_s = 0.296 \text{ in.} \]

Since \( I_y/I_a = 2.50 > 1.0 \), it follows that \( D_s = d'_s = 0.296 \text{ in.} \text{ (i.e. compression stiffener fully effective)} \).

Thus, the section is fully effective.
\( y_{cg} = 6/2 = 3.000 \text{ in.} \text{ (from symmetry)} \)

Full section properties are the same as were found in the first iteration. Thus, as before, top compression fiber may be used in computing \( S_e \).

To check if web is fully effective:
\[ f_1 = [(3.000 - 0.154)/3.000] \times 29.86 = 28.33 \text{ ksi (compression)} \]
\[ f_2 = -28.33 \text{ ksi (tension)} \]
\[ \psi = -28.33/28.33 = -1.000 \]
\[ k = 24.000 \]
\[ \lambda = (1.052/\sqrt[4]{24})(94.87) \sqrt{28.33/29500} = 0.631 < 0.673 \]
\[ b_e = w = 5.692 \text{ in.} \]

Hence, as in first iteration, \( b_1 + b_2 = 2.846 \text{ in.} \) and thus the web is fully effective as assumed.
\[ I_x = 2.975 \text{ in.}^4 \]
\[ S_e = 0.992 \text{ in.}^3 \]
\[ M = f \times S_e = 29.86 \times 0.992 \]
\[ = 29.62 \text{ kip-in.} = M_s \text{ OK} \]

Thus \( I_x = 2.975 \text{ in.}^4 \) using procedure I for deflection determination.
EXAMPLE NO. 3.
BRACED C-SECTION

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ channel with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
1. Design flexural strength, $\phi_bM_{in}$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:
1. Calculation of the design flexural strength, $\phi_bM_{in}$:
   Properties of $90^\circ$ corners:
\( r = R + \frac{t}{2} = \frac{3}{32} + \frac{0.060}{2} = 0.124 \) in.

Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \) in. Distance of c.g. from center of radius, \( c = 0.637r = 0.637 \times 0.124 = 0.079 \) in.

Computation of \( I_z \):
For the first approximation, assume a compression stress of \( f = F_y = 50 \) ksi in the top fibers of the section and that the web is fully effective.

Compression flange:
\[
\begin{align*}
    w &= 1.317 \text{ in.} \\
    \frac{w}{t} &= 1.317/0.060 = 21.95 \\
    S &= 1.28 \sqrt{\frac{E}{f}} \\
    &= 1.28 \sqrt{\frac{29500}{50}} = 31.09 \\
    \frac{S}{3} &= 10.36 < \frac{(w/t)}{S} = 21.95 < S = 31.09 \\
    I_x &= 399t^4 \left[ \frac{((w/t)/S)-0.33}{3} \right]^3 \\
    &= 399(0.060)^4 \left[ (21.95/31.09)-0.33 \right]^3 \\
    &= 0.000275 \text{ in.}^4 \\
    D &= 0.600 \text{ in.} \\
    d &= 0.446 \text{ in.} \quad \frac{d}{t} = 0.446/0.060 = 7.43 < 14 \text{ OK} \\
    (\text{Section B4 of the Commentary}) \\
    I_s &= d^3t/12 \\
    &= (0.446)^3(0.060)/12 = 0.000444 \text{ in.}^4 \\
    \frac{D}{w} &= 0.600/1.317 = 0.456, 0.25 < \frac{D}{w} = 0.456 < 0.80 \quad (\text{Fig. B4-2}) \\
    k &= \left[ 4.82-5(\frac{D}{w}) \right] \left( \frac{I_x}{I_s} \right)^n + 0.43 \leq 5.25-5(\frac{D}{w}) \quad (\text{Eq. B4.2-9}) \\
    n &= 1/2 \\
    &= \left[ 4.82-5(0.456) \right] \left( \frac{0.000444}{0.000275} \right)^{1/2} + 0.43 = 3.657 \\
    &= 5.25-5(0.456) = 2.970 \\
    k &= 2.970 \\
\end{align*}
\]

Since \( I_s > I_a \) and \( \frac{D}{w} < 0.8 \), the stiffener is not considered as a simple lip.

\[
\begin{align*}
    \frac{w}{t} &= 21.95 < 90 \text{ OK} \quad (\text{Section B1.1-(a)-(1)}) \\
    \lambda &= \frac{(1.052/\sqrt{k})(w/t)}{\sqrt{fE}} \quad (\text{Eq. B2.1-4}) \\
    &= (1.052/\sqrt{2.970})(21.95) \sqrt{50/29500} = 0.55 < 0.673 \\
    b &= w \quad (\text{Eq. B2.1-1}) \\
    &= 1.317 \text{ in. (i.e. compression flange fully effective)} \\
\end{align*}
\]

Compression (upper) stiffener:
\[
\begin{align*}
    k &= 0.43 \text{ (unstiffened compression element)} \\
    \frac{d}{t} &= 7.43 \\
\end{align*}
\]
f conservatively taken equal to 50 ksi as in top compression fiber

\[ \lambda = \frac{1.052}{\sqrt{0.43}} \frac{(7.43) \sqrt{50/29500}}{50} = 0.489 < 0.673 \]

dependent on,

\[ d' = d = 0.446 \text{ in.} \]

\[ d_s = d'(\frac{L}{h}) \leq d' \]

\[ = 0.446(\frac{0.000444}{0.000275}) = 0.720 \text{ in.} > 0.446 \text{ in.} \]

\[ d_s = 0.446 \text{ in.} \text{ (i.e. compression stiffener is fully effective)} \]

Thus, one concludes that the section is fully effective.

\[ y_c = \frac{6}{2} = 3.000 \text{ in. (from symmetry)} \]

Full section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L (in.)</th>
<th>Distance from Centerline of Section (in.)</th>
<th>Ly^2 (in.(^3))</th>
<th>1' About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>5.692</td>
<td>2.623</td>
<td>6.137</td>
<td>15.368</td>
</tr>
<tr>
<td>Stiffeners</td>
<td>2 x 0.446 = 0.892</td>
<td>2.925</td>
<td>6.673</td>
<td>-</td>
</tr>
<tr>
<td>Corners</td>
<td>4 x 0.195 = 0.780</td>
<td>6.673</td>
<td>36.044</td>
<td>15.383</td>
</tr>
<tr>
<td>Flanges</td>
<td>2 x 1.317 - 2.634</td>
<td>2.970</td>
<td>23.234</td>
<td>-</td>
</tr>
</tbody>
</table>

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from symmetry).

To check if web is fully effective: (Section B2.3)

\[ f_1 = \frac{(3.000-0.154)}{3.000} \times 50 = 47.43 \text{ ksi (compression)} \]

\[ f_2 = -47.43 \text{ ksi (tension)} \]

\[ \psi = f_2/f_1 = -47.43/47.43 = -1.000 \]

\[ k = 4+2(1-\psi)^3 + 2(1-\psi) \]

\[ = 24.000 \]

\[ h = w = 5.692 \text{ in.}, h/t = w/t = \frac{5.692/0.060}{50} = 94.87 \]

\[ h/t = 94.87 < 200 \text{ OK (Section B1.2-(a))} \]

\[ \lambda = \frac{1.052}{\sqrt{24}}(94.87) \sqrt{47.43/29500} = 0.817 > 0.673 \]
\[ \rho = \frac{[1-(0.22/\lambda)]/\lambda}{\lambda} \quad \text{(Eq.B2.1-3)} \]
\[ \rho w = \frac{1-(0.22/0.817)}{0.817} = 0.894 \]
\[ b_e = \rho w \quad \text{(Eq. B2.1-2)} \]
\[ b_2 = \frac{b_e}{2} \quad \text{(Eq. B2.3-2)} \]
\[ b_1 = \frac{b_e}{(3-\psi)} \quad \text{(Eq. B2.3-1)} \]
\[ b_1 + b_2 = 1.272 + 2.545 = 3.817 \text{ in.} \]

Compression portion of the web = \( y_{cg} - 0.154 \)
\[ = 3.000 - 0.154 \]
\[ = 2.846 \text{ in.} \]

Since \( b_1 + b_2 = 3.817 \text{ in.} > 2.846 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 2.846 in.. This verifies the assumption that the web is fully effective.

\[ I'_x = L y^2 + I'_t \]
\[ = 36.044 + 15.383 \]
\[ = 51.427 \text{ in.}^3 \]

Actual \( I_x = I'_x + t \)
\[ = 51.427 + 0.060 \]
\[ = 51.486 \text{ in.}^3 \]

\[ S_e = \frac{I_x}{y_{cg}} \]
\[ = 3.086/3.000 \]
\[ = 1.029 \text{ in.}^3 \]

\[ M_n = S_e F_y \quad \text{(Eq. C3.1.1-1)} \]
\[ = 1.029 \times 50 \]
\[ = 51.45 \text{ kip-in.} \]
\[ \phi_b = 0.95 \]
\[ \phi_b M_n = 0.95 \times 51.45 = 48.88 \text{ kip-in.} \]

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment \( M_s \):

\[ \phi_b M_n = 1.2 M_{DL} + 1.6 M_{LL} \]
\[ = [1.2(M_{DL}/M_{LL})+1.6]M_{LL} \]
\[ = [1.2(1/5)+1.6]M_{LL} \]
\[ = 1.84 M_{LL} \]

\[ M_{LL} = \phi_b M_s/1.84 = 48.88/1.84 = 26.57 \text{ kip-in.} \]

\[ M_s = M_{DL} + M_{LL} \]
\[ = (1/5+1)M_{LL} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ M_{DL} = 1.2(26.57) = 31.88 \text{ kip-in.} \]

where

- \( M_{DL} \): Moment determined on the basis of nominal dead load
- \( M_{LL} \): Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress \( f \) under this service moment \( M_s \). Knowing \( f \), one proceeds as usual to obtain \( S_e \) and checks to see if \( (f \times S_e) \) is equal to \( M_s \) as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of \( f = 30.98 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

**Compression flange:**

\[
S = 1.28 \sqrt{29500/30.98} = 39.50 \quad \text{(Eq B4-1)}
\]

\[
S/3 = 13.17 < w/t = 21.95 < S = 39.50
\]

\[
I_a = 399(0.060)^4 [(21.95/39.50)-0.33]^3
= 0.000059 \text{ in.}^4
\]

\[
I_w/I_a = 0.000444/0.000059 = 7.525
\]

\[
k = 2.970
\]

\[
\lambda = (1.052/\sqrt{2.970})(21.95) \sqrt{30.98/29500} = 0.434 < 0.673 \quad \text{(Eq. B2.1-4)}
\]

\[
b_d = 1.317 \text{ in. (i.e. compression flange fully effective)}
\]

**Compression (upper) stiffener:**

\[ f \] conservatively taken equal to \( 30.98 \text{ ksi} \) as in top compression fiber

\[
\lambda = (1.052/\sqrt{0.43})(7.43) \sqrt{30.98/29500} = 0.386 < 0.673 \text{ therefore, } d' = 0.446 \text{ in.}
\]

Since \( I_w/I_a = 7.525 > 1.0 \), it follows that \( d = d' = 0.446 \text{ in. (i.e. compression stiffener fully effective).} \)

Thus the section is fully effective.

\[ y_{cg} = 6/2 = 3.000 \text{ in. (from symmetry)} \]

And since the section is singly symmetric about \( x \)-axis, top compression fiber (and also bottom tension fiber) may be used in computing \( S_e \).

To check if web is fully effective:

\[
f_1 = [(3.000-0.154)/3.000]x30.98 = 29.39 \text{ ksi (compression)}
\]

\[
f_2 = -29.39 \text{ ksi (tension)}
\]

\[
\psi = f_2/f_1 = -29.39/29.39 = -1.000
\]

\[
k = 24.0
\]

\[
\lambda = (1.052/\sqrt{24})(94.87) \sqrt{29.39/29500} = 0.643 < 0.673
\]

\[
b_c = w = 5.692 \text{ in.} \quad \text{(Eq. B2.1-1)}
\]

\[
b_2 = 5.692/2 = 2.846 \text{ in.}
\]
b_1 = 5.692 / [3-(-1)] = 1.423 in.

b_1 + b_2 = 4.269 in. > compression portion of the web = 2.846 in. thus b_1 + b_2 shall be taken as 2.846 in. This verifies the assumption that the web is fully effective.

Full section properties are the same as were found in determination of \( f_b M_n \) since the section is fully effective.

\[ I_x = 3.086 \text{ in.}^4 \]
\[ S_e = 1.029 \text{ in.}^3 \]
\[ M = f x S_e = 30.98 \times 1.029 \]
\[ = 31.88 \text{ kip-in.} = M_s \text{ OK} \]

Thus, \( I_x = 3.086 \text{ in.}^4 \) using procedure I for deflection determination.
EXAMPLE NO. 4.
BRACED Z-SECTION

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $6 \times 1.625 \times 0.060$ Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
1. Design flexural strength, $\phi_bM_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:
1. Calculation of the design flexural strength, $\phi_bM_n$: 
Properties of 90° corners:
\[ r = R + \frac{t}{2} = \frac{3}{32} + 0.060/2 = 0.124 \text{ in.} \]

Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \text{ in.} \)

Distance of c.g. from center of radius,
\[ c = 0.637r = 0.637 \times 0.124 = 0.079 \text{ in.} \]

Properties of 135° corners:
\[ r = R + \frac{t}{2} = \frac{3}{32} + 0.060/2 = 0.124 \text{ in.} \]

Length of arc, \( u = (45^\circ/180^\circ)(3.14)r = 0.785r = 0.785 \times 0.124 = 0.097 \text{ in.} \)

Distance of c.g. from center of radius,
\[ c_i = rsin\theta/\theta = \frac{(0.124 \times \sin45^\circ)}{0.785} = 0.112 \text{ in.} \]

Computation of \( I_n \):

For the first approximation, assume a compression stress of \( f = F_y = 50 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:
\[
\begin{align*}
\text{w} & = 1.471 \text{ in.} \\
\text{w/t} & = 1.471/0.060 = 24.52 \\
\text{S} & = 1.28 \sqrt{E/f} \\
& = 1.28 \sqrt{29500/50} = 31.09 \\
\text{S/3} & = 10.36 < \text{w/t} = 24.52 < S = 31.09 \\
\text{I}_n & = 399r^4 \left\{ [(w/t)/S] -0.33 \right\}^3 \quad \text{(Eq. B4.2-6)} \\
& = 399(0.060)^4 \left\{ (24.52/31.09) -0.33 \right\}^3 \\
& = 0.000499 \text{ in.}^4 \\
\text{d} & = 0.600 \text{ in.}, \text{d/t} = 0.600/0.060 = 10 < 14 \text{ OK} \quad \text{(Section B4 of the Commentary)} \\
\text{D} & = d+0.154\tan(\theta/2) = 0.600+0.154\tan(45^\circ/2) = 0.664 \text{ in.} \\
\text{I}_b & = d^4\sin^2\theta/12 \quad \text{(Eq. B4-2)} \\
& = (0.600)^4(0.600)\sin^2(45^\circ)/12 = 0.000540 \text{ in.}^4 \\
\text{I}_b/\text{I}_n & = 0.000540/0.000499 = 1.082 \\
\text{D/w} & = 0.664/1.471 = 0.451, 0.25 < \text{D/w} = 0.451 < 0.80 \\
k & = [4.82-5(D/w)](\text{I}_b/\text{I}_n)^{0.43} \leq 5.25-5(D/w) \quad \text{(Eq. B4.2-9)} \\
\text{n} & = 1/2 \\
& [4.82-5(0.451)](1.082)^{1/2}+0.43 = 3.098 \\
& 5.25-5(0.451) = 2.995 \\
k & = 2.995
Since \( I_s > I_a \) and \( D/w < 0.8 \), the stiffener is not considered as a simple lip.

\[
w/h = 24.52 < 90 \text{ OK (Section B1.1-(a)-(1))}
\]

\[
\lambda = \frac{(1.052/\sqrt{k})(w/h)\sqrt{f/E}}{2.995}(24.52)\sqrt{50/29500} = 0.614 < 0.673
\]

\[
b = w = 1.471 \text{ in. (i.e. compression flange fully effective)}
\]

Compression (upper) stiffener:

\[
k = 0.43 \text{ (unstiffened compression element)}
\]

\[
d/t = 10.00
\]

\( f \) conservatively taken equal to 50 ksi as in top compression fiber

\[
\lambda = \frac{(1.052/\sqrt{0.43})(10.00)\sqrt{50/29500}}{2.995} = 0.660 < 0.673
\]

therefore,

\[
d'_s = d = 0.600 \text{ in.}
\]

\[
d_s = d'_s(1/I_s) = d'_s = 0.600(1.082)
\]

\[
= 0.649 \text{ in.} > 0.600 \text{ in.}
\]

\[
d_s = 0.600 \text{ in. (i.e. compression stiffener is fully effective)}
\]

Thus, one concludes that the section is fully effective.

\( y_{cg} = 6/2 = 3.000 \text{ in. (from symmetry)} \)

Full section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) (in.)</th>
<th>( y ) Distance from Centerline of Section (in.)</th>
<th>( Ly^2 ) (in.(^3))</th>
<th>( I'_1 ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>5.692</td>
<td>—</td>
<td>—</td>
<td>15.368</td>
</tr>
<tr>
<td>Stiffeners</td>
<td>2 x 0.600 = 1.200</td>
<td>2.743</td>
<td>9.029</td>
<td>0.018</td>
</tr>
<tr>
<td>90° corners</td>
<td>2 x 0.195 = 0.390</td>
<td>2.925</td>
<td>3.337</td>
<td>—</td>
</tr>
<tr>
<td>135° corners</td>
<td>2 x 0.097 = 0.194</td>
<td>2.958</td>
<td>1.697</td>
<td>—</td>
</tr>
<tr>
<td>Flanges</td>
<td>2 x 1.471 = 2.942</td>
<td>2.970</td>
<td>25.951</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>40.014</td>
<td></td>
<td>15.386</td>
<td>—</td>
</tr>
</tbody>
</table>

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from geometry).
To check if web is fully effective; (Section B2.3)

\[ f_1 = \frac{(3.000 - 0.154)}{3.000} \times 50 = 47.43 \text{ ksi (compression)} \]

\[ f_2 = -47.43 \text{ ksi (tension)} \]

\[ \psi = \frac{f_2}{f_1} = \frac{-47.43}{47.43} = -1.000 \]

\[ k = 4 + 2(1-\psi)^3 + 2(1-\psi) \]

\[ = 4 + 2[1-(-1)]^3 + 2[1-(-1)] \]

\[ = 24.000 \]

\[ h = w = 5.692 \text{ in., } h/t = w/t = \frac{5.692}{0.060} = 94.87 \]

\[ h/t < 200 \text{ OK (Section B1.2-(a))} \]

\[ \lambda = \left(\frac{1.052}{\sqrt{24}}\right)(94.87) \sqrt{47.43/29500} = 0.817 > 0.673 \]

\[ \rho = \frac{[1-(0.22/\lambda)]}{\lambda} \quad \text{(Eq. B2.1-3)} \]

\[ = \frac{[1-(0.22/0.817)]}{0.817} = 0.894 \]

\[ b_c = \rho w \quad \text{(Eq. B2.1-2)} \]

\[ = 0.894 \times 5.692 = 5.089 \text{ in.} \]

\[ b_2 = b_c/2 \quad \text{(Eq. B2.3-2)} \]

\[ = \frac{5.089}{2} = 2.545 \text{ in.} \]

\[ b_1 = b_c/(3-\psi) \quad \text{(Eq. B2.3-1)} \]

\[ = \frac{5.089}{3-(-1)} = 1.272 \text{ in.} \]

\[ b_1 + b_2 = 1.272 + 2.545 = 3.817 \text{ in.} \]

Compression portion of the web = \( y_{cg} - 0.154 \)

\[ = 3.000 - 0.154 = 2.846 \text{ in.} \]

Since \( b_1 + b_2 = 3.817 \text{ in.} > 2.846 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 2.846 in. This verifies the assumption that the web is fully effective.

\[ I'_{x} = L y'^2 + I'_{1} \]

\[ = 40.014 + 15.386 \]

\[ = 55.400 \text{ in.}^3 \]

Actual \( I_{x} = I'_{x} t \)

\[ = 55.400 \times 0.060 \]

\[ = 3.324 \text{ in.}^4 \]

\[ S_{c} = \frac{I_{x}}{y_{cg}} \]

\[ = \frac{3.324}{3.0} = 1.108 \text{ in.}^3 \]

\[ M_{n} = S_{c} F_{y} \quad \text{(Eq. C3.1.1-1)} \]

\[ = 1.108 \times 50 = 55.40 \text{ kip-in.} \]

\[ \phi_{b} = 0.95 \]

\[ \phi_{b} M_{n} = 0.95 \times 55.400 = 52.63 \text{ kip-in.} \]
2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment $M_s$:

\[
\phi_i n = 1.2M_{DL} + 1.6M_{LL} = 1.2M_{DL} \left( \frac{M_{DL}}{M_{LL}} \right) + 1.6M_{LL} = [1.2(\frac{1}{5}) + 1.6] M_{LL} = 1.84M_{LL}
\]

\[
M_{LL} = \frac{\phi_i n}{1.84} = \frac{52.63}{1.84} = 28.603 \text{ kip-in.}
\]

\[
M_s = M_{DL} + M_{LL} = (\frac{1}{5} + 1)M_{LL} = 1.2(28.603) = 34.324 \text{ kip-in.}
\]

where

- $M_{DL} = \text{Moment determined on the basis of nominal dead load}$
- $M_{LL} = \text{Moment determined on the basis of nominal live load}$

The procedure is iterative: one assumes the actual compressive stress $f$ under this service moment $M_s$. Knowing $f$, one proceeds as usual to obtain $S_e$ and checks to see if $(f \times S_e)$ is equal to $M_s$ as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of $f = 30.99 \text{ ksi}$ in the top fibers of the section and that the web is fully effective.

**Compression flange:**

\[
S = 1.28 \sqrt{29500/30.99} = 39.49
\]

\[
S/3 = 13.16 < w/t = 24.52 < S = 39.49
\]

\[
I_a = 399(0.060)^4 \left[ \left( \frac{24.52}{39.49} \right) - 0.33 \right]^3 = 0.000127 \text{ in.}^4
\]

\[
I_a/I_a = 0.000540/0.000127 = 4.252
\]

\[
k = \frac{[4.82 - 5(0.443)](4.252)^2 + 0.43}{5} = 5.802 > 3.035
\]

\[
k = 3.035
\]

\[
\lambda = \frac{(1.052/\sqrt{3.035})(24.52) \sqrt{30.99/29500}}{0.480 < 0.673}
\]

\[
b_d = 1.471 \text{ in. (i.e. compression flange fully effective)}
\]

**Compression (upper) stiffener:**

$f$ conservatively taken equal to $30.99 \text{ ksi}$ as in top compression fiber

\[
\lambda = \frac{(1.052/\sqrt{0.43})(10.00) \sqrt{30.99/29500}}{0.520 < 0.673}
\]

therefore, $d' = 0.600 \text{ in.}$

Since $I/I_a = 4.252 > 1.0$, it follows that $D_s = d' = 0.600 \text{ in. (i.e. compression stiffener fully effective)}$.

Thus the section is fully effective.

\[y_{ce} = \frac{6}{2} = 3.000 \text{ in. (from symmetry)}\]
And since the section is singly symmetric about x-axis, top compression fiber (and also bottom tension fiber) may be used in computing $S_e$.

To check if web is fully effective:

$$f_1 = \left[\frac{(3.000-0.154)/3.000}{3.000}\right] \times 30.99 = 29.40 \text{ ksi (compression)}$$

$$f_2 = -29.40 \text{ ksi (tension)}$$

$$\psi = \frac{f_2}{f_1} = \frac{-29.40}{29.40} = -1.000$$

$$k = 24.000$$

$$\lambda = \left(\frac{1.052}{\sqrt{24}}\right) \left(\frac{94.87}{\sqrt{29.40/29500}}\right) = 0.643 < 0.673$$

$$b_e = w = 5.692 \text{ in.}$$

$$b_2 = \frac{5.692}{2} = 2.846 \text{ in.}$$

$$b_1 = \frac{5.692}{[3-(-1)]} = 1.423 \text{ in.}$$

$$b_1 + b_2 = 4.269 \text{ in.} > \text{compression portion of the web} = 2.846 \text{ in.} \text{ thus } b_1 + b_2 \text{ shall be taken as } 2.846 \text{ in.} \text{ This verifies the assumption that the web is fully effective.}$$

Full section properties are the same as were found in determination of $\phi_b M_n$ since the section is fully effective.

$$I_x = 3.324 \text{ in.}^4$$

$$S_e = 1.108 \text{ in.}^3$$

$$M = f \times S_e = 30.99 \times 1.108 = 34.34 \text{ kip-in.} = M_e \text{ OK}$$

Thus, $I_x = 3.324 \text{ in.}^4$ using procedure I for deflection determination.
EXAMPLE NO. 4A.
DEEP-Z SECTION WITH STIFFENED FLANGE

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $9.5 \times 1.625 \times 0.060$ Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:
1. Calculation of the design flexural strength, $\phi_b M_n$:
   Properties of $90^\circ$ corners:
\[ r = R + \frac{t}{2} = 3/32 + 0.060/2 = 0.124 \text{ in.} \]

Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \text{ in.} \)

Distance of c.g. from center of radius,
\[ c = 0.637r = 0.637 \times 0.124 = 0.079 \text{ in.} \]

Properties of 135° corners:
\[ r = R + \frac{t}{2} = 3/32 + 0.060/2 = 0.124 \text{ in.} \]

Length of arc, \( u = (45^\circ/180^\circ)(3.14)r = 0.785r = 0.785 \times 0.124 = 0.097 \text{ in.} \)

Distance of c.g. from center of radius,
\[ c_i = rsin\theta/\theta = [(0.124 \times sin45^\circ)/0.785] = 0.112 \text{ in.} \]

Computation of \( I_x \):
For the first approximation, assume a compression stress of \( f = F_y = 50 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:
\[ w = 1.471 \text{ in.} \]
\[ \frac{w}{t} = 1.471/0.060 = 24.52 \]
\[ S = 1.28\sqrt{Ey} \quad (\text{Eq. B4-1}) \]
\[ = 1.28 \sqrt{29500/50} = 31.09 \]
\[ \frac{S}{3} = 10.36 < \frac{w}{t} = 24.52 < S = 31.09 \]
\[ I_x = 399f^4 \left[ \left( \frac{w}{t} \right)/S - 0.33 \right]^3 \quad (\text{Eq. B4-2-6}) \]
\[ = 399(0.060)^4 \left[ (24.52/31.09) - 0.33 \right]^3 \]
\[ = 0.000499 \text{ in.}^4 \]
\[ d = 0.600 \text{ in., } \frac{d}{t} = 0.600/0.060 = 10 < 14 \text{ OK} \]
(Section B4 of the Commentary)
\[ D = d + 0.154\tan(8/2) = 0.600 + 0.154\tan(45^\circ/2) = 0.664 \text{ in.} \]
\[ I_s = d^3\sin^28/12 \quad (\text{Eq. B4-2}) \]
\[ = (0.600)^3(0.060)\sin2(45^\circ)/12 = 0.000540 \text{ in.}^4 \]
\[ I_s/I_{s0} = 0.000540/0.000499 = 1.082 \]
\[ D/w = 0.664/1.471 = 0.451, 0.25 < D/w = 0.451 < 0.80 \]
\[ k = [4.82-5(D/w)](I_s/I_{s0})^{0.43} \leq 5.25-5(D/w) \quad (\text{Eq. B4.2-9}) \]
\[ n = 1/2 \]
\[ [4.82-5(0.451)](1.082)^{0.43} + 0.43 = 3.098 \]
\[ 5.25-5(0.451) = 2.995 \]
\[ k = 2.995 \]

Since \( I_s > I_s \) and \( D/w < 0.8 \), the stiffener is not considered as a simple lip.
\[ \frac{w}{t} = 24.52 < 90 \text{ OK} \quad (\text{Section B1.1-(a)-(1)}) \]
\[ \lambda = \frac{(1.052/ \sqrt{k})(w/t)\sqrt{f/E}}{(1.052/ \sqrt{2.995})(24.52) \sqrt{50/29500}} = 0.614 < 0.673 \]  
\[ b = w = 1.471 \text{ in. (i.e. compression flange fully effective)} \]

Compression (upper) stiffener:
\[ k = 0.43 \text{ (unstiffened compression element)} \]
\[ d/t = 10.00 \]
f conservatively taken equal to 50 ksi as in top compression fiber
\[ \lambda = \frac{(1.052/ \sqrt{0.43})(10.00) \sqrt{50/29500}} = 0.660 < 0.673 \]

therefore,
\[ d' = d = 0.600 \text{ in.} \]
\[ d_s = d'(1/\lambda) = d' = 0.600 \text{ in. (i.e. compression stiffener is fully effective)} \]

Thus, one concludes that the section is fully effective.
\[ y_{cg} = \frac{9.5}{2} = 4.750 \text{ in. (from symmetry)} \]

It follows that a compression stress of 50 ksi will govern as assumed.

To check if web is fully effective (Section B2.3):
\[ f_1 = \left(\frac{4.750-0.154}{4.750}\right) \times 50 = 48.38 \text{ ksi (compression)} \]
\[ f_2 = -48.38 \text{ ksi (tension)} \]
\[ \psi = \frac{f_2}{f_1} = -\frac{48.38}{48.38} = -1.000 \]
\[ k = 4+2(1-\psi)^2 + 2(1-\psi) \]
\[ = 4+2[1-(-1)]^2 + 2[1-(-1)] \]
\[ = 24.00 \]
\[ h = w = 9.192 \text{ in., } h/t = w/t = 9.192/0.060 = 153.20 \]

\[ h/t = 153.20 < 200 \text{ OK (Section B1.2-(a))} \]
\[ \lambda = \frac{(1.052/ \sqrt{24})(153.20) \sqrt{48.38/29500}} = 1.332 > 0.673 \]
\[ \rho = \frac{1-(0.22/\lambda)}{\lambda}; \quad (\text{Eq. B2.1-3}) \]
\[ = \frac{1-(0.22/1.332)}{1.332} = 0.627 \]
\[ b_w = \rho w \quad (\text{Eq. B2.1-2}) \]
\[ = 0.627 \times 9.192 = 5.763 \text{ in.} \]
\[ b_2 = b_w/2 \quad (\text{Eq. B2.3-2}) \]
\[ = 5.763/2 = 2.882 \text{ in.} \]
\[ b_1 = b_w/(3-\psi) \quad (\text{Eq. B2.3-1}) \]
\[ = 5.763/[3-(-1)] = 1.441 \text{ in.} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ b_1 + b_2 = 1.441 + 2.882 = 4.323 \text{ in.} \]

Compression portion of the web = \( y_{eg} - 0.154 \)
\[ = 4.750 - 0.154 \]
\[ = 4.596 \text{ in.} \]

Since \( b_1 + b_2 = 4.323 \text{ in.} < 4.596 \text{ in.} \), it follows that the web is not fully effective. Hence \( y_{eg} \neq 4.750 \) as assumed.

The procedure to determine the location of the neutral axis (N.A.) based on partially effective web is iterative. We start with \( y_{eg} = 4.750 \) and from Figure B2.3-1 scale \( b_1, b_2 \) already computed with respect to \( y_{eg} = 4.750 \) in. Then we proceed to compute a new N.A. and hence \( b_1 + b_2 \). If \( b_1 + b_2 \) same as before, the solution stabilizes and the location of N.A. is calculated according to this \((b_1 + b_2)\). If \((b_1 + b_2)\) differ than before, one reiterates in the same before mentioned manner until \( b_1 + b_2 \) stabilizes.

Thus, for the first iteration, the web is divided into three segments:
\[ b_1 = 1.441 \text{ in.}, \text{ ineffective portion of web, and } b_2 = 2.882 \text{ in.} \]

The compression flange and stiffener remain fully effective since nothing is altered in their calculations.

Effective section properties about x-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L_y ) (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b_1 )</td>
<td>1.441</td>
<td>0.154+(1.441/2) = 0.875</td>
<td>1.261</td>
</tr>
<tr>
<td>( b_2+(9.5-y_{eg})-0.154 )</td>
<td>7.478</td>
<td>9.5-0.154-(7.478/2) = 5.607</td>
<td>41.929</td>
</tr>
<tr>
<td>Compression flange</td>
<td>1.471</td>
<td>0.030</td>
<td>0.044</td>
</tr>
<tr>
<td>Compression stiffener</td>
<td>0.600</td>
<td>0.064-0.124\cos45^\circ+(0.600/2)\cos45^\circ = 0.257</td>
<td>0.154</td>
</tr>
<tr>
<td>Top 90° corner</td>
<td>0.195</td>
<td>0.075</td>
<td>0.015</td>
</tr>
<tr>
<td>Top 135° corner</td>
<td>0.097</td>
<td>0.154-0.112</td>
<td>0.042</td>
</tr>
<tr>
<td>Bottom 135° corner</td>
<td>0.097</td>
<td>9.5-(0.154-0.112) = 9.458</td>
<td>0.917</td>
</tr>
<tr>
<td>Bottom 90° corner</td>
<td>0.195</td>
<td>9.5-0.075</td>
<td>9.425</td>
</tr>
<tr>
<td>Bottom stiffener</td>
<td>0.600</td>
<td>9.5-0.257</td>
<td>9.243</td>
</tr>
<tr>
<td>Tension flange</td>
<td>1.471</td>
<td>9.5-(0.060/2) = 9.470</td>
<td>13.930</td>
</tr>
<tr>
<td>Sum</td>
<td>13.645</td>
<td>13.645</td>
<td>65.638</td>
</tr>
</tbody>
</table>

\[ y_{eg} = \frac{L_y}{L} = \frac{65.638}{13.645} = 4.810 \text{ in. (measured from top compression fiber)} \]

\[ f_1 = \frac{(4.810-0.154)/4.810}{(50)} = 48.40 \text{ ksi (compression)} \]

\[ f_2 = \frac{-(9.5-4.810-0.154)/4.810}{(50)} = -47.15 \text{ ksi (tension)} \]

\[ \psi = -47.15/48.40 = -0.974 \]

\[ k = 4+2 [1-(-0.974)]^2 + 2 [1-(-0.974)] = 23.332 \]
\[ \lambda = \frac{1.052}{\sqrt{23.332}} (153.20) \sqrt{48.40/29500} = 1.351 > 0.673 \]
\[ \rho = \frac{1-(0.22/1.351)}{1.351} = 0.620 \]
\[ b_c = 0.620 \times 9.192 = 5.699 \text{ in.} \]
\[ b_2 = 5.699/2 = 2.850 \text{ in.} \]
\[ b_1 = 5.699 / [3-(-0.974)] = 1.434 \text{ in.} \]
\[ b_1 + b_2 = 4.284 \text{ in.} \neq 4.323 \text{ in.} \text{ therefore need to reiterate} \]

For the second iteration:
\[ b_1 = 1.434 \text{ in.} \]
\[ b_2 + (9.5 - y_{cg}) - 0.154 = 2.850 + 9.5 - 4.810 - 0.154 = 7.386 \text{ in.} \]
\[ \text{ineffective portion of web} = 9.192 - 1.434 - 7.386 = 0.372 \text{ in.} \]

**Effective section properties about x-axis:**

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L_y ) (in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b_1 )</td>
<td>1.434</td>
<td>0.871</td>
<td>1.249</td>
</tr>
<tr>
<td>( b_2 + (9.5 - y_{cg}) - 0.154 )</td>
<td>7.386</td>
<td>5.653</td>
<td>41.753</td>
</tr>
<tr>
<td>Compression flange</td>
<td>1.471</td>
<td>0.030</td>
<td>0.044</td>
</tr>
<tr>
<td>Compression stiffener</td>
<td>0.600</td>
<td>0.257</td>
<td>0.154</td>
</tr>
<tr>
<td>Top 90° corner</td>
<td>0.195</td>
<td>0.075</td>
<td>0.015</td>
</tr>
<tr>
<td>Top 135° corner</td>
<td>0.097</td>
<td>0.042</td>
<td>0.004</td>
</tr>
<tr>
<td>Bottom 135° corner</td>
<td>0.097</td>
<td>9.458</td>
<td>0.917</td>
</tr>
<tr>
<td>Bottom 90° corner</td>
<td>0.195</td>
<td>9.425</td>
<td>1.838</td>
</tr>
<tr>
<td>Bottom stiffener</td>
<td>0.600</td>
<td>9.243</td>
<td>5.546</td>
</tr>
<tr>
<td>Tension flange</td>
<td>1.471</td>
<td>9.470</td>
<td>13.930</td>
</tr>
<tr>
<td>Sum</td>
<td>13.546</td>
<td></td>
<td>65.450</td>
</tr>
</tbody>
</table>

\[ y_{cg} = 65.450/13.546 = 4.832 \text{ in.} \text{ (measured from top compression fiber)} \]
\[ f_1 = [(4.832-0.154)/4.832](50) = 48.41 \text{ ksi} \]
\[ f_2 = - [(9.5-4.832-0.154)/4.832](50) = -46.71 \text{ ksi} \]
\[ \psi = -46.71/48.41 = -0.965 \]
\[ k = 4+2 [1-(-0.965)]^3 + 2 [1-(-0.965)] = 23.105 \]
\[ \lambda = (1.052/\sqrt{23.105})(153.20) \sqrt{48.41/29500} = 1.358 > 0.673 \]
\[ \rho = [1-(0.22/1.358)]/1.358 = 0.617 \]
\[ b_1 = 0.617 \times 9.192 = 5.671 \text{ in.} \]
\[ b_2 = 5.671/2 = 2.836 \text{ in.} \]
\[ b_1 = 5.671 / [3-(-0.965)] = 1.430 \text{ in.} \]
\[ b_1 + b_2 = 4.266 \text{ in.} \neq 4.284 \text{ in. therefore need to reiterate} \]

For the third iteration:
\[ b_1 = 1.430 \text{ in.} \]
\[ b_2 + (9.5 - y_{cg}) - 0.154 = 2.836 + 9.5 - 4.832 - 0.154 = 7.350 \text{ in.} \]
in effective portion of web = 9.192 - 1.430 - 7.350 = 0.412 in.

Effective section properties about x-axis:
\[ L = 13.506 \text{ in.} \]
\[ Ly = 65.373 \text{ in.}^2 \]
\[ y_{cg} = 65.373/13.506 = 4.840 \text{ in.} \]
\[ f_1 = [(4.840 - 0.154)/4.840](50) = 48.41 \text{ ksi} \]
\[ f_2 = -[(9.5 - 4.840 - 0.154)/4.840](50) = -46.55 \text{ ksi} \]
\[ \Psi = -46.55/48.41 = -0.962 \]
\[ k = 4 + 2[1 - (-0.962)]^3 + 2[1 - (-0.962)] = 23.029 \]
\[ \lambda = (1.052/\sqrt{23.029})(153.20) \sqrt{48.41/29500} = 1.360 > 0.673 \]
\[ \rho = [1 - (0.22/1.360)]/1.360 = 0.616 \]
\[ b_e = 0.616 \times 9.192 = 5.662 \text{ in.} \]
\[ b_2 = 5.662/2 = 2.831 \text{ in.} \]
\[ b_1 = 5.662/[3-(-0.962)] = 1.429 \text{ in.} \]
\[ b_1 + b_2 = 4.260 \text{ in.} \neq 4.266 \text{ in. therefore need to reiterate} \]

For the fourth iteration:
\[ b_1 = 1.429 \text{ in.} \]
\[ b_2 + (9.5 - y_{cg}) - 0.154 = 2.831 + 9.5 - 4.840 - 0.154 = 7.337 \text{ in.} \]
in effective portion of web = 9.192 - 1.429 - 7.337 = 0.426 in.

Effective section properties about x-axis:
\[ L = 13.492 \text{ in.} \]
\[ Ly = 65.345 \text{ in.}^2 \]
\[ y_{cg} = 65.345/13.492 = 4.843 \text{ in.} \]
\[ f_1 = [(4.843 - 0.154)/4.843](50) = 48.41 \text{ ksi} \]
\[ f_2 = -[(9.5 - 4.843 - 0.154)/4.843](50) = -46.49 \text{ ksi} \]
\[ \Psi = -46.49/48.41 = -0.960 \]
\[ k = 4 + 2[1 - (-0.960)]^3 + 2[1 - (-0.960)] = 22.979 \]
\[ \lambda = (1.052/\sqrt{22.979})(153.20) \sqrt{48.41/29500} = 1.362 > 0.673 \]
\[ = [1 - (0.22/1.362)]/1.362 = 0.616 \]
\[ b_e = 0.616 \times 9.192 = 5.662 \text{ in.} \]
\[ b_2 = 5.662/2 = 2.831 \text{ in.} \]
\[ b_1 = \frac{5.662}{3-(-0.960)} = 1.430 \text{ in.} \]
\[ b_1 + b_2 = 4.261 \text{ in. close enough to 4.260 in. thus the solution stabilizes.} \]

Hence we now compute the location of N.A. and moment of inertia using \( b_1 = 1.430 \text{ in. and } b_2 = 2.831 \text{ in.} \)

Effective section properties about x-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>( Ly ) (in^2)</th>
<th>( Ly^2 ) (in^4)</th>
<th>( I'_1 ) About Own Axis (in^4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b_1 )</td>
<td>1.430</td>
<td>0.869</td>
<td>1.243</td>
<td>1.080</td>
<td>0.244</td>
</tr>
<tr>
<td>( B_2+(9.5-y_{cg})-0.154)</td>
<td>7.334</td>
<td>5.679</td>
<td>41.650</td>
<td>236.578</td>
<td>32.860</td>
</tr>
<tr>
<td>Compression flange</td>
<td>1.471</td>
<td>0.030</td>
<td>0.044</td>
<td>0.001</td>
<td>-</td>
</tr>
<tr>
<td>Compression stiffener</td>
<td>0.600</td>
<td>0.257</td>
<td>0.154</td>
<td>0.040</td>
<td>0.009</td>
</tr>
<tr>
<td>Top 90° corner</td>
<td>0.195</td>
<td>0.075</td>
<td>0.015</td>
<td>0.001</td>
<td>-</td>
</tr>
<tr>
<td>Top 135° corner</td>
<td>0.097</td>
<td>0.042</td>
<td>0.004</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bottom 135° corner</td>
<td>0.097</td>
<td>9.458</td>
<td>0.917</td>
<td>8.673</td>
<td>-</td>
</tr>
<tr>
<td>Bottom 90° corner</td>
<td>0.195</td>
<td>9.425</td>
<td>1.838</td>
<td>17.322</td>
<td>-</td>
</tr>
<tr>
<td>Bottom stiffener</td>
<td>0.600</td>
<td>9.243</td>
<td>5.546</td>
<td>51.262</td>
<td>0.009</td>
</tr>
<tr>
<td>Tension flange</td>
<td>1.471</td>
<td>9.470</td>
<td>13.930</td>
<td>131.917</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>13.490</td>
<td>65.341</td>
<td>446.875</td>
<td>33.122</td>
<td>-</td>
</tr>
</tbody>
</table>

Distance of x-axis from top fiber is \( y_{cg} = \frac{65.341}{13.490} = 4.844 \text{ in.} \)

Since distance of top compression fiber from neutral axis is greater than one half the beam depth (= 4.750 in.), a compression stress of 50 ksi will govern as assumed.

\[
I'_x = Ly^2 + I'_1 - Ly_{cg}^2 = 446.875 + 33.122 - 13.490(4.844)^2 = 163.487 \text{ in}^4
\]

Actual \( I_x = I'_x t = 163.487 \times 0.060 = 9.809 \text{ in}^4 \)

\( S_e = I_x/y_{cg} = 9.809/4.844 = 2.025 \text{ in}^3 \)

\( M_n = S_e F_y = 2.025 \times 50 = 101.25 \text{ kip-in.} \)

\( \phi_b = 0.95 \)
2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment $M_s$:

$\phi_b M_n = 0.95 \times 101.25 = 96.19$ kip-in.

$M_n = 0.95 \times 101.25 = 96.19$ kip-in.

$2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment $M_s$:

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6] M_{LL}$

$= [1.2(1/5)+1.6] M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \frac{\phi_b M_n}{1.84} = \frac{96.19}{1.84} = 52.28$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(52.28) = 62.73$ kip-in.

where

$M_{DL} = \text{Moment determined on the basis of nominal dead load}$

$M_{LL} = \text{Moment determined on the basis of nominal live load}$

The procedure is iterative: one assumes the actual compressive stress $f$ under this service moment $M_s$. Knowing $f$, one proceeds as usual to obtain $S_e$ and checks to see if $(f \times S_e)$ is equal to $M_s$ as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of $f = 30$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28 \sqrt{29500/30} = 40.14$

$S/3 = 13.38 < w/t = 24.52 < S = 40.14$

$I_a = 399(0.060)^4[(24.52/40.14)-0.33]^3$

$I_a = 0.000115 \text{ in.}^4$

$I/I_a = 0.000540/0.000115 = 4.696$

$k = [4.82-5(0.451)](4.696)^{1/2}+0.43 = 5.988 > 2.995$

$k = 2.995$

$\lambda = (1.052/ \sqrt{2.995})(24.52) \sqrt{30/29500} = 0.475 < 0.673$

$bd = 1.471 \text{ in.} \ (\text{i.e. compression flange fully effective})$

Compression (upper) stiffener:

$f$ conservatively taken equal to 30 ksi as in the top compression fiber.

$\lambda = (1.052/ \sqrt{0.43})(10.00)\sqrt{30/29500} = 0.512 < 0.673$

therefore, $d's = 0.600 \text{ in.}$

Since $I/I_a = 4.696 > 1.0$, it follows that $D_s = d's = 0.600 \text{ in.}$

(i.e. compression stiffener fully effective).

Thus section is fully effective (since web was assumed fully effective).

$y_{cg} = 9.5/2 = 4.750 \text{ in.} \ (\text{from symmetry})$
To check if web is fully effective:

\[ f_1 = \frac{(4.750-0.154)}{4.750} \times 30 = 29.03 \text{ ksi} \]

\[ f_2 = -29.03 \text{ ksi} \]

\[ \psi = \frac{-29.03}{29.03} = 1.000 \]

\[ k = 24.000 \]

\[ \lambda = \frac{(1.052/\sqrt{24})(153.20)}{\sqrt{29.03/29500}} = 1.032 > 0.673 \]

\[ r = \frac{[1-(0.22/1.032)]}{1.032} = 0.762 \]

\[ b_e = 0.762 \times 9.192 = 7.004 \text{ in.} \]

\[ b_2 = \frac{7.004}{2} = 3.502 \text{ in.} \]

\[ b_1 = \frac{7.004}{3-(-1)} = 1.751 \text{ in.} \]

Compression portion of the web = \( y_{cg} - 0.154 \)

\[ = 4.750 - 0.154 = 4.596 \text{ in.} \]

\( b_1 + b_2 = 5.253 \text{ in.} > 4.596 \text{ in.} \)

Thus \( b_1 + b_2 \) shall be taken as 4.596 in. This verifies the assumption that the web is fully effective.

Full section properties about x-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) (in.)</th>
<th>( y ) Distance from Centerline of Section (in.)</th>
<th>( L_y^2 ) (in.(^3))</th>
<th>( I'_l ) About Own Axis (in.(^4))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>9.192</td>
<td>--</td>
<td>--</td>
<td>64.722</td>
</tr>
<tr>
<td>Stiffeners</td>
<td>2 x 0.600 = 1.200</td>
<td>4.493</td>
<td>24.224</td>
<td>0.018</td>
</tr>
<tr>
<td>90° corners</td>
<td>2 x 0.195 = 0.390</td>
<td>4.675</td>
<td>8.524</td>
<td>--</td>
</tr>
<tr>
<td>135° corners</td>
<td>2 x 0.097 = 0.194</td>
<td>4.708</td>
<td>4.300</td>
<td>--</td>
</tr>
<tr>
<td>Flanges</td>
<td>2 x 1.471 = 2.942</td>
<td>4.720</td>
<td>65.543</td>
<td>--</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td>102.591</td>
</tr>
</tbody>
</table>

\[ I' = L_y^2 + I'_1 \]

\[ = 102.591 + 64.740 = 167.331 \text{ in.}^3 \]

\[ I_x = 167.331(0.060) = 10.040 \text{ in.}^4 \]

\[ S_e = 1_x / y_{cg} = 10.040/4.750 = 2.114 \text{ in.}^3 \]

\[ M = f \times S_e = 30 \times 2.114 \]

\[ = 63.42 \text{ kip-in.} \text{ not equal to } M_a = 60.63 \text{ kip-in.} \text{ thus need to reiterate.} \]

However, one sees that we need to assume a smaller stress than 30 ksi and since the section was fully effective for \( f = 30 \text{ ksi} \), it will be fully effective for \( f < 30 \text{ ksi} \).

Thus \( S_e = 2.114 \text{ in.}^3 \).
Therefore, the correct actual \( \tau = \frac{M_A}{S_e} = 60.63/2.114 = 28.68 \text{ ksi} \). And \( I_x = 10.026 \text{ in.}^4 \) using procedure I for deflection determination.

Remark:

It was clearly seen that in the calculation of \( \phi_b M_n \) the assumption of the web being fully effective was not true. However, it would be interesting to see the percentage of error if one neglected the partial effectiveness of the web and proceeded with the assumption of a fully effective web.

To demonstrate: neglect in the first approximation in the calculation of \( \phi_b M_n \) the partial effectiveness of the web

Thus the whole section is fully effective. Full section properties about x-axis (from part 2):

\[
\begin{align*}
I_x &= 10.040 \text{ in.}^4 \\
S_e &= 2.114 \text{ in.}^3 \\
\phi_b M_n &= 0.95(2.114 \times 50) = 100.42 \text{ kip-in.} \\
\% \text{ error} &= \left(\frac{100.42 - 96.00}{96.00}\right) \times 100\% = 4.60\%
\end{align*}
\]

Since the percentage of error is small, one could rationalize that in practical cases to get a first-hand quick answer one could assume the web being fully effective.
EXAMPLE NO. 5
HAT SECTION
Complete Flexural Design,
Stiffened Compression Flange

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown in sketch.
3. Span: $L = 8$ ft., with simple supports, no overhang, and 6-in. support bearing lengths.
4. Loading: Live = 300 lb/ft.; Dead = 20 lb/ft.

Required:
Check adequacy of section for:
1. Bending moment
2. Shear
3. Web Crippling
4. Deflection

Solution:
1. Properties of 90° corners:
Radius to centerline,
\[ r = R + \frac{t}{2} = \frac{3}{32} + \frac{0.060}{2} = 0.124 \text{ in.} \]

Length of arc, \( u = 1.57r = 1.57 \times 0.124 = 0.195 \text{ in.} \)

Distance of c.g. from center of radius,
\[ c = 0.637r = 0.637 \times 0.124 = 0.079 \text{ in.} \]

\( I' \) of corner about its own centroidal axis is negligible

2. Nominal Section Strength, \( M_{n} \) (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding

Computation of \( I_{x} \), first approximation:

* Assume a compressive stress of \( f = F_{y} = 50 \text{ ksi} \) in the top fibers of the section.
* Also assume web is fully effective.

Element 4:

\[ \frac{h}{t} = \frac{3.692}{0.060} = 61.53 < (\frac{h}{t})_{\text{max}} = 200 \text{ OK (Section B1.2-(a))} \]
Assumed fully effective

Element 5:

\[ \frac{w}{t} = \frac{8.692}{0.060} = 144.9 < 500 \text{ OK (Section B1.1-(a)-(2))} \]

\[ k = 4 \]

\[ \lambda = \frac{1.052/\sqrt{k} \sqrt{f/E}}{\sqrt{50/29500}} = 3.138 > 0.673 \]

\[ \rho = \frac{1-(0.22/\lambda)}{\lambda} = \frac{1-(0.22/3.138)}{3.138} = 0.296 \]

\[ b = \rho w = 0.296 \times 8.692 = 2.573 \text{ in.} \]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( \text{Effective Length} )</th>
<th>( y ) Distance from Top Fiber</th>
<th>( Ly ) (in.(^2))</th>
<th>( Ly^2 ) (in.(^3))</th>
<th>( I'_{x} ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 \times 0.596 = 1.192</td>
<td>3.548</td>
<td>4.229</td>
<td>15.005</td>
<td>0.035</td>
</tr>
<tr>
<td>2</td>
<td>4 \times 0.195 = 0.780</td>
<td>3.925</td>
<td>3.062</td>
<td>12.016</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>2 \times 2.692 = 5.384</td>
<td>3.970</td>
<td>21.375</td>
<td>84.857</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>2 \times 3.692 = 7.384</td>
<td>2.000</td>
<td>14.768</td>
<td>29.536</td>
<td>8.388</td>
</tr>
<tr>
<td>5</td>
<td>2.573</td>
<td>0.030</td>
<td>0.077</td>
<td>0.002</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>2 \times 0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>17.703</td>
<td>43.540</td>
<td>141.418</td>
<td>8.423</td>
<td>-</td>
</tr>
</tbody>
</table>
Distance of neutral axis from top fiber, \( y_{eg} = \frac{L_y}{L} = \frac{43.540}{17.703} = 2.460 \text{ in.} \)

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of \( F_y \) will govern as assumed.

\[
I'_x = I_y^2 + I'_1 - L_y^2 y_{eg} = 141.418 + 8.423 - 17.703(2.460)^2 = 42.71 \text{ in.}^3
\]

Actual \( I_x = t I'_x = (0.060)(42.71) = 2.56 \text{ in.}^4 \)

Check Web

\[
f_1 = \frac{(2.306/2.460)(50)}{} = 46.87 \text{ ksi (compression)}
\]

\[
f_2 = -\frac{(1.386/2.460)(50)}{} = -28.17 \text{ ksi (tension)}
\]

\[
\psi = \frac{f_2}{f_1} = -28.17/46.87 = -0.601
\]

\[
k = 4 + 2(1-\psi)^3 + 2(1-\psi) = 15.41
\]

\[
\lambda = \frac{(1.052/\sqrt{E})}{\sqrt{l/E} \cdot f = f_1} = \sqrt{1.541}(61.53) \sqrt{46.87/29500} = 0.657 < 0.673
\]

\[
b = w
\]

\[
b_e = 3.692 \text{ in.}
\]

\[
b_2 = b_e/2
\]

\[
b_1 = b_2/(3-\psi) = 3.692/[3-(-0.601)] = 1.025 \text{ in.}
\]

\[
b_1 + b_2 = 1.025 + 1.846 = 2.871 \text{ in.} > 2.306 \text{ in. (compression portion of web)}
\]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

Therefore, web is fully effective.

\[ S_c = \frac{I_y}{y_e} \]
\[ = 2.56/2.46 = 1.04 \text{ in.}^3 \]

\[ M_n = S_d F_y \]
\[ = (1.04)(50) \]
\[ = 52.0 \text{ kip-in.} \quad \text{(Eq. C3.1.1-1)} \]

b. Procedure II - Based on Inelastic Reserve Capacity

\[ \lambda_1 = \frac{1.11}{\sqrt{\frac{F_y}{E}}} \quad \text{(Eq. C3.1.1-2)} \]
\[ = \frac{1.11}{\sqrt{50/29500}} = 26.96 \]

\[ \lambda_2 = \frac{1.28}{\sqrt{\frac{F_y}{E}}} \quad \text{(Eq. C3.1.1-3)} \]
\[ = \frac{1.28}{\sqrt{50/29500}} = 31.09 \]

\[ \frac{w}{t} = 8.692/0.06 = 144.9 \]

For \( \frac{w}{t} > \lambda_2 \), \( C_y = 1 \)

Maximum compressive strain = \( C_y \epsilon_y = \epsilon_y \)

Therefore, the nominal moment, \( M_n \), is the same as the \( M_n \) determined by procedure I because the compression flange will yield first.

3. Design Flexural Strength, \( \phi_b M_n \) (Section C3.1)

\[ \phi_b = 0.95, \phi_b M_n = 0.95 \times 52.0 = 49.4 \text{ kip-in.} \]

\[ w_u = 1.2w_{DL} + 1.6w_{LL} = 1.2(0.02) + 1.6(0.3) = 0.504 \text{ kip/ft.} \]

Maximum required flexural strength = \( w_u L^2/8 = 0.504(8)^2(12)/8 = 48.38 \text{ kip-in.} \)

\[ M_u = 48.38 \text{ kip-in.} < \phi_b M_n = 49.4 \text{ kip-in.} \text{ OK} \]

4. Strength for Shear Only (Section C3.2)

The required shear strength at any section shall not exceed the design shear strength \( \phi V_n \):

\[ \sqrt{E K_v / F_y} = \sqrt{29500(5.34)/50} = 56.13 \]

\[ 1.415 \sqrt{E K_v / F_y} = 1.415(56.13) = 79.42 \]

\[ h/t = 61.53 \]

For \( \sqrt{E K_v / F_y} < h/t < 1.415 \sqrt{E K_v / F_y} \)

\[ \phi_v = 0.64 \sqrt{K_v F_y / E} \quad \text{(Eq. C3.2-2)} \]
\[ = 0.64(0.06)^2 \sqrt{5.34(50)(29500)} = 6.47 \text{ kips (per web)} \]

Total \( V_n \) for section:

\[ V_n = 2(6.47) = 12.94 \text{ kips} \]

\[ \phi_v V_n = 0.90(12.94) = 11.65 \text{ kips} \]

Maximum Required Shear Strength = Reactant

\[ V_u = w_u L/2 = 0.504(8)/2 = 2.02 \text{ k} < \phi_v V_n = 11.65 \text{ k} \text{ OK} \]
5. Web Crippling Strength (Section C3.4)

R/t = (3/32)/0.06 = 1.563 < 6 OK
h/t = 3.692/0.06 = 61.53 < 200

TABLE C3.4-1

<table>
<thead>
<tr>
<th>Stiffened Flanges</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_n = t^2kC_2C_4f_c [331-0.61(h/t)][1+0.01(N/t)]$</td>
</tr>
<tr>
<td>$k = F_y/33 = 50/33 = 1.515$</td>
</tr>
<tr>
<td>$C_3 = (1.33-0.33k)$</td>
</tr>
<tr>
<td>$C_4 = (1.15-0.15R/t) ≤ 1.0 \text{ but not less than } 0.50$</td>
</tr>
<tr>
<td>$P_n = (2 \text{ webs})(2.43 \text{ k/web}) = 4.86 \text{ k}$</td>
</tr>
<tr>
<td>$\phi_w = 0.75$</td>
</tr>
<tr>
<td>$\phi_wP_n = 0.75(4.86) = 3.65 \text{ k}$</td>
</tr>
<tr>
<td>Reaction = 2.02 k &lt; $\phi_wP_n = 3.65 \text{ k}$ OK</td>
</tr>
</tbody>
</table>

6. Deflection Determination at Service Moment $M_s$

Find $I_{eff}$ at $M_s = wL^2/8 = 0.32(8)^2(12)/8 = 30.72 \text{ kip-in.}$

6a. Procedure I

Computation of $I_{eff}$, first approximation

* Assume a compressive stress of $f = 0.6F_y = 30 \text{ ksi}$ in the top fibers of the section.
* Also assume web is fully effective.
Element 5:

\[ \lambda = \frac{1052}{\sqrt{4}}(144.9)\sqrt{30/29500} = 2.431 > 0.673 \quad \text{(Eq. B2.1-4)} \]

\[ \rho = [1-(0.22/2.431)/2.431] = 0.374 \quad \text{(Eq. B2.1-3)} \]

\[ b_d = \rho w \quad \text{(Eq. B2.1-6)} \]

\[ = 0.374(8.692) = 3.251 \text{ in.} \]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.)</th>
<th>Ly^2 (in.)</th>
<th>I' About Own Axis (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 x 0.596 = 1.192</td>
<td>3.548</td>
<td>4.229</td>
<td>15.005</td>
<td>0.035</td>
</tr>
<tr>
<td>2</td>
<td>4 x 0.195 = 0.780</td>
<td>3.925</td>
<td>3.062</td>
<td>12.016</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2 x 2.692 = 5.384</td>
<td>3.970</td>
<td>21.375</td>
<td>84.857</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2 x 3.692 = 7.384</td>
<td>2.000</td>
<td>14.768</td>
<td>29.536</td>
<td>8.388</td>
</tr>
<tr>
<td>5</td>
<td>3.251</td>
<td>0.030</td>
<td>0.098</td>
<td>0.003</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2 x 0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>Sum</td>
<td>18.381</td>
<td>43.561</td>
<td>141.419</td>
<td>8.423</td>
<td></td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{L y}{L} = \frac{43.561}{18.381} = 2.370 \text{ in.} \]

\[ I'_{eff} = L y^2 + I'_{eg} - L y_{cg}^2 \]

\[ = 141.419 + 8.423 - 18.381(2.370)^2 \]

\[ = 46.60 \text{ in.}^3 \]

Actual \[ I_{eff} = t I'_{eff} \]

\[ = (0.060)(46.60) = 2.80 \text{ in.}^4 \]

Check Web

* Should be fully effective
\( f_1 = \frac{2.216}{2.370}(30) = 28.05 \text{ ksi (compression)} \)

\( f_2 = -\frac{1.476}{2.370}(30) = -18.68 \text{ ksi (tension)} \)

\( \psi = f_2/f_1 = -18.68/28.05 = -0.666 \)

\( k = 4 + 2(1-\psi)^3 + 2(1-\psi) \) \hspace{1cm} (Eq. B2.3-4)

\( \lambda = \left( 1.052\sqrt{k(w/t)} \right) \frac{\sqrt{y}E}{s} \), \( s = f_1 \) \hspace{1cm} (Eq. B2.1-4)

\( b_c = \frac{3.692}{2} = 1.846 \text{ in.} \)

\( b_2 = b_c/2 \) \hspace{1cm} (Eq. B2.3-2)

\( b_1 = b_c/(3-\psi) \) \hspace{1cm} (Eq. B2.3-1)

\( b_1 + b_2 = 1.007 + 1.846 = 2.853 \text{ in.} > 2.216 \text{ in.} \) (compression portion of web)

Therefore, web is fully effective.

\[ S_{\text{eff}} = \frac{I_{\text{eff}}}{y_{cg}} = \frac{2.80}{2.370} = 1.18 \text{ in.}^3 \]

\[ M = S_{\text{eff}}(0.6F_y) \]

\[ = (1.18)(30) = 35.4 \text{ kip-in.} \]

To determine \( I_{\text{eff}} \) at \( M = 30.72 \text{ kip-in.} \), extrapolate using

1. \( M= 52.00 \text{ kip-in.}, I = 2.56 \text{ in.}^4 \)
2. \( M= 35.40 \text{ kip-in.}, I = 2.80 \text{ in.}^4 \)
3. \( M = 30.72 \text{ kip-in.}, I = ? \)

\[ \frac{(30.72-35.4)/(I-2.80)}{(35.4-52.0)/(2.80-2.56)} = -4.68 = -69.17(I-2.80) \]

\[ 0.0676 = I-2.80 \]

\[ I = 2.87 \text{ in.}^4 \]

Use \( I = 2.87 \text{ in.}^4 \) in deflection calculations

\[ \text{Deflection} = 5wL^4/384EI \]

b. Procedure II

\( \lambda = 0.256 + 0.328(w/t) \sqrt{F_y/E} \) \hspace{1cm} (Eq. B2.1-10)

\[ = 0.256 + 0.328(144.9) \sqrt{50/29500} = 2.213 \]

Computation of \( I_{\text{eff}} \): Check case of \( f = F_y \)

\( \lambda = 3.138 > \lambda_c \)

\( \rho = (0.41 + 0.59 \sqrt{F_y/f} - 0.22\lambda) / \lambda \) \hspace{1cm} (Eq. B2.1-9)

\( \rho = (1 - 0.22\lambda) / \lambda \)

\( \rho = 0.296 \), which is the same value for Load Capacity Determination
Computation of $I_{eff}$: Assume a compressive stress $f = 0.6F_y$

$= 30$ ksi in top fiber of section

Note: Web is fully effective

$\lambda = 2.431 > \lambda_c$

$\rho = (0.41 + 0.59 \sqrt{F_y/f} - 0.22/\lambda_c)/\lambda$  \hspace{1cm} (Eq. B2.1-9)

$= (0.41 + 0.59 \sqrt{50/30} - 0.22/2.431)/2.431 = 0.445$

For $\lambda > 0.673$

$b_d = \rho w$  \hspace{1cm} (Eq. B2.1-6)

$= 0.445(8.692) = 3.868$ in.

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>$L_y$ (in.²)</th>
<th>$L_y^2$ (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 x 0.596 = 1.192</td>
<td>3.548</td>
<td>4.229</td>
<td>15.005</td>
</tr>
<tr>
<td>2</td>
<td>4 x 0.195 = 0.780</td>
<td>3.925</td>
<td>3.062</td>
<td>12.016</td>
</tr>
<tr>
<td>3</td>
<td>2 x 2.692 = 5.384</td>
<td>3.970</td>
<td>21.375</td>
<td>84.857</td>
</tr>
<tr>
<td>4</td>
<td>2 x 3.692 = 7.384</td>
<td>2.000</td>
<td>14.768</td>
<td>29.536</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>3.868</td>
<td>0.030</td>
<td>0.004</td>
</tr>
<tr>
<td>6</td>
<td>2 x 0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
</tr>
<tr>
<td>Sum</td>
<td>18.998</td>
<td>43.579</td>
<td>141.420</td>
<td>8.423</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

$y_{cg} = L_y/L = 43.579/18.998 = 2.294$ in.

$I_{eff}' = L_y^2 + I_{1'} - L_y y_{cg}$

$= 141.420 + 8.423 - 18.998(2.294)^2$

$= 49.87$ in.³

Actual $I_{eff} = I_{eff}'$

$= (0.060)(49.87) = 2.99$ in.⁴

$S_{eff} = I_{eff}/y_{cg} = 2.99/2.294 = 1.30$ in.³

$M = S_{eff}(0.6F_y)$

$= (1.30)(30)$

$= 39.0$ kip-in.

To determine $I_{eff}$ at $M = 30.72$ kip-in., extrapolate using
(1) \( M = 52.00 \text{ kip-in., } I = 2.56 \text{ in.}^4 \)

(2) \( M = 39.00 \text{ kip-in., } I = 2.99 \text{ in.}^4 \)

(3) \( M = 30.72 \text{ kip-in., } I = ? \)

\[
\frac{(30.72-39.0)}{(I-2.99)} = \frac{(39.0-52.0)}{(2.99-2.56)} \\
-8.28 = -30.23(I-2.99) \\
0.27 = I-2.99, I = 3.26 \text{ in.}^4
\]

Use \( I = 3.26 \text{ in.}^4 \) in deflection calculations

Deflection = \( \frac{5wL^4}{384EI} \)
EXAMPLE NO. 6
HAT SECTION
Flexural Design, Compression Flange
with Intermediate Stiffener

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown in the sketch.
3. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
1. For the section, determine the design flexural strength, $\phi_b M_n$, and the moment of inertia for deflection calculation.
2. Compare structural economy of this section with an almost identical section without an intermediate stiffener computed in previous example.

Solution:
1. Properties of 90° corners:
   Radius to centerline,
   
   $r = R + v/2 = 3/32 + 0.060/2 = 0.124$ in.

   Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.
Distance of c.g. from center of radius,
\[ c = 0.637r = 0.637 \times 0.124 = 0.079 \text{ in.} \]

I' of corner about its own centroidal axis is negligible

2. Nominal Section Strength, \( M_n \) (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding

Computation of \( I_x \), first approximation:

* Assume a compressive stress of \( f_c = F_y = 50 \text{ ksi} \) in the top fibers of the section.
* Also assume web is fully effective.

Element 4:
\[ h/t = 3.692/0.060 = 61.53 < 200 \text{ OK (Section B1.2-(a))} \]
Assumed fully effective

Element 5:
\[ S = 1.28 \sqrt{E/f} \]  
\[ = 1.28 \sqrt{29500/50} = 31.09 \]
\[ b_o/t = 8.692/0.060 = 144.9 < 500 \text{ OK (Section B1.1-(a)-(2))} \]
\[ 3S = 3(31.09) = 93.27 \]

For \( b_o/t > 3S \) (Case III)
\[ I_a = t^4 \left[ \frac{1}{128(b_o/t)/S} - 285 \right] \]  
\[ = (0.06)^4 \left[ \frac{1}{128(144.9)/31.09} - 285 \right] = 0.004038 \text{ in}^4 \]

Determine full section properties of stiffener 7:

All inner radii = 3/32 in.
\[ r = R + t/2 = 3/32 + 0.060/2 = 0.124 \text{ in.} \]
\[ u = 1.57r = 1.57(0.124) = 0.195 \text{ in.} \]
\[ c = 0.637r = 0.637(0.124) = 0.079 \text{ in.} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

<table>
<thead>
<tr>
<th>Element</th>
<th>Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>(L_y) (in.)</th>
<th>(L_y^2) (in.(^2))</th>
<th>About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>2 x 0.195 = 0.390</td>
<td>0.075</td>
<td>0.0293</td>
<td>0.0022</td>
<td>—</td>
</tr>
<tr>
<td>9</td>
<td>2 x 0.350 = 0.700</td>
<td>0.329</td>
<td>0.2303</td>
<td>0.0758</td>
<td>0.0071</td>
</tr>
<tr>
<td>10</td>
<td>2 x 0.195 = 0.390</td>
<td>0.583</td>
<td>0.2274</td>
<td>0.1326</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>1.480</td>
<td>0.4870</td>
<td>0.2106</td>
<td>0.0071</td>
<td>—</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{L_y}{L} = \frac{0.4870}{1.480} = 0.329 \text{ in.} \]

Total area of section, \(L_t = (1.480)(0.060) = 0.0888 \text{ in.}^2\)

\[ I'_{s} = L_y^2 + I' - L_y y_{cg} \]
\[ = 0.2106 + 0.0071 - 1.480(0.329)^2 \]
\[ = 0.0575 \text{ in.}^3 \]

Actual \(I_s = t l'_{s} = (0.060)(0.0575) = 0.00345 \text{ in.}^4\)

Reduced Area of Stiffener

Element 9:

Stiffened element, \(k = 4\)

\[ f = F_y = 50 \text{ ksi} \]
\[ w/t = 0.350/0.060 = 5.83 < 500 \text{ OK (Section B1.1-(a)-(2))} \]

\[ \lambda = (1.052/\sqrt{k})(w/t) \sqrt{f/E} \]
\[ = (1.052/\sqrt{4})(5.83) \sqrt{50/29500} = 0.126 < 0.673 \]

\[ b = \frac{w}{(Eq. \ B2.1-4)} \]
\[ = 0.350 \text{ in. (fully effective)} \]

\[ A'_{s} = L_t = 0.0888 \text{ in.}^2 \]

\[ A_{s} = A'_{s}(I_s/I_d) \leq A'_{s} \]
\[ = 0.0888(0.00345/0.004038) \]
\[ = 0.0888(0.8544) = 0.0759 \text{ in.}^2 < A'_{s} \text{ OK} \]

\[ L_s = (A_t/I) = (0.0759/0.060) = 1.265 \text{ in.} \]

Continuing with element 5:

\[ k = 3(I_s/I_d)^{1/3} + 1 \leq 4 \]
\[ = 3(0.8544)^{1/3} + 1 = 3.847 < 4 \text{ OK} \]

\[ w/t = 4.098/0.060 = 68.30 \]

\[ \lambda = (1.052/\sqrt{k})(w/t) \sqrt{f/E} \]
\[ = (Eq. \ B2.1-4) \]
\[
\rho = \frac{(1-0.22/\lambda)}{\lambda} = \frac{(1-0.22/1.508)}{1.508} = 0.566
\]

(Eq. B2.1-3)

\[
b = \rho w
\]

(Eq. B2.1-2)

\[
b = 0.566(4.098) = 2.320 \text{ in.}
\]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.²)</th>
<th>Ly² (in.⁴)</th>
<th>(I') About Own Axis (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 x 0.596 = 1.192</td>
<td>3.548</td>
<td>4.229</td>
<td>15.005</td>
<td>0.035</td>
</tr>
<tr>
<td>2</td>
<td>4 x 0.195 = 0.780</td>
<td>3.925</td>
<td>3.062</td>
<td>12.016</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>2 x 2.692 = 5.384</td>
<td>3.970</td>
<td>21.375</td>
<td>84.857</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>2 x 3.692 = 7.384</td>
<td>2.000</td>
<td>14.768</td>
<td>29.536</td>
<td>8.388</td>
</tr>
<tr>
<td>5</td>
<td>2 x 2.320 = 4.640</td>
<td>0.030</td>
<td>0.139</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>6</td>
<td>2 x 0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>Stiffener</td>
<td>1.265</td>
<td>0.329</td>
<td>0.416</td>
<td>0.137</td>
</tr>
<tr>
<td>Sum</td>
<td>21.035</td>
<td></td>
<td>44.018</td>
<td>141.557</td>
<td>8.481</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber, \(y_{cg} = \frac{L_y}{L} = \frac{44.018}{21.035} = 2.093 \text{ in.}\)

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of \(F_y\) will govern as assumed.

\[
I' = \frac{L_y^2 + I'_{1} \cdot L_y^2}{cg}
\]

\[
I' = 141.557 + 8.481 - 21.035(2.093)^2
\]

\[
I' = 57.89 \text{ in.}^4
\]

Actual \(I_x = tl' = (0.060)(57.89) = 3.47 \text{ in.}^4\)

Check Web
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ f_1 = (1.939/2.093)(50) = 46.32 \text{ ksi (compression)} \]

\[ f_2 = -(1.753/2.093)(50) = -41.88 \text{ ksi (tension)} \]

\[ \psi = f_2/f_1 = -41.88/46.32 = -0.904 \]

\[ k = 4+2(1-\psi)^3 + 2(1-\psi) \quad \text{(Eq. B2.3-4)} \]

\[ = 4+2[-(-0.904)]^3 + 2[-(-0.904)] \]

\[ = 21.61 \]

\[ \lambda = (1.052/\sqrt{k}) (w/t) \sqrt{f_{\text{y}}/E}, f = f_1 \quad \text{(Eq. B2.1-4)} \]

\[ = (1.052/\sqrt{21.61})(61.53) \sqrt{46.32/29500} = 0.552 < 0.673 \]

\[ b = w \quad \text{(Eq. B2.1-1)} \]

\[ b_e = 3.692 \text{ in.} \]

\[ b_2 = b_e/2 \quad \text{(Eq. B2.3-2)} \]

\[ = 3.692/2 = 1.846 \text{ in.} \]

\[ b_1 = b_2/(3-\psi) \quad \text{(Eq. B2.3-1)} \]

\[ = 3.692/[3-(-0.904)] = 0.946 \text{ in.} \]

\[ b_1 + b_2 = 0.946 + 1.846 = 2.792 \text{ in.} > 1.939 \text{ in. (compression portion of web)} \]

Therefore, web is fully effective.

\[ S_e = I_e/y_{cg} \]

\[ = 3.47/2.093 = 1.66 \text{ in.}^3 \]

\[ M_n = S_e f_{\text{y}} \quad \text{(Eq. C3.1.1-1)} \]

\[ = (1.66)(50) = 83.0 \text{ kip-in.} \]

b. Procedure II - Based on Inelastic Reserve Capacity

For multiple-stiffened compression elements

\[ C_y = 1 \]

Maximum compressive strain = \( C_y e_y = e_y \)

Therefore, the nominal moment, \( M_n \) is the same as the \( M_n \) determined by procedure I because the compression flange will yield first.

3. Design Flexural Strength, \( \phi_b M_n \) (Section C3.1)

\[ \phi_b = 0.95 \]

\[ \phi_b M_n = 0.95 \times 83.0 = 78.85 \text{ kip-in.} \]

4. For complete design, one should also check:

   a. Shear Strength
   b. Combined Bending and Shear, if applicable
   c. Web Crippling Strength

5. Deflection Determination at Service Moment \( M_s \)

\[ \phi_b M_n = 1.2M_{\text{DL}} + 1.6M_{\text{LL}} \]
IV-54 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ M_{DL} = 1.2(\frac{M_{OL}}{M_{LL}}) + 1.6 \] M_{LL} \\
\phi_{n}M_{n} = 1.84M_{LL} \\
M_{LL} = \phi_{n}M_{n}/1.84 = 78.85/1.84 = 42.85 \text{kip-in.} \\
M_{s} = M_{DL} + M_{LL} \\
= (1/5+1)M_{LL} \\
= 1.2(42.85) = 51.42 \text{kip-in.}

where

M_{DL} = \text{Moment determined on the basis of nominal dead load} \\
M_{LL} = \text{Moment determined on the basis of nominal live load} \\

Find I_{eff} at M_{s} = 51.42 \text{kip-in.}

Note: Procedure II of Section B2.1-(b)-(2) does not apply.

Computation of I_{eff}, first approximation

* Assume a compressive stress of \( f = 0.6F_y = 30 \text{ ksi} \) in the top fibers of the section.
* Web is fully effective, because it was fully effective at a higher stress gradient.
* Element 9 of the stiffener, which was fully effective at \( f = 50 \text{ ksi} \) will also be fully effective at \( f = 30 \text{ ksi} \).

Element 5:

\[ S = 1.28 \sqrt{\frac{E}{f}}, f = 30 \] (Eq. B4-1) \\
\[ S = 1.28 \sqrt{29500/30} = 40.14 \]

\[ \frac{b_0}{t} = 144.9 \]

\[ 3S = 3(40.14) = 120.42 \]

For \( b_0/t > 3S \) (Case III)

\[ I_a = t^4 \left( \frac{1}{128(\frac{b_0}{t})/S} \right) - 285 \] (Eq. B4.1-9) \\
\[ I_a = (0.06)^4 \left( \frac{1}{128(144.9)/40.14} \right) - 285 = 0.002295 \text{ in.}^4 \]

\[ I_s = 0.00345 \text{ in.}^4 \]

\[ k = 3(I_s/I_a)^{1/3} + 1 \leq 4 \] (Eq. B4.1-10) \\
\[ k = 3(0.00345/0.002295)^{1/3} + 1 = 4.437 > 4 \]

\[ w/t = 68.30 \]

\[ \lambda = (1.052/\sqrt{k})(w/t) \sqrt{f/E} = 30 \text{ ksi} \] (Eq. B2.1-4) \\
\[ \lambda = (1.052/\sqrt{4})(68.30)\sqrt{30/29500} = 1.146 > 0.673 \]

\[ \rho = (1-0.22/\lambda)/\lambda \] (Eq. B2.1-3) \\
\[ \rho = (1-0.22/1.146)/1.146 = 0.705 \]

\[ b = \rho w \] (Eq. B2.1-2) \\
\[ b = 0.705(4.098) = 2.889 \text{ in.} \]
Stiffener, Element 7:

\[ A_s = A'_s \left( \frac{L_y}{I_1} \right) \leq A'_s \]  
(Eq. B4.1-11)

\[ = 0.0888(0.00345/0.002295) \]
\[ = 0.133 \text{ in.}^2 > A'_s \]

\[ A_s = A'_s = 0.0888 \text{ in.}^2 \]

\[ L_s = A_s/t = 0.0888/0.060 = 1.480 \text{ in.} \]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.²)</th>
<th>Ly² (in.⁴)</th>
<th>I' About Own Axis (in.⁵)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 x 0.596 = 1.192</td>
<td>3.548</td>
<td>4.229</td>
<td>15.005</td>
<td>0.035</td>
</tr>
<tr>
<td>2</td>
<td>4 x 0.195 = 0.780</td>
<td>3.925</td>
<td>3.062</td>
<td>12.016</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>2 x 2.692 = 5.384</td>
<td>3.970</td>
<td>21.375</td>
<td>84.857</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>2 x 3.692 = 7.384</td>
<td>2.000</td>
<td>14.768</td>
<td>29.536</td>
<td>8.388</td>
</tr>
<tr>
<td>5</td>
<td>2 x 2.889 = 5.778</td>
<td>0.030</td>
<td>0.173</td>
<td>0.005</td>
<td>—</td>
</tr>
<tr>
<td>6</td>
<td>2 x 0.195 = 0.390</td>
<td>0.075</td>
<td>0.029</td>
<td>0.002</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>Stiffener 1.480</td>
<td>0.329</td>
<td>0.487</td>
<td>0.160</td>
<td>0.058</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td>22.388</td>
<td>44.123</td>
<td>141.581</td>
<td>8.481</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{L_y}{L} = 44.123/22.388 = 1.971 \text{ in.} \]

\[ I'_{eff} = \frac{L_y^2 + I'_1 - L_y^2_{cg}}{4} \]
\[ = 141.581 + 8.481 - 22.388(1.971)^2 \]
\[ = 63.09 \text{ in.}^3 \]

Actual \( I_{eff} = tI'_{eff} \)
\[ = (0.060)(63.09) = 3.79 \text{ in.}^4 \]

\[ S_{eff} = \frac{I_{eff}}{y_{cg}} = 3.79/1.971 = 1.92 \text{ in.}^3 \]

\[ M = S_{eff}(0.6F_y) \]
\[ = (1.92)(30) \]
\[ = 57.6 \text{ kip-in.} > M_s = 51.42 \text{ kip-in.} \]

Computation of \( I_{eff} \): second approximation

Extrapolate to obtain the stress value
(1) \( M = 83.00 \text{ kip-in.}, f = F_y = 50 \text{ ksi} \)

(2) \( M = 57.60 \text{ kip-in.}, f = 0.6F_y = 30 \text{ ksi} \)

(3) \( M = 51.42 \text{ kip-in.}, f = ? \)

\[
(f-30)/(30-50) = (51.42-57.6)/(57.6-83.0)
\]

\( f = 25.13 \text{ ksi} \)

* Compressive stress of \( f = 25.13 \text{ ksi} \) in the top fiber of section

* Web is fully effective

* Element 9 of stiffener is fully effective

**Element 5:**

\[
S = 1.28 \sqrt{E/f}, f = 25.13 \text{ ksi} \quad \text{(Eq. B4-1)}
\]

\[
= 1.28 \sqrt{29500/25.13} = 43.86
\]

\[
b_o/t = 144.9
\]

\[
3S = 3(43.86) = 131.58
\]

For \( b_o/t > 3S \) (Case III)

\[
I_a = t^4 \left[ \frac{128(b_o/t)/S}{285} \right] \quad \text{(Eq. B4.1-9)}
\]

\[
= (0.06)^4 \left[ \frac{128(144.9)/43.86}{285} \right] = 0.00179 \text{ in.}^4
\]

\[
I_a = 0.00345 \text{ in.}^4
\]

\[
k = 3(I_a/I_a)^{1/3} + 1 \leq 4 \quad \text{(Eq. B4.1-10)}
\]

Since \( L/I_a > 1 \), \( k = 4 \)

\[
w/t = 68.30
\]

\[
\lambda = (1.052/\sqrt{k})(w/t) \sqrt{f/E} = 25.13 \text{ ksi} \quad \text{(Eq. B2.1-4)}
\]

\[
= (1.052/\sqrt{4})(68.30) \sqrt{25.13/29500} = 1.049 > 0.673
\]

\[
\rho = (1-0.22/\lambda)/\lambda \quad \text{(Eq. B2.1-3)}
\]

\[
= (1-0.22/1.049)/1.049 = 0.753
\]

\[
b = \rho w \quad \text{(Eq. B2.1-2)}
\]

\[
= 0.753(4.09) = 3.086 \text{ in.}
\]

**Stiffener, Element 7:**

\[
A_s = A'_s(L/I_a) \leq A'_s \quad \text{(Eq. B4.1-11)}
\]

Since \( L/I_a > 1 \)

\[
A_s = A'_s = 0.0888 \text{ in.}^2
\]

\[
L_s = A_s/t = 0.0888/0.060 = 1.480 \text{ in.}
\]

Effective section properties about x axis:
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{L_y}{L} = \frac{44.135}{22.782} = 1.937 \text{ in.} \]

\[ I'_{\text{eff}} = L_y^2 + I'_1 - L_y^2_y \]

\[ = 141.582 + 8.481 - 22.782(1.937)^2 \]

\[ = 64.59 \text{ in.}^3 \]

Actual \( I_{\text{eff}} = t I'_{\text{eff}} = (0.060)(64.59) = 3.88 \text{ in.}^4 \)

\[ S_{\text{eff}} = \frac{I_{\text{eff}}}{y_{cg}} = \frac{3.88}{1.937} = 2.00 \text{ in.}^3 \]

\[ M = (2.00)(25.13) = 50.26 \text{ kip-in. Close enough OK} \]

Use \( I = 3.88 \text{ in.}^4 \) in deflection calculations

7. Comparison of sections with and without intermediate stiffeners

<table>
<thead>
<tr>
<th>Section</th>
<th>Total Area (in.²)</th>
<th>Design Flexural Strength (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without Stiffener</td>
<td>1.43</td>
<td>49.40</td>
</tr>
<tr>
<td>With Stiffener</td>
<td>1.49</td>
<td>78.85</td>
</tr>
</tbody>
</table>

Increase in weight = \( [(1.49-1.43)/1.43] \times 100\% = 4.2\% \)

Increase in moment capacity = \( [(78.85-49.40)/49.40] \times 100\% = 59.6\% \)
EXAMPLE NO. 7
HAT SECTION
Using Inelastic Reserve Capacity

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown in sketch.
3. Top flange continuously supported.
4. Span = 8 ft. (simply supported).

Required:
Determine design flexural strength, $\phi_b M_n$.

Solution:
1. Properties of 90° corners:
   Radius to centerline,
   \[ r = R + \frac{t}{2} = \frac{3}{16} + \frac{0.135}{2} = 0.255 \text{ in.} \]
   Length of arc, \( u = 1.57r = 1.57 \times 0.255 = 0.400 \text{ in.} \)
   Distance of c.g. from center of radius,
   \[ c = 0.637r = 0.637 \times 0.255 = 0.162 \text{ in.} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

IV-59

\[ I' \] of corner about its own centroidal axis = 0.149\( r^2 \) = 0.149(0.255)^3 = 0.003 in.\(^3 \). This is negligible.

2. Nominal Section Strength (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding

Computation of \( I_x \), first approximation:

* Assume a compressive stress of \( f = F_y = 50 \) ksi in the top fibers of the section.

* Assume web is fully effective.

Element 3:

\[ h/t = 2.354/0.135 = 17.44 < 200 \text{ OK (Section B1.2-(a)). Assumed fully effective} \]

Element 5:

\[ w/t = 3.854/0.135 = 28.55 < 500 \text{ OK (Section B1.1-(a)-(2))} \]

\[ k = 4 \]

\[ \lambda = \frac{(1.052/\sqrt{4})(w/t)}{\sqrt{50/29500}} \]

\[ = \frac{(1.052/\sqrt{4})(28.55)}{\sqrt{50/29500}} = 0.618 < 0.673 \]

\[ b = w \]

\[ = 3.854 \text{ in. (Fully effective)} \]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>( Ly ) (in.(^2 ))</th>
<th>( Ly^2 ) (in.(^3 ))</th>
<th>( I'_x ) About Own Axis (in.(^3 ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 x 1.347 = 2.694</td>
<td>2.933</td>
<td>7.902</td>
<td>23.175</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>2 x 0.400 = 0.800</td>
<td>2.839</td>
<td>2.271</td>
<td>6.448</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>2 x 2.354 = 4.708</td>
<td>1.500</td>
<td>7.062</td>
<td>10.593</td>
<td>2.174</td>
</tr>
<tr>
<td>4</td>
<td>2 x 0.400 = 0.800</td>
<td>0.161</td>
<td>0.129</td>
<td>0.021</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>3.854</td>
<td>0.068</td>
<td>0.262</td>
<td>0.018</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>12.856</td>
<td>17.626</td>
<td>40.255</td>
<td>2.174</td>
<td>—</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{Ly}{L} = \frac{17.626}{12.856} = 1.371 \text{ in.} \]

Since the distance of the top compression fiber from the neutral axis is less than one half of the beam depth, a compressive stress of \( F_y \) will not govern as assumed. The actual compressive stress will be less than \( F_y \) and so the flange will still be fully effective. The tension flange will yield first. Section properties will not change.

Therefore,

\[ I'_x = Ly^2 + I'_1 - Ly^2_{cg} \]
\[ I_x = 40.255 + 2 \cdot 174 \cdot 12.856(1.371)^2 \]
\[ = 18.26 \text{ in.}^3 \]

Actual \( I_x \) = \( t' \cdot x = (0.135)(18.26) = 2.47 \text{ in.}^4 \)

Check Web

\[ \lambda = \frac{(1.052/\sqrt{k})(w/t)}{\sqrt{E}}, \quad f = f_1 \]
\[ = \frac{(1.052/\sqrt{31.15})}{(17.44)} \cdot \sqrt{32.17/29500} = 0.109 < 0.673 \]

\[ f_1 = \left( \frac{1.048}{1.629} \right)(50) = 32.17 \text{ ksi (compression)} \]

\[ f_2 = \left( \frac{-1.306}{1.629} \right)(50) = -40.09 \text{ ksi (tension)} \]

\[ \psi = \frac{f_2}{f_1} = \frac{-40.09}{32.17} = -1.246 \]

\[ k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \]
\[ = 4 + 2[1 - (-1.246)]^3 + 2[1 - (-1.246)] \]
\[ = 31.15 \]

\[ \lambda = \left( \frac{1.052/\sqrt{k}}{w/t} \right) \sqrt{E}, \quad f = f_1 \]
\[ = \left( \frac{1.052/\sqrt{31.15}}{17.44} \right) \sqrt{32.17/29500} = 0.109 < 0.673 \]

\[ b = w \]

\[ b_e = 2.354 \text{ in.} \]

\[ b_2 = b_e/2 \]
\[ = 2.354/2 = 1.177 \text{ in.} \]

\[ b_1 = b_e/(3 - \psi) \]
\[ = 2.354/[3 - (-1.246)] = 0.554 \text{ in.} \]

\[ b_1 + b_2 = 0.554 + 1.177 = 1.731 \text{ in.} > 1.048 \text{ in. (compression portion of web)} \]

Therefore, web is fully effective.

\[ S_e = l_y/(d - y_{cg}) = 2.47/(3 - 1.371) = 1.516 \text{ in.}^3 \]

\[ M_n = S_e F_y \]
\[ = (1.516)(50) \]
b. Procedure II - Based on Inelastic Reserve Capacity

\[ \lambda_1 = \frac{(1.11)\sqrt{F_y/E}}{\sqrt{50/29500}} = 26.96 \quad \text{(Eq. C3.1.1-2)} \]
\[ \lambda_2 = \frac{(1.28)\sqrt{F_y/E}}{\sqrt{50/29500}} = 31.09 \quad \text{(Eq. C3.1.1-3)} \]

w/t = 28.55

For 26.96 < w/t < 31.09

\[ C_y = 3 - 2\left[ \frac{(w/t-\lambda_1)}{\lambda_2 - \lambda_1} \right] \]
\[ = 3 - 2\left[ \frac{(28.55-26.96)}{31.09 - 26.96} \right] = 2.23 \]

Compute location of \( e \), on strain diagram, the summation of longitudinal forces should be zero. Using equations from Reck, Pekoz, and Winter, "Inelastic Strength of Cold-Formed Steel Beams," Journal of the Structural Division, November 1975, ASCE.

Distance from neutral axis to the outer compression fiber, \( y_c \):

\[ t = 0.135 \text{ in.} \]
\[ b_t = 2(1.670) = 3.340 \text{ in.} \]
\[ b_c = 4.500 \text{ in.} \]
\[ d = 3.000 \text{ in.} \]
\[ y_c = \frac{(1/4)(b_t-b_c+2d)}{b_t} \]
\[ y_c = \frac{(1/4)[3.340-4.500+2(3.000)]}{3.340} = 1.210 \text{ in.} \]
\[ y_p = \frac{y_c}{C_y} \]
\[ = 1.210/2.23 = 0.543 \text{ in.} \]
\[ y_t = d-y_c \]
\[ = 3.000 - 1.210 = 1.790 \text{ in.} \]
\[ y_{cp} = y_c-y_p \]
\[ = 1.210 - 0.543 = 0.667 \text{ in.} \]
\[ y_{tp} = y_t-y_p = 1.790 - 0.543 = 1.247 \text{ in.} \]

Summing moments of stresses in component plates:

\[ M_n = F_y\left[ b_c\,y_c + 2y_{cp}\left[y_p + (y_{cp}/2)\right] + (4/3)y_p^2 \right. \]
\[ + 2y_{tp}\left[y_p + (y_{tp}/2)\right] + b_t\,y_t \]
$M_n = 50(0.135) \{4.500(1.210)+2(0.667)[0.543+(0.667/2)] + (4/3)(0.543)² + 2(1.247)[0.543+(1.247/2)] + 3.340(1.790)\}$

$M_n = 107.3 \text{ kip-in.}$

$M_n$ shall not exceed $1.25S_e F_y = 1.25(75.8) = 94.75 \text{ kip-in.}$

Therefore

$M_n = 1.25S_e F_y = 94.75 \text{ kip-in.}$

The inelastic reserve capacity can be used because:

1. Member is not subject to twisting, lateral, torsional, or torsional-flexural buckling.
2. The effect of cold-forming is not included in determining the yield point, $F_y$.
3. The ratio of depth of the compressed portion of the web to its thickness does not exceed $\lambda_1 = (1.210-0.323)/0.135 = 6.57 < \lambda_1 = 26.96 \text{ OK}$
4. The shear force does not exceed $0.35F_y$ times the web area, $h \times t$.
   This still needs to be checked for a complete design.
5. The angle between any web and the vertical does not exceed $30°$.

3. Design Flexural Strength, $\phi_b M_n$

$\phi_b = 0.95$

$\phi_b M_n = 0.95 \times 94.75 = 90.01 \text{ kip-in.}$
EXAMPLE NO. 8
DECK SECTION

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.
3. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
1. Design flexural strengths, $\phi_M$, for positive and negative bending.
2. Factored uniform load, $w_u$, as controlled either by bending or deflection when deck is continuous over three 10'-0" spans. Deflection due to service live load is to be limited to 1/240 of the span.
Corner Properties:

\[ \theta = 75.96^\circ \]
\[ R = 1/8'' \]
\[ r = 0.155'' \]
\[ a = r \sin(90^\circ - 75.96^\circ) \]
\[ = 0.155'' \sin 14.04^\circ \]
\[ = 0.0376'' \]
\[ b = t/2 + r - a \]
\[ = 0.060''/2 + 0.155'' - 0.0376'' \]
\[ = 0.147'' \]

\[ b' = b - t/2 \]
\[ = 0.147'' - 0.060''/2 \]
\[ = 0.117'' \]
\[ b'/b'' = \cos(90^\circ - 75.96^\circ) \]
\[ b'' = b'/\cos 14.04^\circ \]
\[ = 0.117''/\cos 14.04^\circ \]
\[ = 0.112'' \]
Solution:

1. Full Section Properties:
   Elements 2 and 6:
   Radius to centerline,
   \[ r = R + \frac{t}{2} = \frac{1}{8} + \frac{0.060}{2} = 0.155 \text{ in.} \]
   Angle, \( \theta = 75.96^\circ = 1.326 \text{ rad} \)
   Length of arc, \( u = \theta r = 1.326(0.155) = 0.206 \text{ in.} \)
   Distance of c.g. from center of radius,
   \[ c_1 = rsin\theta/\theta = 0.155sin75.96^\circ/1.326 = 0.113 \text{ in.} \]
   \( I'_{1} \) of arc element about its own centroidal axis is negligible.

   Element 3:
   \( I = 3.819 \text{ in.} \)
   \[ \theta = 14.04^\circ \]
   \[ \cos\theta = 0.9701 \]
   \[ I'_{1} = (\cos^2\theta l^3)/12 = \left( (0.9701)^2 (3.819) \right)/12 = 4.368 \text{ in.}^3 \]

   Element 7:
   \( l = 1.000 \text{ in.} \)
   \[ \theta = 14.04^\circ \]
\[ \cos \theta = 0.9701 \]
\[ T_1' = \frac{(\cos^2 \theta)^3}{12} = \frac{(0.9701)^2 (1)^3}{12} = 0.0784 \text{ in.}^3 \]

Distance of centroid of full section from top fiber,
\[ y = 4 - 0.147 - \frac{(1.000/2) \cos 14.04^\circ}{2} = 3.368 \text{ in.} \]

2. Section Modulus for Load Determination - Positive Bending (Based on Procedure I)

Since the effective design width of flat compressive elements is a function of stress, iteration is required.

Computation of \( I_x \), first approximation:

* Assume a compressive stress of \( f = F_y = 50 \text{ ksi} \) in the top fibers of the section.
* Assume web is fully effective.

Element 3:
\[ h/t = 3.819/0.060 = 63.65 \quad 200 \text{ OK (Section B1.2-(a)). Assumed fully effective} \]

Element 4:
\[ \frac{w}{t} = 2.000/0.060 = 33.33 < 500 \text{ OK (Section B1.1-(a)-(2))} \]
\[ k = 4 \]
\[ \lambda = \frac{(1.052/\sqrt{E})(w/t)}{\sqrt{f}} \quad f = F_y \quad \text{(Eq. B2.1-4)} \]
\[ \lambda = \frac{(1.052/\sqrt{E})(33.33)}{\sqrt{50/29500}} = 0.722 > 0.673 \]
\[ \rho = (1-0.22/\lambda)/\lambda \quad \text{(Eq. B2.1-3)} \]
\[ \rho = (1-0.22/0.722)/0.722 = 0.963 \]
\[ b = \rho w \quad \text{(Eq. B2.1-2)} \]
\[ b = 0.963 \times 2 = 1.926 \text{ in.} \]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( \text{Effective Length} ) (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L_y ) (in.(^2))</th>
<th>( L_y^2 ) (in.(^3))</th>
<th>( I'_1 ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.000</td>
<td>3.970</td>
<td>3.970</td>
<td>15.761</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>5 \times 0.206 = 1.030</td>
<td>3.928</td>
<td>4.046</td>
<td>15.892</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>4 \times 3.819 = 15.276</td>
<td>2.000</td>
<td>30.552</td>
<td>61.104</td>
<td>17.472</td>
</tr>
<tr>
<td>4</td>
<td>2 \times 1.926 = 3.852</td>
<td>0.030</td>
<td>0.116</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>5 &amp; 8</td>
<td>2 \times 2.000 = 4.000</td>
<td>3.970</td>
<td>15.880</td>
<td>63.044</td>
<td>—</td>
</tr>
<tr>
<td>6</td>
<td>4 \times 0.206 = 0.824</td>
<td>0.072</td>
<td>0.059</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>\text{1.000}</td>
<td>3.368</td>
<td>3.368</td>
<td>11.343</td>
<td>0.078</td>
</tr>
<tr>
<td>Sum</td>
<td>26.982</td>
<td>57.991</td>
<td>167.152</td>
<td>17.550</td>
<td></td>
</tr>
</tbody>
</table>
\[ y_{cg} = \frac{L_y}{L} = \frac{57.991}{26.982} = 2.149 \text{ in.} \]

Since the distance of the top compression fiber from the neutral axis is greater than one half of the deck depth, a compressive stress of \( F_y \) will govern as assumed.

\[ I'_x = L_y^2 + I'_y - L_y y_{cg} = 167.152 + 17.550 - 26.982(2.149)^2 \]
\[ = 60.09 \text{ in.}^3 \]

Actual \( I'_x = \left(0.060\right)(60.09) = 3.61 \text{ in.}^4 \)

Check Web

\[ f_1 = \frac{(2.002/2.149)(50)}{} = 46.58 \text{ ksi (compression)} \]
\[ f_2 = -\frac{(1.704/2.149)(50)}{} = -39.65 \text{ ksi (tension)} \]
\[ \psi = f_2/f_1 = -39.65/46.58 = -0.851 \]
\[ k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad \text{(Eq. B2.3-4)} \]
\[ = 4 + 2\left[1 - \left(-0.851\right)\right]^3 + 2\left[1 - \left(-0.851\right)\right] = 20.39 \]
\[ \lambda = \frac{1.052/\sqrt{k}}{(w/t)} \quad \text{(Eq. B2.1-4)} \]
\[ = \frac{1.052/(\sqrt{20.39})}{(63.65)\sqrt{46.58}/29500} = 0.589 < 0.673 \]

\[ b = w \quad \text{(Eq. B2.1-1)} \]
\[ b_e = 3.819 \text{ in.} \]
\[ b_2 = b_e/2 \quad \text{(Eq. B2.3-2)} \]
\[ = 3.819/2 = 1.910 \text{ in.} \]
\[ b_1 = b_e/(3 - \psi) \quad \text{(Eq. B2.3-1)} \]
\[ = 3.819/(3 - (-0.851)) = 0.992 \text{ in.} \]
\[ b_1 + b_2 = 0.992 + 1.910 = 2.902 \text{ in.} > 2.002 \text{ in. (compression portion of web)} \]

Therefore, web is fully effective.
S_e = I_s/y_cg
= 3.61/2.149 = 1.68 in.³

M_n = S_eF_y
= (1.68)(50)
= 84.0 kip·in

φ_b = 0.95
φ_bM_n = 0.95 x 84.0 = 79.8 kip·in.

3. Moment of Inertia for Deflection Determination - Positive Bending

φ_bM_n = 1.2M_DL + 1.6M_LL
= [1.2(M_DL/M_LL) + 1.6]M_LL
= [1.2(1/5) + 1.6]M_LL
= 1.84M_LL

M_LL = φ_bM_n/1.84 = 79.80/1.84 = 43.37 kip·in.

M_s = M_DL + M_LL
= (1/5 + 1)M_LL
= 1.2(43.37) = 52.04 kip·in.

where

M_DL = Moment determined on the basis of nominal dead load
M_LL = Moment determined on the basis of nominal live load

Computation of I_eff, first approximation:

* Assume a compressive stress of f = 31 ksi in the top fibers of the section.
* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 4:

w/t = 33.33
k = 4

λ = (1.052/√k)(w/t)√f/E
= (1.052/√4)(33.33)√31/29500 = 0.568 < 0.673

b_d = w
= 2.000 in. (Fully effective)

Note: All elements are fully effective.

Effective section properties about x axis:
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>$L_y$ (in.$^2$)</th>
<th>$L_y^2$ (in.$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.000</td>
<td>3.970</td>
<td>3.970</td>
<td>15.761</td>
</tr>
<tr>
<td>2</td>
<td>5 x 0.206 = 1.030</td>
<td>3.928</td>
<td>4.046</td>
<td>15.892</td>
</tr>
<tr>
<td>3</td>
<td>4 x 3.819 = 15.276</td>
<td>2.000</td>
<td>30.552</td>
<td>61.104</td>
</tr>
<tr>
<td>4</td>
<td>2 x 2.000 = 4.000</td>
<td>0.030</td>
<td>0.120</td>
<td>0.004</td>
</tr>
<tr>
<td>5 &amp; 8</td>
<td>2 x 2.000 = 4.000</td>
<td>3.970</td>
<td>15.880</td>
<td>63.044</td>
</tr>
<tr>
<td>6</td>
<td>4 x 0.206 = 0.824</td>
<td>0.072</td>
<td>0.059</td>
<td>0.004</td>
</tr>
<tr>
<td>7</td>
<td>1.000</td>
<td>3.368</td>
<td>3.368</td>
<td>11.343</td>
</tr>
<tr>
<td>Sum</td>
<td>27.130</td>
<td>57.995</td>
<td>167.152</td>
<td>17.550</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = \frac{L_y}{L} = \frac{57.995}{27.130} = 2.138 \text{ in.} \]

Since the distance of the top compression fiber from the neutral axis is greater than one half the deck depth, the compressive stress of 31 ksi will govern as assumed.

\[ I'_{eff} = L_y^2 + I'_1 - L_y^2_{cg} = 167.152 - 27.130(2.138)^2 \]

\[ = 60.69 \text{ in.}^3 \]

Actual \( I_{eff} = \frac{1}{t} I'_{eff} \)

\[ = (0.060)(60.69) = 3.64 \text{ in.}^4 \]

\[ S_{eff} = \frac{L_y}{y_{cg}} = \frac{3.64}{2.138} = 1.70 \text{ in.}^3 \]

\[ M = S_{eff}(31) = (1.70)(31) \]

\[ = 52.7 \text{ kip-in.} > M_s = 52.04 \]

\( I_{eff} = 3.64 \text{ in.}^4 \) can be used for deflection calculations for the following reasons:

1. \( M(I_{eff} = 3.64) = M_s \)

2. In order to achieve \( M(I_{eff}) = M_s \) a lower compressive stress would be required, but the section is already fully effective.

\( I_{eff} = 3.64 \text{ in.}^4 \)

4. Section Modulus for Load Determination - Negative Bending (Based on Procedure I)

Following a similar procedure as in positive bending.

Computation of \( I_n \), first approximation:

* Assume a compressive stress of \( f = F_y = 50 \text{ ksi} \) in the bottom fiber of the section.

* Assume web is fully effective.

Element 3:

\( h/t = 3.819/0.060 = 63.65 < 200 \text{ OK (Section B1.2-(a))} \). Assumed fully effective
Element 1:
\[
\frac{w}{t} = \frac{1.000}{0.060} = 16.67 \quad 60 \text{ OK (Section B1.1-(a)-(3))}
\]
\[
k = 0.43
\]
\[
\lambda = \frac{(1.052/\sqrt{k})(w/t) \sqrt{f/E}}{(1.052/\sqrt{0.43})(16.67) \sqrt{50/29500}} = 1.101 > 0.673
\]
\[
\rho = (1-0.22/\lambda)/\lambda
\]
\[
= (1-0.22/1.101)/1.101 = 0.727
\]
\[
b = \rho w
\]
\[
= 0.727 \times 1.000
\]
\[
= 0.727 \text{ in.}
\]

Element 5:
Same as element 4 in positive bending case
\[
b = 1.926 \text{ in.}
\]

Element 8:
\[
\frac{w}{t} = \frac{2.000}{0.060} = 33.33 < 60 \text{ OK (Section B1.1-(a)-(3))}
\]
\[
S = 1.28 \sqrt{E/f}
\]
\[
= 1.28 \sqrt{29500/50} = 31.09
\]

For \( \frac{w}{t} > S \)
\[
I_s = t^4 \left( \frac{[115(w/t)/S] + 5}{115(33.33)/31.09} + 5 \right)
\]
\[
= (0.060)^4 \left( \frac{[115(33.33)/31.09] + 5}{0.00166} \right)
\]
\[
= 0.00166 \text{ in.}^4
\]
\[
I_s = d^3 \sin^2 \theta/12
\]
\[
= (1.000)^3(0.060)(\sin75.96^\circ)/12 = 0.00471 \text{ in.}^4
\]
\[
D = 1.000+0.185\tan(75.96^\circ/2) = 1.144 \text{ in.}
\]
\[
D/w = 1.144/2.000 = 0.572
\]

For \( 0.25 < D/w < 0.80 \)
\[
k = \left[ 4.82-5(D/w) \right] \left( \frac{(1s/la)}{1/3} + 0.43 \leq 5.25-5(D/w) \right)
\]
\[
[4.82-5(0.572)](0.00471/0.00166)^{1/3}+0.43 = 3.205
\]
\[
5.25-5(0.572) = 2.390
\]
\[
k = 2.390
\]
\[
\lambda = \frac{(1.052/\sqrt{k})(w/t) \sqrt{f/E}}{(1.052/\sqrt{2.390})(33.33) \sqrt{50/29500}} = 0.934 > 0.673
\]
\[
\rho = (1-0.22/\lambda)/\lambda
\]
\[
= (1-0.22/0.934)/0.934 = 0.818
\]
\[
b = \rho w
\]
Example Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ = 0.818(2.000) = 1.636 \text{ in.} \]

Element 7:
\[ I_s = 0.00471 \text{ in.}^4 \text{ (calculated previously)} \]
\[ I_a = 0.00166 \text{ in.}^4 \text{ (calculated previously)} \]
\[ d = 1.000 \text{ in.} \]

Assume max. stress in element, \( f = F_y = 50 \text{ ksi} \) although it will be actually less.

\[ k = 0.43 \]

\[ w/t = 1.000/0.060 = 16.67 > 14 \text{ (Section B4 of the Commentary)} \]

\[ \lambda = (1.052/ \sqrt{k})(w/t)\sqrt{f/E} \]
\[ = (1.052/ \sqrt{0.43})(16.67)\sqrt{50/29500} = 1.101 > 0.673 \]

\[ \rho = (1-0.22/\lambda)/\lambda \]
\[ = (1-0.22/1.101)/1.101 = 0.727 \]

\[ b = \rho w \]
\[ = 0.727(1.000) = 0.727 \text{ in.} \]

\[ d' = 0.727 \text{ in.} \]

\[ d_s = d'(I_s/I_a) \leq d' \]

Since \( I/I_a > 1 \)
\[ d_s = 0.727 \text{ in.} \]

\[ \Gamma' = (d_s)^3\sin^2\theta/12 = (0.727)^3(\sin75.96^\circ)/2/12 = 0.030 \text{ in.}^3 \]

Distance of centroid of reduced section from top fiber,
\[ y = 4-0.147-(0.727/2)\cos14.04^\circ = 3.500 \text{ in.} \]

Effective section properties about x axis:
Effective Length

<table>
<thead>
<tr>
<th>Element</th>
<th>L (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly \text{(in.}^3)</th>
<th>Ly^2 (in.}^3)</th>
<th>I_y About Own Axis (in.}^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.727</td>
<td>3.970</td>
<td>2.886</td>
<td>11.458</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>5 x 0.206 = 1.030</td>
<td>3.928</td>
<td>4.046</td>
<td>15.892</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>4 x 3.819 = 15.276</td>
<td>2.000</td>
<td>30.552</td>
<td>61.104</td>
<td>17.472</td>
</tr>
<tr>
<td>4</td>
<td>2 x 2.000 = 4.000</td>
<td>0.030</td>
<td>0.120</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>1.926</td>
<td>3.970</td>
<td>7.646</td>
<td>30.355</td>
<td>—</td>
</tr>
<tr>
<td>6</td>
<td>4 x 0.206 = 0.824</td>
<td>0.072</td>
<td>0.059</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>0.727</td>
<td>3.500</td>
<td>2.545</td>
<td>8.906</td>
<td>0.030</td>
</tr>
<tr>
<td>8</td>
<td>1.636</td>
<td>3.970</td>
<td>6.495</td>
<td>25.785</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>26.146</td>
<td>54.349</td>
<td>153.508</td>
<td>17.502</td>
<td></td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{ce} = \frac{L_y}{L} = \frac{54.349}{26.146} = 2.079 \text{ in.} \]

Since the distance of the top fiber from the neutral axis is greater than one half the deck depth, a compressive stress of \( F_y \) will not govern as assumed. The compressive stress will be slightly less.

\[ f_t = \left( \frac{2.079}{1.921} \right) (50) = 54.11 \text{ ksi} > F_y \]

Computation of \( I_y \), second approximation:

* Assume a compressive stress of \( f = 45 \text{ ksi} \)
* Assume web is fully effective.

Element 1:

\[ \frac{w}{t} = 16.67 \]
\[ k = 0.43 \]
\[ \lambda = \left( \frac{1.052}{\sqrt{k}} \right) \left( \frac{w}{t} \right) \sqrt{f/E} \]  
(Eq. B2.1-4)
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ \rho = \frac{1 - 0.22/\lambda}{\lambda} \]
\[ \lambda = \frac{1.052/\sqrt{k}}{1.045} \]
\[ \frac{b}{pw} = 0.756 \times 1.000 = 0.756 \text{ in.} \]

Element 5:
\[ \frac{w}{t} = 33.33 \]
\[ k = 4 \]
\[ \lambda = \frac{1.052}{\sqrt{k}} \left( \frac{w}{t} \right) \sqrt{\frac{E}{f}} \]
\[ \rho = \frac{1 - 0.22/\lambda}{\lambda} \]
\[ b = \frac{\rho w}{1.045} = 0.991 \times 2.000 = 1.982 \text{ in.} \]

Element 8:
\[ \frac{w}{t} = 33.33 \]
\[ S = 1.28 \sqrt{\frac{E}{f}} \]
\[ = 1.28 \sqrt{\frac{29500}{45}} = 32.77 \text{ in.} \]

For \( w/t > S \)
\[ I_s = t^4 \left\{ \frac{115}{115(33.33)/32.77} + 5 \right\} = 0.00158 \text{ in.}^4 \]
\[ I_s = 0.00471 \text{ in.}^4 \text{ (calculated previously)} \]
\[ D = 1.144 \text{ in.} \text{ (calculated previously)} \]
\[ \frac{D}{w} = 0.572 \text{ (calculated previously)} \]

For \( 0.25 < D/w < 0.80 \)
\[ k = \left[ 4.82 - 5(D/w) \right] \left( I_s/I_s \right)^{1/2} + 0.43 \leq 5.25 - 5(D/w) \]
\[ = 5.25 - 5(0.572)(0.00471/0.00158)^{1/2} + 0.43 = 3.251 \]
\[ 5.25 - 5(0.572) = 2.390 \]
\[ k = 2.390 \]
\[ \lambda = \frac{1.052}{\sqrt{k}} \left( \frac{w}{t} \right) \sqrt{\frac{E}{f}} \]
\[ = \frac{1.052}{\sqrt{2.390}} \left( \frac{33.33}{1.045} \right) \sqrt{\frac{29500}{45}} = 0.886 > 0.673 \]
\[ \rho = \frac{1 - 0.22/\lambda}{\lambda} \]
\[ = \frac{1 - 0.22/0.886}{0.886} = 0.848 \]
\[ b = \frac{\rho w}{1.045} = 0.848(2.000) = 1.696 \text{ in.} \]
Element 7:

\[ I_\alpha = 0.00471 \text{ in.}^4 \text{ (calculated previously)} \]
\[ I_\alpha = 0.00158 \text{ in.}^4 \text{ (calculated previously)} \]
\[ d = 1.000 \text{ in.} \]

Assume max. stress in element, \( f = 45 \text{ ksi} \) although it will be actually less.

\[ k = 0.43 \]

\[ \frac{w}{t} = 16.67 \]

\[ \lambda = \frac{(1.052/\sqrt{k})(w/t)\sqrt{f/E}}{1.052/\sqrt{0.43}}(16.67) \sqrt{45/29500} = 1.045 > 0.673 \]  
\[ (Eq. B2.1-4) \]

\[ \rho = (1-0.22/\lambda)/\lambda \]
\[ = (1-0.22/1.045)/1.045 = 0.756 \]  
\[ (Eq. B2.1-3) \]

\[ b = \rho w \]
\[ = 0.756(1.000) = 0.756 \text{ in.} \]

\[ d'_s = 0.756 \text{ in.} \]

\[ d_s = d'_s(\lambda/I_\alpha) \leq d'_s \]  
\[ (Eq. B2.1-2) \]

Since \( I/I_\alpha > 1 \)

\[ d_s = d'_s = 0.756 \text{ in.} \]

\[ \Gamma'_1 = (d_s)^3 \sin^2 \theta/12 = (0.756)^3 (\sin 75.96^\circ)^2/12 = 0.034 \text{ in.}^3 \]

Distance of centroid of reduced section from top fiber,

\[ y = 4 \cdot 0.147 - (0.756/2) \cos 14.04^\circ = 3.486 \text{ in.} \]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) Effective Length (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( \Gamma'_1 ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.756</td>
<td>3.970</td>
<td>3.001</td>
</tr>
<tr>
<td>2</td>
<td>( 5 \times 0.206 = 1.030 )</td>
<td>3.928</td>
<td>4.046</td>
</tr>
<tr>
<td>3</td>
<td>( 4 \times 3.819 = 15.276 )</td>
<td>2.000</td>
<td>30.552</td>
</tr>
<tr>
<td>4</td>
<td>( 2 \times 2.000 = 4.000 )</td>
<td>0.030</td>
<td>0.120</td>
</tr>
<tr>
<td>5</td>
<td>1.982</td>
<td>3.970</td>
<td>7.869</td>
</tr>
<tr>
<td>6</td>
<td>( 4 \times 0.206 = 0.824 )</td>
<td>0.072</td>
<td>0.059</td>
</tr>
<tr>
<td>7</td>
<td>0.756</td>
<td>3.486</td>
<td>2.635</td>
</tr>
<tr>
<td>8</td>
<td>1.696</td>
<td>3.970</td>
<td>6.733</td>
</tr>
<tr>
<td>Sum</td>
<td>26.320</td>
<td>55.015</td>
<td>156.074</td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,
\[ y_{eg} = \frac{L_y}{L} = \frac{55.015}{26.320} = 2.090 \text{ in.} \]

\[ f_t = \frac{(2.090/1.910)(45)}{1.910} = 49.24 \text{ ksi Satisfactory} \]

Check Web

\[ f_1 = \frac{(1.763/1.910)(45)}{1.910} = 41.54 \text{ ksi (compression)} \]
\[ f_2 = -\frac{(1.943/1.910)(45)}{41.54} = -45.78 \text{ ksi (tension)} \]
\[ \psi = f_2 / f_1 = -45.78 / 41.54 = -1.102 \]
\[ k = 4 + 2(1-\psi)^3 + 2(1-\psi) \]
\[ = 4 + 2[1 - (-1.102)]^3 + 2[1 - (-1.102)] \]
\[ = 26.78 \]
\[ \lambda = \frac{(1.052/\sqrt{26.78})(63.65)}{\sqrt{f/E}} \cdot f_t = f_1 \]
\[ = (1.052/\sqrt{26.78})(63.65) \sqrt{41.54/29500} = 0.486 < 0.673 \]
\[ b = w \]
\[ b_e = 3.819 \text{ in.} \]
\[ b_2 = b_e / 2 \quad \text{(Eq. B2.3-2)} \]
\[ b_2 = 3.819 / 2 = 1.910 \text{ in.} \]
\[ b_1 = b_e / (3 - \psi) \quad \text{(Eq. B2.3-1)} \]
\[ b_1 = 3.819 / (3 - (-1.102)) = 0.931 \text{ in.} \]
\[ b_1 + b_2 = 0.931 + 1.910 = 2.841 \text{ in.} > 1.763 \text{ in. (compression portion of web)} \]
Therefore, web is fully effective.

Check Element 7:

Maximum stress in element, \( f = 41.54 \text{ ksi} \)
\[
\begin{align*}
  k &= 0.43, \quad \text{w/t} = 16.67 \\
  \lambda &= (1.052 / \sqrt{k})(\text{w/t}) \sqrt{E} \\
  &= (1.052 / \sqrt{0.43})(16.67) \sqrt{41.54 / 29500} = 1.004 > 0.673 \\
  \rho &= (1 - 0.22 / \lambda) / \lambda \\
  &= (1 - 0.22 / 1.004) / 1.004 = 0.778 \\
  b &= \rho w \\
  &= 0.778(1.000) = 0.778 \text{ in.} \\
  d_s' &= 0.778 \text{ in.} \\
  d_s &= d_s'(I_s / I_s) \leq d_s' \quad \text{(Eq. B4.2-11)}
\end{align*}
\]

Since \( I_s / I_s > 1 \)
\[ d_s = d_s' = 0.778 \text{ in.} \]
\[ I_1' = (d_s')^3 \sin^2 \theta / 12 = (0.778)^3 \sin(75.96^\circ) / 12 = 0.037 \text{ in.}^3 \]
Distance of centroid of reduced section from top fiber,
\[ y = 4 - 0.147 - (0.778 / 2) \cos 14.04^\circ = 3.476 \text{ in.} \]

Determine section properties, but only the properties of element 7 have changed
\[ \Delta L = 0.778 - 0.756 = 0.022 \text{ in.} \]
\[ \Delta L y = (0.778)(3.476) - 2.635 = 0.069 \text{ in.}^2 \]
\[ \Delta L y^2 = 0.778(3.476)^2 - 9.187 = 0.213 \text{ in.}^3 \]
\[ \Delta I_1' = 0.037 - 0.034 = 0.003 \text{ in.}^3 \]

Therefore,
\[ L = 26.320 + 0.022 = 26.342 \text{ in.} \]
\[ L y = 55.015 + 0.069 = 55.084 \text{ in.}^2 \]
\[ L y^2 = 156.074 + 0.213 = 156.287 \text{ in.}^3 \]
\[ I_1' = 17.506 + 0.003 = 17.509 \text{ in.}^3 \]

Distance of neutral axis from top fiber,
\[ y_{cg} = L y / L = 55.084 / 26.342 = 2.091 \text{ in.} \]
\[ f_t = (2.091 / 1.909)(45) = 49.29 \text{ ksi} \approx 50 \text{ ksi OK} \]
\[ I'_x = I_y^2 + I_y - Ly_{eg}^2 \]
\[ = 156.287 + 17.509 - 26.342(2.091)^2 \]
\[ = 58.62 \text{ in.}^3 \]

Actual \( I_x = tI'_x \)
\[ = (0.060)(58.62) = 3.52 \text{ in.}^4 \]

\[ S_c = \frac{I_x}{y_{eg}} \]
\[ = 3.52/2.091 \]
\[ = 1.68 \text{ in.}^3 \]

\[ M_n = S_c F_y \] (Eq. C3.1.1-1)
\[ = (1.68)(50) \]
\[ = 84.0 \text{ kip-in.} \]

\[ \phi_b = 0.90 \]

\[ \phi_b M_n = 0.90 \times 84.0 = 75.6 \text{ kip-in.} \]

5. Moment of Inertia for Deflection Determination - Negative Bending
\[ \phi_b M_n = 1.2 M_{DL} + 1.6 M_{LL} \]
\[ = [1.2(M_{DL}/M_{LL}) + 1.6] M_{LL} \]
\[ = 1.84 M_{LL} \]

\[ M_{LL} = \phi_b M_n / 1.84 = 75.60 / 1.84 = 41.09 \text{ kip-in.} \]

\[ M_s = M_{DL} + M_{LL} = (1/5 + 1) M_{LL} = 1.2(41.09) = 49.31 \text{ kip-in.} \]

Computation of \( I_{eff} \), first approximation:

* Assume a compressive stress of \( f = 27 \text{ ksi} \) in the bottom fiber of the section.

* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 1:

\[ \frac{w}{t} = 16.67 \]

\[ k = 0.43 \]

\[ \lambda = \frac{(1.052/\sqrt{k})(w/t)}{\sqrt{f/E}} \] (Eq. B2.1-4)
\[ = (1.052/\sqrt{0.43})(16.67) \sqrt{27/29500} = 0.809 > 0.673 \]

\[ \rho = (1 - 0.22/\lambda)/\lambda \] (Eq. B2.1-3)
\[ = (1 - 0.22/0.809)/0.809 = 0.900 \]

\[ b = \rho w \] (Eq. B2.1-2)
\[ = 0.900 \times 1.000 \]
\[ = 0.900 \text{ in.} \]

Element 5:

\[ \frac{w}{t} = 33.33 \]
k = 4
\lambda = (1.052/ \sqrt{k(w/t)} \sqrt{f/E})
= (1.052/ \sqrt{4})(33.33) \sqrt{27/29500} = 0.530 > 0.673
\text{(Eq. B2.1-4)}

b_d = w
= 2.000 (Fully effective)
\text{(Eq. B2.1-5)}

Element 8:
\frac{w}{t} = 33.33
\text{(Eq. B2.1-4)}
S = 1.28 \sqrt{E/f}
= 1.28 \sqrt{29500/27} = 42.31
\text{(Eq. B4-1)}

For S/3 < \frac{w}{t} < S,
I_s = t^{4.399}[\frac{(w/t)}{S}-0.33]^3
= (0.060)^4(399)[(33.33/42.31)-0.33]^3
= 0.000496 \text{ in.}^4
\text{(Eq. B4.2-6)}

I_s = 0.00471 \text{ in.}^4 \text{ (calculated previously)}
I_s/I_a = 0.00471/0.000496 = 9.5 > 1
D = 1.144 \text{ in.} \text{ (calculated previously)}
D/w = 0.572 \text{ (calculated previously)}
\text{(Eq. B4.2-9)}

For 0.25 < \frac{D}{w} < 0.80
k = [4.82 - 5(\frac{D}{w})] \frac{(I_s/I_a)^{\frac{1}{2}}+0.43}{5.25 - 5(\frac{D}{w})}
\text{(Eq. B2.1-3)}

Since I_s/I_a < 1
k = 5.25 - 5(\frac{D}{w}) = 5.25 - 5(0.572) = 2.390
\lambda = (1.052/ \sqrt{k})(w/t) \sqrt{f/E}
= (1.052/ \sqrt{2.390})(33.33) \sqrt{27/29500} = 0.680 > 0.673
\rho = (1-0.22/\lambda)/\lambda
=(1-0.22/0.680)/0.680 = 0.994
b = \rho w
= 0.994(2.000) = 1.988 \text{ in.}
\text{(Eq. B2.1-2)}

Element 7:
I_s/I_a > 1
d = 1.000 \text{ in.}
\text{(Eq. B2.1-4)}

Assume max. stress in element, f = 27 ksi although it will be actually less.
k = 0.43
\frac{w}{t} = 16.67
\lambda = (1.052/ \sqrt{k})(w/t) \sqrt{f/E}
= (1.052/ \sqrt{0.43})(16.67) \sqrt{27/29500} = 0.809 > 0.673
\rho = (1-0.22/\lambda)/\lambda
\text{(Eq. B2.1-3)}
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ b = \rho w = 0.900 \times 1.000 = 0.900 \text{ in.} \]  

(Eq. B2.1-2)

\[ d'_s = 0.900 \text{ in.} \]

\[ d_s = d'_s \left( \frac{I_s}{I_s'} \right) \leq d'_s \]  

(Eq. B4.2-1)

Since \( I_s/I_s' > 1 \)

\[ d_s = d'_s = 0.900 \text{ in.} \]

\[ I'_1 = (d_s)^3 \sin^2 \theta / 12 = (0.900)^3 \sin(75.96^\circ)2/12 = 0.057 \text{ in}^3 \]

Distance of centroid of reduced section from top fiber,

\[ y = 4 - 0.147 - (0.900/2)\cos14.04^\circ = 3.416 \text{ in.} \]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.)</th>
<th>Ly^2 (in.)</th>
<th>I'_1 About Own Axis (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.900</td>
<td>3.970</td>
<td>3.573</td>
<td>14.185</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5 x 0.206 = 1.030</td>
<td>3.928</td>
<td>4.046</td>
<td>15.892</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>4 x 3.819 = 15.276</td>
<td>2.000</td>
<td>30.552</td>
<td>61.104</td>
<td>17.472</td>
</tr>
<tr>
<td>4</td>
<td>2 x 2.000 = 4.000</td>
<td>0.030</td>
<td>0.120</td>
<td>0.004</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2.000</td>
<td>3.970</td>
<td>7.940</td>
<td>31.522</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>4 x 0.206 = 0.824</td>
<td>0.072</td>
<td>0.059</td>
<td>0.004</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.900</td>
<td>3.416</td>
<td>3.074</td>
<td>10.50</td>
<td>0.057</td>
</tr>
<tr>
<td>8</td>
<td>1.988</td>
<td>3.970</td>
<td>7.892</td>
<td>31.331</td>
<td></td>
</tr>
<tr>
<td>Sum</td>
<td>26.918</td>
<td>57.256</td>
<td>164.544</td>
<td>17.529</td>
<td></td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,

\[ y_{cg} = Ly/L = 57.256/26.918 = 2.127 \text{ in.} \]

\[ I'_{eff} = Ly^2 + I'_1 - Ly^2_{cg} \]

\[ = 164.544 + 17.529 - 26.918(2.127)^2 = 60.29 \text{ in}^4 \]

Actual \( I_{eff} = \frac{tI'_{eff}}{(0.060)(60.29)} = 3.62 \text{ in}^4 \)

\[ S_{eff} = \frac{I_{eff}}{(d-y_{cg})} = 3.62/(4-2.127) = 1.93 \text{ in}^3 \]

\[ M = (1.93)(27) = 52.11 \text{ ksi} > M_s = 49.31 \text{ ksi N.G.} \]

Computation of \( I_{eff} \), second approximation:

\* Assume a compressive stress in the bottom fiber of the section using extrapolation.

(1) \( f = 45 \text{ ksi}, \quad M = 84.00 \text{ kip-in.} \)

(2) \( f = 27 \text{ ksi}, \quad M = 52.11 \text{ kip-in.} \)

(3) \( f = ?, \quad M = 49.31 \text{ kip-in.} \)
$$\frac{f-27}{(49.31-52.11)} = \frac{27-45}{(52.11-84.00)}$$

$$f - = -1.58 \quad f = 27 - 1.58$$

$$f = 25.42 \text{ ksi}$$

**Element 1:**

$\frac{w}{t} = 16.67$

$$k = 0.43$$

$$\lambda = \frac{(1.052/\sqrt{k})(w/t) \sqrt{f/E}}{\lambda} = \frac{(1.052/0.43)(16.67) \sqrt{25.42/29500}}{0.785 > .673}$$

$$\rho = \frac{1-0.22}{\lambda} \\lambda = \frac{1-0.22/0.785}{0.785} = 0.917$$

$$b = \rho w$$

$$= 0.917 \times 1 = 0.917 \text{ in.}$$

**Element 5:**

Fully effective at $f = 27 \text{ ksi}$

It will also be fully effective at $f = 25.42 \text{ ksi}$

$$b = 2.000 \text{ in.}$$

**Element 8:**

$\frac{w}{t} = 33.33$

$$S = 1.28 \sqrt{f/E}$$

$$= 1.28 \sqrt{29500/25.42} = 43.60$$

For $S/3 < \frac{w}{t} < S,$

$I/I_a > 1$ by observation

$$D/w = 0.572$$

Since $I/I_a > 1$

$$k = 2.390$$

$$\lambda = \frac{(1.052/\sqrt{k})(w/t) \sqrt{f/E}}{\lambda} = \frac{(1.052/0.43)(33.33) \sqrt{25.42/29500}}{0.666 < 0.673}$$

$$b = w$$

$$= 2.000 \text{ in. (Fully effective)}$$

**Element 7:**

$I/I_a > 1$

$$d = 1.000 \text{ in.}$$

Assume max. stress in element, $f = 25.42 \text{ ksi}$ although it will be actually less.

$$k = 0.43$$

$$\frac{w}{t} = 16.67$$
\[ \lambda = \frac{(1.052/\sqrt{k})(w/t)}{\sqrt{f/E}} \quad \text{(Eq. B2.1-4)} \]
\[ = \frac{(1.052/\sqrt{0.43})(16.67)\sqrt{25.42/29500}}{0.785} = 0.785 > 0.673 \]
\[ \rho = \frac{-0.22/\lambda}{\lambda} \quad \text{(Eq. B2.1-3)} \]
\[ = \frac{-0.22/0.785}{0.785} = 0.917 \]
\[ b = \rho w \quad \text{(Eq. B2.1-2)} \]
\[ = 0.917(1.000) = 0.917 \text{ in.} \]
\[ d'_{s} = 0.917 \text{ in.} \]

Since \( I/I_{g} > 1 \)
\[ d_{i} = d'_{s} = 0.917 \text{ in.} \]
\[ I'_{1} = (d_{i})^{3}\sin^{2}\theta/12 = (0.917)^{3}(\sin75.96^{\circ})2/12 = 0.060 \text{ in}^{3} \]

Distance of centroid of reduced section from top fiber,
\[ y = 4-0.147-(0.917/2)\cos14.04^{\circ} = 3.408 \text{ in.} \]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( y )</th>
<th>( L_{\text{Effective Length}} ) (in.)</th>
<th>( L_{\text{from Top Fiber}} ) (in.)</th>
<th>( L_{y} ) (in.(^{2}))</th>
<th>( L_{y}^{2} ) (in.(^{3}))</th>
<th>About Own Axis ((\text{in.}^{3}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.917</td>
<td>3.970</td>
<td>3.640</td>
<td>14.453</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>5 x 0.206 = 1.030</td>
<td>3.928</td>
<td>4.046</td>
<td>15.892</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>4 x 3.819 =15.276</td>
<td>2.000</td>
<td>30.552</td>
<td>61.104</td>
<td>17.472</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2 x 2.000 = 4.000</td>
<td>0.030</td>
<td>0.120</td>
<td>0.004</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2.000</td>
<td>3.970</td>
<td>7.940</td>
<td>31.522</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>4 x 0.206 = 0.824</td>
<td>0.072</td>
<td>0.059</td>
<td>0.004</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.917</td>
<td>3.408</td>
<td>3.125</td>
<td>10.650</td>
<td>0.060</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2.000</td>
<td>3.970</td>
<td>7.940</td>
<td>31.522</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sum</td>
<td>26.964</td>
<td>57.422</td>
<td>165.151</td>
<td>17.532</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Distance of neutral axis from top fiber,
\[ y_{cg} = \frac{L_{y}}{L} = 57.422/26.964 = 2.130 \text{ in.} \]
\[ I'_{\text{eff}} = L_{y}^{2} + I'_{1} - L_{y}^{2} \] \[= 165.151 + 17.532 - 26.964(2.130)^{2} = 60.35 \text{ in}^{3} \]

Actual \( I_{\text{eff}} = tl'_{\text{eff}} = (0.060)(60.35) = 3.62 \text{ in}^{4} \)
\[ S'_{\text{eff}} = \frac{I_{\text{eff}}(d-y_{cg})}{3.62/(4-2.130)} = 1.94 \text{ in}^{3} \]
\[ M = (1.94)(25.42) = 49.31 \text{ ksi} = M_{t} \text{ OK} \]

Note: A slight adjustment could be made for element 7 since the actual maximum stress is less than \( f = 25.42 \text{ ksi} \), but the net effect will be negligible.
6. Summary Positive Bending: $\phi_p M_n = 79.8$ kip-in.
   $L_{eff} = 3.64$ in.$^4$

Negative Bending: $\phi_p M_n = 75.6$ kip-in.
   $L_{eff} = 3.62$ in.$^4$

7. Compute Factored Uniform Load for a continuous deck over three equal spans, the maximum bending moment is negative and occurs over the interior supports. It is given by:
   $$M_u = 0.100 w_u L^2$$

Therefore, the maximum factored uniform load is
   $$w_u = M_u / 0.100 L^2 = 75.6 / 0.100 (10 \times 12)^2 = 0.0525$$ kip/in.
   $$w_u = 0.630$$ kip/ft

The maximum deflection occurs at a distance of $0.446 L$ from the exterior supports. It is given by:
   $$\Delta = 0.0069 w L^4 / E I$$

This deflection is limited to $\Delta = L/240$ for live load. Therefore, the maximum live load which will satisfy the deflection requirement is
   $$w_{LL} = E I / [240 (0.0069) L^4] = 29500 (3.64) / [240 (0.0069) (10 \times 12)^3]$$
   $$= 0.0375$$ kip/in.
   $$w_{LL} = 0.450$$ kip/ft
   $$w_u = 1.2 w_{LL} + 1.6 w_{LL}$$
   $$= [1.2(w_{DL} / w_{LL}) + 1.6] w_{LL}$$
   $$= [1.2(1/5) + 1.6] w_{LL}$$
   $$= 1.84 w_{LL} = 1.84 \times 0.450 = 0.828$$ kip/ft $>$ 0.630 kip/ft

Therefore, design flexural strength governs.
Factored Uniform Load = 0.630 kip/ft.

8. Check Shear Strength (Section C3.2)
   $$k_v = 5.34$$, unreinforced web
   $$\sqrt{E k_v F_y} = \sqrt{29500 (5.34/50)} = 56.13$$
   $$1.415 \sqrt{E k_v F_y} = 1.415 (56.13) = 79.42$$
   $$h/t = 3.819/0.060 = 63.65$$

For $\sqrt{E k_v F_y} < h/t < 1.415 \sqrt{E k_v F_y}$
   $$\theta_v = 0.90$$
   $$V_n = 0.64 t^2 \sqrt{E k_v F_y}$$
   $$= 0.64 (0.06)^2 \sqrt{5.34 (50) (29500)} = 6.47$$ kips (per web)

Total $V_n$ for section:
   $$V_n = 4 (6.47) = 25.88$$ kips
   $$\phi_v V_n = 0.90 (25.88) = 23.29$$ kips

Vertical component ($\phi_v V_n_v = 23.29 \cos 14.04^\circ = 22.59$ kips
The maximum required shear strength is given by

\[ V_u = 0.600 \omega_u L \]

\[ = (0.600)(0.630)(10) = 3.78 \text{ kips} < (\phi, V_n) = 22.59 \text{ kips OK} \]

9. Check Strength for Combined Bending and Shear (Section C3.3) At the interior supports there is a combination of web bending and web shear:

\[ \phi_u M_{x_o} = 75.6 \text{ kip-in.} \quad M_u = 0.100 \omega_u L^2 \]

\[ (\phi, V_n) = 22.59 \text{ kips} \quad V_u = 0.600 \omega_u L \]

For unreinforced webs,

\[ (M_u/\phi_u) M_{x_o}^2 + \left[ \frac{V_u}{(\phi, V_n)} \right] \leq 1.0 \quad \text{(Eq. C3.3-1)} \]

Solve for \( \omega_u \):

\[ \left[ \frac{0.100 \omega_u (10 \times 12)^2}{75.6} \right]^2 + \left[ \frac{0.600 \omega_u (10 \times 12)}{22.59} \right]^2 = 1.0 \]

\[ 362.81 \omega_u^2 + 10.16 \omega_u^2 = 1.0 \]

\[ 372.97 \omega_u^2 = 1.0 \]

\[ \omega_u = 0.0518 \text{ kip/in.} \]

\[ = 0.621 \text{ kip/ft.} \]

Factored Uniform Load = 0.621 kip/ft.

10. Check Web Crippling Strength (Section C3.4)

\[ h = 3.819 \text{ in.} \]

\[ t = 0.060 \text{ in.} \]

\[ h/t = 3.819/0.06 = 63.65 < 200 \text{ OK} \]

\[ R = 1/8 \text{ in.} \]

\[ R/t = 0.125/0.06 = 2.083 < 7 \text{ OK} \]

Let \( N = 6 \text{ in.} \)

\[ N/t = 6/0.06 = 100 < 210 \text{ OK} \]

\[ N/h = 6/3.819 = 1.57 < 3.5 \text{ OK} \]

Table C3.4-1 applies

For end reactions: \( \text{(Eq. C3.4-2)} \)

For interior reaction: \( \text{(Eq. C3.4-4)} \)

\[ k = F_y/33 = 50/33 = 1.515 \quad \text{(Eq. C3.4-21)} \]

\[ C_1 = (1.22 - 0.22k) \quad \text{(Eq. C3.4-10)} \]

\[ = [1.22 - 0.22(1.515)] = 0.887 \]

\[ C_2 = (1.06 - 0.06R/t) \quad \text{(Eq. C3.4-11)} \]

\[ = [1.06 - 0.06(2.083)] = 0.935 < 1.0 \text{ OK} \]

\[ C_3 = (1.33 - 0.33k) \quad \text{(Eq. C3.4-12)} \]

\[ = [1.33 - 0.33(1.515)] = 0.830 \]

\[ C_4 = (1.15 - 0.15R/t) \leq 1.0 \text{ but not less than 0.50} \quad \text{(Eq. C3.4-13)} \]
(1.15-0.15R/t) = [1.15-0.15(2.083)] = 0.838 ≤ 1.0 OK

C₄ = 0.838

θ = 75.96°

C₉ = 0.7+0.3(θ/90)²

= 0.7+0.3(75.96/90)² = 0.914

For end reaction:

Pₙ = (t²C₁C₂C₉[ 17-0.28(h/t)][ 1+0.01(N/t)]

= (0.06)²(1.515)(0.830)(0.838)(0.914)[217-0.28(63.65)]

x[ 1+0.01(100)] = 1.38 kips/web

Total Pₙ for section:

Pₙ = (4 webs)(1.38 k/web) = 5.52 kips

θₜₕ = 0.75

θₜₕPₙ = 0.75(5.52) = 4.14 kips

End reaction is given by

R = 0.400wₒL

= (0.400)(0.621)(10) = 2.48 kips < θₜₕPₙ 4.14 kips OK

For interior reaction:

Pₙ = (t²C₁C₂C₉[ 538-0.74(h/t)][ 1+0.007(N/t)]

= (0.06)²(1.515)(0.887)(0.935)(0.914)[538-0.74(63.65)]

x[ 1+0.007(100)] = 3.45 kips/web

Total Pₙ for section:

Pₙ = (4 webs)(3.45 k/web) = 13.80 kips

θₜₕ = 0.75

θₜₕPₙ = 0.75(13.80) = 10.35 kips

Interior reaction is given by

R = 1.10wₒL

= (1.10)(0.621)(10) = 6.83 kips < θₜₕPₙ = 10.35 kips OK
**EXAMPLE NO. 9.**
CYLINDRICAL TUBULAR SECTION

*Given:*
1. Steel: \( F_y = 50 \text{ ksi.} \)
2. Section: Shown in sketch above.

*Required:*
Design flexural strength, \( \phi_b M_n \).

*Solution:*

Ratio of outside diameter to wall thickness,
\[
\frac{D}{t} = \frac{8.000}{0.125} = 64.00
\]
\[
\frac{D}{t} < 0.441\frac{E}{F_y} = 0.441(29500/50) = 260.2 \text{ OK}
\]
\[
S_f = \pi \left[ \left( \frac{\text{O.D.}}{2} \right)^4 - \left( \frac{\text{I.D.}}{2} \right)^4 \right]/32(\text{O.D.})
\]
\[
= \pi \left[ (8)^4 - (7.75)^4 \right]/32(8)
\]
\[
= 5.995 \text{ in.}^3
\]

\[
0.070\frac{E}{F_y} = 0.070(29500/50) = 41.30
\]
\[
0.319\frac{E}{F_y} = 0.319(29500/50) = 188.2
\]

For \( 0.070\frac{E}{F_y} < \frac{D}{t} < 0.319\frac{E}{F_y} \)
\[
M_n = \left[ 0.97 + 0.02(\frac{E}{F_y})/(D/t) \right]F_yS_f
\]

(Eq. C6.1-2)
= [0.97+0.02(29500/50)/64.00](50)(5.995)
= 346.02 kip-in.

$\phi_0 = 0.95$

$\phi_0 M_0 = 0.95 \times 346.02 = 328.72$ kip-in.
EXAMPLE NO. 10.
C-SECTION

Given:
1. Steel: \( F_y = 50 \text{ ksi} \).
2. Section: \( 3.5 \times 2 \times 0.105 \) channel with stiffened flanges.
3. \( K_xL_x = K_yL_y = K_zL_z = 6 \text{ ft} \).

Required:
Design axial strength, \( \phi \alpha P_0 \).

Solution:
1. Basic parameters:
   \[
   r = R + t/2 = 3/16 + 0.105/2 = 0.240 \text{ in.}
   \]
   From the sketch and Section 1.2.2 of Part III of the Manual, \( a = 2.914 \text{ in.}, b = 1.414 \text{ in.}, c = 0.607 \text{ in.}, \alpha = 1.00 \) (Since the section has lips)
\( \bar{a} = A' - t = 3.5 - 0.105 = 3.395 \text{ in.} \)
\( \bar{b} = B' - [t/2 + \alpha u/2] = B' - t = 2 - 0.105 = 1.895 \text{ in.} \)
\( \bar{c} = \alpha [C' - t/2] = C' - t/2 = 0.9 - 0.105/2 = 0.848 \text{ in.} \)
\( u = 1.57 \bar{r} = 1.57 \times 0.240 = 0.377 \text{ in.} \)

2. Area:
\( A = t[ a + 2b + 2u + \alpha(2c + 2u)] = t[ a + 2b + 2c + 4u] \)
\[ A = 0.105[2.914 + 2 \times 1.414 + 2 \times 0.607 + 4 \times 0.377] = 0.889 \text{ in.}^2 \]

3. Moment of inertia about x-axis:
\[ I_x = 2t\left(0.0417a^3 + b(a/2 + \bar{r})^2 + u(a/2 + 0.637\bar{r})^2 + 0.149\bar{r}^3 + \alpha[0.0833c^3 + (c/4)(a-c)^2 + u(0.363\bar{r}) + 0.377(1.414 + 1.637\bar{r})^2 + 0.298(0.240)^3 + 0.0833(0.607)^3 + (0.607/4)(2.914 - 0.607)^3]\right) \]
\[ I_x = 2 \times 0.105 \times 1.414(1.414/2 + 0.240)^2 + 0.377(0.363 \times 0.240) + 0.377(1.414 + 1.637 \times 0.240) + 0.607(1.414 + 2 \times 0.240) \]
\[ I_x = 0.889(0.757)^2 \]
\[ I_x = 1.657 \text{ in.}^4 \]

4. Distance from centroid of section to centerline of web:
\[ \bar{x} = (2t/A)\{b(b/2 + \bar{r}) + u(0.363\bar{r}) + \alpha[u(b + 1.537\bar{r}) + c(b + 2\bar{r})]\} \]
\[ \bar{x} = [(2 \times 0.105)/0.889] \times \{1.414(1.414/2 + 0.240) + 0.377(0.363 \times 0.240) + 0.377(1.414 + 1.637 \times 0.240) + 0.607(1.414 + 2 \times 0.240)\} \]
\[ I_x = 0.757 \text{ in.} \]

5. Moment of inertia about y-axis:
\[ I_y = 2t\{b(b/2 + \bar{r})^2 + 0.0833b^3 + 0.356\bar{r}^3 + \alpha[c(b + 2\bar{r})^2 + u(b + 1.637\bar{r})^2 + 0.149\bar{r}^3] - A(\bar{x})^2 \}
\[ I_y = 2 \times 0.105 \times 1.414(1.414/2 + 0.240)^2 + 0.356(0.240)^3 + 0.607(1.414 + 2 \times 0.240)^2 + 0.377(1.414 + 1.637 \times 0.240)^2 + 0.149(0.240)^3 - 0.889(0.757)^2 \]
\[ I_y = 0.524 \text{ in.}^4 \]

6. Distance from shear center to centerline of web:
\[ m = (6t/12I_x)[6 \bar{c}(\bar{a})^2 + 3 \bar{b}(\bar{a})^2 - 8(\bar{c})^3] \]
\[ m = [(1.895 \times 0.105)/(12 \times 1.657)][6 \times 0.848(3.395)^2 + 3 \times 1.895(3.395)^2 - 8(0.848)^3] \]
\[ m = 1.194 \text{ in.} \]

7. Distance from centroid to shear center:
\[ x_o = - (\bar{x} + m) = -(0.757 + 1.194) \]
8. St. Venant torsion constant:

\[ J = \frac{(l^3)}{3} [a+2b+2u+\alpha(2c+2u)] \]
\[ = [(0.105)^3/3][2.914+2x1.414+4x0.377+2x0.607] \]
\[ = 0.003266 \text{ in.}^4 \]

9. Warping Constant:

\[ C_w = \left(\frac{l^3}{A}\right) \left[ \frac{\bar{A}(\bar{a})^2}{l} \right] \left[ \frac{(b^2)}{3+m^2-mb} \right] + (A/3\alpha) \left[ \frac{(m)^2}{(a)^3} \right] \]
\[ + (5/3) \left( \bar{c} \right)^2 \frac{(2/3+\bar{a})}{l(2\bar{a}+4\bar{c})} + \left[ \frac{m(b^2)}{(2/3)\bar{c}} \right] \]
\[ + 2m(2\bar{c}(\bar{c}-\bar{a})+6(2\bar{c}-3\bar{a})) + \left[ \frac{(b)^2}{(a)^2} \right] \left[ \frac{(3\bar{c}+b)(4\bar{c}+a)-6(\bar{b})^3}{(a)^3} \right] \]
\[ - \left[ \frac{m^2}{(\bar{a})^4/4} \right] \]
\[ = \left[ (0.105)^2/0.889 \right] \left[ \frac{0.757x0.89x(3.395)^2}{0.105} \right] \left[ (1.895)^2/3 \right] \]
\[ + (1.194)^2 - 1.194x1.895 + 0.898/(3x0.105)[(1.194)^2 (3.395)^3 \]
\[ + (1.895)^2 (0.848)^2 (2x0.848+3x3.395)] \]
\[ - [1.657x(1.194)^2/0.105/(2x3.395+4x0.848)] \]
\[ + (1.194(0.848)^2/3) \left[ (2x1.895)^2 (0.848) \right] \]
\[ + 2x1.194(2x0.848+3x3.395+1.895(2x0.848-3x3.395))] \]
\[ + [ (1.895)^2 (3.395)^2 /6] [(3x0.848+1.895)(4x0.848+3x3.95) \]
\[ - 6(0.848)^2 -(1.194)^2 (3.395)^4/4] \]
\[ = 2.050 \text{ in.}^6 \]

10. Radii of gyration:

\[ r_x = \sqrt{\frac{I_x}{A}} = \sqrt{1.657/0.889} = 1.365 \text{ in} \]
\[ r_y = \sqrt{\frac{I_y}{A}} = \sqrt{0.524/0.889} = 0.768 \text{ in} \]
\[ \langle K_rL_x \rangle/r = (6x12)/0.768 = 93.75 < 200 \]
\[ r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{(1.365)^2 + (0.768)^2 + (-1.951)^2} \]
\[ = 2.502 \text{ in.}^2 \]

11. Torsional-flexural constant:

\[ \beta = 1 - \frac{(x_o/r_o)^2}{(Eq. \ C4.2-3)} \]
\[ = 1 - (-1.951/2.502)^2 = 0.392 \]

12. Determination of \( F_c \):

For this singly symmetric section (x-axis is the axis of symmetry), \( F_c \) shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):

\[ (F_c)_1 = \left( \frac{\pi^2E}{(K_rL_x/r)} \right)^2 \quad (Eq. \ C4.1-1) \]
\[ = \left( \frac{\pi^2x29500}{(6x12/0.768)^2} \right)^2 = 33.13 \text{ ksi} \]
\[ \sigma_{ex} = \left( \frac{\pi^2E}{(K_rL_x/r)} \right)^2 \quad (Eq. \ C3.1.2-7) \]
IV-90 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[
\sigma_1 = \frac{1}{(4.03^2)\left[ \frac{G J + (\pi^2 E c_w \times 0.003266)}{(K_t L_t)^2} \right]} \quad \text{(Eq. C3.1.2-9)}
\]

\[
= \frac{1}{(0.889 \times 2.502^2)(11300 \times 0.003266 + (\pi^2 \times 29500 \times 2.050)/(6 \times 12)^2)} = 0.392
\]

\[
\sigma_1 = 0.392 \times (104.65 + 27.32) - \frac{0.392 \times 104.65 \times 27.32}{2} = 27.32 \text{ ksi}
\]

\[
\left( \frac{F_y}{2} \right)^2 = \frac{1}{(2 \times 0.392)} \left[ \frac{0.392 \times (104.65 + 27.32) - 4 \times 0.392 \times 104.65 \times 27.32}{2} \right] = 23.27 \text{ ksi}
\]

\[
F_e = 23.27 \text{ ksi}
\]

13. Determination of \( F_n \):

\[
F_y/2 = 50/2 = 25.00 \text{ ksi}
\]

For \( F_e < F_y/2 \)

\[
F_n = F_e = 23.27 \text{ ksi.} \quad \text{(Eq. C4-3)}
\]

14. Determination of \( A_c \):

Flanges:
\[
d = 0.607 \text{ in.}
\]

\[
I_s = \frac{d^3 t}{12} = \frac{(0.607)^3 \times 0.105}{12} = 0.001957 \text{ in.}^4
\]

\[
D = 0.9 \text{ in.}
\]

\[
w = 1.414 \text{ in.}
\]

\[
D/w = 0.9/1.414 = 0.636 < 0.80
\]

\[
S = \frac{1.28 \sqrt{\frac{t}{E}} \times f = F_e}{f = \frac{F_e}{23.27}} = \frac{1.28 \sqrt{29500/23.27}}{23.27} = 45.57
\]

\[
w/t = 1.414/0.105 = 13.47 < S/3 = 15.19 \quad \text{(Eq. B4.2-1)}
\]

\[
I_s = 0 \quad \text{(no edge stiffener needed)} \quad \text{(Eq. B4.2-2)}
\]

\[
b = w \quad \text{(Eq. B4.2-3)}
\]

\[
w/t = 13.47 < 90 \quad \text{(Section B1.1-(a)-(1))}
\]

Web:
\[
w = 2.914 \text{ in., } k = 4.00
\]

\[
\lambda = \frac{(1.052/\sqrt{k}) \times (w/t) \sqrt{t/E}, f = F_e}{(1.052/\sqrt{4})(2.914/0.105) \sqrt{23.27/29500}} = 0.410 < 0.673 \quad \text{(Eq. B2.1-1)}
\]

\[
b = w
\]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

= 2.914 in. (web fully effective)

\[ w/t = \frac{2.914}{0.105} = 27.75 < 500 \text{ (Section B1.1-(a)-(2))} \]

Lips:

- \( d = 0.607 \text{ in.} \)
- \( k = 0.43 \) (unstiffened compression element)
- \( d_s = d' \)
- \( d/t = 5.78 < 14 \) (Section B4 of the Commentary)

Since flanges, web, and lips are fully effective

\[ A_e = A_e = 0.889 \text{ in.}^2 \]

15. Determination of \( \phi_c P_n \):

\[ P_n = A_e F_n \]

\[ = 0.889 \times 23.27 \]

\[ = 20.69 \text{ kips} \]

\[ \phi_c = 0.85 \]

\[ \phi_c P_n = 0.85 \times 20.69 = 17.59 \text{ kips} \]
EXAMPLE NO. 11
C-SECTION WITH HOLES

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $3.5 \times 2 \times 0.105$ channel with stiffened flanges.
3. $K_x L_x = K_y L_y = K_t L_t = 6$ ft.
4. Web is perforated with holes for bolts of 1/2-in. diameter (in standard hole) at 4 in. spacing along the height of the column.

Required:
Design axial strength, $\phi_c P_n$

Solution:
1. Basic parameters:
   \[ r = R + t/2 = \frac{3}{16} + 0.105/2 = 0.240 \text{ in.} \]

   From the sketch and Section 1.2.2 of Part III of the Manual, $a = 2.914$ in., $b = 1.414$ in., $c = 0.607$ in.,
\( \alpha = 1.00 \) (Since the section has lips)

\( \tilde{a} = A' \cdot t = 3.5 - 0.105 = 3.395 \text{ in.} \)

\( \bar{b} = B' \cdot t = 2 - 0.105 = 1.895 \text{ in.} \)

\( \bar{c} = C' \cdot t/2 = 0.9 - 0.105/2 = 0.848 \text{ in.} \)

\( u = 1.57 \cdot r = 1.57 \times 0.240 = 0.377 \text{ in.} \)

2. Area:

\[ A = t \left( a + 2b + 2c + 4u \right) \]

\[ = 0.105 \left( 2.914 + 2 \times 1.414 + 2 \times 0.607 + 4 \times 0.377 \right) \]

\[ = 0.889 \text{ in.}^2 \]

3. Moment of inertia about x-axis:

\[ I_x = 2t \left[ 0.0417 \cdot a^3 + b \left( a/2 + r \right)^2 + 2u \left( a/2 + 0.637r \right)^2 + 0.298r^3 + 0.0833 \cdot c^3 + \left( c/4 \right) \left( a - c \right)^2 \right] \]

\[ = 2 \times 0.105 \left[ 0.0417 \cdot (2.914)^3 + 1.414 \cdot (2.914/2 + 0.240)^2 + 2 \times 0.377 \cdot (2.914/2 + 0.637 \times 0.240)^2 + 0.298 \cdot (0.240)^3 + 0.0833 \cdot (0.607)^3 + (0.607/4) \cdot (2.914 - 0.607)^2 \right] \]

\[ = 1.657 \text{ in.}^4 \]

4. Distance from centroid of section to centerline of web:

\[ \bar{x} = \left( 2t/A \right) \left[ b \left( b/2 + r \right) + u \left( 0.363r \right) + u \left( b + 1.637r \right) + c \left( b + 2r \right) \right] \]

\[ = \left( 2 \times 0.105/0.889 \right) \left[ 1.414 \left( 1.414/2 + 0.240 \right) + 0.377 \left( 0.363 \times 0.240 \right) + 0.377 \left( 1.414 + 1.637 \times 0.240 \right) + 0.607 \left( 1.414 + 2 \times 0.240 \right) \right] \]

\[ = 0.757 \text{ in.} \]

5. Moment of inertia about y-axis:

\[ I_y = 2t \left[ b \left( b/2 + r \right)^2 + 0.0833 \cdot b^3 + 0.505r^3 + c \left( b + 2r \right)^2 + u \left( b + 1.637r \right)^2 - A \left( \bar{x} \right)^2 \right] \]

\[ = 2 \times 0.105 \left[ 1.414 \left( 1.414/2 + 0.240 \right)^2 + 0.0833 \left( 1.414 \right)^3 + 0.505 \left( 0.240 \right)^3 + 0.607 \left( 1.414 + 2 \times 0.240 \right)^2 + 0.377 \left( 1.414 + 1.637 \times 0.240 \right)^2 \right] - 0.889 \left( 0.757 \right)^2 \]

\[ = 0.524 \text{ in.}^4 \]

6. Distance from shear center to centerline of web:

\[ m = \left( 5t/12I_x \right) \left[ 6 \bar{c} \left( \bar{a} \right)^2 + 3 \bar{b} \left( \bar{a} \right)^2 - 8 \left( \bar{c} \right)^3 \right] \]

\[ = \left[ (1.895 \times 0.105)/(12 \times 1.657) \right] \left[ 6 \times 0.848 \left( 3.395 \right)^2 + 3 \times 1.895 \left( 3.395 \right)^2 - 8 \left( 0.848 \right)^3 \right] \]

\[ = 1.194 \text{ in.} \]

7. Distance from centroid to shear center:

\[ x_o = -(\bar{x} + m) = -(0.757 + 1.194) \]
IV-94 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

8. St. Venant torsion constant:
\[ J = \left( \frac{t^3}{3} \right) [a + 2b + 2c + 4u] \]
\[ = [0.105]^3/3 \times [2.914 + 2 \times 1.414 + 2 \times 0.607 + 4 \times 0.377] \]
\[ = 0.003266 \text{ in.}^4 \]

9. Warping Constant:
\[ C_w = \left( \frac{t^2}{A} \right) \left\{ \begin{array}{l}
\frac{1}{3} (2 \bar{a}^2 + 3 \bar{a}) - m \left( \frac{\bar{a}}{t} \right) \left[ \frac{m}{2} \left( \bar{a} \right)^2 - 1 + \left( \frac{3 \bar{a}}{4} \right) \left( \frac{\bar{a}}{t} \right) \right] + \left( \frac{5}{3} \bar{a} \right)^2 + \left( \frac{1}{3} \right) \left( \bar{a} \right)^2 \left( \frac{3 \bar{a}}{4} \right) \\
+ \frac{2m}{t} \left( \frac{\bar{a}}{t} \right) \left[ \left( \frac{3 \bar{a}}{4} \right) \left( \frac{\bar{a}}{t} \right) - \left( \frac{\bar{a}}{t} \right)^2 \right] \left( \frac{3 \bar{a}}{4} \right) + \left( \frac{\bar{a}}{t} \right)^2 \left( \frac{3 \bar{a}}{4} \right)
\end{array} \right\} \]
\[ = [0.105]^2/0.889 \left\{ [0.757 \times 0.889 \times (3.395)^2 / 0.105] [1.895]^2/3 + (1.194)^2 - 1.194 \times 2 \times 1.895 + 0.889 / (3 \times 0.105) [1.895]^2 (3.395) \right. \]
\[ + (1.951)^2 - 1.951 / 2 \times 1.895 + 2 \times 1.194 \times 1.895 + 1.951 / 2 \times 1.895 - (1.895)^2/3 \]
\[ + (0.848)^2/6 \left[ (3 \times 0.848 + 1.895) / (2 \times 0.848 + 3 \times 3.395) \right] \]
\[ + (0.848)^2 \left[ (1.194)^2 (3.395)^2 / 4 \right] \}
\[ = 2.050 \text{ in.}^6 \]

10. Radii of gyration:
\[ r_x = \sqrt{(I_x / A)} = \sqrt{(1.657 / 0.889)} = 1.365 \text{ in.} \]
\[ r_y = \sqrt{(I_y / A)} = \sqrt{(0.524 / 0.889)} = 0.768 \text{ in.} \]
\[ (K_y r_y) / r_y = (6 \times 12) / 0.768 = 93.75 < 200 \]
\[ r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{(1.365)^2 + (0.768)^2 + (-1.951)^2} \]
\[ = 2.502 \text{ in.}^2 \]

11. Torsional-flexural constant:
\[ \beta = 1 - (x_o / r_o)^2 \quad \text{(Eq. C4.2-3)} \]
\[ = 1 - (-1.951 / 2.502)^2 \]
\[ = 0.392 \]

12. Determination of \( F_e \):
For this singly symmetric section (x-axis is the axis of symmetry), \( F_e \) shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):
\[ (F_e)_1 = \left( \frac{\pi^2 E}{K_y r_y} \right)^2 \quad \text{(Eq. C4.1-1)} \]
\[ = (\pi^2 \times 29500) / (93.75)^2 = 33.13 \text{ ksi} \]
\[ \sigma_{ex} = \frac{(\pi^2 E)}{(K x L x/r x)^2} \]  
\[ = \frac{(\pi^2 \times 29500)}{(6x12/1.365)^2} = 104.65 \text{ ksi} \]  
\[ \sigma_t = \frac{1}{(A_r^2)^{\frac{1}{6}}} \]  
\[ \frac{GJ}{+\left(\pi^2 E C_w\right)}(K x L x)^2 \]  
\[ = \frac{1}{(0.889x2.502)^{\frac{1}{6}}}[11300x0.003266+(\pi^2 x 29500x2.050)/(6x12)^2] \]  
\[ = 27.32 \text{ ksi}. \]

\[ (F_e) = \left(\frac{1}{2\beta}\right)[(\sigma_{ex} + \sigma_t) - \sqrt{\sigma_{ex} + \sigma_t^2 - 4\beta\sigma_{ex}\sigma_t}] \]  
\[ = \frac{1}{(2x0.392)}[(104.65+27.32) - \sqrt{(104.65+27.32)^2 - 4x0.392x104.65x27.32}] \]  
\[ = 23.27 \text{ ksi}. \]

\[ F_e = 23.27 \text{ ksi}. \]

13. Determination of \( F_n \):
\[ F_y/2 = 50/2 = 25.00 \text{ ksi} \]

For \( F_e < F_y/2 \)
\[ F_n = F_e \]  
\[ = 23.27 \text{ ksi}. \]  

14. Determination of \( A_c \):
Flanges:
\[ d = 0.607 \text{ in.} \]
\[ I_s = d^3 t/12 = (0.607)^3(0.105)/12 = 0.001957 \text{ in.}^4 \]
\[ D = 0.9 \text{ in.} \]
\[ w = 1.414 \text{ in.} \]
\[ D/w = 0.9/1.414 = 0.636 < 0.80 \]
\[ S = 1.28 \sqrt{E/f}, \]  
\[ f = F_n \]  
\[ = 1.28 \sqrt{29500/23.27} = 45.57 \]  
\[ w/t = 1.414/0.105 = 13.47 < S/3 = 15.19 \]  
\[ I_s = 0 \] (no edge stiffener needed)
\[ b = w \]  
\[ = 1.414 \text{ in.} \] (flanges fully effective)
\[ w/t = 13.47 < 90 \] (Section B1.1-(a)-(1))

Web:
\[ w = 2.914 \text{ in.}, k = 4.00 \]
\[ \lambda = (1.052/\sqrt{k})(w/t) \sqrt{E/f}, \]  
\[ f = F_n \]  
\[ = (1.052/\sqrt{4})(2.914/0.105) \sqrt{23.27/29500} \]  
\[ = 0.410 < 0.673 \]  
\[ d_h = d+1/16 \]  
\[ = 0.607+1/16 \]  
\[ = 0.645 \text{ in.} \]

(Table E3)
\[ d = \text{diameter of bolt} = 0.5 \text{ in.} \]
\[ d_h = 0.5 + 1/16 = 0.563 \text{ in.} \]

Number of holes in the effective length = 17
\[ (17 \times 0.563)/(6 \times 12) = 0.133 > 0.015 \text{ then } A_e \text{ must be determined with holes accounted for. (Section C4-(a))} \]
\[ d_h/w = 0.563/2.914 = 0.193 < 0.50 \]
\[ w/t = 2.914/0.105 = 27.75 < 70 \]
\[ 0.5w = 0.5 \times 2.914 = 1.457 \text{ in.} \]
\[ 3d_h = 3 \times 0.563 = 1.689 \text{ in.} \]

Spacing of holes = 4 in. greater than 0.5w and 3d_h

Effective width, b, shall be determined by using (Eq. B2.2-1)
\[ b = w - d_h \]
\[ = 2.914 - 0.563 \]
\[ = 2.351 \text{ in.} \]
\[ w/t = 27.75 < 500 \text{ (Section B1.1-(a)-(2))} \]

Lips:
\[ d = w \]
\[ = 0.607 \text{ in.} \]
\[ k = 0.43 \text{ (unstiffened compression element)} \]
\[ \lambda = (1.052/\sqrt{0.43})(0.607/0.105) \sqrt{23.27/29500} \]
\[ = 0.260 < 0.673 \]
\[ d' = d = 0.607 \text{ in.} \]
\[ d_s = d' \]
\[ = 0.607 \text{ in. (No reduction in lips area)} \]
\[ w/t = 0.607/0.105 = 5.78 < 14 \text{ (Section B4 of the Commentary)} \]
\[ A_e = A - t(w - b)_{web} \]
\[ = 0.889 - 0.105(2.914 - 2.351) \]
\[ = 0.830 \text{ in.}^2 \]

15. Determination of \( \phi c P_n \):
\[ P_n = A_eF_n \]
\[ = 0.830 \times 23.27 \]
\[ = 19.31 \text{ kips} \]
\[ \phi = 0.85 \]
\[ \phi c P_n = 0.85 \times 19.31 \]
\[ = 16.41 \text{ kips} \]
EXAMPLE NO. 12
C-SECTION WITH WIDE FLANGE

Given:
1. Steel: \( F_y = 50 \text{ ksi} \).
2. Section: 3.5 x 3.5 x 0.105 channel with stiffened flanges.
3. \( K_x L_x = K_y L_y = K_t L_t = 6 \text{ ft} \).

Required:
Design axial strength, \( f_r P_n \).

Solution:
1. Basic parameters:
\[ r = R + t/2 = 3/16 + 0.105/2 = 0.240 \text{ in}. \]

From the sketch and Section 1.2.2 of Part III of the Manual, \( a = 2.914 \text{ in.}, b = 2.914 \text{ in.}, c = 0.607 \text{ in.} \),
\[ \alpha = 1.00 \] (Since the section has lips)
\[ \bar{a} = A' - t = 3.5 - 0.105 = 3.395 \text{ in.} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ B = B' - t = 3.5 - 0.105 = 3.395 \text{ in.} \]
\[ C = C' - t/2 = 0.9 - 0.105/2 = 0.848 \text{ in.} \]
\[ u = 1.57r = 1.57 \times 0.240 = 0.377 \text{ in.} \]

2. Area:
\[ A = t[a + 2b + 2c + 4u] \]
\[ = 0.105[2.914 + 2 \times 2.914 + 2 \times 0.607 + 4 \times 0.377] \]
\[ = 1.204 \text{ in.}^2 \]

3. Moment of inertia about x-axis:
\[ I_x = 2t\left[0.0417a^3 + b(a/2+r)^2 + 2u(a/2+0.637r)^2 + 0.298r^3 + 0.0833c^3 + (c/4)(a-c)^2\right] \]
\[ = 2 \times 0.105\left[0.0417(2.914)^3 + 2.914(2.914/2+0.240)^2 + 0.377(2.914/2+0.637 \times 0.240)^2 + 0.0833(0.607)^3 + (0.607/4)(2.914-0.607)^2\right] \]
\[ = 2.564 \text{ in.}^4 \]

4. Distance from centroid of section to centerline of web:
\[ \bar{x} = (2t/A)[b(b/2+r)+u(0.363r)+u(b+1.637r)+c(b+2r)] \]
\[ = [(2 \times 0.105)/1.204][2.914(2.914/2+0.240)+0.377(0.363 \times 0.240) + 0.377(2.914+1.637 \times 0.240)+0.607(2.914+2 \times 0.240)] \]
\[ = 1.445 \text{ in.} \]

5. Moment of inertia about y-axis:
\[ I_y = 2t[b(b/2+r)^2 + 0.0833b^3 + 0.505r^3 + c(b+2r)^2 + u(b+1.637r)^3 - A(\bar{x})^3] \]
\[ = 2 \times 0.105[2.914(2.914/2+0.240)^2 + 0.0833(2.914)^3 + 0.505(0.240)^3 + 0.607(2.914+2 \times 0.240)^2 + 0.377(2.914+1.637 \times 0.240)^2 - 1.204(1.445)^3] \]
\[ = 2.017 \text{ in.}^4 \]

6. Distance from shear center to centerline of web:
\[ m = (5t/12I_x)[6C(\bar{a})^2 + 3B(\bar{a})^2 - 8(\bar{c})^3] \]
\[ = [(3.395 \times 0.105)/(12 \times 2.564)]\times 0.848(3.395)^2 + 3 \times 3.395(3.395)^2 - 8(0.848)^3 \]
\[ = 1.983 \text{ in.} \]

7. Distance from centroid to shear center:
\[ x_o = -(\bar{x} + m) = -(1.445 + 1.983) = -3.428 \text{ in.} \]

8. St. Venant torsion constant:
\[ J = (t^3/3)[a + 2b + 2c + 4u] \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[
= \left[ (0.105)^2 / 3 \right] [2.914 + 2 \times 2.914 + 2 \times 0.607 + 4 \times 0.377]
= 0.004424 \text{ in.}^4
\]

9. Warping Constant:
\[
C_w = \left( \frac{t}{A} \right) \left[ \bar{x} A (\bar{a})^2 / A \right] + \left( \frac{b}{b} \right)^2 \left( \bar{c} + 3 \bar{a} \right) - \left( \frac{t}{t} \right) \left( m / t \right) (2 \bar{a} + 4 \bar{a}) + \left( \frac{m}{m} \right) \left( \bar{c}^2 / 3 \right) + \left( \frac{m}{m} \right) \left( \bar{a}^2 / 6 \right) + \left( \frac{m}{m} \right) \left( \bar{c} + \bar{b} \right) (4 \bar{c} + \bar{a}) - 6 (\bar{c}^2)
- \left[ m^2 / (\bar{a}^4 / 4) \right]
\]
\[
= \left[ (0.105)^2 / 1.204 \right] \left[ (1.445 \times 1.204 \times (3.395)^2 / 0.105) \right] (3.395)^2 / 3
+(1.983)^2 - 1.983 \times 3.395) + 1.204 / (3 \times 0.105) \left[ (1.983)^2 (3.395)^3
+(3.395)^2 (0.848)^2 (2 \times 0.848 + 3 \times 3.395) \right]
- \left[ 2.564 \times (1.983)^2 / 0.105 \right] (2 \times 3.395 + 4 \times 0.848)
+ \left[ 1.983 (0.848)^2 / 3 \right] (8 (3.395)^2 (0.848)
+2 \times 1.983 \times (2 \times 0.848 + 3.395) + 3.395 \times (2 \times 0.848 + 3 \times 3.395))
+ \left[ (3.395)^2 (3.395)^2 / 6 \right] (3 \times 0.848 + 3.395) (4 \times 0.848 + 3.395)
-6 (0.848)^2 \right] - \left[ (1.983)^2 (3.395)^4 / 4 \right]
= 7.572 \text{ in.}^6
\]

10. Radii of gyration:
\[
x = \sqrt{\frac{I}{A}} = \sqrt{(2.564 / 1.204)} = 1.459 \text{ in.}
\]
\[
y = \sqrt{\frac{I}{A}} = \sqrt{(2.017 / 1.204)} = 1.294 \text{ in.}
\]
\[
(K_y L_y) / r_y = (6 \times 12) / 1.294 = 55.64 < 200
\]
\[
r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{(1.459)^2 + (1.294)^2 + (-3.428)^2}
= 3.944 \text{ in.}^2
\]

11. Torsional-flexural constant:
\[
\beta = 1 - (x_o / r_o)^2 \quad \text{ (Eq. C4.2-3)}
\]
\[
= 1 - (-3.428 / 3.944)^2
= 0.244
\]

12. Determination of \( F_e \):
For this singly symmetric section (x-axis is the axis of symmetry), \( F_e \) shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):
\[
(F_e)_1 = \frac{\pi^2 E}{(K_y L_y / r_y)^2}
= \frac{(\pi^2 \times 29500)}{(55.64)^2} = 94.05 \text{ ksi}
\]
\[
\sigma_{w} = \frac{\pi^2 E}{(K_y L_y / r_o)^2}
= \frac{(\pi^2 \times 29500)}{(6 \times 12 / 1.459)^2} = 119.55 \text{ ksi}
\]
\[
\sigma_t = 1 / (A_o^2) \left[ G + (\pi^2 E C_w) / (K_t L_t)^2 \right]
= 1 / (1.204 \times 3.944) \left[ 11300 \times 0.004424 + (\pi^2 \times 29500 \times 7.572) / (6 \times 12)^2 \right]
\]
= 25.38 ksi.

\[(F_e)_2 = (1/2\beta)[(\sigma_{ex} + \sigma_{s}) - \sqrt{(\sigma_{ex} + \sigma_{i})^2 - 4\beta \sigma_{ex} \sigma_{i}})]\]  
\[(\text{Eq. C4.2-1})\]

\[= 1/(2x0.244) \{ (119.55+25.38) -\sqrt{(119.55 + 25.38)^2 - 4x0.244x119.55x25.38} \}\]
\[= 21.73 \text{ ksi} \]

\[F_e = 21.73 \text{ ksi}\]

13. Determination of \(F_n\):

\[F_y/2 = 50/2 = 25.00 \text{ ksi}\]

For \(F_s < F_y/2\)

\[F_n = F_e \]
\[= 21.73 \text{ ksi}\]

\[(\text{Eq. C4-3})\]

14. Determination of \(A_c\):

Flanges:

\[d = 0.607 \text{ in.}\]
\[I_s = d^4/12 = (0.607)^4(0.105)/12 \]
\[= 0.001957 \text{ in.}^4\]

\[D = 0.9 \text{ in.}\]
\[w = 2.914 \text{ in.} \text{ (for flange)}\]
\[D/w = 0.9/2.914 = 0.309 < 0.80\]

\[S = 1.28 \sqrt{E/f}, f = F_n \]
\[= 1.28 \sqrt{29500/21.73} = 47.16, S/3 = 15.72\]

\[w/t = 2.914/0.105 = 27.75\]

\[S/3 < w/t < S\]

\[I_a = 399t^4 \left[ \left( \frac{w}{t} / S \right) -0.33 \right]^3\]  
\[(\text{Eq. B4.2-6})\]

\[= 399(0.105)^4[(27.75/47.16)-0.33]^3\]
\[= 0.000837 \text{ in.}^4 < I_s = 0.001957 \text{ in.}^4\]

\[C_1 = 2-(I_s/I_a) \geq 1.0\]  
\[= 2-(0.001957/0.000837) = -0.34 < 1.0\]  
\[(\text{Eq. B4.2-8})\]

\[C_1 = 1.0\]

\[C_2 = I_s/I_a \leq 1.0\]  
\[(\text{Eq. B4.2-7})\]

\[I_s/I_a = (0.001957/0.000837) = 2.34 > 1.0\]

\[C_2 = 1.0\]

\[0.25 < D/w = 0.309 < 0.8\]

\[k = [4.82-5(D/w)](I_s/I_a)^{0.43} \leq 5.25-5(D/w)\]  
\[(\text{Eq. B4.2-9})\]

\[n = 1/2\]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[
[4.82 - 5(0.309)(0.001957/0.000837)^2]^{0.43} + 0.43 = 5.438
\]

\[
5.25 - 5(0.309) = 3.705 < 5.438
\]

\[
k = 3.705
\]

\[
\lambda = (1.052/\sqrt{4})(w/t) \sqrt{E_f}, f = F_n
\]

\[
= (1.052/\sqrt{3.705})(27.75) \sqrt{21.73/29500} = 0.412 < 0.673
\]

\[
b = w = 2.914 \text{ in. (flanges fully effective)}
\]

\[
w/t = 27.75 < 90 \text{ (Section B1.1-(a)-(1))}
\]

Web:

\[
w = 2.914 \text{ in., } k = 4.00
\]

\[
\lambda = (1.052/\sqrt{4})(2.914/0.105) \sqrt{21.73/29500}
\]

\[
= 0.396 < 0.673
\]

\[
b = w = 2.914 \text{ in. (web fully effective)}
\]

\[
w/t = 2.914/0.105 = 27.75 < 500 \text{ (Section B1.1-(a)-(2))}
\]

Lips:

\[
d = 0.607 \text{ in.}
\]

\[
k = 0.43 \text{ (unstiffened compression element)}
\]

\[
\lambda = (1.052/\sqrt{0.43})(0.607/0.105) \sqrt{21.73/29500}
\]

\[
= 0.252 < 0.673
\]

\[
d'_s = d = 0.607 \text{ in.}
\]

\[
d_s = d'_s(l/l_w) \leq d'_s
\]

\[
= 0.607(2.34) = 1.420 > d'_s = 0.607 \text{ in.}
\]

\[
d_s = 0.607 \text{ in. (Lip fully effective in computing the overall effective area)}
\]

\[
d/t = 5.78 < 14 \text{ (Section B4 of the Commentary)}
\]

Since flanges, web, and lips are fully effective

\[
A_e = A = 1.204 \text{ in.}^2
\]

15. Determination of \(\phi_c P_n\):

\[
P_n = A_e F_n
\]

\[
= 1.204 \times 21.73
\]

\[
= 26.16 \text{ kips}
\]

\[
\phi_c = 0.85
\]

\[
\phi_c P_n = 0.85 \times 26.16
\]

\[
= 22.24 \text{ kips}
\]
EXAMPLE NO. 13
TUBULAR SECTION - SQUARE

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $4 \times 4 \times 0.065$ Square Tube.
3. $K_x L_x = K_y L_y = 10$ ft.

Required:
Design axial strength, $\phi P_n$.

Solution:
1. Properties of 90° corners:
   $r = R + t/2 = 1/16 + 0.065/2 = 0.095$ in.
   Length of arc, $u = 1.57r = 1.57 \times 0.095 = 0.149$ in.
   Distance of c.g. from center of radius,
   $c = 0.637r = 0.637 \times 0.095 = 0.061$ in.
I_x = I_y = I (doubly symmetric section)

<table>
<thead>
<tr>
<th>Element</th>
<th>(in.)</th>
<th>y Distance to Center of Section (in.)</th>
<th>Ly^2 (in.^2)</th>
<th>I' About Own Axis (in.^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges</td>
<td>2 x 3.744 = 7.488</td>
<td>2 - 0.065/2 = 1.968</td>
<td>29.001</td>
<td>—</td>
</tr>
<tr>
<td>Corners</td>
<td>4 x 0.149 = 0.596</td>
<td>3.744/2+0.061 = 1.933</td>
<td>2.227</td>
<td>—</td>
</tr>
<tr>
<td>Web</td>
<td>2 x 3.744 = 7.488</td>
<td>—</td>
<td>—</td>
<td>8.747</td>
</tr>
<tr>
<td>Sum</td>
<td>15.572</td>
<td>31.228</td>
<td>8.747</td>
<td></td>
</tr>
</tbody>
</table>

w/t = 3.744/0.065 = 57.60 < 500 (Section B1.1-(a)-(2))

A = Lt = 15.572 x 0.065 = 1.012 in.^2

I' = Ly^2 + I'_1 = 31.228 + 8.747 = 39.975 in.^3

I = I't = 39.975 x 0.065 = 2.598 in.^4

r = √(I/A) = √(2.598/1.012) = 1.602 in.

KL/r = 10 x 12/1.602 = 74.91 < 200 (Section C4-(d))

2. Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

\[ F_e = \pi^2 E/(KL/r)^2 \]  
\[ = \pi^2 x 29500/(74.91)^2 = 51.89 \text{ ksi} \]  

\[ F_y/2 = 50/2 = 25.00 \text{ ksi} \]

For \( F_e > F_y/2 \):

\[ F_n = F_e (1-F_e/4F_y) \]  
\[ = 50(1-50/(4x51.89)) = 37.96 \text{ ksi} \]

3. \( \lambda = (1.052/\sqrt{k})(w/t) \sqrt{k/E}, \ f = F_n \)  
\[ = (1.052/\sqrt{4})(3.744/0.065)^{\sqrt{29500/37.96/29500}} = 1.087 > 0.673 \]  
(Section not fully effective)

\[ \rho = (1-0.22/\lambda)/\lambda \]  
\[ = (1-0.22/1.087)/1.087 = 0.734 \]

\[ b = \rho w \]  
\[ = 0.734 x 3.744 = 2.748 \text{ in.} \]

\[ A_e = A-4(w-b) \]  
\[ = 1.012-4(3.744-2.748)x0.065 \]  
\[ = 0.753 \text{ in.}^2 \]

4. \( P_n = A_e F_n \)  
\[ (Eq. C4-1) \]
\[ = 0.753 \times 37.96 = 28.58 \text{ kips} \]
\[ \phi_c = 0.85 \]
\[ \phi_c P_n = 0.85 \times 28.58 = 24.29 \text{ kips} \]
EXAMPLE NO. 14
TUBULAR SECTION - ROUND

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.
3. Height: $L = 10'-0''$, simply supported at each end.

Required:
Design axial strength, $\phi P_n$.

Solution:

Ratio of outside diameter to wall thickness,

$$\frac{D}{t} = \frac{8.000}{0.125} = 64.00$$

$\frac{D}{t} < 0.441E/F_y = 0.441(29500/50) = 260.2$ OK

$$F_e = \frac{\pi^2E/(KL)^2}{(\text{Eq. C4.1-1})}$$

$$I = \frac{1}{4}\pi\left[(\text{O.R.})^4-(\text{I.R.})^4\right]$$
$$= \frac{1}{4}\pi\left[(4)^4-(3.875)^4\right]$$
$$= 23.98 \text{ in.}^4$$

$$A = \frac{1}{4}\pi\left[(\text{O.D.})^2-(\text{I.D.})^2\right]$$
$$= \frac{1}{4}\pi\left[(8)^2-(7.75)^2\right]$$
$$= 3.093 \text{ in.}^2$$

$$r = \sqrt[4]{\frac{I}{A}}$$
$$= \sqrt[4]{23.98/3.093} = 2.784 \text{ in.}$$

$$F_e = \frac{\pi^2(29500)[10(12)/2.784]^2}{156.71 \text{ ksi}}$$
Since $F_e > F_y/2$

\[ F_a = F_y(1-F_y/4F_e) \quad (Eq. \text{C6.2-2}) \]
\[ = 50(1-50/(4\times156.71)) = 46.01 \text{ ksi} \]

\[ A_o = \frac{0.037}{(Df_y/\mu) + 0.667}A \leq A \quad (Eq. \text{C6.2-5}) \]
\[ = \frac{0.037}{(8\times50/(0.125\times29500)) + 0.667}(3.093) \]
\[ = 3.118 \text{ in.}^2 \]

Therefore, $A_o = A = 3.093 \text{ in.}^2$

\[ A_e = \frac{1-(1-R^2)(1-A_o/A)}{A} \quad (Eq. \text{C6.2-3}) \]

Since $A_y/A = 1$, $A_e = A$

\[ A_e = 3.093 \text{ in.}^2 \]

\[ P_n = F_aA_e \quad (Eq. \text{C6.2-1}) \]
\[ = (46.01)(3.093) \]
\[ = 142.31 \text{ kips} \]

\[ \phi_e = 0.85 \]

\[ \phi_eP_n = 0.85 \times 142.31 = 120.96 \text{ kips} \]
EXAMPLE NO. 15
C-SECTION

Given:
1. Steel: \( F_y = 50 \text{ ksi} \).
2. Section: Channel as shown.
3. Length of Section = 16 ft.
4. \( L_x = L_y = L_t = 16 \text{ ft} \).
5. \( K_x = K_y = K_t = 1.0 \).
6. Axial loads: \( P_{DL} = 0.4 \text{ kips}, P_{LL} = 2.0 \text{ kips} \).
7. Eccentricity at both ends:
   (a) Axial loads are applied 2 in. to the left of the c.g. of the full section.
   (b) Axial loads are applied 2 in. to the left and 4 in. above the c.g. of the full section.
Required:

Check the adequacy of the given section for both cases.

Solution: Part (a)

1. Full section properties:

   - Full section properties:
     
     \[
     \begin{align*}
     r &= R + t/2 = \frac{3}{16} + 0.105/2 = 0.240 \text{ in.} \\
     a &= A' - (2r + t) = 8.000 - (2 \times 0.240 + 0.105) = 7.415 \text{ in.} \\
     \bar{a} &= A' - t = 8.000 - 0.105 = 7.895 \text{ in.} \\
     b &= B' - (2r + t) = 3.000 - (2 \times 0.240 + 0.105) = 2.415 \text{ in.} \\
     \bar{b} &= B' - t = 3.000 - 0.105 = 2.895 \text{ in.} \\
     c &= C' - (r + t/2) = 0.800 - (0.240 + 0.105/2) = 0.508 \text{ in.} \\
     \bar{c} &= C' - t/2 = 0.800 - (0.105/2) = 0.748 \text{ in.} \\
     u &= 1.57r = 1.57 \times 0.240 = 0.377 \text{ in.}
     \end{align*}
     \]

   Distance of corner's c.g. from center of radius = 0.637 \( r = 0.637(0.240) = 0.153 \) in.

   \[
   \begin{align*}
   A &= t[a + 2b + 2c + 4u] = 0.105[7.415 + 2 \times 2.415 + 2 \times 0.508 + 4 \times 0.377] \\
   &= 1.551 \text{ in.}^2 \\
   I_x &= 2t[0.0417a^3 + b(a/2 + r)^2 + 2u(a/2 + 0.637r)^2 + 0.298r^3 \\
   &+ 0.0833c^3 + (c/4)(a - c)^2] \\
   &= 2 \times 0.105[0.0417(7.415)^3 + 2.415(7.415/2 + 0.240)^2 + 2 \times 0.377(7.415/2 + 0.637 \times 0.240)^3 + 0.0833(0.508)^3 + (0.508/4)(7.415 - 0.508)^2] \\
   &= 15.108 \text{ in.}^4 \\
   \bar{x} &= (2t/A)[b(b/2 + r) + u(0.363r) + u(b + 1.637r) + c(b + 2r)] \\
   &= (2 \times 0.105/1.551)[2.415(2.415/2 + 0.240) + 0.377(0.363 \times 0.240) + 0.377(2.415 + 1.637 \times 0.240) + 0.508(2.415 + 2 \times 0.240)] \\
   &= 0.820 \text{ in.} \\
   I_y &= 2t[b(b/2 + r)^2 + 0.0833b^3 + 0.505r^3 + c(b + 2r)^2 \\
   &+ u(b + 1.637r)^2] - A(\bar{x})^2 \\
   &= 2 \times 0.105[2.415(2.415/2 + 0.240)^2 + 0.0833(2.415)^3 + 0.505(0.240)^3 + 0.508(2.415 + 2 \times 0.240)^2 + 0.377(2.415 + 1.637 \times 0.240) - 1.551(0.820)^2] \\
   &= 1.786 \text{ in.}^4 \\
   m &= (5t/12)[6c(\bar{a})^3 + 3b(\bar{a})^3 - 8(\bar{c})^3] \\
   &= [(2.895 \times 0.105)/(12 \times 15.108)][6 \times 0.748(7.895)^2 + 3 \times 2.895(7.895)^2 - 8(0.748)^3] \\
   &= 1.371 \text{ in.}
   \end{align*}
   \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ x_0 = -(\bar{x} + m) = -(0.820 + 1.371) \]
\[ = -2.191 \text{ in.} \]

\[ J = (t^3/3)[a + 2b + 2c + 4u] \]
\[ = [(0.105)^3/3][7.415 + 2x2.415 + 2x0.508 + 4x0.377] \]
\[ = 0.005699 \text{ in.}^4 \]

\[ C_w = (t^3/A)[(\bar{x}A(\bar{a})^2/2)[(B^2/3 + m^2 - m A/A/3t)][(m^2/\bar{a})^3 \]
\[ + (B^2/\bar{a}^2)(2c + 3\bar{a})] - (L_x m^2/2)(2\bar{a} + 4\bar{c}) + [m(\bar{c}^2/3)][(8B^2/\bar{c})^2 + 2m(2\bar{c}(\bar{c} - a) + B(2\bar{c} - 3\bar{a})) + [(B^2/\bar{a}^3)/6][3c + B(4\bar{c} + \bar{a}) - 6(\bar{c})^2] \]
\[ - [m^2(\bar{a})^4/4]] \]
\[ = [(0.105)^2/1.551][[(0.820x1.551x(7.895)^2/0.105][{(2.895)^2/3} \]
\[ + (1.371)^2 - 1.371x2.895 + 1.551/(3x0.105)]((1.371)^2/7.895)] \]
\[ + (2.895)^2(0.748)^2(2x0.748 + 3x7.895)] \]
\[ - [15.108x(1.371)^2/0.105)(2x7.895 + 4x0.748) \]
\[ + [1.371(0.748)^2/3][8(2.895)^2(0.748) \]
\[ + 2x1.371(2x0.748 + 3x7.895 + 2.895(2x0.748 - 3x7.895))] \]
\[ + [(2.895)^2(7.895)^2/6][(3x0.748 + 2.895)(4x0.748 + 7.895) \]
\[ - 6(0.748)^3 - [(1.371)^2(7.895)^4/4]] \]
\[ = 23.468 \text{ in.}^6 \]

\[ \beta_w = - |0.0833[t \bar{x}(\bar{a})^3 + t(\bar{a})^3] \]
\[ = - |0.0833[0.105x0.820(7.895)^3] + 0.105(0.820)^3x7.895] \]
\[ = -3.987 \]

\[ B_f = (t/2)[(B - \bar{x})^4 - (\bar{x})^4] + [t(\bar{a})^2/4][(B - \bar{x})^2 - (\bar{x})^2] \]
\[ = (0.105/2)((2.895-0.820)^4 - (0.820)^4) \]
\[ + [0.105(7.895)^2/4][(2.895-0.820)^2 - (0.820)^2] \]
\[ = 6.894 \]

\[ \beta_h = 2\bar{c}(B - \bar{x})3 + (2/3)t(B - \bar{x})[(\bar{a}/2)^3 - (\bar{a}/2 - \bar{c})^3] \]
\[ = 2x0.748x0.105(2.895 - 0.820)^3 + (2/3)x0.105(2.895 \]
\[ - 0.820)(7.895/2)^3 - [(7.895/2) - 0.748]^3] \]
\[ = 5.581 \]

\[ J = (1/2I_y)(\beta_w + \beta_f + \beta_h)x_o \quad (\text{Eq. C3.1.2-11}) \]
\[ = [1/(2x1.786)](-3.987 + 6.894 + 5.581) - (-2.191) \]
\[ = 4.567 \]

\[ r_x = \sqrt{I_y/A} = \sqrt{15.108/1.551} = 3.121 \text{ in.} \]
\[ K_{xL_r} = [1(16x12)]/3.121 = 61.52 \]

\[ r_y = \sqrt{I_y/A} = \sqrt{1.786/1.551} = 1.073 \text{ in.} \]
\[ K_{yL_r} = [1(16x12)]/1.073 = 178.94 < 200 \text{ (Section C4-(d))} \]
IV-110  Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ r_0 = \sqrt{r_{x_0}^2 + r_{y_0}^2 + x_0^2} \]  
\[ = \sqrt{(3.121)^2 + (1.073)^2 + (-2.191)^2} = 3.961 \text{ in.} \]  
\[ \beta = 1 - \frac{x_0}{r_0}^2 \]  
\[ = 1 - \frac{-2.191}{3.961}^2 = 0.694 \]  

2. Determination of \( \phi_Pn \) (Section C4):

Since the channel is singly symmetric, \( F_n \) shall be taken as the smaller of \( F_n \) calculated according to Section C4.1 or \( F_n \) calculated according to Section C4.2.

Section C4.1:

\[ (F_{n1}) = \frac{(\pi^2E)}{(K_yL_y/r_y)^2} \]  
\[ = \frac{(\pi^2\times29500)}{(178.94)^2} = 9.093 \text{ ksi} \]  

Section C4.2:

\[ \sigma_{e1} = \frac{(\pi^2E)}{(K_xL_x/r_x)^2} \]  
\[ = \frac{(\pi^2\times29500)}{(116\times12/3.121)^2} = 76.93 \text{ ksi} \]  
\[ \sigma_1 = 1/\{Ar_o^2 \left[ GJ + (\pi^2EC_w)/(K_xL_x)^2 \right] \} \]  
\[ = [1/1.551(3.961)^2][11300\times0.005699] \]  
\[ + (\pi^2\times29500\times23.468)/[(1\times16\times12)^2] \]  
\[ = 10.26 \text{ ksi.} \]  
\[ (F_{n2}) = \frac{(1/2\beta)((\sigma_{e1} + \sigma_1) - \sqrt{(\sigma_{e1} + \sigma_1)^2 - 4\beta\sigma_{e1}\sigma_1})}{(1/2x0.694)[(76.93+10.26)]} \]  
\[ = \frac{-\sqrt{76.93+10.26}^2 - 4x0.694\times76.93\times10.26}{(76.93+10.26)^2} \]  
\[ = 9.820 \text{ ksi} \]  

Therefore

\[ F_n = 9.093 \text{ ksi} \]  
\[ F_{y/2} = 50/2 = 25.00 \text{ ksi} \]  

Since \( F_n < F_{y/2} \) it follows that

\[ F_n = F_e \]  
\[ = 9.093 \text{ ksi.} \]  

For element 1:

\[ w = 7.415 \text{ in.} \]  
\[ w/t = 7.415/0.105 = 70.62 < 500 \text{ OK (Section B1.1-(a)-(2))} \]  
\[ k = 4.0 \text{ (Since connected to two stiffened elements)} \]  
\[ \lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}, \ f = F_n \]  
\[ = (1.052/\sqrt{4})(70.62)\sqrt{9.093/29500} \]  
\[ = 0.652 < 0.673 \]  
\[ b = w \]  
\[ \text{(Eq. B2.1-4)} \]  
\[ \text{(Eq. B2.1-1)} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

= 7.415 in. (Element 1 fully effective)

For element 2:
\[ w = 2.415 \text{ in.} \]
\[ w/t = 2.415/0.105 = 23.00 \]
\[ S = 1.28 \sqrt{\frac{E}{f}}, \quad f = F_n \quad \text{ (Eq. B4-1)} \]
\[ = 1.28 \sqrt{29500/9.093} = 72.91 \]
\[ w/t = 23.00 < S/3 = 24.30 \]
\[ b = w \quad \text{ (Eq. B4.2-3)} \]
\[ = 2.415 \text{ in. (Element 2 fully effective)} \]

For element 3:
\[ d = 0.508 \text{ in.} \]
\[ d/t = 0.508/0.105 = 4.84 < 14 \text{ OK (Section B4 of the Commentary)} \]
\[ k = 0.43 \text{ (unstiffened compression element)} \]
\[ \lambda = \frac{1.052/ \sqrt{0.43}(4.84) \sqrt{9.093/29500}}{1.052/ 0.43 } = 0.136 < 0.673 \]
\[ d_s' = d = 0.508 \text{ in.} \]
\[ d_s = d_s' \quad \text{ (Eq. B4.2-4)} \]
\[ = 0.508 \text{ in. (Element 3 fully effective)} \]

Thus the whole section is fully effective.

\[ A_e = A = 1.551 \text{ in.}^2 \]
\[ P_n = A_e F_n \]
\[ = 1.551 \times 9.093 \]
\[ = 14.10 \text{ kips} \]
\[ \phi_c = 0.85 \]
\[ \phi_c P_n = 0.85 \times 14.10 \]
\[ = 11.985 \text{ kips} \]

3. \[ P_u = 1.2 \times 0.4 + 1.6 \times 2.0 = 3.68 \text{ kips} \]
\[ P_u/\phi_c P_n = 3.68/11.985 = 0.307 > 0.15 \]

Must check both interaction equations (Eq. C5-1), (Eq. C5-2)

4. Determination of \[ \phi_c P_{no} \] (Section C4 for \( F_n = F_y \)):

For element 1:
\[ \lambda = \frac{1.052/ \sqrt{4.00} (70.62)/\sqrt{50/29500}}{1.529} = 1.529 > 0.673 \quad \text{ (Eq. B2.1-3)} \]
\[ \rho = \frac{1-0.22/\lambda}{\lambda} \]
\[ = (1-0.22/1.529)/1.529 = 0.560 \]
\[ b = \rho w \quad \text{ (Eq. B2.1-2)} \]
\[ = 0.560 \times 7.415 = 4.152 \text{ in.} \]
For element 2:

\[ S = 1.28 \sqrt{29500/50} = 31.09 \]
\[ S/3 = 10.36 \]
\[ S/3 < w/t = 23.00 < S = 31.09 \]
\[ I_a = 399t^4 \left\{ \left( \frac{w/t}{S} \right) - 0.33 \right\}^3 \]
\[ = 399(0.105)^4 \left( \frac{23/31.09 - 0.33}{0.6} \right) = 0.0003337 \text{ in.}^4 \]
\[ I_t = d^4/12 = (0.508)^4(0.105)/12 \]
\[ = 0.001147 \text{ in.}^4 \]
\[ I_x/I_a = 0.001147/0.0003337 = 0.344 \]
\[ D/w = 0.8/2.415 = 0.331 \]
\[ n = 1/2 \]
\[ k = \left( 4.82 - 5(D/w) \right) \left[ (I/I_a)^2 + 0.43 \right] \leq 2.286 \]
\[ = \left( 4.82 - 5(0.331) \right) \left( 0.344 \right)^2 + 0.43 = 2.286 \]
\[ = 5.25 - 5(D/w) = 3.595 > 2.286 \]
\[ k = 2.286 \]
\[ \lambda = (1.052/ \sqrt{2.286})(23.00) \sqrt{50/29500} = 0.659 < 0.673 \]
\[ b = w = 2.415 \text{ in. (Element 2 fully effective)} \]

For element 3:

\[ \lambda = (1.052/ \sqrt{0.43})(4.84) \sqrt{50/29500} = 0.320 < 0.673 \]
\[ d_{v}' = d = 0.508 \text{ in.} \]
\[ d_{s} = d_{v}'(I/I_a) \leq d_{v}' \]
\[ \text{Since } I/I_a = 0.344 < 1.0 \]
\[ d_{s} = 0.508(0.344) = 0.175 \text{ in.} \]
\[ A_e = 1.551 - 0.105(7.415 - 4.152) - 0.105(0.508 - 0.175)x2 \]
\[ = 1.138 \text{ in.}^2 \]
\[ P_{no} = 1.138 \times 50 = 56.90 \text{ kips} \]
\[ \phi_c = 0.85 \]
\[ \phi_c P_{no} = 0.85 \times 56.90 \]
\[ = 48.37 \text{ kips} \]

5. Determination of \( M_{uy} \) (required flexural strength about y-axis): \( (M_{ux} = 0 \text{ since } e_y = 0) \) \( M_{uy} \) will be with respect to the centroidal axes of the effective section determined for the required axial strength alone.

\[ A_c = 1.551 \text{ in.}^2 \text{ under required axial strength alone} \]

Since \( A_c = A \), the centroidal axes for the effective section are the same as those for the full section. Therefore, \( e_c \) did not change.

\[ M_{uy} = 3.68(2.00) = 7.36 \text{ kip-in. (Required Flexural Strength)} \]

The interaction equations (Eq. C5-1) and (Eq. C5-2) reduce to the following:
6. Determination of \( \phi_b M_{ny} \) (Section C3.1):

\( \phi_b M_{ny} \) shall be taken as the smaller of the design flexural strengths calculated according to sections C3.1.1 and C3.1.2:

a. Section C3.1.1: \( M_{ny} \) will be calculated on the basis of initiation of yielding.

Here it is evident that the initial yielding will not be in the compression flange, rather it will be in the tension flange.

The procedure is iterative: one assumes the actual compressive stress \( f \) under \( M_{ny} \). Knowing \( f \) one proceeds as usual to obtain \( x_{cg} \) (measured from top fiber) to neutral axis. Then one obtains \( f = F_y \left( \frac{x_{cg}}{3-x_{cg}} \right) \) and checks if it equals to the assumed value. If not, one reiterates by assuming another \( f \) until finally it checks. Then for this condition one obtains \( I_y \) and \( M_{ny} = f(I/y/x_{cg}) = F_y \left( \frac{I_y}{(3-x_{cg})} \right) \).

For the first iteration assume a compressive stress \( f = 20 \text{ ksi} \) in the top compression fibers and that the webs are fully effective.

Compression flange:

\[
\begin{align*}
\text{k} & = 4.00 \\
\text{w/t} & = \frac{7.415}{0.105} = 70.62 \\
\lambda & = \left(1.052/\sqrt{4.00}\right)\left(70.62\right)\sqrt{20/29500} = 0.967 > 0.673 \\
\rho & = \left[1 -(0.22/0.967)\right]/0.967 = 0.799 \\
b & = 0.799 \times 7.415 = 5.925 \text{ in.}
\end{align*}
\]
To calculate effective section properties about y-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>x Distance from Top Fiber (in.)</th>
<th>Lx (in.)</th>
<th>Lx^2 (in.^3)</th>
<th>L' About Own Axis (in.^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>2x2.415 = 4.830</td>
<td>1.500</td>
<td>7.245</td>
<td>10.868</td>
<td>2.347</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>2x0.377 = 0.754</td>
<td>0.140</td>
<td>0.106</td>
<td>0.015</td>
<td>—</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>2x0.377 = 0.754</td>
<td>2.860</td>
<td>2.156</td>
<td>6.167</td>
<td>—</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>5.925</td>
<td>0.053</td>
<td>0.314</td>
<td>0.017</td>
<td>—</td>
</tr>
<tr>
<td>Tension Flanges</td>
<td>2x0.508 = 1.016</td>
<td>2.948</td>
<td>2.995</td>
<td>8.830</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>13.279</td>
<td>12.816</td>
<td>25.897</td>
<td>2.347</td>
<td></td>
</tr>
</tbody>
</table>

Distance from top fiber to y-axis is
\[ x_{cg} = \frac{12.816}{13.279} = 0.965 \text{ in.} \]
\[ f = f_y \left( \frac{x_{cg}}{3-x_{cg}} \right) \]
\[ = 50 \left( \frac{0.965}{3.00-0.965} \right) = 23.71 \text{ ksi} > 20 \text{ ksi} \]
need to do another iteration.

For the second iteration assume a compressive stress \( f = 24.95 \text{ ksi} \) in the top compression fibers, and that the webs are fully effective.

Compression flange:
\[ \lambda = \frac{1.052}{\sqrt{4.00}} \cdot (70.62) \sqrt{24.95/29500} = 1.080 > 0.673 \]
\[ \rho = \frac{1-(0.22/1.080)}{1.080} = 0.737 \]
\[ b = 0.737 \times 7.415 = 5.465 \text{ in.} \]

Effective section properties about y-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>x Distance from Top Fiber (in.)</th>
<th>Lx (in.)</th>
<th>Lx^2 (in.^3)</th>
<th>L' About Own Axis (in.^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>2x2.415 = 4.830</td>
<td>1.500</td>
<td>7.245</td>
<td>10.868</td>
<td>2.347</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>2x0.377 = 0.754</td>
<td>0.140</td>
<td>0.106</td>
<td>0.015</td>
<td>—</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>2x0.377 = 0.754</td>
<td>2.860</td>
<td>2.156</td>
<td>6.167</td>
<td>—</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>5.465</td>
<td>0.053</td>
<td>0.290</td>
<td>0.015</td>
<td>—</td>
</tr>
<tr>
<td>Tension Flanges</td>
<td>2x0.508 = 1.016</td>
<td>2.948</td>
<td>2.995</td>
<td>8.830</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>12.819</td>
<td>12.792</td>
<td>25.895</td>
<td>2.347</td>
<td></td>
</tr>
</tbody>
</table>

Distance from top fiber to y-axis is
\[ x_{cg} = \frac{12.819}{12.792} = 0.996 \text{ in.} \]
Xc_g = 12.792/12.819 = 0.998 in.

\[ f = 50(\frac{0.998}{3.00-0.998}) = 24.93 \text{ ksi (close enough)} \]

Thus actual compressive stress \( f = 24.95 \text{ ksi} \)

To check if the webs are fully effective (Section B2.3):

\[ f_1 = \frac{(0.998-0.293)/2.002)(50) = 17.61 \text{ ksi (compression)} \]

\[ f_2 = -\frac{(2.002-0.293)/2.002)(50) = -42.68 \text{ ksi (tension)} \]

\[ \psi = \frac{f_2}{f_1} = -42.68/17.61 = -2.424 \]

\[ k = 4+2(1-\psi)^3+2(1-\psi) \quad \text{(Eq. B2.3-4)} \]

\[ = 4+2[1-(-2.424)]^3+2[1-(-2.424)] = 91.132 \]

\[ h = w = 2.415 \text{ in.} \]

\[ \frac{w}{t} = 2.415/0.105 = 23.00 < 200 \text{ OK (Section B1.2-(a))} \]

\[ \lambda = (1.052/\sqrt{91.132})(23.00) \sqrt{17.61/29500} = 0.062 < 0.673 \]

\[ b_x = 2.415 \text{ in.} \]

\[ b_2 = b_x/2 \quad \text{(Eq. B2.3-2)} \]

\[ = 2.415/2 = 1.208 \text{ in.} \]

\[ b_1 = b_x/(3-\psi) \quad \text{(Eq. B2.3-1)} \]

\[ = 2.415/[3-(-2.424)] = 0.445 \text{ in.} \]

Compression portion of each web calculated on the basis of the effective section

\[ x_{cg} = 0.998-0.293 = 0.705 \text{ in.} \]

Since \( b_1+b_2 = 1.653 \text{ in.} > 0.705 \text{ in.}, b_1+b_2 \) shall be taken as 0.705 in.. This verifies the assumption that the web is fully effective.

\[ I_y' = Lx^2 + I_1 - Lx^2_{cg} \]

\[ = 25.895 + 2.347 - 12.819(0.998)^2 \]

\[ = 15.474 \text{ in.}^3 \]

Actual \( I_y = I_y' + 15.474(0.105) = 1.625 \text{ in.}^3 \)

\[ S_e = I_y/(3.000-x_{cg}) \]

\[ = 1.625/(3.000-0.998) \]

\[ = 0.812 \text{ in.}^3 \]

\[ M_{ny} = S_eF_y \quad \text{(Eq. C3.1.1-1)} \]

\[ = 0.812(50) \]

\[ = 40.60 \text{ kip-in.} \]

\[ \phi_b = 0.95 \]

\[ \phi_b M_{ny} = 0.95 \times 40.65 = 38.57 \text{ kip-in.} \]
b. Section C3.1.2: \( M_{ny} \) will be calculated on the basis of the lateral buckling strength. (y-axis is the axis of bending).

For the full section:

\[
\begin{align*}
I_y &= 1.786 \text{ in.}^4 \\
\bar{x}_g &= \frac{\bar{x} + t/2}{0.820 + 0.105/2} = 0.873 \text{ in.} \\
S_f &= I_y/\bar{x}_g = 1.786/0.873 = 2.046 \text{ in.}^3 \\
M_y &= S_fF_y = 2.046(50) = 102.30 \text{ kip-in.} \\
C_s &= +1.00 \\
A &= 1.551 \text{ in.}^2 \\
\sigma_{ex} &= 76.93 \text{ ksi} \\
\sigma_t &= 10.26 \text{ ksi} \\
M_1/M_2 &= -1.00 \text{ (single curvature)} \\
C_{TF} &= 0.6 - 0.4(M_1/M_2) \\
&= 0.6 - 0.4(-1.00) = 1.00 \\
r_o &= 3.961 \text{ in.} \\
j &= 4.567 \\
M_c &= C_sA\sigma_{ex} \left[ j + C_s \sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex})^2} \right]/C_{TF} \\
&= 1.0(1.551)(76.93)(4.567) \\
&= 1116.54 \text{ kip-in.} \\
M_e &= 1116.54 \text{ kip-in.} > 0.5M_y = 51.15 \text{ kip-in} \\
M_c &= M_y[1 - (M_y/4M_e)] \\
&= 102.30[1 - (102.30/(4 \times 1116.54))] = 99.96 \text{ kip-in.} \\
M_c/S_f &= 99.96/2.046 = 48.86 \text{ ksi} \\
\end{align*}
\]

To calculate effective section properties to obtain \( S_c \) at stress 48.86 ksi, we assume that the webs are fully effective.

Compression flange:

\[
\begin{align*}
\lambda &= (1.052/\sqrt{4.00})(70.62)\sqrt{48.86/29500} = 1.512 > 0.673 \\
\rho &= [1 - (0.22/1.512)]/1.512 = 0.565 \\
b &= 0.565 \times 7.415 = 4.189 \text{ in.}
\end{align*}
\]

Effective section properties about y-axis:
**Effective Length**

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>(L_x) (in.³)</th>
<th>(L_x^2) (in.⁶)</th>
<th>(I'_y) About Own Axis (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>2x2.415 = 4.830</td>
<td>1.500</td>
<td>7.245</td>
<td>10.868</td>
<td>2.347</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>2x0.377 = 0.754</td>
<td>0.140</td>
<td>0.106</td>
<td>0.015</td>
<td>-</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>2x0.377 = 0.754</td>
<td>2.860</td>
<td>2.156</td>
<td>6.167</td>
<td>-</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>4.189</td>
<td>0.053</td>
<td>0.222</td>
<td>0.012</td>
<td>-</td>
</tr>
<tr>
<td>Tension Flanges</td>
<td>2x0.508 = 1.016</td>
<td>2.948</td>
<td>2.995</td>
<td>8.830</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>11.543</td>
<td>12.724</td>
<td>25.892</td>
<td>2.347</td>
<td></td>
</tr>
</tbody>
</table>

Distance from top fiber to y-axis is

\[ x_{cg} = \frac{12.724}{11.543} = 1.102 \text{ in.} \]

To check if the webs are fully effective (Section B2.3):

\[ f_1 = \frac{(1.102-0.293)/1.102}{48.86} = 35.87 \text{ ksi (compression)} \]

\[ f_2 = \frac{-(1.898-0.293)/1.102}{48.86} = -71.16 \text{ ksi (tension)} \]

\[ \psi = -71.16/35.87 = -1.984 \]

\[ k = 4+2\left[1-(-1.984)^3\right] + 2\left[1-(-1.984)^3\right] = 63.109 \]

\[ \lambda = (1.052/\sqrt[3]{63.109})(23.00)\sqrt[3]{35.87/29500} = 0.106 < 0.673 \]

\[ b_e = 2.415 \text{ in.} \]

\[ b_2 = 2.415/2 = 1.208 \text{ in.} \]

\[ b_1 = 2.415/(3-(-1.984)) = 0.485 \text{ in.} \]

Compression portion of each web calculated on the basis of the effective section = 1.102-0.293 = 0.809 in.

Since \( b_1 + b_2 = 1.693 \text{ in.} > 0.809 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 0.809 in. This verifies the assumption that the web is fully effective.

\[ I'_y = 25.892 + 2.347 - 11.543(1.102)^2 \]

\[ = 14.221 \text{ in.}^3 \]

Actual \( I_y = 14.221(0.105) = 1.493 \text{ in.}^4 \]

\[ S_c = I_y/x_{cg} = 1.493/1.102 = 1.355 \text{ in.}^3 \]

\[ M_{ny} = M_cS_c/S_f \]

\[ = 0.996(1.355)/2.046 \]

\[ = 66.20 \text{ kip-in.} \]

\[ \phi_b = 0.90 \]

\[ \phi_bM_{ny} = 0.90 \times 66.20 = 59.58 \text{ kip-in.} \]

\( \phi_bM_{ny} \) shall be the smaller of 38.57 kip-in. and 59.58 kip-in.
Thus
\[ \phi_b M_{ny} = 38.57 \text{ kip-in.} \]

7. \[ C_{my} = 0.6-0.4(M_1/M_2) \]
\[ M_1/M_2 = -1.00 \text{ (single curvature)} \]
\[ 0.6-0.4(-1.00) = 1.00 \]
\[ C_{my} = 1.00 \]

8. Determination of \( 1/\alpha_{ny} \):
\[ \phi_c = 0.85 \]
\[ P_E = \pi^2 EI_y/(K_y L_y)^2 \] (Eq. C5-5)
\[ I_y = 1.786 \text{ in.}^4 \]
\[ K_y L_y = 1.0(16\times12) = 192 \text{ in.} \]
\[ P_E = \left[ \pi^2 (29500)(1.786)/(192)^2 \right] = 14.11 \text{ kips} \]
\[ 1/\alpha_{ny} = 1/[1-P_u/(\phi_c P_E)] \] (Eq. C5-4)
\[ = 1/[1-3.68/(0.85\times14.11)] = 1.443 \]
\[ \alpha_{ny} = 0.693 \]

9. Check interaction equations:
\[ P_u/\phi_c P_n+C_{my} M_{uy}/\phi_b M_{ny} \alpha_{ny} \leq 1.0 \] (Eq. C5-1)
\[ 3.68/11.985+1.00\times7.36/(38.57\times0.693) = 0.307+0.275 \]
\[ = 0.582 < 1.0 \text{ OK} \]
\[ P_u/\phi_c P_{no}+M_{ux}/\phi_b M_{ny} \leq 1.0 \] (Eq. C5-2)
\[ 3.68/48.37+7.36/38.57 = 0.076+0.191 = 0.267 < 1.0 \text{ OK} \]
Therefore the section is adequate for the applied loads.

Solution: Part (b)

1. Full section properties are the same as previously calculated in part (a.1).

2. \[ \phi_c P_n = 11.985 \text{ kips (calculated in part (a))}. \]

3. \[ P_u/\phi_c P_n = 3.68/11.985 = 0.307 > 0.15 \]

Therefore the following interaction equations must be satisfied.
\[ P_u/\phi_c P_n+C_{mx} M_{ux}/\phi_b M_{nx} \alpha_{nx} +C_{my} M_{uy}/\phi_b M_{ny} \alpha_{ny} \leq 1.0 \] (Eq. C5-1)
\[ P_u/\phi_c P_{no}+M_{ux}/\phi_b M_{nx}+M_{uy}/\phi_b M_{ny} \leq 1.0 \] (Eq. C5-2)

4. \[ \phi_c P_{no} = 48.37 \text{ kips (calculated in part (a.4))}. \]

5. Determination of \( M_{ux} \) (Section C5):
The centroidal x-axis is the same for both the full and effective sections.

\[ e_y = 4.000 \text{ in.} \]
\[ M_{ux} = P_e e_y = 3.68(4.000) = 14.72 \text{ kip-in.} \]

6. Determination of \( \phi_b M_{ux} \) (Section C3.1):

\( \phi_b M_{ux} \) shall be taken as the smaller of the design flexural strengths calculated according to Sections C3.1.1 and C3.1.2.

a. Section C3.1.1: \( M_{ux} \) will be calculated based on the initiation of yielding.

First approximation:

* Assume a compressive stress of \( f = F_y = 50 \text{ ksi} \) in the top fiber of the section.
* Assume that the web is fully effective.

Compression flange:

\[ w = 2.415 \text{ in.} \]
\[ w/t = 2.415/0.105 = 23.00 \]
\[ S = 1.28 \sqrt{E/f} \]
\[ = 1.28 \sqrt{29500/50} = 31.09 \]

For \( S/3 = 10.36 < w/t = 23.00 < S = 31.09 \)

\[
I_a = t^4 399\{(w/t)/S -0.33\}^3 \]
\[ = (0.105)^4(399)[(23.00/31.09)-0.33] \]
\[ = 0.003337 \text{ in.}^4 \]

\[
I_s = d^3 t/12 \]
\[ = (0.508)^3(0.105)/12 = 0.001147 \text{ in.}^4 \]

\[
I_d/I_a = 0.001147/0.003337 = 0.344 \]

\[ D = 0.800 \text{ in.} \]

\[ D/w = 0.800/2.415 = 0.331 \]

\[ w/t = 23.00 < 14 \text{ OK (Section B4 of the Commentary)} \]

For \( 0.25 < D/w = 0.331 < 0.8 \)

\[
k = [4.82-5(D/w)](I_d/I_a)^{1/2}+0.43\leq5.25-5(D/w) \]
\[ = [4.82-5(0.331))(0.344)^{1/2}+0.43 = 2.286 \]

\[ 5.25-5(0.331) = 3.595 > 2.286 \]

\[ k = 2.286 \]

\[
\lambda = (1.052/\sqrt{k})(w/t) \sqrt{E/f} \]
\[ = (1.052/\sqrt{2.286})(23.00)\sqrt{50/29500} = 0.659 < 0.673 \]

\[ b = w \]
\[ = 2.415 \text{ in. (fully effective)} \]

Compression stiffener:

\[ d = 0.508 \text{ in.} \]
d/t = 0.508/0.105 = 4.84 < 14 OK (Section B4 of the Commentary)

k = 0.43

Assume max. stress in element, \( f = F_y = 50 \text{ ksi} \) although it will be actually less.

\[ \lambda = \frac{1.052}{\sqrt{k}}(w/t) \frac{\sqrt{f/E}}{V_k} \]  
(Eq. B2.1-4)

\[ = \frac{1.052}{\sqrt{0.43}}(4.84) \sqrt{50/29500} = 0.320 \]

For \( \lambda < 0.673 \)

\( b = w \)  
(Eq. B2.1-1)

\( d_s' = 0.508 \text{ in.} \)

\( d_s = d_s'(I_x/I_x) \leq d_s' \)  
(Eq. B4.2-11)

\[ = 0.508(0.344) \]

\[ = 0.175 \text{ in.} \]

Effective section properties about x-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.)</th>
<th>Ly^2 (in.^3)</th>
<th>t'x About Own Axis (in.^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression Flange</td>
<td>2.415</td>
<td>0.053</td>
<td>0.128</td>
<td>0.007</td>
<td>—</td>
</tr>
<tr>
<td>Compression Stiffener</td>
<td>0.175</td>
<td>0.381</td>
<td>0.067</td>
<td>0.025</td>
<td>—</td>
</tr>
<tr>
<td>Compression Corners</td>
<td>2x0.377 = 0.754</td>
<td>0.140</td>
<td>0.106</td>
<td>0.015</td>
<td>—</td>
</tr>
<tr>
<td>Web</td>
<td>7.415</td>
<td>4.000</td>
<td>29.660</td>
<td>118.640</td>
<td>33.974</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>2.415</td>
<td>7.948</td>
<td>19.194</td>
<td>152.557</td>
<td>—</td>
</tr>
<tr>
<td>Tension Stiffener</td>
<td>0.508</td>
<td>7.453</td>
<td>3.786</td>
<td>28.218</td>
<td>0.011</td>
</tr>
<tr>
<td>Tension Corners</td>
<td>2x0.377 = 0.754</td>
<td>7.860</td>
<td>5.926</td>
<td>46.582</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>14.436</td>
<td>58.867</td>
<td>346.044</td>
<td>33.985</td>
<td></td>
</tr>
</tbody>
</table>

Distance from neutral axis to top fiber,

\( y_{cg} = \frac{Ly}{L} = \frac{58.867}{14.436} = 4.078 \text{ in.} \)

Since the distance from the neutral axis to the top compression fiber is greater than half the depth of the section, a compressive stress of \( F_y = 50 \text{ ksi} \) governs as assumed.

\[
I_x' = Ly^2 + I_x - Ly_{cg}^2
\]

\[
= 346.044 + 33.985 - 14.436(4.078)^2
\]

\[
= 140.0 \text{ in.}^3
\]

Actual \( I_x = tl_x' \)

\[
= (0.105)(140.0) = 14.70 \text{ in.}^4
\]

Check Web

\( w/t = 7.415/0.105 = 70.62 < 200 \text{ OK (Section B1.2-(a))} \)
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ f_1 = \frac{(4.078 - 0.293)}{4.078} \times 50 = 46.41 \text{ ksi (compression)} \]

\[ f_2 = -\frac{(3.922 - 0.293)}{4.078} \times 50 = -44.50 \text{ ksi (tension)} \]

\[ \psi = \frac{f_2}{f_1} = -\frac{-44.50}{46.41} = -0.959 \]

\[ k = 4 + 2\left[ 1 - (-0.959) \right] + 2\left[ 1 - (-0.959) \right] = 22.95 \]

\[ \lambda = \frac{(1.052/\sqrt{22.95})(70.62)\sqrt{46.41/29500}}{0.615} \]

For \( \lambda < 0.673 \)

\[ b = w \]

\[ b_e = 7.415 \text{ in.} \]

\[ b_2 = \frac{7.415}{2} = 3.708 \text{ in.} \]

\[ b_1 = \frac{7.415}{[3 - (-0.959)]} = 1.873 \text{ in.} \]

\[ b_1 + b_2 = 1.873 + 3.708 = 5.581 \text{ in.} > 3.785 \text{ in. (compression portion of web)} \]

Therefore web is fully effective as assumed.

Check Compression Stiffener

Actual maximum stress in stiffener = 46.41 ksi

\[ \lambda = \frac{(1.052/\sqrt{0.43})(4.84)\sqrt{46.41/29500}}{0.308} \]

For \( \lambda < 0.673 \)

\[ d'_s = 0.508 \text{ in.} \]

Since \( I_y/I_s \) is unchanged

\[ d_s = 0.175 \text{ in.} \]

Conservative assumption OK

\[ S_c = I_s/y_{cg} = 14.70/4.078 = 3.605 \text{ in.}^3 \]

\[ M_{nx} = S_c F_y \quad \text{(Eq. C3.1.1-1)} \]

\[ = (3.605)(50) = 180.25 \text{ kip-in.} \]

\[ \phi_b = 0.95 \]

\[ \phi_b M_{nx} = 0.95 \times 180.25 = 171.24 \text{ kip-in.} \]

b. Section C3.1.2: \( M_{nx} \) will be calculated based on the lateral buckling strength.

For the full section:

\[ I_x = 15.108 \text{ in.}^4 \]

\[ y_{cg} = 4.000 \text{ in.} \]

\[ S_f = I_s/y_{cg} = 15.108/4.000 = 3.777 \text{ in.}^3 \]

\[ M_y = S_f F_y \quad \text{(Eq. C3.1.2-4)} \]

\[ = 3.777(50) = 188.85 \text{ kip-in.} \]

\[ C_b = 1.00 \quad \text{(for members subject to combined axial load and bending moment)} \]

\[ r_o = 3.961 \text{ in.} \]
A = 1.551 in.²

\[ \sigma_{ey} = \pi^2 E / (K_y L_y / t_y)^2 \]

\[ = \pi^2 (29500) / (178.94)^2 = 9.093 \text{ ksi} \]

\[ \sigma_t = 10.26 \text{ ksi} \]

\[ M_e = C_p f A \sqrt{\sigma_{ey} \sigma_t} \quad \text{(Eq. C3.1.2-5)} \]

\[ = (1.000)(3.961)(1.551) \sqrt{(9.093)(10.26)} \]

\[ = 59.34 \text{ kip-in.} \]

\[ 0.5M_y = 0.5(188.85) = 94.43 \text{ kip-in.} \]

For \( M_e < 0.5M_y \)

\[ M_e = M_e \quad \text{(Eq. C3.1.2-3)} \]

\[ = 59.34 \text{ kip-in.} \]

\[ M_d/S_f = 59.34/3.777 = 15.71 \text{ ksi} \]

Determine \( S_e \), the elastic section modulus of the effective section calculated at a stress of \( M_e/S_f \) in the extreme compression fiber.

For compression flange:

\[ w = 2.415 \text{ in.} \]

\[ w/t = 2.415 / 0.105 = 23.00 \]

\[ S = 1.28 \sqrt{E/f} \]

\[ = 1.28 \sqrt{29500 / 15.71} = 55.47 \]

\[ S/3 = 18.49 < w/t = 23.00 < S = 55.47 \]

\[ I_a = 399(0.105)^4 [(23.00/55.47)-0.33]^3 \]

\[ = 0.000029 \text{ in.}^4 \]

\[ I_s = 0.001147 \text{ in.}^4 \]

\[ I_s/I_a = 0.001147 / 0.000029 = 39.55 \]

\[ k = [4.82-5(0.331)](39.55)^{1/2} + 0.43 = 20.334 > 3.595 \]

\[ \lambda = (1.052/\sqrt{3.595})(23.00)\sqrt{15.71/29500} = 0.294 < 0.673 \]

\[ b = w = 2.415 \text{ in. (compression flange fully effective)} \]

For compression stiffener:

\( f \) is taken conservatively 15.71 ksi as in the top compression fiber.

\[ d/t = 4.84 \]

\[ \lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{15.71/29500} = 0.179 < 0.673 \]

\[ d'_s = d = 0.508 \text{ in.} \]

And since \( I_s/I_a = 39.55 > 1.0 \)

\[ d_s = d'_s = 0.508 \text{ in. (compression stiffener fully effective)} \]

And since the web was fully effective at the stress \( f = F_y = 50 \text{ ksi} \), it will be fully effective for \( f = 15.71 \text{ ksi} \). Thus the whole section is fully effective at \( M_y/S_f = 15.71 \text{ ksi} \)
Therefore
\[ S_c = S_e = 3.777 \text{ in.}^3 \]
\[ M_{nx} = M_c S_e / S_t \]
\[ = 59.34(3.777)/3.777 \]
\[ = 59.34 \text{ kip-in.} \]
\[ \phi_b = 0.90 \]
\[ \phi_b M_{nx} = 0.90 \times 59.34 = 53.41 \text{ kip-in.} \]
\( \phi_b M_{nx} \) shall be the smaller of 171.24 kip-in. and 53.41 kip-in.
Therefore
\[ \phi_b M_{nx} = 53.41 \text{ kip-in.} \]

7. Determination of \( C_{mx} \) (Section C5):
\[ M_1/M_2 = -1.00 \text{ (single curvature)} \]
\[ C_{mx} = 0.6-0.4(-1.0) = 1.00 \]

8. Determination of \( \alpha_{nx} \) (Section C5):
\[ P_u = 3.68 \text{ kips} \]
\[ P_E = \pi^2 EI_s / (K_s L_o)^2 \]
\[ = \pi^2 (29500)(15.108) / [1(16)x12]^2 = 119.32 \text{ kips} \]
\[ \phi_c = 0.85 \]
\[ 1/\alpha_{nx} = 1/[1-P_u/(\phi_c P_E)] \]
\[ = 1/[1-3.68/(0.85\times119.32)] = 1.038 \]
\[ \alpha_{nx} = 0.964 \]

9. \( M_{uy} = 7.36 \text{ kip-in.} \) (calculated in part (a.5))

10. \( \phi_b M_{my} = 38.57 \text{ kip-in.} \) (calculated in part (a.6))

11. \( C_{my} = 1.0 \) (calculated in part (a.7))

12. \( \alpha_{ny} = 0.693 \) (calculated in part (a.8))

13. Interaction equations (Section C5):
\[ P_u/\phi_c P_n + C_{ax} M_{ax} / \phi_b M_{ax} \alpha_{ax} + C_{my} M_{uy} / \phi_b M_{my} \alpha_{ny} \leq 1.0 \]
\[ (Eq. C5-1) \]
\[ 3.68/11.985 + 1.0 \times 14.72/(53.41 \times 0.964) + 1.0 \times 7.36/(38.57 \times 0.693) \]
\[ 0.307 + 0.286 + 0.275 = 0.868 < 1.0 \text{ OK} \]
\[ P_u/\phi_c P_n + M_{ux} / \phi_b M_{ux} + M_{uy} / \phi_b M_{uy} \leq 1.0 \]
\[ (Eq. C5-2) \]
\[ 3.68/48.37 + 14.72/53.41 + 7.36/38.57 \]
\[ 0.076 + 0.276 + 0.191 = 0.543 < 1.0 \text{ OK} \]

Therefore the section is adequate for the applied loads.
EXAMPLE NO. 16
C-SECTION - WALL STUD
Stud Cross Section/Properties

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: As shown, spacing 24 in. O.C.
3. Length: 15 ft.-0 in.
4. Cladding: On both sides, 1/2 in. gypsum board with No. 6 Type S-12 self-drilling screws @ 12 in. O.C. vertically.
5. Dead load to live load ratio $D/L = 1/5$.

Required:
1. The design axial strength, $\phi_c P_n$.
2. The permitted service axial load in combination with 5 psf lateral load.
Solution:

1. No Lateral Load (Section D4.1)
   a. Check column buckling between fasteners

\[
KL/r_y = (2)(12)/0.999 = 24.0
\]

For flexural buckling about the y-y axis

\[
F_{e1} = \frac{\pi^2E}{(KL/r_y)^2} \quad \text{(Eq. C4.1-1)}
\]

\[
= \left( \frac{\pi^2 (29500)}{(24.0)^2} \right) = 505 \text{ ksi}
\]

For torsional-flexural buckling

\[
F_{e2} = \frac{(1/2\beta)[(\sigma_{ex}+ \sigma_t) - \sqrt{(\sigma_{ex}+ \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}]}{\text{Eq. C4.2-1}}
\]

\[
\sigma_{ex} = \frac{\pi^2E}{(K_L/r_x)^2} \quad \text{(Eq. C3.1.2-7)}
\]

\[
= \pi^2 (29500)/(12)(2.76)^2 = 3850 \text{ ksi}
\]

\[
\sigma_t = \frac{(1/A_o)^2}{(GJ + (\pi^2 EC_w)/(K_t L_t))(\beta)} \quad \text{(Eq. C3.1.2-9)}
\]

\[
= \left( \frac{1}{1.003(3.57)^2} \right) \left( \frac{11300x0.00188 + (\pi^2x29500x10.10)/(2x12^2)}{401} \right) = 401 \text{ ksi}
\]

\[
\beta = 1-(x/\rho)^2 \quad \text{(Eq. C4.2-3)}
\]

\[
= 1-(2.03/3.57)^2 = 0.677
\]

\[
F_{e2} = \frac{(1/2x0.677)[(3850+401) - \sqrt{(3850+401)^2 - 4(0.677)(3850)(401)}]}{387 \text{ ksi} < 505 \text{ ksi}}
\]

Since \( F_{e2} > F_y/2 = 25 \text{ ksi} \)

\[
F_{n1} = F_y(1-F_y/4F_y) \quad \text{(Eq. C4.2-2)}
\]

\[
= 50[1-0.5/(4x387)] = 48.4 \text{ ksi}
\]

b. Check flexural and/or torsional overall column buckling

\[
\sigma_{cr} = \sigma_{cy} + Q \quad \text{(Eq. D4.1-2)}
\]

\[
\sigma_{cr} = \frac{(1/2\beta)[(\sigma_{ex}+ \sigma_t) - \sqrt{(\sigma_{ex}+ \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}]}{\text{Eq. D4.1-3}}
\]

\[
\sigma_{ex} = \frac{\pi^2E}{(K_l/r_x)^2} \quad \text{(Eq. D4.1-8)}
\]

\[
= \pi^2 (29500)/(15)(12)(2.76)^2 = 68.5 \text{ ksi}
\]

\[
\sigma_{t} = \frac{(1/A_o)^2}{(GJ + (\pi^2 EC_w)/(L^2))} \quad \text{(Eq. D4.1-11)}
\]

\[
= \left( \frac{1}{1.003(3.57)^2} \right) \left( \frac{11300x0.00188 + (\pi^2x29500x10.10)/(15x12^2)}{8.76 \text{ ksi}} \right)
\]

\[
= 8.76 \text{ ksi}
\]

\[
Q = \frac{(Qd^2)}{(4A_o^2)} \quad \text{(Eq. D4.1-14)}
\]

\[
Q = qB
\]
IV-126 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ B = 24 \text{ in. O.C.} \]  
\[ \bar{q} = \bar{q}_d(2-s/12) \quad \text{(Eq. D4.1-26)} \]  
\[ \bar{q}_o = 2.0 \text{ kip/in.} \]  
\[ s = 12 \text{ in.} \]  
\[ \bar{q} = 2.0(2-12/12) = 2.0 \text{ kip/in.} \]  
\[ \bar{Q} = 2.0(24) = 48 \text{ kips} \]  
\[ \bar{Q}_t = 48(7)^2/(4)(1.003)(3.57)^2 = 46.0 \text{ ksi} \]  
\[ \sigma_{eq} = 8.76 + 46.0 = 54.8 \text{ ksi} \]  
\[ \sigma_{uy} = \pi^2 E/(L/r_y)^2 \quad \text{(Eq. D4.1-10)} \]  
\[ = \pi^2 (29500)/[(15)/(0.999)]^2 = 8.97 \text{ ksi} \]  
\[ \bar{Q}_a = \bar{Q}/A \quad \text{(Eq. D4.1-13)} \]  
\[ = 48/1.003 = 47.9 \text{ ksi} \]  
\[ \beta = 0.677 \text{ (calculated previously)} \]  
\[ \sigma_{cr} = 8.97 + 47.9 = 56.9 \text{ ksi} \]  
\[ \sigma_{cr} = \frac{1}{2(0.677)} \left[ (68.5+54.8) - \sqrt{(68.5 + 54.8)^2 - 4(0.677)(68.5)(54.8)} \right] = 38.6 \text{ ksi} \]  
Use \( \sigma_{cr} = 38.6 \text{ ksi} = F_e \)  
\( F_e > F_j/2 = 25 \text{ ksi} \) so,  
\( F_{n2} = F_y(1-F_y/4F_e) \quad \text{(Eq. C4-2)} \]  
\[ = 50[1-50/(4\times38.6)] = 33.8 \text{ ksi} \]  
c. Check shear strain of wall material  
\[ C_o = L/350 \quad \text{(Eq. D4.1-21)} \]  
\[ = (12)(15)/350 = 0.514 \text{ in.} \]  
\[ D_o = L/700 \quad \text{(Eq. D4.1-22)} \]  
\[ = (12)(15)/700 = 0.257 \text{ in.} \]  
\[ E_o = L/(dx10,000) \quad \text{(Eq. D4.1-23)} \]  
\[ = (12)(15)/(7\times10,000) = 0.00257 \text{ rad} \]  
Let initial trial value of \( F_{n0} \) be based on largest stress for elastic values of \( E \) and \( G \)  
Assume \( F_n = 0.5F_y = 0.5(50) = 25 \text{ ksi} \)  
\[ E = 29500 \text{ ksi} \]  
\[ G = 11300 \text{ ksi} \]  
\[ C_1 = (F_n C_o)/([\sigma_{eq} F_n + \bar{Q}_a]) \quad \text{(Eq. D4.1-16)} \]
\[ \sigma_{eq} = 8.97 \text{ ksi} \]
\[ \bar{Q}_d = 47.9 \text{ ksi} \]

\[ C_1 = \frac{(25)(0.514)}{(8.97-25+47.9)} = 0.403 \]

\[ E_1 = \frac{\{F_n \left[ (\sigma_{eq} - F_n)(r_o^2 \sigma_o - x_o D_o) - F_n x_o (D_o - x_o \sigma_o) \right] \}}{\left[ (\sigma_{eq} - F_n)^2 (\sigma_Q - F_n) - (F_n x_o)^2 \right]} \] (Eq. D4.1-17)

\[ \sigma_{eq} = 68.5 \text{ ksi} \]
\[ \sigma_1 Q = 54.8 \text{ ksi} \]

\[ E_1 = \frac{(25)\{(68.5-25)[(3.57)^2(0.00257)-(2.03)(0.257)] -\(25)(2.03)[0.257-(2.03)(0.00257)]\}}{\{(68.5 -\(25)(3.57)^2(54.8-25)-\(25)(2.03)^2 \}} \]

\[ E_1 = -0.0610, \text{ use absolute value of 0.0610} \]

\[ \bar{\gamma} = 0.008 \text{ in./in. (Table D4)} \]
\[ \gamma = \frac{\pi}{L} [C_1 + (E_1 d/2)] \] (Eq D4.1-15)

\[ \gamma = \frac{\pi}{(15)(12)}[0.403 + (0.0610)(7)/2] \]
\[ = 0.0108 \bar{\gamma} = 0.008 \]

Now, try a new value of \( F_{n3} = 21 \text{ ksi} \)

\[ C_1 = \frac{(21)(0.514)}{(8.97-21+47.9)} = 0.301 \]

\[ E_1 = \frac{(21)\{(68.5-21)[(3.57)^2(0.00257)-(2.03)(0.257)] -\(21)(2.03)[0.257-(2.03)(0.00257)]\}}{\{(68.5 -\(21)(3.57)^2(54.8-21)-\(21)(2.03)^2 \}} \]

\[ E_1 = -0.0382, \text{ use absolute value of 0.0382} \]

\[ \gamma = \frac{\pi}{(15)(12)}[0.301 + (0.0382)(7)/2] \]
\[ = 0.0076 \bar{\gamma} = 0.008 \]

Interpolating, and trying a value of \( F_{n3} = 21.5 \text{ ksi} \)

\[ C_1 = \frac{(21.5)(0.514)}{(8.97-21.5+47.9)} = 0.312 \]

\[ E_1 = \frac{(21.5)\{(68.5-21.5)[(3.57)^2(0.00257)-(2.03)(0.257)] -\(21.5)(2.03)[0.257-(2.03)(0.00257)]\}}{\{(68.5 -\(21.5)(3.57)^2(54.8-21.5)-\(21.5)(2.03)^2 \}} \]

\[ E_1 = -0.0405, \text{ use absolute value of 0.0405} \]

\[ \gamma = \frac{\pi}{(15)(12)}[0.312 + (0.0405)(7)/2] \]
\[ = 0.0079 \bar{\gamma} = 0.008 \]

Trying one final value of \( F_{n3} = 21.6 \text{ ksi} \)

\[ C_1 = \frac{(21.6)(0.514)}{(8.97-21.6+47.9)} = 0.315 \]

\[ E_1 = \frac{(21.6)\{(68.5-21.6)[(3.57)^2(0.00257)-(2.03)(0.257)] -\(21.6)(2.03)[0.257-(2.03)(0.00257)]\}}{\{(68.5 -\(21.6)(3.57)^2(54.8-21.6)-\(21.6)(2.03)^2 \}} \]
Calculate $A_e$ at $F_n = 21.6$ ksi, the smallest value of $F_{n1}$, $F_{n2}$, and $F_{n3}$

Web:

$$w = 6.663 \text{ in.}$$

$$\lambda = (1.052/\sqrt{4})(6.663/0.075) \sqrt{21.6/29500} = 16.264 \text{ (Eq. B2.1-4)}$$

For $\lambda > 0.673$,

$$\rho = (1-0.22/1.264)/1.264 = 0.653 \text{ (Eq. B2.1-3)}$$

$$\beta = \rho w = (0.653)(6.663) = 4.351 \text{ in.} \text{ (Eq. B2.1-2)}$$

Flange:

$$w = 2.413 \text{ in.}$$

$$w/t = 32.17$$

$$S = 1.28 \sqrt{29500/21.6} = 47.3 \text{ (Eq. B4-1)}$$

$$S/3 < w/t < S \text{ so,}$$

$$I_a = 399w^4[((w/t)/S) - 0.33]^3$$

$$= 399(0.075)^4[(32.17/47.3)-0.33]^3$$

$$= 0.000542 \text{ in.}^4$$

Stiffener:

$$w = 0.531 \text{ in.}$$

$$I_s = w^4t/12 = (0.531)^3(0.075)/12$$

$$= 0.000936 \text{ in.}^4$$

$$\lambda = (1.052/\sqrt{4.03} )(0.531/0.075) \sqrt{21.6/29500}$$

$$= 0.307 < 0.673$$

$$b = d's = 0.531 \text{ in.}$$

$$d_s = d's = 0.531 \text{ in. (for } I_s \geq I_a)$$

$$D/w = 0.7/2.413 = 0.290$$

$$k = [4.82-5(0.290)](0.000936/0.000542)^{1/2}+0.43$$

$$= 4.86$$

$5.25-5(0.290) = 3.80 < 4.86$, so use $k = 3.80$

$$\lambda = (1.052/\sqrt{3.80} )(32.17)\sqrt{21.6/29500}= 0.470 < 0.673 \text{ (Eq. B2.1-1)}$$

$$b = w = 2.413 \text{ in.}$$

$$A_e = 1.003(6.663-4.351)(0.075) = 0.830 \text{ in.}^2$$

$$P_n = A_e F_n \text{ (Eq. D4.1-1)}$$

$$= 0.830 \times 21.6 = 17.93 \text{ kips}$$
\[ \phi_c = 0.85 \]
\[ \phi_c P_n = 0.85 \times 17.93 = 15.24 \text{ kips} \]
\[ \phi_c P_n = P_c = 1.2P_{DL} + 1.6P_{LL} \]
\[ = (1.2P_{DL} / P_{LL}) + 1.6P_{LL} \]
\[ = 1.2(1/5) + 1.6P_{LL} \]
\[ = 1.84P_{LL} \]
\[ P_{LL} = \phi_c P_n / 1.84 = 15.24 / 1.84 = 8.28 \text{ kips} \]
\[ P_s = P_{DL} + P_{LL} \]
\[ = (1/5 + 1)P_{LL} \]
\[ = 1.2(8.28) = 9.94 \text{ kips} \]

Where

\[ P_u = \text{Required axial strength} \]
\[ P_s = \text{Service axial load} \]
\[ P_{DL} = \text{Axial load determined on the basis of nominal dead load} \]
\[ P_{LL} = \text{Axial load determined on the basis of nominal live load} \]

2. Permitted Service Axial Load with 5 psf Lateral Load (Section D4.3)

\[ M_{ux} = 1.3(0.9)(5 \text{ psf})(2 \text{ ft O.C.})(15 \text{ ft})(12 \text{ in.}/\text{ft})/8 \]
\[ = 3948.75 \text{ in.-lbs} \]
\[ P_u / \phi_c P_n + C_{mr} M_{ux} / \phi_b M_{xy} \alpha_{xy} + C_{my} M_{uy} / \phi_b M_{xy} \alpha_{xy} \leq 1.0 \quad \text{(Eq. C5-1)} \]
\[ M_{uy} = 0 \]

Assume \( C_{nx} = 1.0 \) (braced against joint translation in the plane of loading, subject to transverse loading between supports with member ends unrestrained)

\[ 1/\alpha_{ax} = 1/(1 - Pu / \phi_c P_n) \quad \text{(Eq. C5-4)} \]
\[ \phi_c = 0.85 \]
\[ P_e = \pi^2 E I_y / (K_w L_d)^3 \quad \text{(Eq. C5-5)} \]
\[ = [\pi^2 (29500)(7.66)] / (12)(15) = 68.8 \text{ kips} \]
\[ \phi_c P_e = 0.85(68.8) = 58.48 \text{ kips} \]
\[ \phi_c P_n = 15.24 \text{ kips} \]

\[ M_{ax} = M_{xxx} = \text{Nominal moments about the centroidal axes determined in accordance with Section C3.1 except lateral buckling provisions} \]

Following Procedure I - Based on Initiation of Yielding

\[ \phi_b = 0.95 \text{ for section with stiffened or partially stiffened compression flanges} \]
\[ M_{ax} = S_{ax} F_y \quad \text{(Eq. C3.1.1-1)} \]
\[ F_y = 50 \text{ ksi} \]

Calculation of \( S_{ax} \) with extreme compression fiber at \( F_y \)

Stiffener (compression): Assume maximum stress is \( F_y \) initially, although actually it will be less.
\[ \begin{align*}
    w &= 0.531 \text{ in.} \\
    \lambda &= \frac{1.052}{\sqrt{0.43}} \cdot (0.531/0.075) \cdot \sqrt{50/29500} \quad \text{(Eq. B2.1-4)} \\
    &= 0.468 < 0.673 \\
    I_s &= 0.000936 \text{ in.}^4 \\
    b &= d' = 0.531 \text{ in.} \quad \text{(max. stress assumption OK)} \\
    d_s &= d'(I/L) \quad \text{(Eq. B4.2-11)} \\
    &= 0.531\left(\frac{0.00936}{0.00392}\right) = 0.127 \text{ in.} \\
    
    \text{See below for calculation of } I_s. \\
    
    \text{Flange (compression):} \\
    w &= 2.413 \text{ in.} \\
    w/t &= 32.17 \\
    S &= 1.28 \sqrt{29500/50} = 31.1 \quad \text{(Eq. B4-1)} \\
    w/t &\geq S \text{ so,} \\
    I_s &= t^4 \left\{ \frac{115(w/t)}{S} + 5 \right\} \quad \text{(Eq. B4.2-13)} \\
    &= (0.075)^4 \left\{ \frac{115(32.17)/31.1}{5} + 5 \right\} = 0.00392 \text{ in.}^4 \\
    D/w &= 0.290 \\
    k &= \left[ 4.82 - 5(0.290) \right] \left( 0.00936/0.00392 \right)^{1/2} + 0.43 = 2.52 \quad \text{(Eq. B4.2-9)} \\
    5.25 - 5(0.290) &= 3.80 > 2.52 \text{ OK} \\
    \lambda &= (1.052/\sqrt{k})(w/t) \sqrt{f/E} \quad \text{(Eq. B2.1-4)} \\
    &= (1.052/\sqrt{2.52})(32.17)\sqrt{50/29500} = 0.878 \\
    \rho &= (1-0.22/0.878)/0.878 = 0.854 \quad \text{(Eq. B2.1-3)} \\
    b &= (0.854)(2.413) = 2.061 \text{ in.} \quad \text{(Eq. B2.1-2)} \\
    
    \text{Assume the web is fully effective initially.} \\
\end{align*} \]

<table>
<thead>
<tr>
<th>Element</th>
<th>L</th>
<th>( y ) Distance from Top Fiber</th>
<th>( L_y ) (in.(^2))</th>
<th>( L_y^2 ) (in.(^3))</th>
<th>( I_y' ) About Own Axis (in.(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.127</td>
<td>0.232</td>
<td>0.029</td>
<td>0.007</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>0.206</td>
<td>0.085</td>
<td>0.018</td>
<td>0.001</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>2.061</td>
<td>0.038</td>
<td>0.078</td>
<td>0.003</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>0.206</td>
<td>0.085</td>
<td>0.018</td>
<td>0.001</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>6.663</td>
<td>3.500</td>
<td>23.321</td>
<td>81.622</td>
<td>24.651</td>
</tr>
<tr>
<td>6</td>
<td>0.206</td>
<td>6.915</td>
<td>1.424</td>
<td>9.850</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>2.413</td>
<td>6.963</td>
<td>16.802</td>
<td>116.990</td>
<td>—</td>
</tr>
<tr>
<td>8</td>
<td>0.206</td>
<td>6.915</td>
<td>1.424</td>
<td>9.850</td>
<td>—</td>
</tr>
<tr>
<td>9</td>
<td>0.531</td>
<td>6.566</td>
<td>3.487</td>
<td>22.893</td>
<td>0.012</td>
</tr>
<tr>
<td>Sum</td>
<td>12.619</td>
<td>6.566</td>
<td>3.487</td>
<td>22.893</td>
<td>0.012</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
    \text{Sum} &= 46.601 \quad 241.217 \quad 24.663 \\
\end{align*}
\]
\[ Y_{cg} = \frac{46.601}{12.619} = 3.693 \text{ in.} \]

\[ I_{\text{eff}} = 0.075(241.217 + 24.663 - 12.619 \times 3.693^2) = 7.03 \text{ in.}^4 \]

Now, check to see if the web is fully effective.

\[ f_1 = \left(\frac{3.693 - 0.169}{3.693}\right) \times 50 = 47.71 \text{ ksi (compression)} \]

\[ f_2 = -\left(\frac{3.307 - 0.169}{3.693}\right) \times 50 = -42.49 \text{ ksi (tension)} \]

\[ \psi = \frac{f_2}{f_1} = \frac{-42.49}{47.71} = -0.8906 \]

\[ k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad \text{(Eq. B2.3-4)} \]

\[ = 4 + 2[1 - (-0.8906)]^3 + 2[1 - (-0.8906)] = 21.30 \]

\[ \lambda = \frac{1.052}{\sqrt{21.30}} \times \frac{6.663 / 0.075}{\sqrt{47.71 / 29500}} = 0.814 > 0.673 \quad \text{(Eq. B2.1-4)} \]

\[ \rho = (1 - 0.22 / 0.814) / 0.814 = 0.896 \quad \text{(Eq. B2.1-3)} \]

\[ b_x = 0.896 \times 6.663 = 5.970 \text{ in.} \quad \text{(Eq. B2.1-2)} \]

\[ b_2 = 5.970 / 2 = 2.985 \text{ in.} \quad \text{(Eq. B2.3-2)} \]

\[ b_1 = b_x / (3 - \psi) = 5.970 / (3 + 0.8906) = 1.534 \text{ in.} \quad \text{(Eq. B2.3-1)} \]

\[ b_1 + b_2 = 4.519 > 3.524 \text{ (compressed portion of the web)} \]

So, the web is fully effective.

\[ S_{cx} = 7.03 / 3.693 = 1.904 \text{ in.}^3 \]

\[ M_{ax} = (1.904) \times 50 = 95.2 \text{ kip-in.} \]

\[ \phi_b M_{ax} = 0.95(95.2) = 90.44 \text{ kip-in.} \]

Using the interaction equation,

\[ P_u / 15.24 + (394.75 / 1000) / (90.44(1 - P_u / 58.48)) \leq 1.0 \]

Iterating with different values of \( P_u \) on the left hand side yields a required axial strength,

\[ P_u \text{ (with 5 psf wind load)} = 14.36 \text{ kips} \]

\[ P_u = 1.2P_{DL} + 0.5P_{LL} \]

\[ = [1.2(P_{DL} / P_{LL}) + 0.5]P_{LL} \]

\[ = 0.74P_{LL} \]
\[ P_{LL} = P_{D}/0.74 = 14.36/0.74 = 19.41 \text{ kips} \]

\[ P_s = P_{DL} + P_{LL} \]
\[ = (1/5+1)P_{LL} \]
\[ = 1.2(19.41) = 23.29 \text{ kips} \]

The axial load alone controls, so \( P_s = 9.94 \text{ kips} \)

Note that (Eq. C5-2) does not control in this case since the simply supported stud has zero end moments (\( M_{ux} = 0 \)) and \( \phi_c P_n < \phi_c P_{no} \) (design axial strength for zero length stud).
EXAMPLE NO. 17
TUBULAR SECTION

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $8 \times 8 \times 0.105$ Square Tube.
3. Unbraced length of column: 10 ft.
4. $K_x = K_y = 1.0$
5. Service axial load: $P = 15$ kips.
6. The eccentricity of axial load at each end of member, $e_y$, is 4 in. and member is bent in single curvature about $x$-axis.
7. $e_x = 0$.
8. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
Check the adequacy of the given section.

Solution:
1. Full section properties:
   $r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.
Length of arc, \( u = 1.57r = 1.57 \times 0.240 = 0.377 \text{ in.} \)

Distance of c.g. from center of radius,
\( c = 0.637r = 0.637 \times 0.240 = 0.153 \text{ in.} \)

\( I_x = I_y \) (doubly symmetric section)

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) (in.)</th>
<th>( \text{Distance to Center of Section} ) (in.)</th>
<th>( L_y^2 ) (in.(^4))</th>
<th>( I' ) about Own Axis (in.(^4))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges</td>
<td>2 x 7.414 = 14.828</td>
<td>3.948</td>
<td>231.120</td>
<td>—</td>
</tr>
<tr>
<td>Corners</td>
<td>4 x 0.377 = 1.508</td>
<td>3.860</td>
<td>22.469</td>
<td>—</td>
</tr>
<tr>
<td>Webs</td>
<td>2 x 7.414 = 14.828</td>
<td>—</td>
<td>—</td>
<td>67.921</td>
</tr>
<tr>
<td>Sum</td>
<td>31.164</td>
<td>253.589</td>
<td>67.921</td>
<td></td>
</tr>
</tbody>
</table>

\( A = Lt = 31.164 \times 0.105 = 3.272 \text{ in.}\(^2\) \)

\( I' = Ly^2 + I' = 253.589 + 67.921 = 321.510 \text{ in.}\(^4\) \)

\( I_x = I_y = I't = 321.510 \times 0.105 = 33.759 \text{ in.}\(^4\) \)

\( r_x = r_y = \sqrt{33.759/3.272} = 3.212 \text{ in.} \)

\( S_x = I_x/4.000 = 33.759/4.000 = 8.440 \text{ in.}\(^3\) \)

\( K_xL_x/r_x = 1.0(10 \times 12)/3.212 = 37.36 < 200 \text{ OK (Section C4-(d))} \)

2. Determination of \( \phi \) (Section C4):
   Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

\( F_c = \frac{\pi^2 E}{(K_xL_x/r_x)^2} \)  \( \text{Eq. C4.1-1} \)

\( = (\pi^2 \times 29500)/(37.36)^2 = 208.60 \text{ ksi} \)

\( F_y/2 = 50/2 = 25.00 \text{ ksi} \)

For \( F_c > F_y/2 \):

\( F_n = F_y(1-F_y/4F_c) \)  \( \text{Eq. C4-2} \)

\( = 50[1-50.00/(4 \times 208.60)] = 47.00 \text{ ksi} \)

\( w = 7.414 \text{ in.} \)

\( w/t = 7.414/0.105 = 70.61 < 500 \text{ OK (Section B1.1-(a)-(2))} \)

\( k = 4.00 \) (Section B2.1-(a))

\( \lambda = (1.052/\sqrt{k})(w/t) \sqrt{E/\pi}, f = F_n \)  \( \text{Eq. B2.1-4} \)

\( = (1.052/\sqrt{4.00})(70.61) \sqrt{47/29500} = 1.482 > 0.673 \)

\( \rho = (1-0.22/\lambda)/\lambda \)  \( \text{Eq. B2.1-3} \)

\( = (1-0.22/1.482)/1.482 = 0.575 \)
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ b = \rho w \]  \hspace{1cm} (Eq. B2.1-2)
\[ = 0.575 \times 7.414 = 4.263 \text{ in.} \]

\[ A_e = A - 4(w - b)t \]
\[ = 3.272 - 4(7.414 - 4.263)(0.105) = 1.949 \text{ in}^2 \]

\[ P_n = A_e F_n \]  \hspace{1cm} (Eq. C4-1)
\[ = 1.949 \times 47.00 = 91.60 \text{ kips} \]
\[ \phi_c = 0.85 \]
\[ \phi_c P_n = 0.85 \times 91.60 = 77.86 \text{ kips} \]

3. \[ P_{D_L} + P_{L_L} = \left( \frac{P_{D_L}}{P_{L_L} + 1} \right) P_{L_L} \]
\[ = \left( \frac{1}{5} + 1 \right) P_{L_L} = 1.2 P_{L_L} = P \]
\[ P_{L_L} = \frac{P}{1.2} = 15/1.2 = 12.5 \text{ kips} \]
\[ P_u = 1.2 P_{D_L} + 1.6 P_{L_L} = (1.2 P_{D_L}/P_{L_L} + 1.6) P_{L_L} = \left[ 1.2 \left( \frac{1}{5} \right) + 1.6 \right] (12.5) = 23 \text{ kips} \]

where
- \( P_{D_L} \) = Axial load determined on the basis of nominal dead load
- \( P_{L_L} \) = Axial load determined on the basis of nominal live load
- \( P_u/\phi_c P_n = \frac{23}{77.86} = 0.295 > 0.15 \)

Must check both interaction equations (Eq. C5-1), (Eq. C5-2).

4. Determination of \( \phi_c P_{n_0} \) (Section C4 for \( F_n = F_y \))
\[ \lambda = \frac{1.052}{\sqrt{4.00}} \sqrt{70.61/29500} = 1.529 > 0.673 \]
\[ \rho = \frac{1-0.22/1.529}{1.529} = 0.560 \]
\[ b = 0.560 \times 7.414 = 4.152 \text{ in.} \]
\[ A_e = 3.272 - 4(7.414 - 4.152)(0.105) = 1.902 \text{ in}^2 \]
\[ P_{n_0} = 1.902 \times 50.00 = 95.10 \text{ kips} \]
\[ \phi_c P_{n_0} = 0.85 \times 95.10 = 80.84 \text{ kips} \]

5. Determination of \( M_{ux}, M_{uy} \) (Section C5):
Since the section is doubly symmetric, the centroidal axes of the effective section at \( \phi_c P_n \) are the same as those of the full section.
\[ M_{ux} = P_{uc} e_x = 23 \times 4 = 92 \text{ kip-in.} \]
\[ M_{uy} = P_{uc} e_y = 0 \]

Since \( M_{uy} = 0 \), the interaction equations (Eq. C5-1) and (Eq. C5-2) reduce to the following:
\[ P_u/\phi_c P_n + C_{u_x} M_{ux}/\phi_c M_{ux} a_{ux} \leq 1.0 \]  \hspace{1cm} (Eq. C5-1)
\[ P_u/\phi_c P_{n_0} + M_{ux}/\phi_c M_{ux} \leq 1.0 \]  \hspace{1cm} (Eq. C5-2)

6. Determination of \( \phi_p M_{ux} \) (Section C3.1):
\( \phi_p M_{ux} \) shall be taken as the smaller of the design flexural strengths calculated according to Sections C3.1.1 and C3.1.2:

a. Section C3.1.1: \( M_{ux} \) will be calculated on the basis of initiation of yielding.
Computation of $I_x$:
For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the compression flange, and that the web is fully effective.

Compression flange: $k = 4.00$ (stiffened compression element supported by a web on each longitudinal edge)

\[
\frac{w}{t} = \frac{7.414}{0.105} = 70.61 < 500 \text{ OK (Section B1.1-(a)-(2))}
\]

\[
\lambda = \frac{(1.052/\sqrt{4.00})(70.61)/\sqrt{50/29500}}{1.529} = 1.529 > 0.673
\]

\[
\rho = \frac{(1-0.22/1.529)}{1.529} = 0.560
\]

\[
b = 0.560 \times 7.414 = 4.152 \text{ in.}
\]

Effective section properties about $x$-axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>$L$ Effective Length (in.)</th>
<th>$y$ Distance from Top Fiber (in.)</th>
<th>$L_y$ (in.$^2$)</th>
<th>$L_y^2$ (in.$^3$)</th>
<th>$I_y$' About Own Axis (in.$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>14.828</td>
<td>4.000</td>
<td>59.312</td>
<td>237.248</td>
<td>67.921</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>0.754</td>
<td>0.140</td>
<td>0.106</td>
<td>0.015</td>
<td>-</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>0.754</td>
<td>7.860</td>
<td>5.926</td>
<td>46.582</td>
<td>-</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>4.152</td>
<td>0.053</td>
<td>0.220</td>
<td>0.012</td>
<td>-</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>7.414</td>
<td>7.948</td>
<td>58.926</td>
<td>468.348</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>27.902</td>
<td>124.490</td>
<td>752.205</td>
<td>67.921</td>
<td></td>
</tr>
</tbody>
</table>

Distance from top fiber to $x$-axis is

\[
y_{cg} = \frac{L_y}{L} = \frac{124.490}{27.902} = 4.462 \text{ in.}
\]

Since the distance of top compression fiber from neutral axis is greater than one half the section depth (i.e., $4.462 > 4.000$), a compression stress of 50 ksi will govern as assumed (i.e., initial yielding is in compression).

To check if the web is fully effective (Section B2.3)

\[
f_1 = [(4.462-0.293)/4.462](50) = 46.72 \text{ ksi (compression)}
\]

\[
f_2 = -(3.538-0.293)/4.462(50) = -36.36 \text{ ksi (tension)}
\]

\[
\psi = \frac{f_2}{f_1} = \frac{-36.36}{46.72} = -0.778
\]

\[
k = 4+2[1-(-0.778)]^3 + 2[1-(-0.778)] = 18.798
\]

\[
h = w = 7.414 \text{ in., } h/t = \frac{w}{t} = \frac{7.414}{0.105} = 70.61
\]

\[
h/t = 70.61 < 200 \text{ OK (Section B1.2-(a))}
\]

\[
\lambda = (1.052/\sqrt{18.798})(70.61)/\sqrt{46.72/29500} = 0.682 > 0.673
\]

\[
\rho = (1-0.22/0.682)/0.682 = 0.993
\]

\[
b_c = 0.993 \times 7.414 = 7.362 \text{ in.}
\]
b_1 = b_0/(3-\psi) \quad \text{Eq. B2.3-1}
= 7.362/[3-(0.778)] = 1.949 \text{ in.}

Compression portion of the web calculated on the basis of the effective section = y_{cg} \cdot 0.293 = 4.462 \cdot 0.293 = 4.169 \text{ in.}

Since b_1 + b_2 = 5.630 \text{ in.} > 4.169 \text{ in.}, b_1 + b_2 shall be taken as 4.169 \text{ in.}

This verifies the assumption that the web is fully effective.

\[ \text{I}_x' = \text{Ly}^2 + I_i' - \text{Ly}_{cg}^2 \]
\[ = 752.205 + 67.921 - 27.902(4.462)^2 \]
\[ = 264.613 \text{ in.}^3 \]

Actual \( \text{I}_x \) = \( t' \text{I}_x' \)
\[ = (0.105)(264.613) = 27.784 \text{ in.}^4 \]

\[ S_c = \frac{\text{I}_x}{y_{cg}} = 27.784/4.462 = 6.227 \text{ in.}^3 \]

\[ M_{nx} = S_c F_y \]
\[ = (6.227)(50) = 311.35 \text{ kip-in.} \]

\[ \phi_b = 0.95 \]

\[ \phi_b M_{nx} = 0.95 \times 311.35 = 295.78 \text{ kip-in.} \]

b. Section C3.1.2: \( M_{na} \) will be calculated on the basis of lateral buckling strength. However for this square tube (closed box-type member) the provisions of Section C3.1.2 do not apply.

Therefore

\[ \phi_b M_{na} = 295.78 \text{ kip-in.} \]

7. \( C_{mx} = 0.6 - 0.4(M_1/M_2) \)
\[ M_1/M_2 = -(92/92) = -1.0 \text{ (single curvature)} \]
0.6 - 0.4(0.6 - 1.0) = 0.6

8. Determination of \( 1/\alpha_{mx} \):

\[ \phi_c = 0.85 \]

\[ P_E = \pi^2 EI_x/(K_x L_x)^2 \quad \text{Eq. C5-5} \]
\[ = 33.759 \text{ in.}^4 \]

\[ K_x L_x = 1.0(10 \times 12) = 120 \text{ in.} \]

\[ P_E = (\pi^2(29500)(33.759))/(120)^2 = 682.57 \text{ kips} \]

\[ 1/\alpha_{mx} = 1/(1-Pu/\phi_c P_E) \quad \text{Eq. C5-4} \]
\[ = 1/[1-23/(0.85 \times 682.57)] = 1.041 \]

\[ \alpha_{mx} = 0.960 \]

9. Check interaction equations:

\[ P_{c} / \phi_c P_{E} + C_{mx} M_{mf} / \phi_b M_{nx} \alpha_{mx} \leq 1.0 \quad \text{Eq. C5-1} \]
23/77.86+1\times92/(295.78\times0.960) = 0.295+0.324 = 0.619 < 1.0 \text{ OK}

\frac{P_v}{\Phi P_{mo}} + \frac{M_{w}}{\Phi M_{nkr}} \leq 1.0 \quad (\text{Eq. C5-2})

23/80.84+92/295.78 = 0.285+0.311 = 0.596 < 1.0 \text{ OK}

Therefore the section is adequate for the applied loads.
EXAMPLE NO. 18
FLAT SECTION WITH BOLTED CONNECTION

Given:
1. Steel: \( F_y = 33 \, \text{ksi}, \, F_u = 45 \, \text{ksi} \).
2. Bolts conforming to ASTM A307 with washers under bolt head and nut.
3. Detail of connection shown in sketch.

**Required:**
Determine the maximum design strength, $\phi P_n$.

**Solution:**
Thickness of thinnest part connected, $t$

\[ t = 0.105 \text{ in.} < \frac{3}{16} = 0.188 \text{ in.} \]

Therefore, Section E3 applies.

1. Design strength based on spacing and edge distance (Section E3.1)
   a. $F_y/F_y = 45/33 = 1.36 > 1.15$
      For $F_y/F_y > 1.15$, $\phi = 0.70$
      \[ P_n = teF_u \]  \hspace{1cm} (Eq. E3.1-1)
      \[ P_n = 0.105(1)(45) = 4.73 \text{ kips/bolt} \]
      \[ \phi P_n = 0.7(2 \text{ bolts})(4.73 \text{ kips/bolt}) = 6.62 \text{ kips} \]
   b. Distance between bolt hole centers must be greater than 3d.
      \[ 3d = 3(0.5) = 1.5 \text{ in.} < 2 \text{ in.} \text{ OK} \]
   c. Distance between bolt hole center and edge of connecting member must be greater than 1.5d.
      \[ 1.5d = 1.5(0.5) = 0.75 \text{ in.} < 1 \text{ in.} \text{ OK} \]

2. Design strength based on tension on net section.

   Required tension strength on net section of bolted connection shall not exceed $\phi_t T_n$ from Section C2:

   \[ A_n = 0.105 \left[ 4-2(1/2+1/16) \right] = 0.302 \text{ in.}^2 \]
   \[ T_n = A_nF_y \] \hspace{1cm} (Eq. C2-1)
   \[ T_n = (0.302)(33) = 9.97 \text{ kips} \]
   \[ \phi_t = 0.95 \]
   \[ \phi_t T_n = 0.95(9.97) = 9.47 \text{ kips} \]
   or $\phi P_n$ from Section E3.2:

   Since $t = 0.105 \text{ in.} < \frac{3}{16} \text{ in.}$ and washers are provided under both bolt head and nut
   \[ P_n = (1.0-0.9r+3rd/s)F_uA_n \leq F_uA_n \] \hspace{1cm} (Eq. E3.2-1)

where in this case:
\[ r = 2(\phi P_n/2)/\phi P_n = 1 \]
\[ d = 0.5 \text{ in.} \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ s = 2 \text{ in.} \]

\[ P_n = [1.0-0.9(1)+3(1)(0.5)/2] (45)(0.302) \]
\[ = 11.55 \text{ kips} < 45(0.302) = 13.59 \text{ kips OK} \]

\[ \phi = 0.55 \text{ for single shear connection} \]

\[ \phi P_n = 0.55(11.55) = 6.35 \text{ kips} \]

Therefore, design strength based on tension on net section is 6.35 kips.

3. Design strength based on bearing (Section E3.3)

For single shear with washers under bolt head and nut; 0.024 in. \( \leq t = 0.105 \text{ in.} < 3/16 \text{ in.} \)

From Table E3.3-1

\[ P_n = 3.00F_{ud}t = 3.00(45)(0.5)(0.105) = 7.09 \text{ kips/bolt} \]

\[ \phi = 0.60 \]

\[ \phi P_n = 0.60(2 \text{ bolts})(7.09 \text{ kips/bolt}) = 8.51 \text{ kips} \]

4. Design strength based on bolt shear (Section E3.4)

\[ P_n = A_bF_n \quad \text{(Eq. E3.4-1)} \]

\[ A_b = (\pi/4)(0.5)^2 = 0.196 \text{ in.}^2 \]

\[ F_n = F_{nv} = 27 \text{ ksi (Table E3.4-1, d} \geq 1/2 \text{ in.)} \]

\[ P_n = (27)(0.196) = 5.29 \text{ kips/bolt} \]

\[ \phi = 0.65 \]

\[ \phi P_n = 0.65(2 \text{ bolts})(5.29 \text{ kips/bolt}) = 6.88 \text{ kips} \]

5. Comparing the values from 1, 2, 3, and 4 above, the design tensile strength on the net section of the connected part controls and

\[ \phi P_n = 6.35 \text{ kips} \]
EXAMPLE NO. 19
FLAT SECTION WITH ARC SPOT WELDED CONNECTION

Given:
1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, $F = 6.8$ kips
3. Detail of connection shown in sketch.

Required:
Design the connection to transmit \( F = 6.8 \text{ kips} \) using arc spot welds having \( 3/4 \text{ in.} \) visible diameter.

Solution:
1. Weld Dimensions
\[
\begin{align*}
\text{d} & = 0.75 \text{ in.} \\
\text{d}_a & = \text{d}-\text{t} = 0.75-0.06 = 0.69 \text{ in.} \\
\text{d}_c & = 0.7\text{d}-1.5\text{t} \quad \text{but } \leq 0.55\text{d} \\
& = 0.7(0.75)-1.5(0.06) = 0.44 \\
0.55\text{d} & = 0.55(0.75) = 0.41 \text{ in.} \\
\text{0.44 in. } > 0.41 \text{ in.}, \text{ use } \text{d}_c = 0.41 \text{ in.} > 3/8 \text{ in.} \text{ OK}
\end{align*}
\]

2. Determine number of arc spot welds required.
   a. \( P_n = 0.589\text{d}_a^2F_{xx} \) \hspace{1cm} (Eq. E2.2-1)

   Using E60 electrode, \( F_{xx} = 60 \text{ ksi} \)

   \[
   \begin{align*}
   P_n & = 0.589(0.69)^2(60) = 5.94 \text{ kips/weld} \\
   \phi & = 0.60 \\
   \phi P_n & = 0.60(5.94) = 3.56 \text{ kips/weld}
   \end{align*}
   \]

   b. Compute \( d_u/t = 0.69/0.06 = 11.5 \)

   Compute \( \sqrt{E/F_u} = \sqrt{29500/65} = 21.3 \)

   For \( d_u/t = 11.5 < 0.815 \sqrt{E/F_u} = 17.4 \)

   \[
   \begin{align*}
   P_n & = 2.20\text{d}_a F_u \\
   & = 2.20(0.06)(0.69)(65) = 5.92 \text{ kips/weld} \\
   \phi & = 0.60 \\
   \phi P_n & = 0.60(5.92) = 3.55 \text{ kips/weld (control)}
   \end{align*}
   \]

   Number of welds = \( 6.8 \text{ kips}/(3.55 \text{ kips/weld}) = 1.92 \text{ welds, use } 2. \)

3. Check the edge distance and spacing requirements
   a. \( F_u/F_y = 65/50 = 1.3 > 1.15 \)

   For \( F_u/F_y > 1.15, \phi = 0.70 \)

   \[
   \begin{align*}
   e_{\text{min}} & = P_u/\phi F_ut \\
   & = (6.8/2)/ [ (0.70)(65)(0.06) ] = 1.25 \text{ in.}
   \end{align*}
   \]

   1.25 in. edge distance = \( e_{\text{min}} = 1.25 \text{ in.} \text{ OK} \)
b. Edge distance shall not be less than 1.5d.

\[ 1.5d = 1.5(0.75) = 1.13 \text{ in.} < 1.25 \text{ in. } \text{OK} \]

c. Clear distance between weld and end of member shall not be less than 1.0d.

\[ 1.0d = 1.0(0.75) = 0.75 \text{ in.} \]

Clear distance \( = 1.25 - 0.375 = 0.875 \text{ in.} > 0.75 \text{ in. } \text{OK} \)

d. Thinnest connected part, \( t = 0.06 \text{ in.} < 0.15 \text{ in. } \text{OK} \)

4. Use 2-3/4 in. diameter spot welds in the configuration shown. No weld washers required because \( t = 0.06 \text{ in.} > 0.028 \text{ in.} \)
EXAMPLE NO. 20.
FLAT SECTION WITH ARC SEAM WELDED CONNECTION

Given:
1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, $F = 4.1$ kips
3. Detail of connection shown in sketch.

**Required:**

Design the connection to transmit $F = 4.1$ kips using arc seam welds.

Try $d = \frac{1}{2}$ in..

**Solution:**

1. Required strength shall not exceed either

$$\phi P_n = \phi (\pi d_e^2 / 4 + Ld_e)(0.75F_{xx}) \quad (Eq. E2.3-1)$$

Try E60 electrode, $F_{xx} = 60$ ksi

$$L = 1.5 \text{ in., or maximum } 3d, 3(0.5) = 1.5 \text{ in. OK}$$

$$d_a = 0.5 - 0.06 = 0.44 \text{ in.} \quad (Eq. E2.3-3)$$

$$d_e = 0.7d - 1.5t$$

$$= 0.7(0.5) - 1.5(0.06) = 0.260 \text{ in.}$$

$$P_n = \left[ \frac{\pi (0.26)^2}{4} + (1.5)(0.26) \right] \left[ 0.75(60) \right]$$

$$= 19.94 \text{ kips}$$

$$\phi P_n = 0.60(19.94) = 11.96 \text{ kips.}$$

OR

$$\phi P_n = 2.5t F_u (0.25L + 0.96d_e) \quad (Eq. E2.3-2)$$

$$= 2.5(0.06)(65) \left[ 0.25(1.5) + 0.96(0.44) \right] = 7.77 \text{ kips}$$

$$\phi P_n = 0.60(7.77) = 4.66 \text{ kips (control)}$$

$$\phi P_n = 4.66 \text{ kips} > F = 4.10 \text{ kips OK}$$

2. Determine minimum edge distance in line of force.

a. $F_u/F_y = 65/50 = 1.3 > 1.15$

For $F_u/F_y > 1.15$, $\phi = 0.70$

$$e_{\min} = P_u/\phi F_u t \quad (Eq. E2.2-6)$$

$$= 4.1 / \left[ (0.70)(65)(0.06) \right] = 1.50 \text{ in.}$$

1.50 in. edge distance = $e_{\min}$ = 1.50 in. OK

b. Edge distance shall not be less than 1.5d.

$$1.5d = 1.5(0.50) = 0.75 \text{ in.} < 1.50 \text{ in. OK}$$
c. Clear distance between weld and end of member shall not be less than 1.0d.

\[ 1.0d = 1.0(0.50) = 0.50 \text{ in.} \]

Clear distance = 1.50 - 0.25 = 1.25 in. > 0.50 in. OK

3. Use arc seam welded connection per sketch with E60 electrode and \( d = \frac{1}{2} \text{ in.} \).
EXAMPLE NO. 21
FLAT SECTION WITH LAP FILLET WELDED CONNECTION

Given:
1. Steel: \( F_y = 50 \text{ ksi}, F_u = 65 \text{ ksi} \).
2. Total Required Strength, \( F = 6.1 \text{ kips} \).
3. Detail of connection shown in sketch.

Required:
Check to see if longitudinal fillet welded connection is adequate to transmit \( F = 6.1 \text{ kips} \).

Solution:
1. \( L/t = 2/0.06 = 33.33 > 25 \)
   For \( L/t \geq 25 \),
   \[
   \phi = 0.55 \\
   P_n = 0.75tLF_u 
   \]  
   (Eq. E2.4-2)
\[
= 0.75(0.06)(2)(65) = 5.85 \text{ kips}
\]

\[
\phi P_n = 0.55(5.85) = 3.22 \text{ kips/weld}
\]

2. Note: \( t = 0.06 \text{ in.} < 0.150 \text{ in.} \). Therefore, (Eq. E2.4-4) does not apply.

3. \((3.22 \text{ kips/weld})(2 \text{ welds}) = 6.44 \text{ kips} < 6.1 \text{ kips OK}\)
EXAMPLE NO. 22
FLAT SECTION WITH SINGLE FLARE BEVEL GROOVE WELDED CONNECTION

Given:
1. Steel: \( F_y = 50 \text{ ksi}, F_u = 65 \text{ ksi} \).
2. Total Required Strength, \( F = 6.1 \text{ kips} \).
3. Detail of connection shown in sketch.
4. Transverse loading.

Required:
Design the welded connection to transfer \( F = 6.1 \text{ kips} \).

Solution:
1. For flare-bevel groove welds, transverse loading, the required strength \( F \) shall not exceed \( \phi P_n \)
   \[
   \phi = 0.55 \\
   P_n = 0.833tL F_u 
   \]
   (Eq. E2.5-1)
Solve for $L$

$$L = \frac{F}{0.833}$$

$$\phi_t F_u = \frac{6.1}{0.833(0.55)(0.06)(65)} = 3.41 \text{ in.}$$

2. Use 3.5 in. long flare bevel groove weld per sketch.

3. Size of weld 1/8 in. (1/16 in. min)
EXAMPLE NO. 23
C-SECTION BRACING UNDER GRAVITY LOADING

Given:
1. Steel: $F_y = 50$ ksi.
2. 60 ft. wide building, 20 ft. bays, simply supported purlins on 5-foot centers.
4. Same channel as in ex. # 15.
5. Dead load = 3 psf; live load = 15 psf

Required:
Design of bracings of the roof system under gravity loads, using Section D3.2.1 of Specification.

Solution:

\[
P = 0.05W
\]

\[
W = (1.2 \times 3 + 1.6 \times 15) \times 30 \times 20 = 16560 \text{ lbs.}
\]
\[ P = 0.05(16560) = 828 \text{ lbs} \]

A total restraint force of 828 lbs. must be supplied in each bay. It is up the designer to decide what devices can be used to supply this force.
EXAMPLE NO. 24
Z-SECTION BRACING UNDER GRAVITY LOADING

Given:
Same roof system as Example 23, with Z-section instead of channel.
Z-section is the one used in Example 4.

Required:
Design of bracings of the roof system under gravity loads, using Section D3.2.1 of Specification.

Solution:

\[ b = (3/32 + 0.06)(\tan 22.5°) + 1.625 = 1.689 \text{ in.} \]

\[ d = 6.000 \text{ in.} \]

\[ t = 0.060 \text{ in.} \]

\[ n_p = 7 \]

\[ \theta = 4.76°, \sin \theta = 0.083 \]

\[ W = (1.2 \times 3 + 1.6 \times 15) \times 30 \times 20 = 16560 \text{ lbs.} \]

1. Single span system with restraints at supports:

\[ P_L = 0.5 \times \frac{0.22 b^{1.50}}{(n_p^{0.72} d^{0.90} t^{0.60}) - \sin \theta} W \]  

\[ (\text{Eq. D3.2.1-1}) \]

\[ P_L = 0.5 \times \frac{0.474 (1.689)^{1.52}}{(7^{0.72} x 6.000^{0.9} x 0.060^{0.6}) - 0.083} \times 16560 \]

\[ P_L = 373 \text{ lbs.} \]

2. Single span system with third point restraints:

\[ P_L = \frac{0.224 b^{1.32}}{(n_p^{0.65} d^{0.81} t^{0.50}) - \sin \theta} W \]  

\[ (\text{Eq. D3.2.1-2}) \]

\[ P_L = 0.224 (1.689)^{1.32} / (7^{0.65} \times 6.000^{0.83} \times 0.060^{0.50} - 0.083) \times 16560 \]

\[ P_L = 573 \text{ lbs.} \]

3. Single span system with midspan restraint:

\[ P_L = \frac{0.224 b^{1.32}}{(n_p^{0.65} d^{0.81} t^{0.50}) - \sin \theta} W \]  

\[ (\text{Eq. D3.2.1-3}) \]

\[ P_L = 0.224 (1.689)^{1.32} / (7^{0.65} \times 6.000^{0.83} \times 0.060^{0.50} - 0.083) \times 16560 \]

\[ P_L = 553 \text{ lbs.} \]
Each brace will be designed to resist one of the $P_L$ forces, determined above, depending on the restraint condition of the span.
EXAMPLE NO. 25
WALL PANEL

Given:
1. Steel: \( F_y = 50 \text{ ksi} \).
2. Section: Shown in sketch above.
3. Dead load to live load ratio \( D/L = 1/5 \) and \( 1.2D + 1.6L \) governs the design.

Required:
Section properties for positive and negative bending.

Solution:
1. Linear Properties.
   Elements 4 and 10
   90° corners, \( r = R + t/2 = 0.125 + 0.030/2 = 0.140 \text{ in.} \)
   Length of arc, \( u = 1.57r = 1.57 	imes 0.140 = 0.220 \text{ in.} \)
   Distance of c.g. from center of radius,
\[ c_1 = 0.637r = 0.637 \times 0.140 = 0.089 \text{ in.} \]

Element 7:
\[ r = 0.140 \text{ in., } \theta = 45^\circ = 0.785 \text{ rad.} \]
\[ c_1 = rsin\theta/\theta = 0.140 \times 0.707 / 0.785 = 0.126 \text{ in.} \]
\[ n = 0.350 - 2 \times 0.140(1 - \cos 45^\circ) = 0.350 - 0.082 = 0.268 \text{ in.} \]
\[ l_b = \frac{0.268}{\sin 45^\circ} = 0.379 \text{ in.} \]
\[ l_a = 0.785 \times 0.140 = 0.110 \text{ in.} \]

\[ I'(\text{straight portions}) = 2 \times \frac{1}{12} x l_b x n^2 \]
\[ = 2 \times \frac{1}{12} x 0.379 x 0.268^2 = 0.0045 \text{ in.}^3 \]

\[ I'(\text{arcs}) = 4 \times 0.110 \times (0.350/2 - 0.140 + 0.126)^2 = 0.0114 \text{ in.}^3 \]

\[ I' \geq I'(\text{straight portions}) + I'(\text{arcs}) = 0.0045 + 0.0114 = 0.0159 \text{ in.}^3 \]

Check adequacy of intermediate stiffener according to Section B5.

For \( w/t = (3.000 - 0.140 - 0.932/2)/0.030 = 79.8 \) (Element 6)

\[ I_{\text{min}} = \left[ 3.66 \sqrt{(w/t)^2 - (0.136E/F_y)} \right] t^4 \]  
\[ = \left[ 3.66 \sqrt{(79.8)^2 - (0.136 \times 29500/50)} \right] (0.030)^4 \]
\[ = 0.000235 \text{ in.}^4 < 0.000477 \text{ in.}^4 \text{ satisfactory} \]

\[ L_{sf} = 4l_b + 2l_b = 0.440 + 0.758 = 1.198 \text{ in.} \]
Element 1

<table>
<thead>
<tr>
<th></th>
<th>L</th>
<th>Distance from Center of Top Flange</th>
<th>Ly</th>
<th>Ly'</th>
<th>Ly'²</th>
<th>Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(in.)</td>
<td>(in.)</td>
<td>(in.)²</td>
<td>(in.)³</td>
<td>(in.)³</td>
</tr>
<tr>
<td>90° Corner</td>
<td>0.220</td>
<td>0.140-0.089 =0.051</td>
<td>0.011</td>
<td>—</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Straight Segments</td>
<td>0.500</td>
<td>—</td>
<td>0.265</td>
<td>0.133</td>
<td>0.035</td>
<td>0.003</td>
</tr>
<tr>
<td>Semi-Circled</td>
<td>0.440</td>
<td>0.390+0.089 =0.479</td>
<td>0.211</td>
<td>0.101</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Sum</td>
<td>1.160</td>
<td>0.355</td>
<td>0.136</td>
<td>0.003</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

\[y_{eg} = \frac{0.355}{1.160} = 0.306 \text{ in.}\]

\[I'_{x} = I'_{1} + Ly'^{2} - Ly_{eg}^{2} = 0.003 + 0.136 - 1.160 \times 0.306^2 = 0.139 - 0.109 = 0.030 \text{ in.}^3\]

\[I_{x} = I'_{xt} = 0.030 \times 0.030 = 0.00090 \text{ in.}^4\]

2. Section Modulus for Load Determination - Positive Bending
Since the neutral axis will be below the center of the cross section, the compression stress will govern.

Element 1 from Section B3.1(a)

\[w = 0.25 \text{ in.}\]

\[k = 0.43\]

\[f = F_y\]

\[\lambda = \frac{(1.052/\sqrt{0.43})(0.25/0.030)\sqrt{50/29500}}{31.091} = 0.550\]

(Eq. B2.1-4)

\[\rho = 1 \text{ for } \lambda \leq 0.673\]

\[b = w = 0.25 \text{ in.}\]

(Eq. B2.1-1)

Element 2 from Section B4.2(a)

\[w/t = \frac{[3-3(0.140)]/0.030 = 86}{f = F_y}\]

\[S = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/50} = 31.091\]

(Eq. B4-1)

\[D/w = \frac{[0.25+2(0.125)+1.5(0.030)]/[3-3(0.140)] = 0.211}{n = 1/3 \text{ for } w/t > S}\]

\[I_a = (0.030)^4 [115(86/31.091)+5] = 0.000262 \text{ in.}^4\]

(Eq. B4.2-13)

\[I_s = I_t = 0.00090 \text{ in.}^4\]

\[k = 3.57(0.00090/0.000262)^{1/3} + 0.43 = 5.814 > 4\]

(Eq. B4.2-10)

\[\lambda = \frac{(1.052/\sqrt{0.43})(86)/50/29500}{1.862}\]

(Eq. B2.1-4)
\[ p = \frac{(1-0.22/1.862)/1.862}{1.862} = 0.474 \quad (\text{Eq. B2.1-3}) \]

\[ b = pw = 0.474 \left[ 3-3(0.140) \right] = 1.223 \text{ in.} \quad (\text{Eq. B2.1-2}) \]

\[ A_s = A'_s = 1.160 \times 0.030 = 0.0348 \text{ in.}^2 \text{ for } I_a \geq I_a \quad (\text{Eq. B4.2-12}) \]

**Element 9 from Section B3.2(a)**

\[ w = 0.415 - 0.030 - 0.125 = 0.26 \text{ in.} \]

\[ k = 0.43 \]

\[ f < F_y, \text{ use } F_y \text{ as conservative value} \]

\[ \lambda = (1.052/\sqrt{0.43})(0.26/0.030) \sqrt{50/29500} = 0.572 \quad (\text{Eq. B2.1-4}) \]

\[ \rho = 1 \text{ for } \lambda \leq 0.673 \]

\[ b_p = w = 0.26 \text{ in.} \quad (\text{Eq. B2.1-1}) \]

**Element 3 from Section B4.2(a)**

\[ w/t = [2-2(0.140)]/0.030 = 57.333, f = F_y \]

\[ S = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/50} = 31.091 \quad (\text{Eq. B4-1}) \]

\[ D/w = [0.415-0.030]/[2-2(0.140)] = 0.233 \]

\[ n = 1/3 \text{ for } w/t > S \]

\[ I_a = (0.030)^4 \left[ \frac{115(57.333/31.091)+5}{115(57.333/31.091)+5} \right] = 0.000176 \text{ in.}^4 \quad (\text{Eq. B4.2-13}) \]

\[ I_s = bh^3/12 = (0.030)(0.415-0.125-0.030)^3/12 = 0.000044 \text{ in.}^4 \]

\[ k = 3.57(0.000044/0.000176)^{1/3}+0.43 = 2.679 < 4 \quad (\text{Eq. B4.2-10}) \]

\[ \lambda = (1.052/\sqrt{2.679})(57.333) \sqrt{50/29500} = 1.517 \quad (\text{Eq. B2.1-4}) \]

\[ \rho = (1-0.22/1.517)/1.517 = 0.564 \quad (\text{Eq. B2.1-3}) \]

\[ b = pw = 0.564 \left[ 2-2(0.140) \right] = 0.970 \text{ in.} \quad (\text{Eq. B2.1-2}) \]

\[ d_s = (I_s/I_a)d'_s \]

\[ = (0.000044/0.000176)(0.26) = 0.065 \text{ in.} \quad (\text{Eq. B4.2-11}) \]

**Effective section properties about x axis:**
**Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification**

<table>
<thead>
<tr>
<th>Element</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L Effective Length (in.)</td>
<td>y Distance from Top Fiber (in.)</td>
<td>Ly (in.²)</td>
<td>Ly² (in.⁴)</td>
</tr>
<tr>
<td>1</td>
<td>1.160</td>
<td>0.321</td>
<td>0.372</td>
<td>0.120</td>
</tr>
<tr>
<td>2</td>
<td>1.223</td>
<td>0.015</td>
<td>0.018</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>0.970</td>
<td>0.015</td>
<td>0.015</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>0.660</td>
<td>0.066</td>
<td>0.044</td>
<td>0.003</td>
</tr>
<tr>
<td>5</td>
<td>3.440</td>
<td>1.015</td>
<td>3.492</td>
<td>3.544</td>
</tr>
<tr>
<td>6</td>
<td>4.788</td>
<td>2.015</td>
<td>9.648</td>
<td>19.440</td>
</tr>
<tr>
<td>7</td>
<td>2.396</td>
<td>1.840</td>
<td>4.409</td>
<td>8.112</td>
</tr>
<tr>
<td>8</td>
<td>2.068</td>
<td>2.015</td>
<td>4.167</td>
<td>8.397</td>
</tr>
<tr>
<td>9</td>
<td>0.065</td>
<td>0.188</td>
<td>0.012</td>
<td>0.002</td>
</tr>
<tr>
<td>10</td>
<td>0.440</td>
<td>1.964</td>
<td>0.864</td>
<td>1.697</td>
</tr>
<tr>
<td>Sum</td>
<td>17.210</td>
<td>23.041</td>
<td>41.315</td>
<td>0.878</td>
</tr>
</tbody>
</table>

\[
y_{cg} = \frac{Ly}{L} = \frac{23.041}{17.210} = 1.339 \text{ in.}
\]

\[
I_x' = Ly^2 + I_x - Ly^2_{cg}
\]

\[
= 41.315 + 0.878 - 17.210(1.339)^2 = 11.337 \text{ in.}^3
\]

\[
I_x = I_x' \times t
\]

\[
= 11.337(0.030) = 0.340 \text{ in.}^4
\]

\[
S_x = I_x/y_{cg} = 0.340/1.339 = 0.254 \text{ in.}^3
\]

\[
M_n = S_x F_y = 0.254(50) = 12.70 \text{ kip-in.}
\]

\[
\phi_b = 0.95
\]

\[
\phi_b M_n = 0.95 \times 12.70 = 12.07 \text{ kip-in.}
\]

**Element 5 from Section B2.3(a)**

\[
y_{cg} = 1.339 \text{ in.}
\]

\[
f_1 = \left(\frac{(1.339-0.125-0.030)/1.339}{(50)}\right) = 44.212 \text{ ksi}
\]

\[
f_2 = \left(\frac{(2.030-0.125-0.030-1.339)/1.339}{(50)}\right) = -20.015 \text{ ksi}
\]

\[
\psi = f_2/f_1 = -20.015/44.212 = -0.453
\]

\[
k = 4+2(1-\psi)^2+2(1-\psi)
\]

\[
= 4+2[1-(-0.453)]^2+2[1-(-0.453)] = 13.041
\]

\[
\lambda = (1.052/\sqrt{k})(w/t) \sqrt{F/E}, \ f = f_1
\]

\[
= (1.052/\sqrt{13.041})(2.030)
\]
For \( \lambda = 0.647 < 0.673 \)

\[
\begin{align*}
\beta & = w = 1.720 \text{ in.} \\
\beta_2 & = \beta /2 = 1.720/2 = 0.860 \text{ in.} \\
\beta_1 & = \beta / (3-\psi) = 1.720/ (3-(-0.453)) = 0.498 \text{ in.} \\
w_c & = 1.339 - 0.030 - 0.125 = 1.184 \text{ in.} \\
b_1 + b_2 & = 0.498 + 0.860 = 1.358 \text{ in.} > w_c = 1.184 \text{ in.}
\end{align*}
\]

Thus element 5 is fully effective so properties above are correct.

3. Moment of Inertia for Deflection Determination - Positive Bending

\[
\phi_b M_n = 1.2 M_{DL} + 1.6 M_{LL} = [1.2( M_{DL} / M_{LL} ) + 1.6 ] M_{LL} = [1.2(1/5) + 1.6 ] M_{LL} = 1.84 M_{LL}
\]

\[
M_{LL} = \phi_b M_n / 1.84 = 12.07 / 1.84 = 6.56 \text{ kip-in.}
\]

\[
M_k = M_{DL} + M_{LL} = (1/5 + 1) M_{LL} = 1.2 (6.56) = 7.87 \text{ kip-in.}
\]

where

\[
M_{DL} = \text{Moment determined on the basis of nominal dead load}
\]

\[
M_{LL} = \text{Moment determined on the basis of nominal live load}
\]

Computation of \( I_{eff} \), first approximation:

* Assume a compressive stress of \( f = 30 \text{ ksi} \) in the top fibers of the section.

* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 2 from Section B4.2(b)

\[
\begin{align*}
\lambda_c & = 0.256 + 0.328(2.580/0.030) \sqrt{50/29500} = 1.417 \quad \text{(Eq. B2.1-10)} \\
S & = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/30} = 40.138 \quad \text{(Eq. B4-1)} \\
I_a & = (0.030)^4 [115(86/40.138) + 5 ] \times 0.000204 \text{ in.}^4 \quad \text{(Eq. B4.2-13)} \\
k & = 3.57(0.00090/0.000204)^{1/3} + 0.43 = 6.282 > 4 \quad \text{(Eq. B4.2-10)}
\end{align*}
\]
\( k = 4 \)

\[ \lambda = \frac{(1.052/\sqrt{4})(86)\sqrt{30/29500}}{29500} = 1.443 \]  
\( \text{(Eq. B2.1-4)} \)

\[ \rho = \frac{(0.41+0.59 \sqrt{50/30} -0.22/1.443)/1.443}{29500} = 0.706 \]  
\( \text{(Eq. B2.1-9)} \)

\[ b = \rho w = 0.706(2.580) = 1.821 \text{ in.} \]  
\( \text{(Eq. B2.1-2)} \)

\[ A_s = A'_s = 0.0348 \text{ in.}^2 \]  
\( \text{(Eq. B4.2-12)} \)

Element 3 from Section B4.2(b)

\[ w = 2.2(0.140) = 1.720 \text{ in.} \]

\[ \lambda_c = 0.256+0.328(1.720/0.030)\sqrt{50/29500} = 1.030 \]  
\( \text{(Eq. B2.1-10)} \)

\[ S = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/30} = 40.138 \]  
\( \text{(Eq. B4.1)} \)

\[ I_s = (0.030)^4 \left[ 115(57.333/40.138)+5 \right] = 0.000137 \text{ in.}^4 \]  
\( \text{(Eq. B4.2-13)} \)

\[ k = 3.57(0.000044/0.000137)^{1/3} + 0.43 = 2.875 < 4 \]  
\( \text{(Eq. B4.2-10)} \)

\[ k = 2.875 \]

\[ \lambda = \frac{(1.052/\sqrt{2.875})(57.333)\sqrt{30/29500}}{29500} = 1.134 \]  
\( \text{(Eq. B2.1-4)} \)

\[ \rho = \frac{(0.41+0.59 \sqrt{50/30} -0.22/1.134)/1.134}{29500} = 0.862 \]  
\( \text{(Eq. B2.1-9)} \)

\[ b = \rho w = 0.862(1.720) = 1.483 \text{ in.} \]  
\( \text{(Eq. B2.1-2)} \)

\[ d_s = (I_s/I_a)d'_s \]  
\( \text{(Eq. B4.2-11)} \)

\[ = (0.000044/0.000137)(0.26) = 0.084 \text{ in.} \]

Effective section properties about x axis:
<table>
<thead>
<tr>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.²)</th>
<th>Ly² (in.⁴)</th>
<th>L from Own Axis (in.)</th>
<th>Ly/L</th>
<th>Ly²/L²</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.321</td>
<td>0.372</td>
<td>0.120</td>
<td>1.160</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.015</td>
<td>0.027</td>
<td></td>
<td>1.821</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.015</td>
<td>0.022</td>
<td></td>
<td>1.483</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.066</td>
<td>0.044</td>
<td>0.003</td>
<td>0.660</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.015</td>
<td>3.492</td>
<td>3.544</td>
<td>3.440</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.015</td>
<td>9.648</td>
<td>19.440</td>
<td>4.788</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.840</td>
<td>4.409</td>
<td>8.112</td>
<td>2.396</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.015</td>
<td>4.167</td>
<td>8.397</td>
<td>2.068</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.197</td>
<td>0.017</td>
<td>0.003</td>
<td>0.084</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.964</td>
<td>0.864</td>
<td>1.697</td>
<td>0.440</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>23.062</td>
<td>41.316</td>
<td>18.340</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ y_{cg} = \frac{L_y}{L} = 23.062/18.340 = 1.257 \text{ in.} \]

\[ I'_{x} = L_y^2 + I'_{L} - L_y^2 y_{cg} \]
\[ = 41.316 + 0.878 - 18.340(1.257)^2 = 13.216 \text{ in.}^3 \]

\[ I_x = I'_{xt} = 13.216(0.030) = 0.396 \text{ in.}^4 \]

\[ S_x = I_x/y_{cg} = 0.396/1.257 = 0.315 \text{ in.}^3 \]

\[ M = S_c f = 0.315(30) = 9.45 \text{ kip-in.} > M_c = 7.87 \text{ kip-in.} \]

To determine compression stress \( f \) in the top fibers of the section at \( M = 7.87 \text{ kip-in.} \), extrapolate using

\[ M = 12.70 \text{ kip-in., } f = 50 \text{ ksi} \]
\[ M = 9.45 \text{ kip-in., } f = 30 \text{ ksi} \]
\[ M = 7.87 \text{ kip-in., } f = ? \]

\[ \frac{(12.70-9.45)/(50-30)}{(9.45-7.87)/(30-f)} = \frac{0.1625(30-f)}{1.58} = 1.58 \]

\[ f = \frac{(0.1625 \times 30 - 1.58)}{0.1625} = 20.28 \text{ ksi} \]

For the second approximation, assume a compression stress of \( f = 20.28 \text{ ksi} \) in the top fibers of the section.

Element 2 from Section B4.2(b)

\[ w = 2.580 \text{ in.} \]

\[ \lambda_c = 1.417 \]
IV-164 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ S = 1.28 \sqrt{E/\ell} = 1.28 \sqrt{29500/20.28} = 48.819 \]  
\[ I_a = (0.030)^4 \left[ \frac{115}{86} \right] \text{in.}^4 \]  
\[ k = 3.57 \left( \frac{0.00090}{0.000168} \right)^{1/2} + 0.43 = 6.677 > 4 \]  
\[ k = 4 \]  
\[ \lambda = (1.052/\sqrt{4})(86) \sqrt{20.28/29500} = 1.186 < \lambda_c \]  
\[ \rho = (1.358-0.461/1.186)/1.186 = 0.817 \]  
\[ b = \rho w = 0.817(2.580) = 2.108 \text{in.} \]  
\[ A_s = A_s' = 0.0348 \text{in.}^2 \]  

Element 3 from Section B4.2(b)

\[ w = 1.720 \text{in.} \]  
\[ \lambda_c = 1.030 \]  
\[ S = 1.28 \sqrt{E/\ell} = 1.28 \sqrt{29500/20.28} = 48.819 \]  
\[ I_a = (0.030)^4 \left[ \frac{115}{86} \right] \text{in.}^4 \]  
\[ k = 3.57 \left( \frac{0.00044}{0.000113} \right)^{1/2} + 0.43 = 3.037 < 4 \]  
\[ k = 3.037 \]  
\[ \lambda = (1.052/\sqrt{3.037})(57.333) \sqrt{20.28/29500} = 0.907 < \lambda_c \]  
\[ \rho = (1.358-0.461/0.907)/0.907 = 0.937 \]  
\[ b = \rho w = 0.937(1.720) = 1.612 \text{in.} \]  
\[ d_s = (I_s/I_a)d_s' \]  
\[ d = (0.000044/0.000113)(0.26) = 0.101 \text{in.} \]

Effective section properties about x axis:
### ExamJ21es Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification IV-165

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.)</th>
<th>Ly² (in.²)</th>
<th>I'₁ (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.160</td>
<td>0.321</td>
<td>0.372</td>
<td>0.120</td>
<td>0.030</td>
</tr>
<tr>
<td>2</td>
<td>2.108</td>
<td>0.015</td>
<td>0.032</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>1.612</td>
<td>0.015</td>
<td>0.024</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>0.660</td>
<td>0.066</td>
<td>0.044</td>
<td>0.003</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>3.440</td>
<td>1.015</td>
<td>3.492</td>
<td>3.544</td>
<td>0.848</td>
</tr>
<tr>
<td>6</td>
<td>4.788</td>
<td>2.015</td>
<td>9.648</td>
<td>19.440</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>2.396</td>
<td>1.840</td>
<td>4.409</td>
<td>8.112</td>
<td>—</td>
</tr>
<tr>
<td>8</td>
<td>2.068</td>
<td>2.015</td>
<td>4.167</td>
<td>8.397</td>
<td>—</td>
</tr>
<tr>
<td>9</td>
<td>0.101</td>
<td>0.026</td>
<td>0.021</td>
<td>0.004</td>
<td>—</td>
</tr>
<tr>
<td>10</td>
<td>0.440</td>
<td>1.964</td>
<td>0.864</td>
<td>1.697</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>18.773</td>
<td>23.073</td>
<td>41.317</td>
<td>70.878</td>
<td>—</td>
</tr>
</tbody>
</table>

\[ y_{cg} = \frac{L}{L} = \frac{23.073}{18.773} = 1.229 \text{ in.} \]

\[ I'_{x} = L_{y}^{2} + L - L_{y_{cg}}^{2} \]
\[ = 41.317 + 0.878 - 18.773(1.229)^2 = 13.839 \text{ in.}^3 \]

\[ I_{x} = I'_{x}t \]
\[ = 13.839(0.030) = 0.415 \text{ in.}^4 \]

\[ S_{x} = I_{x}/y_{cg} = 0.415/1.229 = 0.338 \text{ in.}^3 \]

\[ M = S_{x}f = 0.338(20.28) = 6.85 \text{ kip-in.} < M_{e} = 7.87 \text{ kip-in.} \]

To determine compression stress \( f \) in the top fibers of the section at \( M = 7.87 \text{ kip-in.} \), interpolate using

\[ M = 9.45 \text{ kip-in., } f = 30 \text{ ksi} \]
\[ M = 6.85 \text{ kip-in., } f = 20.28 \text{ ksi} \]
\[ M = 7.87 \text{ kip-in., } f = ? \]

\[ \frac{(9.45-6.85)/(30-20.28)}{0.2675(20.28)} = \frac{(7.87-6.85)/(f-20.28)}{1.02} \]

\[ f = \frac{(1.02+0.2675x20.28)}{0.2675} = 24.09 \text{ ksi} \]

For the third approximation, assume a compression stress of \( f = 24.09 \text{ ksi} \) in the top fibers of the section.

### Element 2 from Section B4.2(b)

\[ w = 2.580 \text{ in.} \]
\[ \lambda_c = 1.417 \]
\[ S = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/24.09} = 44.792 \] (Eq. B4-1)
\[ I_a = (0.030)^4 \left[ 115(86/44.792)+5 \right] = 0.000183 \text{ in.}^4 \] (Eq. B4.2-13)
\[ k = 3.57(0.000090/0.000183)^{1/3} + 0.43 = 6.501 > 4 \] (Eq. B4.2-10)
\[ k = 4 \]
\[ \lambda = (1.052/\sqrt{4})(86) \sqrt{24.09/29500} = 1.293 < \lambda_c \] (Eq. B2.1-4)
\[ \rho = (1.358-0.461/1.293)/1.293 = 0.775 \] (Eq. B2.1-8)
\[ b = \rho w = 0.775(2.580) = 2.000 \text{ in.} \] (Eq. B2.1-2)
\[ A_s = A'_s = 0.0348 \text{ in.}^2 \] (Eq. B4.2-12)

Element 3 from Section B4.2(b)
\[ w = 1.720 \text{ in.} \]
\[ \lambda_c = 1.030 \]
\[ S = 1.28 \sqrt{E/f} = 1.28 \sqrt{29500/24.09} = 44.792 \] (Eq. B4-1)
\[ I_a = (0.030)^4 \left[ 115(57.333/44.792)+5 \right] = 0.000123 \text{ in.}^4 \] (Eq. B4.2-13)
\[ k = 3.57(0.000044/0.000123)^{1/3} + 0.43 = 2.964 < 4 \] (Eq. B4.2-10)
\[ k = 2.964 \]
\[ \lambda = (1.052/\sqrt{2.964})(57.333) \sqrt{24.09/29500} = 1.001 < \lambda_c \] (Eq. B2.1-4)
\[ \rho = (1.358-0.461/1.001)/1.001 = 0.897 \] (Eq. B2.1-8)
\[ b = \rho w = 0.897(1.720) = 1.543 \text{ in.} \] (Eq. B2.1-2)
\[ d_s = (I_y/I_a)d'_s \] (Eq. B4.2-11)
\[ d = (0.000044/0.000123)(0.26) = 0.093 \text{ in.} \]

Effective section properties about x axis:
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>Ly (in.)</th>
<th>Ly² (in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.160</td>
<td>0.321</td>
<td>0.372</td>
<td>0.120</td>
</tr>
<tr>
<td>2</td>
<td>2.000</td>
<td>0.015</td>
<td>0.030</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>1.543</td>
<td>0.015</td>
<td>0.023</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>0.660</td>
<td>0.066</td>
<td>0.044</td>
<td>0.003</td>
</tr>
<tr>
<td>5</td>
<td>3.440</td>
<td>1.015</td>
<td>3.492</td>
<td>3.544</td>
</tr>
<tr>
<td>6</td>
<td>4.788</td>
<td>2.015</td>
<td>9.648</td>
<td>19.440</td>
</tr>
<tr>
<td>7</td>
<td>2.396</td>
<td>1.840</td>
<td>4.409</td>
<td>8.112</td>
</tr>
<tr>
<td>8</td>
<td>2.068</td>
<td>2.015</td>
<td>4.167</td>
<td>8.397</td>
</tr>
<tr>
<td>9</td>
<td>0.093</td>
<td>0.202</td>
<td>0.019</td>
<td>0.004</td>
</tr>
<tr>
<td>10</td>
<td>0.440</td>
<td>1.964</td>
<td>0.864</td>
<td>1.697</td>
</tr>
<tr>
<td>Sum</td>
<td>18.588</td>
<td></td>
<td>23.068</td>
<td>41.317</td>
</tr>
</tbody>
</table>

\[ y_{eg} = \frac{L}{L} = 23.068/18.588 = 1.241 \text{ in.} \]

\[ I'_{x} = \frac{L^{2} + I'_{y} - Ly_{eg}^{2}}{I'_{y}} = 41.317 + 0.878 - 18.588(1.241)^{2} = 13.568 \text{ in.}^{3} \]

\[ I_{x} = \frac{I'_{x}y}{y_{eg}} = 13.568(0.030) = 0.407 \text{ in.}^{4} \]

\[ S_{x} = \frac{I_{x}/y_{eg}}{0.407/1.241} = 0.328 \text{ in.}^{3} \]

\[ M = S_{x}f = 0.328(24.09) = 7.90 \text{ kip-in.} = M_{x} = 7.87 \text{ kip-in.} \]

Therefore

\[ I_{x} = 0.407 \text{ in.}^{4} \]

4. Section Modulus for Load Determination - Negative Bending

Since the N.A. may be closer to the compression flange than to the tension flange, the compression stress is unknown, and therefore the effective width of the compression flange and section properties must be determined by an iterative method.

Elements 1, 2, 3, 4, 5, 9, and 10 do not vary with stress level. Assume compression stress will govern, i.e., \( f = F_{y} = 50 \text{ ksi} \) in the bottom compression fibers of the section.

Element 6 from Section B5(d)

\[ \frac{w}{t} = \frac{[3-0.140-0.5(0.932)]}{0.030} = 79.8 > 60 \]

\[ k = 4 \]

\[ \lambda = \frac{(1.052/\sqrt{4})(79.8)\sqrt{50/29500}}{1.728} = 1.728 \quad \text{(Eq. B2.1-4)} \]

\[ \rho = \frac{(1-0.22/1.728)}{1.728} = 0.505 \quad \text{(Eq. B2.1-3)} \]
\( b = \rho w = 0.505(2.394) = 1.209 \text{ in.} \) Since \( w/t > 60 \),
\( b_e = 0.030 \left[ \frac{(1.209/0.030)-0.10(79.8-60)}{0.10} \right] = 1.150 \text{ in.} \) \( \text{(Eq. B2.1-2)} \)

Element 7 from Section B5(d)
\( 60 < w/t = 79.8 < 90 \)
\( \alpha = 3 - \frac{2(1.150/2.394)}{} \)
\( -\left\{ \frac{1}{30} \right\} \left[ 1 - \frac{1.150}{2.394} \right] \right] (79.8) = 0.657 \) \( \text{(Eq. B5-3)} \)
\( L_{ef} = \alpha L_{st} = 0.657(2.396) = 1.574 \text{ in.} \) \( \text{(Eq. B5-4)} \)

Element 8 from Section B5(d)
\( w/t = (3-0.932)/0.030 = 68.933 > 60 \)
\( k = 4 \)
\( \lambda =\frac{(1.052/\sqrt{4})(68.933)}{\sqrt{50/29500}} = 1.493 \) \( \text{(Eq. B2.1-4)} \)
\( \rho = \frac{(1-0.22/1.493)}{1.493} = 0.571 \) \( \text{(Eq. B2.1-3)} \)
\( b = \rho w = 0.571(2.068) = 1.181 \text{ in.} \) \( \text{(Eq. B2.1-2)} \)
\( b_e = 0.030 \left[ (1.181/0.030)-0.10(68.933-60) \right] = 1.154 \text{ in.} \) \( \text{(Eq. B5-3)} \)

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>Effective Length (in.)</th>
<th>Distance from Top Fiber (in.)</th>
<th>( Ly ) (in.²)</th>
<th>( Ly^2 ) (in.⁴)</th>
<th>( I'_1 ) About Own Axis (in.³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.160</td>
<td>0.321</td>
<td>0.372</td>
<td>0.120</td>
<td>0.030</td>
</tr>
<tr>
<td>2</td>
<td>2.580</td>
<td>0.015</td>
<td>0.039</td>
<td>0.001</td>
<td>—</td>
</tr>
<tr>
<td>3</td>
<td>1.720</td>
<td>0.015</td>
<td>0.026</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>4</td>
<td>0.660</td>
<td>0.066</td>
<td>0.044</td>
<td>0.003</td>
<td>—</td>
</tr>
<tr>
<td>5</td>
<td>3.440</td>
<td>1.015</td>
<td>3.492</td>
<td>3.544</td>
<td>0.848</td>
</tr>
<tr>
<td>6</td>
<td>2.300</td>
<td>2.015</td>
<td>4.635</td>
<td>9.339</td>
<td>—</td>
</tr>
<tr>
<td>7</td>
<td>1.574</td>
<td>1.840</td>
<td>2.896</td>
<td>5.329</td>
<td>—</td>
</tr>
<tr>
<td>8</td>
<td>1.154</td>
<td>2.015</td>
<td>2.325</td>
<td>4.685</td>
<td>—</td>
</tr>
<tr>
<td>9</td>
<td>0.260</td>
<td>0.285</td>
<td>0.074</td>
<td>0.021</td>
<td>—</td>
</tr>
<tr>
<td>10</td>
<td>0.440</td>
<td>1.964</td>
<td>0.864</td>
<td>1.697</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>15.288</td>
<td>14.767</td>
<td>24.739</td>
<td>0.878</td>
<td>—</td>
</tr>
</tbody>
</table>

\( y_{cg} = \frac{Ly}{L} = \frac{14.767}{15.288} = 0.966 \text{ in.} \)

\( y_c = 2.030-0.966 = 1.064 \text{ in.} > 2.030/2 = 1.015 \text{ in.} \)

Therefore, compression stress controls as assumed.
$I'_{x} = L_{y}^{2} + I'_{1} - Ly_{c}^{2} = 24.739 + 0.878 - 15.288(0.966)^{2} = 11.351 \text{ in}^{3}$

$I_{x} = I'_{x}t$

$= 11.351(0.030) = 0.341 \text{ in}^{4}$

$S_{x} = I_{x}/y_{c} = 0.341/1.064 = 0.320 \text{ in}^{2}$

$M_{n} = S_{x}F_{y} = 0.320(50) = 16.00 \text{ kip-in.}$

$\phi_{b} = 0.95$

$\phi_{b}M_{n} = 0.95x16.00 = 15.20 \text{ kip-in.}$

Element 5 from Section B2.3(a)

$f_{1} = [(1.064-0.125-0.030)/1.064] (50) = 42.716 \text{ ksi}$

$f_{2} = -[(0.966-0.125-0.030)/1.064] (50) = -38.111 \text{ ksi}$

$\psi = f_{2}/f_{1} = -38.111/42.716 = -0.892 < -0.236$

$k = 4+2(1-\psi)^{3}+2(1-\psi)$

$= 4+2 [1-(-0.892)]^{3}+2 [1-(-0.892)] = 21.329$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f_{y}/E}, f = f_{1}$

$= (1.052/\sqrt{21.329})(2.030)$

$-2(0.155)/0.030 \sqrt{42.716/29500} = 0.497$

For $\lambda = 0.497 < 0.673$

$b_{e} = w = 1.720 \text{ in.}$

$= b_{y}/2$

$= 1.720/2 = 0.860 \text{ in.}$

$b_{1} = b_{y}/3-\psi$

$= 1.720/[3-(-0.892)] = 0.442 \text{ in.}$

$w_{c} = 1.064-0.030-0.125 = 0.909 \text{ in.}$

$b_{1}+b_{2} = 0.442+0.860 = 1.302 \text{ in.} > w_{c} = 0.909 \text{ in.}$

Thus element 5 is fully effective so properties above are correct.

5. Moment of Inertia for Deflection Determination - Negative Bending

$\phi_{b}M_{n} = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}$

$= [1.2(1/5)+1.6]M_{LL}$

$= 1.84M_{LL}$
\[ M_{LL} = \phi_b M_p / 1.84 = 15.20 / 1.84 = 8.26 \text{ kip-in.} \]

\[ M_s = M_{DL} + M_{LL} = (1/5+1)M_{LL} = 1.2(8.26) = 9.91 \text{ kip-in.} \]

For deflection determination, the procedure is iterative: one assume the actual compressive stress \( f \) under this service moment \( M_s \). Knowing \( f \), one proceeds as usual to obtain \( S_e \) and checks to see if \( f \times S_e \) is equal to \( M_s \) as it should. If not, reiterate until one obtains the desired level of accuracy.

(Refer to Example No. 8 for example of procedure to follow.)
EXAMPLE NO. 26

BUILT-UP SECTION - CONNECTING TWO CHANNELS

Given:
1. Steel: $F_y = 50$ ksi.
2. Sections: 2 - 6 x 2.5 x 0.060 channels with stiffened flanges.

Required:
1. Determine the maximum permissible longitudinal spacing of connectors joining two channels to form an I-section used as a compression member with unbraced length of 12 ft.
2. Design resistance welds connecting the two channels to form an I-section used as a beam with the following load, span, and support conditions:
   a. Span: 10'-0"
   b. Total uniformly distributed factored load including factored dead load: 0.520 kips per lin. ft.
   c. Length of bearing at end support: 3 in.

Solution:
1. Maximum longitudinal spacing of connectors for compression member [Section D1.1(a)]

   For compression members, the maximum permissible longitudinal spacing of connectors is

   $$s_{max} = \frac{L_{ref}}{2r_y}$$

   (Eq. D1.1-1)
IV-172  Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[(12 \times 12) \times 0.909/(2 \times 1.18) = 55.46 \text{ in.}\]

\(i_e\) is from Table 1 and \(i_1\) is from Table 5 of Part V of the Manual.

Therefore, the maximum spacing of connectors used for connecting these two channels as a compression member is 55 in.

2. Design resistance welds connecting the two channels to form an I-section used as a beam [Section D1.1(b)]

a. Spacing of welds between end supports:

The maximum permissible longitudinal spacing of welds for a flexural member is

\[s_{\text{max}} = \frac{L}{6}\]  
\[= \frac{12 \times 10}{6} = 20 \text{ in.}\]

Maximum spacing is also limited by

\[s_{\text{max}} = \frac{2gT_s}{(mq)}\]  
(Eq. D1.1-3)

in which

\[g = 5.0 \text{ in. (assumed for 6 in. deep section)}\]

\[T_s = 0.65 \times 2.28 \times 0.25 = 0.371 \text{ kips (Section E2.6)}\]

\[m = 1.148 \text{ in. (from Table 1 of Part V of the Manual)}\]

\[q = 3 \times 0.520/12 = 0.130 \text{ kips per lin. in.}\]

Therefore

\[s_{\text{max}} = \frac{2 \times 5 \times 0 \times 0.371}{(1.148 \times 0.130)} = 24.86 \text{ in.}\]

\[s_{\text{max}} = \frac{L}{6} \text{ controls. Use a spacing of 20 in. throughout the span.}\]

b. Strength of welds at end supports: Since the weld spacing is larger than the bearing length of 3.0 in., the required design strength of the welds directly at the reaction is

\[T_s = \frac{P_m}{(2g)}\]  
(Eq. D1.1-6)

\[= 0.52 \times 5 \times 1.148/(2 \times 5) = 0.298 \text{ kips}\]

which is less than 0.370 kips as provided.
EXAMPLE NO. 27
STRENGTH INCREASE FROM COLD WORK OF FORMING

Given:
1. Steel: $F_y = 50$ ksi and $F_u = 65$ ksi.
2. Section: $8 \times 3 \times 0.135$ channel with stiffened flanges.
3. Section to be used as a beam.

Required:
Determine the average tensile yield point of steel, $F_{ya'}$, for the flange considering the increase in strength resulting from the cold work of forming.

Solution:
1. Check the limitations.
   
a. In order to use Eq. A5.2.2-1 for computing the average tensile yield point for the beam flange, the channel section must be that the quantity $\rho$ is unity as determined according to Section B2 for each
IV-174 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

of the component element of the section, i.e., the channel section must be fully effective (Section-A5.2.2).

Whether the channel section is fully effective or not can be determined either by computations which follow the flow charts for Section B2 provided in Part VI of the Manual (See Examples 2 and 3), or by comparison of $S_x$ from Table 1 and $S_e$ from Table 10 (based on $F_y = 50$ ksi) of Part V the Manual, if $S_e = S_x$ then the channel section is fully effective. The second method is used in this example.

$S_e = 4.843$ in. $^3$ (from Table 10)

$S_x = 4.843$ in. $^3$ (from Table 1)

Therefore, section is fully effective. Eq. A5.2.2-1 can be used to determine $F_{ya}$.

b. Eq. A5.2.2-2 is applicable only when $F_{uv}/F_{yv} \geq 1.2, R/t \leq 7$, $\theta \leq 120^\circ$.

Since $F_{uv}/F_{yv} = 65/50 = 1.3 > 1.2$

$R/t = 0.1875/0.135 = 1.389 < 7$

$\theta = 90^\circ < 120^\circ$

Therefore, Eq. A5.2.2-2 can be used to determine $F_{yc}$.

2. Determination of $F_{yc}$:

$B_c = 3.69(F_{uv}/F_{yv})-0.819(F_{uv}/F_{yv})^2-1.79$  \hspace{1cm} (Eq. A5.2.2-3)

$= 3.69(1.3)-0.819(1.3)^2-1.79 = 1.623$

$m = 0.192(F_{uv}/F_{yv})-0.068$  \hspace{1cm} (Eq. A5.2.2-4)

$= 0.192(1.3)-0.068 = 0.182$

$F_{yc} = B_cF_{yv}/(R/t)^m$  \hspace{1cm} (Eq. A5.2.2-2)

$= 1.623(50)/(0.1875/0.135)^{1.82} = 76.44$ ksi.in

3. Determination of $F_{ya}$:

$r = R+t/2 = 0.1875+0.135/2 = 0.255$ in.

Cross-sectional area of corner = $1.57x0.255x0.135 = 0.054$ in. $^2$

Total corner cross-sectional area of the controlling flange

$= 0.054 \times 2 = 0.108$ in. $^2$

Full cross-sectional area of the controlling flange

$= 0.108 + (2.354 \times 0.135) = 0.426$ in. $^2$

$C = 0.108/0.426 = 0.254$

$F_{ya} = F_{yc}+(1-C)F_{yf}$  \hspace{1cm} (Eq. A5.2.2-1)

$= 0.254(76.44)+(1-0.254)(50) = 56.72$ ksi
EXAMPLE NO. 28
FLANGE CURLING

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $8 \times 2 \times 0.105$ channel with unstiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio $D/L = 1/5$ and $1.2D + 1.6L$ governs the design.

Required:
Determine the amount of curling for the compression flange.

Solution:
1. Determination of the design flexural strength, $\phi M_n$:

   The elastic section modulus of the effective section, $S_e$, calculated with the extreme compression or tension fiber at $F_y$ can be determined by either computations which follow the flow charts for Sec-
2. Determination of the average stress in compression flange, \( f_{av} \), at the service moment \( M_s \):

\[
\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}
\]

\[
= [1.2(M_{DL} / M_{LL}) + 1.6] M_{LL}
\]

\[
= [1.2(1/5) + 1.6] M_{LL}
\]

\[
= 1.84M_{LL}
\]

\[
M_{LL} = \phi_b M_n / 1.84 = 103.68 / 1.84 = 56.35 \text{ kip-in.}
\]

\[
M_s = M_{DL} + M_{LL}
\]

\[
= (1/5 + 1) M_{LL}
\]

\[
= 1.2(56.35) = 67.62 \text{ kip-in.}
\]

where

\( M_{DL} \) = Moment determined on the basis of nominal dead load

\( M_{LL} \) = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress \( f \) under this service moment \( M_s \). Knowing \( f \), proceeds as usual to obtain \( S_e \) and checks to see if \((f \times S_e)\) is equal to \( M_s \) as it should. If not, reiterate until one obtains the desired level of accuracy.

Properties of 90° corners:

\[
r = R + t/2 = 3/16 + 0.105/2 = 0.240 \text{ in.}
\]

Length of arc, \( u = 1.57r = 1.57 \times 0.240 = 0.377 \text{ in.}

Distance of c.g. from center of radius,

\[
c = 0.637r = 0.637 \times 0.240 = 0.153 \text{ in.}
\]

a. For the first iteration, assume a compression stress of \( f = F/2 = 25 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange: \( k = 0.43 \) (unstiffened compression element)

\[
w/t = 1.707/0.105 = 16.26 < 60 \text{ OK (Section B1.1-(a)-(3))}
\]

\[
\lambda = (1.052/ \sqrt{k})(w/t) \sqrt{f/E} \quad \text{(Eq. B2.1-4)}
\]
\[
\rho = \frac{w}{\lambda} = \frac{[1-(0.22/0.759)]}{0.759} = 0.936 \\
\text{(Eq. B2.1-2)}
\]

\[
b = \rho w = 0.936 \times 1.707 = 1.598 \text{ in.}
\]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>L Effective Length (in.)</th>
<th>y Distance from Top Fiber (in.)</th>
<th>Ly (in.²)</th>
<th>Ly² (in.⁴)</th>
<th>Ly' About Own Axis (in.²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web</td>
<td>7.414</td>
<td>4.000</td>
<td>29.656</td>
<td>118.624</td>
<td>33.961</td>
</tr>
<tr>
<td>Upper Corner</td>
<td>0.377</td>
<td>0.140</td>
<td>0.053</td>
<td>0.007</td>
<td>-</td>
</tr>
<tr>
<td>Lower Corner</td>
<td>0.377</td>
<td>7.860</td>
<td>2.963</td>
<td>23.291</td>
<td>-</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>1.598</td>
<td>0.053</td>
<td>0.085</td>
<td>0.004</td>
<td>-</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>1.707</td>
<td>7.948</td>
<td>13.567</td>
<td>107.832</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>11.473</td>
<td>46.324</td>
<td>249.758</td>
<td>33.961</td>
<td>-</td>
</tr>
</tbody>
</table>

Distance from top fiber to x-axis is

\[
y_{cg} = \frac{46.324}{11.473} = 4.038 \text{ in.}
\]

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 25 ksi will control as assumed.

To check if web is fully effective (Section B2.3):

\[
f_1 = \frac{[(4.038-0.293)/4.038] \times 25}{25} = 23.19 \text{ ksi (compression)}
\]

\[
f_2 = -\frac{[(3.962-0.293)/4.038] \times 25}{25} = -22.72 \text{ ksi (tension)}
\]

\[
\psi = f_2/f_1 = -22.72/23.19 = -0.980
\]

\[
k = 4+2(1-\psi)^3+2(1-\psi) \quad \text{(Eq. B2.3-4)}
\]

\[
k = 4+2[1-(-0.980)]^3+2[1-(-0.980)]
\]

\[
k = 23.48
\]

\[
h = w = 7.414 \text{ in.}, h/t = w/t = 7.414/0.105 = 70.61
\]

\[
h/t = 70.61 < 200 \text{ OK (Section B1.2-(e))}
\]

\[
\lambda = \frac{(1.052/\sqrt{23.48})(70.61)}{\sqrt{23.19/29500}} = 0.430 < 0.673
\]

\[
b_e = w \quad \text{(Eq. B2.1-1)}
\]
\[ b_2 = \frac{b_0}{2} = \frac{7.414}{2} = 3.707 \text{ in.} \quad \text{(Eq. B2.3-2)} \]

\[ b_1 = \frac{b_0}{(3-\psi)} = \frac{7.414}{3-(-0.980)} = 1.863 \text{ in.} \quad \text{(Eq. B2.3-1)} \]

\[ b_1 + b_2 = 1.863 + 3.707 = 5.570 \text{ in.} \]

Compression portion of the web calculated on the basis of the effective section:

\[ y_{cg} - 0.293 = 4.038 - 0.293 = 3.745 \text{ in.} \]

Since \( b_1 + b_2 = 5.570 \text{ in.} < 3.745 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 3.745 in. This verifies the assumption that the web is fully effective.

\[ I'_x = I_y^2 + I'_1 - L_y^2 \frac{y_{cg}}{1 - \frac{y_{cg}}{0.053}} \]

Actual \( I_x = I'_x \cdot t = 96.647 \times 0.105 = 10.148 \text{ in.}^4 \)

\[ S_c = \frac{I_x}{y_{cg}} = \frac{10.148}{4.038} = 2.513 \text{ in.}^3 \]

\[ M = f \times S_c = 25 \times 2.513 = 62.83 \text{ kip-in.} < M_s = 67.62 \text{ kip-in.} \]

Need to do another iteration and also to increase \( f \).

b. For the second iteration, assume \( f = 27.18 \text{ ksi} \) in the top fibers of the section and that the web is fully effective.

Compression flange:

\[ \lambda = \frac{(1.052/\sqrt{0.43})(16.26)\sqrt{27.18/29500}}{0.792} = 0.792 > 0.673 \]

\[ \rho = \frac{1-(0.22/0.792)}{0.792} = 0.912 \]

\[ b = 0.912 \times 1.707 = 1.557 \text{ in.} \]

Effective section properties about \( x \)-axis:

\[ L = 11.473 - 1.598 + 1.569 = 11.444 \text{ in.} \]

\[ L_y = 46.324 - 0.085 + 1.569 \times 0.053 = 46.322 \text{ in.}^2 \]

\[ L_y^2 = 249.755 - 0.004 + 1.569(0.053)^2 = 249.758 \text{ in.}^3 \]

\[ I'_1 = 33.961 \text{ in.}^3 \]
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

To check if web is fully effective:

\[ f_1 = \left( 4.047 - 0.293 \right) / 4.047 \times 27.18 = 25.21 \text{ ksi} \]

\[ f_2 = -\left( 3.953 - 0.293 \right) / 4.047 \times 27.18 = -24.51 \text{ ksi} \]

\[ \psi = -24.51 / 25.21 = -0.972 \]

\[ k = 4 + 2 \left[ 1 - (-0.972) \right] - 2 \left[ 1 - (-0.972) \right] = 23.28 \]

\[ \lambda = (1.052 / \sqrt{23.28})(70.61) / \sqrt{25.21 / 29500} = 0.450 < 0.673 \]

\[ b_e = 7.414 \text{ in.} \]

\[ b_2 = 7.414 / 2 = 3.707 \text{ in.} \]

\[ b_1 = 7.414 / [3 - (-0.972)] = 1.866 \text{ in.} \]

Compression portion of the web calculated on the basis of the effective section = 4.047 - 0.293 = 3.754 in.

Since \( b_1 + b_2 = 5.573 \text{ in.} > 3.759 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 3.759 in. This verifies the assumption that the web is fully effective.

\[ I'_x = 249.755 + 33.961 - 11.432(4.047)^2 = 96.48 \text{ in.}^3 \]

Actual \( I_x = 96.48 \times 0.105 \)

\[ = 10.130 \text{ in.}^4 \]

\[ S_e = 10.130 / 4.047 = 2.503 \text{ in.}^3 \]

\[ M = f \times S_e = 27.18 \times 2.503 \]

\[ = 68.034 \text{ kip-in.} = M_s \text{OK} \]

\[ f_{aw} = f(b/w) = 27.18 \times (1.569 / 1.707) = 24.98 \text{ ksi} \]

3. Determination of the curling of the compression flange, \( c_f \).

\[ w_f = 2.000 - 0.105 = 1.895 \text{ in.} \]

\[ w_f = \sqrt{0.061 \times d / f_{aw}} \times \sqrt{100 \times c_f / d} \]

\[ 1.895 = \sqrt{0.061 \times (0.105) \times (8)(29500)} / 24.98 \times \sqrt{100 \times c_f / 8} \]

\[ = 7.779 \times \sqrt{12.5 \times c_f} \]

\[ 4 \sqrt{12.5 \times c_f} = 1.895 / 7.779 \]

\[ 12.5 \times c_f = (1.895 / 7.779)^4 \]

\[ c_f = (1.895 / 7.779)^4 / 12.5 = 0.00078 \text{ in.} \]
EXAMPLE NO. 29
SHEAR LAG

Given:
1. Steel: $F_y = 50$ ksi.
2. Section: $8 \times 8 \times 0.135$ square tube.
4. Loading: Concentrated load at midspan.

Required:
Determine the design flexural strength, $\phi_b M_n$.

Solution:
1. Determination of the nominal moment, $M_n$, based on initiation of yielding (Section C3.1.1).

Properties of 90° corners:

$r = R + \frac{t}{2} = \frac{3}{16} + \frac{0.135}{2} = 0.255$ in.
Length of arc, \( u = 1.57r = 1.57 \times 0.255 = 0.400 \) in.

Distance of c.g. from center of radius,
\( c = 0.637r = 0.637 \times 0.255 = 0.162 \) in.

Computation of \( I_z \):
For the first approximation, assume a compression stress of \( f = F_y = 50 \) ksi in the compression flange, and that the webs are fully effective.

Compression flange: \( k = 4.00 \) (stiffened compression element supported by a web on each longitudinal edge)

\[
\frac{w}{t} = \frac{7.354}{0.135} = 54.47 < 500 \text{ OK (Section B1.1-(a)-(2))}
\]

\[
\lambda = \frac{(1.052/\sqrt{4.00})(54.47)}{\sqrt{50/29500}} = 1.180 > 0.673
\]

\[
\rho = \frac{(1-0.22/\lambda)}{\lambda} = \frac{(1-0.22/1.180)}{1.180} = 0.689
\]

\[
b = \rho w = 0.689 \times 7.354
\]

\[
= 5.067 \text{ in.}
\]

Effective section properties about \( x \) axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( \frac{L}{t} ) Effective Length (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L_y ) (in.²)</th>
<th>( L_y^2 ) (in.⁴)</th>
<th>( I'_1 ) About Own Axis (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>14.708</td>
<td>4.000</td>
<td>58.832</td>
<td>235.328</td>
<td>66.286</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>0.800</td>
<td>0.161</td>
<td>0.129</td>
<td>0.021</td>
<td>-</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>0.800</td>
<td>7.839</td>
<td>6.271</td>
<td>49.160</td>
<td>-</td>
</tr>
<tr>
<td>Compression Flange</td>
<td>5.067</td>
<td>0.068</td>
<td>0.345</td>
<td>0.023</td>
<td>-</td>
</tr>
<tr>
<td>Tension Flange</td>
<td>7.354</td>
<td>7.933</td>
<td>58.339</td>
<td>462.806</td>
<td>-</td>
</tr>
<tr>
<td>Sum</td>
<td>28.729</td>
<td>123.916</td>
<td>747.338</td>
<td>-</td>
<td>66.286</td>
</tr>
</tbody>
</table>

Distance from top fiber to \( x \)-axis is

\[
y_{eg} = \frac{123.916}{28.729} = 4.313 \text{ in.}
\]

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of \( 50 \) ksi will govern as assumed (i.e., initial yielding is in compression).

To check if webs are fully effective (Section B2.3):
$f_1 = \frac{(4.313-0.323)}{4.313} \times 50 = 46.26 \text{ ksi (compression)}$

$f_2 = -\frac{(3.687-0.323)}{4.313} \times 50 = -39.00 \text{ ksi (tension)}$

$\psi = \frac{f_2}{f_1} = \frac{-39.00}{46.26} = -0.843$

$k = 4 + 2(1-\psi)^3 + 2(1-\psi) (\text{Eq. B2.3-4})$

$= 4 + 2 \left[ 1 - (-0.843) \right]^3 + 2 \left[ 1 - (-0.843) \right] = 20.206$

$h = w = 7.354 \text{ in.}, h/t = w/t = 7.354/0.135 = 54.47$

$h/t = 54.47 < 200 \text{ OK (Section B1.2-(a))}$

$\lambda = (1.052/\sqrt{20.206}) \times (54.47) \sqrt{46.26/29500} = 0.505 < 0.673$

$b_e = w (\text{Eq. B2.1-1})$

$= 7.354 \text{ in.}$

$b_2 = b_e/2 (\text{Eq. B2.3-2})$

$= 7.354/2 = 3.677 \text{ in.}$

$b_1 = b_e/(3-\psi) (\text{Eq. B2.3-1})$

$= 7.354/ [3-(-0.843)] = 1.914 \text{ in.}$

Compression portion of the web calculated on the basis of the effective section $= y_{cg} - 0.323 = 4.313 - 0.323 = 3.990 \text{ in.}$

Since $b_1 + b_2 = 5.591 \text{ in.} > 3.990 \text{ in.}, b_1 + b_2 \text{ shall be taken as } 3.990 \text{ in.}. \text{ This verifies the assumption that the webs are fully effective.}$

$\Gamma'_{x} = \Gamma_{x} + \Gamma'_{x} - \Gamma_{x}y_{cg}^2$

$= 747.338 + 66.286 - 28.729(4.313)^2$

$= 279.208 \text{ in.}^3$

Actual $I_x = \Gamma'_{x} t$

$= 279.208 \times 0.135$

$= 37.693 \text{ in.}^4$

$S_e = I_x/y_{cg}$

$= 37.693/4.313$

$= 8.739 \text{ in.}^3$

$M_n = S_e F_y = 8.739 \times 50$

$= 436.95 \text{ kip-in.}$

2. Determination of the nominal moment, $M_n$, based on shear lag consideration (Section B1.1(c)).

$w_f = (8-2\times0.135)/2 = 3.865 \text{ in.}$

$L/w_f = 3 \times 12/3.865 = 9.314 < 30$
Because the L/w_f ratio is less than 30, and the member carries a concentrated load, consideration for shear lag is needed.

From Table B1.1(c):
L/w_f = 10, effective design width/actual width = 0.73
L/w_f = 8, effective design width/actual width = 0.67
L/w_f = 9.314, effective design width/actual width = ?

\[
\frac{(10-9.314)}{(9.314-8)} = \frac{(0.73-x)}{(x-0.67)}
\]
\[
0.686(x-0.67) = 1.314(0.73-x)
\]
\[
x = 0.709
\]

Therefore, the effective design widths of compression and tension flanges between webs are

0.709(8-2\times0.135) = 5.481 in.

b = 5.481 - 2R = 5.481 - 2(3/16) = 5.106 in.

Because of symmetry and assume webs are fully effective,

y_{cg} = 4.000 in.

Effective section properties about x-axis:

\[
\]
\[
Ly^2 = 747.338 - 0.023 - 462.806 + 5.106(0.068)^2 + 5.106(7.933)^2
\]
\[
= 605.866 in.^3
\]
\[
I_x = 66.286 in.^3
\]

To check if webs are fully effective:

\[
f_1 = [(4.000 - 0.323)/4.000] \times 50 = 45.96 ksi
\]
\[
f_2 = -45.96 ksi
\]
\[
\psi = -45.96/45.96 = -1.000
\]
\[
k = 4 + 2 \left[ 1 - (-1.000) \right]^3 + 2 \left[ 1 - (-1.000) \right] = 24.000
\]
\[
\lambda = (1.052/\sqrt{24.000 \times (54.47) \times \sqrt{45.96/29500}} = 0.462 < 0.673
\]
\[
b_1 = 7.354 in.
\]
\[
b_2 = 7.354/2 = 3.677 in.
\]
\[
b_3 = 7.354/\left[ 3 - (-1.000) \right] = 1.839 in.
\]

Compression portion of the web calculated on the basis of the effective section = 4.000 - 0.323 = 3.677 in.

Since b_1 + b_2 = 5.516 in. > 3.677 in., b_1 + b_2 shall be taken as 3.677 in. This verifies the assumption that the webs are fully effective.

\[
I_x' = 605.866 + 66.286 - 26.520(4.000)^2
\]
Actual $I_x = 247.832 \times 0.135 = 33.457 \text{ in.}^4$

$S_e = 33.457/4.000 = 8.364 \text{ in.}^3$

$M_n = 8.364 \times 50 = 418.20 \text{ kip-in.} < 436.95 \text{ kip-in. (initial yielding)}$

3. Determination of the design flexural strength, $\phi_b M_n$.

$M_n = 418.20 \text{ kip-in.}$

$\phi_b = 0.95$

$\phi_b M_n = 0.95 \times 418.20 = 397.29 \text{ kip-in.}$
EXAMPLE NO. 30
FLAT SECTION WITH GROOVE WELDED CONNECTION IN BUTT JOINT

Given:
1. Steel: $F_y = 50$ ksi.
2. Electrode: $F_{xx} = 60$ ksi.
3. Detail of connection shown in sketch.

Required:
1. Determine the design tensile strength, $\phi P_n$, normal to the effective area.
2. Determine the design shear strength, $\phi P_n$, on the effective area.

Solution:
1. Determination of the design tensile strength, $\phi P_n$, normal to the effective area (Section E2.1(a)).

\[
P_n = L t_c F_y
\]
\[
= (8.000)(0.135)(50)
\]
\[
= 54.00 \text{ kips}
\]

$\phi = 0.90$

$\phi P_n = 0.90 \times 54.00$
= 48.60 kips

2. Determination of the design shear strength, \( \Phi P_n \), on the effective area (Section E2.1(b))

\[
(P_n)_1 = L(0.6F_{xx}) \\
= (8.000)(0.135)(0.6x60) \\
= 38.88 \text{ kips}
\]

\[ \phi = 0.80 \]

\[ \phi(P_n)_1 = 0.80 \times 38.88 \]

\[ = 31.10 \text{ kips} \]

\[
(P_n)_2 = L'(F_{yy}/\sqrt{3}) \\
= (8.000)(0.135)(50/\sqrt{3}) \\
= 31.18 \text{ kips}
\]

\[ \phi = 0.90 \]

\[ \phi(P_n)_2 = 0.90 \times 31.18 \]

\[ = 28.06 \text{ kips.} \]

Since \( \phi(P_n)_2 < \phi(P_n)_1 \), therefore \( \Phi P_n = 28.06 \text{ kips.} \)
EXAMPLE NO. 31
I-SECTION

Given:
1. Steel: \( F_y = 50 \text{ ksi} \).
2. Section: \( 8 \times 4 \times 0.135 \) I-section with unstiffened flanges.
3. Span: \( L = 12 \text{ ft.} \), with simple supports, and carries uniform load.
4. Beam is laterally braced at both ends and midspan.

Required:
Determine the design flexural strength, \( \phi \beta M_n \).

Solution:
1. Nominal section strength (Section C3.1.1).
   a. Procedure I - based on initiation of yielding.
The elastic section modulus of the effective section, $S_e$, based on initiation of yielding can be determined by either computations which follow the flow charts provided in Part VI of the Manual (see Example 1), or by taking the $S_e$ value provided in Table 15 of Part V of the Manual. The latter method is used in this example.

$$S_e = 6.246 \text{ in.}^3$$

$$M_n = S_e F_y$$

$$= 6.246 \times 50 = 312.30 \text{ kip-in.}$$  \hspace{1cm} (Eq. C3.1.1-1)

b. Procedure II - based on inelastic reserve capacity

Since the member is subjected to lateral buckling, therefore this provision does not apply in this example. Then,

$$M_n I_{yc} = 312.30 \text{ kip-in.}$$

$$\phi_b = 0.90$$

$$\phi_b(M_n)_{1} = 0.90 \times 312.30 = 281.07 \text{ kip-in.}$$

2. Lateral buckling strength (Section C3.1.2).

From Table 6 of Part V of the Manual, $S_f = 6.538 \text{ in.}^3$, $I_y = 1.4499 \text{ in.}^4$

$$M_y = S_f F_y$$

$$= 6.538 \times 50 = 326.90 \text{ kip-in.}$$  \hspace{1cm} (Eq. C3.1.2-4)

$$C_b = 1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2$$

$$= 1.75 + 1.05(0/M_{max}) + 0.3(0/M_{max})^2 = 1.75 < 2.3$$

$$I_{yc} = I_y/2 = 1.4499/2 = 0.725 \text{ in.}^4$$

$$M_e = \pi^2 E S_b d I_{yc}/L^2$$

$$= \pi^2 (29500)(1.75)(8)(0.725)/(6 \times 12)^2$$

$$= 570.06 \text{ kip-in.}$$

$$0.56M_y = 0.56 \times 326.90 = 183.06 \text{ kip-in.}$$

$$2.78M_y = 2.78 \times 326.90 = 908.78 \text{ kip-in.}$$

Since $2.78M_y > M_e > 0.56M_y$, therefore.

$$M_c = (10/9)M_y (1 - 10M_y/3M_e)$$

$$= (10/9)(326.90) [1 - 10(326.90)/(36 \times 570.06)]$$

$$= 305.36 \text{ kip-in.}$$

$$M_n = S_e (M/y)$$

$$= S_e f$$  \hspace{1cm} (Eq. C3.1.2-1)

where
Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ f = \frac{M}{S_f} = \frac{305.36}{6.538} = 46.71 \text{ ksi} \]

Properties of 90° corners:

\[ r = \frac{R + t/2}{3/16 + 0.135/2} = 0.255 \text{ in.} \]

Length of arc, \( u = 1.57r = 1.57 \times 0.255 = 0.400 \text{ in.} \)

Distance of c.g. from center of radius,

\[ c = 0.637r = 0.637 \times 0.255 = 0.162 \text{ in.} \]

Determination of elastic section modulus of the effective section calculated at a stress of \( f = 46.71 \text{ ksi} \) in the extreme compression fiber (assume the webs are fully effective):

Compression flange: \( k = 0.43 \) (unstiffened compression element)

\[ \frac{w}{t} = \frac{1.677}{0.135} = 12.42 < 60 \text{ OK (Section B1.1-(a)-(3))} \]

\[ \lambda = \frac{(1.052/\sqrt{k})(w/t)}{\sqrt{E}} \]

\[ = \frac{(1.052/0.43)(12.42)\sqrt{46.71/29500}}{0.793} > 0.673 \]

\[ \rho = \frac{(1-0.22/\lambda)}{\rho} \]

\[ = \frac{(1-0.22/0.793)}{0.793} = 0.911 \]

\[ b = \rho w \]

\[ = 0.911 \times 1.677 = 1.528 \text{ in.} \]

Effective section properties about x axis:

<table>
<thead>
<tr>
<th>Element</th>
<th>( L ) Effective Length (in.)</th>
<th>( y ) Distance from Top Fiber (in.)</th>
<th>( L_y ) (in.²)</th>
<th>( L_y^2 ) (in.⁴)</th>
<th>( I'_{1} ) About Own Axis (in.⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Webs</td>
<td>14.708</td>
<td>4.000</td>
<td>58.832</td>
<td>235.328</td>
<td>66.286</td>
</tr>
<tr>
<td>Upper Corners</td>
<td>0.800</td>
<td>0.161</td>
<td>0.129</td>
<td>0.021</td>
<td>—</td>
</tr>
<tr>
<td>Lower Corners</td>
<td>0.800</td>
<td>7.839</td>
<td>6.271</td>
<td>49.158</td>
<td>—</td>
</tr>
<tr>
<td>Compression Flanges</td>
<td>3.056</td>
<td>0.068</td>
<td>0.208</td>
<td>0.014</td>
<td>—</td>
</tr>
<tr>
<td>Tension Flanges</td>
<td>3.354</td>
<td>7.933</td>
<td>26.607</td>
<td>211.076</td>
<td>—</td>
</tr>
<tr>
<td>Sum</td>
<td>22.718</td>
<td>92.047</td>
<td>495.597</td>
<td>66.286</td>
<td>—</td>
</tr>
</tbody>
</table>

Distance from top fiber to x-axis is

\[ y_{eg} = \frac{92.047}{22.718} = 4.052 \text{ in.} \]

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of \( f = 46.71 \text{ ksi} \) will govern.

To check if webs are fully effective (Section B2.3):
\[ f_1 = \left(\frac{(4.052-0.323)/4.052}{46.71}\right) \times 46.71 = 42.99 \text{ ksi (compression)} \]

\[ f_2 = -\left(\frac{(3.948-0.323)/4.052}{46.71}\right) \times 46.71 = -41.79 \text{ ksi (tension)} \]

\[ \psi = f_2/f_1 = -41.79/42.99 = -0.972 \]

\[ k = 4+2(1-\psi)^3 + 2(1-\psi) \quad \text{(Eq. B2.3-4)} \]

\[ = 4+2 \left[ 1-(-0.972) \right]^3 + 2 \left[ 1-(-0.972) \right] \]

\[ = 23.281 \]

\[ h = w = 7.354 \text{ in.}, \quad h/t = w/t = 7.354/0.135 = 54.47 \]

\[ h/t = 54.47 < 200 \text{ OK (Section B1.2-(a))} \]

\[ \lambda = \left(\frac{1.052}{\sqrt{23.281}}\right)\left(\frac{54.47}{46.71}\right) = 0.453 < 0.673 \]

\[ b_e = w = 7.354 \text{ in.} \quad \text{(Eq. B2.1-1)} \]

\[ = 7.354 \text{ in.} \]

\[ b_2 = b_e/2 \quad \text{(Eq. B2.3-2)} \]

\[ = 7.354/2 = 3.677 \text{ in.} \]

\[ b_1 = b_e/(3-\psi) \quad \text{(Eq. B2.3-1)} \]

\[ = 7.354/\left[ 3-(-0.972) \right] = 1.851 \text{ in.} \]

Compression portion of the web calculated on the basis of the effective section = \( y_{cg} - 0.323 = 4.052 - 0.323 = 3.729 \text{ in.} \)

Since \( b_1 + b_2 = 5.528 \text{ in.} > 3.729 \text{ in.} \), \( b_1 + b_2 \) shall be taken as 3.729 in. This verifies the assumption that the webs are fully effective.

\[ I'_x = Ly^2 + I'_{1y} - Ly_{cg}^2 \]

\[ = 495.599 + 66.286 - 22.718(4.052)^2 \]

\[ = 188.885 \text{ in.}^3 \]

Actual \( I_x = I'_x t \)

\[ = 188.885 \times 0.135 \]

\[ = 25.499 \text{ in.}^4 \]

\[ S_e = I_x/y_{cg} \]

\[ = 25.499/4.052 \]

\[ = 6.293 \text{ in.}^3 \]

\[ (M_n)_2 = S_e f = 6.293 \times 46.71 \]

\[ = 293.95 \text{ kip-in.} \]

\[ \phi_b = 0.90 \]

\[ \phi_b(M_n)_2 = 0.90 \times 293.95 \]

\[ = 264.56 \text{ kip-in.} < \phi_b(M_n)_1 = 2.8107 \text{ kip-in.} \]
Therefore, $\phi M_n = 264.56$ kip-in. (i.e., lateral buckling strength controls).
EXAMPLE NO. 32
CHANNEL SECTION BRACED
Complete Flexural Design,
Unstiffened Compression Flange

Given:
1. Steel: \( F_y = 50 \) ksi.
2. Section: 7 x 1.5 x 0.135 channel with unstiffened flanges.
3. Span: Section is continuous over three 10 ft. spans with 6 in support bearing lengths.
4. Loading: Live load = 360 lb/ft. Dead load = 40 lb/ft..
5. Deflection due to service live load is to be limited to 1/240 of the span.

Required:
Check adequacy of the section.
Solution:

1. Nominal section strength, $M_n$ (Section C3.1.1).

   a. Procedure I - based on initiation of yielding. The elastic section modulus of the effective section, $S_e$, calculated with the extreme compression or tension fiber at $F_y$ can be determined by either computations which follow the flow charts for Section B2 in Part VI of the Manual (see Example 1), or by taking the $S_e$ value provided in Table 11 of Part V of the Manual. The latter method is used in this example.

   \[ S_e = 2.240 \text{ in.}^3 \]

   \[ M_n = S_e F_y \quad \text{(Eq. C3.1.1-1)} \]

   \[ = 2.240 \times 50 = 112.00 \text{ kip-in.} \]

   b. Procedure II - based on inelastic reserve capacity

   For unstiffened compression element, $C_y = 1$. Maximum compressive strain = $C_y e_y = e_y$. Therefore, the nominal ultimate moment, $M_u$, is the same as the $M_n$ determined by procedure I because the compression flange will yield first.

2. Lateral buckling strength, $M_u$ (Section C3.1.2). Since the compression flange is braced against lateral buckling, this provision does not apply.

3. Design flexural strength, $\phi_b M_n$ (based on nominal section strength).

   \[ \phi_b = 0.90 \]

   \[ \phi_b M_n = 0.90 \times 112.00 = 100.80 \text{ kip-in.} \]

   This value can be used for both positive and negative bending.

   \[ w_u = 1.2 w_d + 1.6 w_{LL} = 1.2(0.04) + 1.6(0.36) = 0.624 \text{ kip/ft.} \]

   For a continuous beam over three equal spans, the maximum bending moment is negative and occurs over the interior supports. It is given by

   \[ M_u = 0.100 w_u L^2 = 0.100(0.624)(10)^2(12) \]

   \[ = 74.88 \text{ kip-in.} < \phi_b M_n = 100.80 \text{ kip-in. OK} \]

4. Strength for shear only (Section C3.2). The required shear strength at any section shall not exceed the design shear strength, $\phi_v V_n$:

   \[ K_v = 5.34 \text{ (unreinforced web)} \]

   \[ \sqrt{E K_v} / F_y = \sqrt{29500(5.34)/50} = 56.13 \]

   \[ h/t = 6.354/0.135 = 47.07 \]

   For $h/t < \sqrt{E K_v} / F_y$

   \[ \phi_v = 1.00 \]

   \[ V_n = 0.577 F_y h t \quad \text{(Eq. C3.2-1)} \]

   \[ = 0.577(50)(6.354)(0.135) = 24.75 \text{ kips} \]
IV-194 Examples Based on the March 16, 1991 Edition of the LRFD Cold-Formed Specification

\[ \phi_v V_n = 1.00(24.75) = 24.75 \text{ kips} \]

The maximum required shear strength is given by

\[ V_u = 0.600w_u L \]
\[ = (0.600)(0.624)(10) = 3.74 \text{ kips} < \phi_v V_n = 24.75 \text{ kips OK} \]

5. Strength for combined bending and shear (Section C3.3). At the interior supports there is a combination of web bending and web shear:

\[ \phi_b M_{n xo} = 100.80 \text{ kip-in.} \quad M_u = 74.88 \text{ kip-in.} \]
\[ \phi_v V_n = 24.75 \text{ kips} \quad V_u = 3.74 \text{ kips} \]

For unreinforced webs,

\[ \frac{(M_u/\phi_b M_{n xo})^2 + (V_u/\phi_v V_n)^2}{(74.88/100.80)^2 + (3.74/24.75)^2} = 0.575 < 1.0 \text{ OK} \]  

(Eq. C3.3-1)

6. Web crippling strength (Section C3.4)

\[ R/t = (3/16)/0.135 = 1.389 < 6 \text{ OK} \]
\[ h/t = 6.354/0.135 = 47.07 < 200 \]

Table C3.4-1 applies:

For end reactions: Eq. C3.4-2

For interior reactions: Eq. C3.4-4

\[ k = F_y/33 = 50/33 = 1.515 \text{ (Eq. C3.4-21)} \]
\[ C_1 = (1.22-0.22k) \text{ (Eq. C3.4-10)} \]
\[ = 1.22-0.22(1.515) = 0.887 \]
\[ C_2 = (1.06-0.06R/t) \leq 1.0 \text{ (Eq. C3.4-11)} \]
\[ = 1.06-0.06(1.389) = 0.977 < 1.0 \text{ OK} \]
\[ C_3 = (1.33-0.33k) \text{ (Eq. C3.4-12)} \]
\[ = 1.33-0.33(1.515) = 0.830 \]
\[ C_4 = (1.15-0.15R/t) \leq 1.0 \text{ but not less than 0.50 (Eq. C3.4-13)} \]
\[ 1.15-0.15R/t = 1.15-0.15(1.389) = 0.942 \leq 1.0 \text{ OK} \]
\[ > 0.50 \text{ OK} \]
\[ C_4 = 0.942 \text{ (Eq. C3.4-20)} \]
\[ C_8 = 0.7+0.3(6/90)^2 \]
\[ = 0.7+0.3(90/90)^2 = 1.0 \]
For end reaction:

\[ P_n = t^2kC_3C_4C_5 \left[ 217-0.28(h/t) \right] \left[ 1+0.01(N/t) \right] \]

\[ = (0.135)^2(1.515)(0.830)(0.942)(1.0) \left[ 217-0.28(47.07) \right] \times \left[ 1+0.01(6/0.135) \right] \]

\[ = 6.36 \text{kips} \]

\[ \phi_w = 0.75 \]

\[ \phi_wP_n = 0.75(6.36) = 4.77 \text{kips} \]

End reaction is given by

\[ R = 0.400w_uL \]

\[ = (0.400)(0.624)(10) = 2.50 \text{kips} < \phi_wP_n = 4.77 \text{kips} \text{OK} \]

For interior reaction:

\[ P_n = t^2kC_1C_2C_6 \left[ 538-0.74(h/t) \right] \left[ 1+0.007(N/t) \right] \]

\[ = (0.135)^2(1.515)(0.887)(0.977)(1.0) \left[ 538-0.74(47.07) \right] \times \left[ 1+0.007(6/0.135) \right] \]

\[ = 15.77 \text{kips} \]

\[ \phi_w = 0.75 \]

\[ \phi_wP_n = 0.75(15.77) = 11.83 \text{kips} \]

Interior reaction is given by

\[ R = 1.10w_uL \]

\[ = (1.10)(0.624)(10) = 6.86 \text{kips} < \phi_wP_n = 11.84 \text{kips} \text{OK} \]

7. Combined bending and web crippling strength (Section C3.5). At the interior supports there is a combination of web bending and web crippling:

\[ \phi_bM_{nko} = 100.80 \text{kip-in.} \quad M_u = 74.88 \text{kip-in.} \]

\[ \phi_wP_n = 11.84 \text{kips} \quad R = 6.86 \text{kips} \]

For shapes having single unreinforced webs:

\[ 1.07(P_u/\phi_wP_n)+(M_u/\phi_bM_{nko}) \leq 1.42 \quad (\text{Eq. C3.5-1}) \]

\[ 1.07(6.86/11.83)+(74.88/100.80) = 1.36 < 1.42 \text{OK} \]

8. Deflection due to service live load.

\[ S_x = 2.240 \text{in.}^3 \text{ (Table 2)} \]

\[ S_e = 2.240 \text{in.}^3 \text{ (Table 11)} \]

Since \( S_e = S_x \), therefore section is fully effective at \( F_y = 50 \text{ ksi} \). For any stress \( f \) which is less than \( F_y = 50 \text{ ksi} \), the section will be fully effective, i.e.,

\[ I_x = 7.840 \text{in.}^4 \text{ (Table 2)} \]
This value can be used for deflection determination.

The maximum deflection occurs at a distance of 0.446L from the exterior supports. It is given by

\[ \Delta = \frac{0.0069wL^4}{EI} \]

\[ = 0.0069(0.36)(10)^4(12)^3/(29500\times7.84) \]

\[ = 0.186 \text{ in.} \]

This deflection is limited to 1/240 of the span, i.e.,

\[ \frac{L}{240} = \frac{10\times12}{240} = 0.5 \text{ in.} > 0.186 \text{ in. OK} \]

From the above calculations, it can be concluded that the section is adequate.
CHARTS AND TABLES

FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE
LOAD AND
RESISTANCE FACTOR
DESIGN SPECIFICATION
FOR COLD-FORMED
STEEL STRUCTURAL
MEMBERS

LRFD Cold-Formed Steel Design Manual-Part V

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE

This document, *Part V of the LRFD Cold-Formed Steel Design Manual*, contains two groups of design aids: (A) charts and tables prepared to assist in the use of particular design provisions of the *Specification*, and (B) tables of section properties. Included in Group A is an extensive series of graphs related to the compression member design procedures contained in *Part III, Supplementary Information*.

These *Charts and Tables* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary (Part II), Supplementary Information (Part III)*, and *Illustrative Examples (Part IV)*, in addition to the *Specification (Part I)*.

American Iron and Steel Institute
December 1991
# TABLE OF CONTENTS

**PART V**

**CHARTS AND TABLES FOR USE WITH THE**

**MARCH 16, 1991, EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION**

**FOR COLD-FORMED STEEL STRUCTURAL MEMBERS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PREFACE</strong></td>
<td>3</td>
</tr>
<tr>
<td><strong>GROUP A—CHARTS RELATED TO PARTICULAR SPECIFICATION PROVISIONS</strong></td>
<td>6</td>
</tr>
<tr>
<td><strong>General Notes</strong></td>
<td>6</td>
</tr>
<tr>
<td><strong>TORSIONAL—FLEXURAL DESIGN CHARTS</strong></td>
<td></td>
</tr>
<tr>
<td>Equal Angles (Singly-Symmetric) With and Without Lips</td>
<td></td>
</tr>
<tr>
<td>Chart V-1.1 Buckling Mode</td>
<td>7</td>
</tr>
<tr>
<td>Chart V-1.2 $C_s/a^2$</td>
<td>8</td>
</tr>
<tr>
<td>Chart V-1.3 $\sigma_{tu} (a/t)^{1.8}$</td>
<td>8</td>
</tr>
<tr>
<td>Chart V-1.4 $C_T/a^2$</td>
<td>8</td>
</tr>
<tr>
<td>Chart V-1.5 F-Factor</td>
<td>9</td>
</tr>
<tr>
<td>Chart V-1.6 W-Factor</td>
<td>10</td>
</tr>
<tr>
<td>Chart V-1.7 $G_f$ Factor</td>
<td></td>
</tr>
<tr>
<td>Channels (Singly-Symmetric) With &amp; Without Lips</td>
<td></td>
</tr>
<tr>
<td>Chart V-2.1 Buckling Mode</td>
<td>11</td>
</tr>
<tr>
<td>Chart V-2.2 $C_s/a^2$</td>
<td>12</td>
</tr>
<tr>
<td>Chart V-2.3 $\sigma_{tu} (a/t)^{1.8}$</td>
<td>13</td>
</tr>
<tr>
<td>Chart V-2.4 $C_T/a^2$</td>
<td>14</td>
</tr>
<tr>
<td>Chart V-2.5 F-Factor</td>
<td>15</td>
</tr>
<tr>
<td>Chart V-2.6 W-Factor</td>
<td>16</td>
</tr>
<tr>
<td>Chart V-2.7 $G_f$ Factor</td>
<td>17</td>
</tr>
<tr>
<td>Hat Sections (Singly-Symmetric)</td>
<td></td>
</tr>
<tr>
<td>Chart V-3.1 Buckling Mode</td>
<td>18</td>
</tr>
<tr>
<td>Chart V-3.2 $C_s/a^2$</td>
<td>19</td>
</tr>
<tr>
<td>Chart V-3.3 $\sigma_{tu} (a/t)^{1.8}$</td>
<td>20</td>
</tr>
<tr>
<td>Chart V-3.4 $C_T/a^2$</td>
<td>21</td>
</tr>
<tr>
<td>Chart V-3.5 F-Factor</td>
<td>22</td>
</tr>
<tr>
<td>Chart V-3.6 W-Factor</td>
<td>23</td>
</tr>
<tr>
<td>Chart V-3.7 $G_f$ Factor</td>
<td>24</td>
</tr>
</tbody>
</table>
GROUP B—TABLES OF SECTION PROPERTIES

General Notes ............................................................... 25

FULL AREA TABLES
Table 1 Channel with Stiffened Flanges ........................................ 26
Table 2 Channel with Unstiffened Flanges ........................................ 28
Table 3 Z-Section with Stiffened Flanges ....................................... 29
Table 4 Z-Section with Unstiffened Flanges .................................... 30
Table 5 Two Channels with Stiffened Flanges Back to Back .................... 31
Table 6 Two Channels with Unstiffened Flanges Back to Back ................... 32
Table 7 Equal Leg Angle with Stiffened Legs ................................... 33
Table 8 Equal Leg Angle with Unstiffened Legs .................................. 33
Table 9 Hat Sections ........................................................... 34

EFFECTIVE AREA TABLES
Table 10 Channel with Stiffened Flanges ...................................... 35
Table 11 Channel with Unstiffened Flanges ...................................... 36
Table 12 Z-Section with Stiffened Flanges ..................................... 37
Table 13 Z-Section with Unstiffened Flanges ................................. 38
Table 14 Two Channels with Stiffened Flanges Back to Back .................. 39
Table 15 Two Channels with Unstiffened Flanges Back to Back ............... 40
GROUP A

CHARTS RELATED TO PARTICULAR SPECIFICATION PROVISIONS

GENERAL NOTES

(a) The appropriate equations from the Specification are generally shown on each design aid.

(b) The definitions of the terms used in these charts and tables can be found in the Specification.

(c) The torsional-flexural buckling charts are grouped together by cross-section type for convenience; that is, singly-symmetric angle sections, singly-symmetric channel sections, and singly-symmetric hat sections.

(d) The torsional-flexural buckling design charts are based on a square corner approximation for all section properties.

(e) In the titles and labels for the torsional-flexural buckling design charts, "a" and "ô" are used interchangeably.
If effective length, KL, of member is longer than $L_{cr}$, flexural mode is critical; if effective length is shorter than $L_{cr}$, torsional-flexural mode is critical.
TORSIONAL-FLEXURAL BUCKLING
(See Part III, Section 2 for application)
CHART V-1.7
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G Factor for Equal Angles (Single-Symmetric), With and Without Lips
Critical length, \( L_{cr} \) (in.)

If effective length, \( KL \), of member is longer than \( L_{cr} \), flexural mode is critical; if effective length is shorter than \( L_{cr} \), torsional-flexural mode is critical.

**CHART V.2.1**
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Channels (Singly-Symmetric), With and Without Lips
CHART V-2.2
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_v/\bar{a}^2$ for Channels (Singly-Symmetric), With and Without Lips
CHART V-2.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
\( \sigma_\text{cr} (b/t)^2 \) for Channels (Singly-Symmetric), With and Without Lips
CHART V-2.4
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_{t}/a^2$ for Channels (Singly-Symmetric), With and Without Lips
CHART V-2.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Channels (Singly-Symmetric), With and Without Lips
CHART V-3.1
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Hat Sections (Singly-Symmetric)

If effective length, KL, of member is longer than Lcr, flexural mode is critical; if effective length is shorter than Lcr, torsional-flexural mode is critical.

Critical length, Lcr (in.)

\( \delta/a \)

\( b/a \)

\( c/a \)

\( L/\bar{a}^2 \)
CHART V-3.2
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_{r}/\bar{a}^r$ for Hat Sections (Singly-Symmetric)
NOTE: Use lower curves and left ordinate for all small $\delta/\bar{a}$ and for large $\delta/\bar{a}$ if $\bar{c}/\bar{a}$ is below 0.4. Use upper curves and right ordinate for large $\delta/\bar{a}$ if $\bar{c}/\bar{a}$ is above 0.4.

CHART V-3.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$\sigma_0 (\bar{a}/t)^2$ for Hat Sections (Singly-Symmetric)
Charts and Tables for use with the March 16, 1991 Edition of the LRFD Cold-Formed Specification

**CHART V-3.4**

Torsional-Flexural Buckling
(See Part III, Section 2 for application)

$C_r/\bar{a}$ for Hat Sections (Singly-Symmetric)
CHART V-3.5
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Hat Sections (Singly-Symmetric)

R = \sigma_t / \sigma_{ss}

F

\bar{b}/\bar{a}

1.0 0.8 0.6 0.4 0.2 0.0 1.0 2.0 3.0 4.0
CHART V-3.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Hat Sections (Singly-Symmetric)
CHART V-3.7
Torsonal-Flexural Buckling
(See Part III, Section 2 for application)
Gₜₐₓ-Factor for Hat Section (Singly-Symmetric)
GROUP B
TABLES OF SECTION PROPERTIES
GENERAL NOTES

(a) The specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The effective section modulus values are calculated as the ratio of effective moment of inertia at the indicated stress level and the distance to the extreme fiber. In calculating the maximum moment capacity of these sections, additional checks such as the provisions of Chapter C of the Specification and the information on laterally unbraced compression flanges in Part III should also be taken into account where applicable.

(c) As a general rule, tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places. However, in some cases space limitations made it impractical to adhere strictly to this guideline.

(d) The weight of these sections is calculated based on steel as weighing 40.8 pounds per square foot per inch thickness.

(e) Where they apply, the algebraic formulae presented in Part III formed the basis of the calculations for these tables.

(f) The properties of Tables 5, 6, 14, and 15 apply only when the channels are adequately joined together. See Chapter D of the Specification.

(g) Tables 1-9 incl. are Full Area Tables. The effective section properties listed in Tables 10-15 incl. were computed for bending about x-axis using $F_y = 33$ksi and $50$ksi.

(h) The Full Area Tables were prepared at Cornell University by Shyh Hann Ji.
### TABLE 1

**CHANNEL WITH STIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size</th>
<th>D</th>
<th>B</th>
<th>R</th>
<th>Area</th>
<th>Wgt. per Foot</th>
<th>Axis x-x</th>
<th>Axis y-y</th>
<th>(m)</th>
<th>J</th>
<th>C</th>
<th>(j)</th>
<th>(r_e)</th>
<th>(x_e)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In.</td>
<td>ln.</td>
<td>ln.</td>
<td>ln.</td>
<td>ln.</td>
<td>Lb.</td>
<td>in.</td>
<td>in.</td>
<td>Lb.</td>
<td>in.</td>
<td>in.</td>
<td>Lb.</td>
<td>in.</td>
<td>in.</td>
</tr>
<tr>
<td>12.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>2.706</td>
<td>9.20</td>
<td>56.357</td>
<td>9.378</td>
<td>4.057</td>
<td>1.599</td>
<td>1.229</td>
<td>0.912</td>
<td>1.497</td>
</tr>
<tr>
<td>10.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>2.057</td>
<td>7.18</td>
<td>45.836</td>
<td>7.305</td>
<td>3.872</td>
<td>1.583</td>
<td>1.233</td>
<td>1.096</td>
<td>1.582</td>
</tr>
<tr>
<td>9.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.887</td>
<td>6.41</td>
<td>36.256</td>
<td>7.011</td>
<td>3.895</td>
<td>1.629</td>
<td>1.246</td>
<td>0.976</td>
<td>1.663</td>
</tr>
<tr>
<td>8.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>2.009</td>
<td>6.93</td>
<td>27.158</td>
<td>6.035</td>
<td>3.49</td>
<td>1.925</td>
<td>1.344</td>
<td>0.985</td>
<td>1.730</td>
</tr>
<tr>
<td>7.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.973</td>
<td>6.32</td>
<td>20.533</td>
<td>4.107</td>
<td>3.91</td>
<td>3.129</td>
<td>1.187</td>
<td>0.932</td>
<td>1.356</td>
</tr>
<tr>
<td>6.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.867</td>
<td>5.74</td>
<td>15.297</td>
<td>3.977</td>
<td>3.53</td>
<td>4.557</td>
<td>1.151</td>
<td>0.882</td>
<td>1.299</td>
</tr>
<tr>
<td>5.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.751</td>
<td>5.16</td>
<td>12.182</td>
<td>2.767</td>
<td>3.15</td>
<td>6.256</td>
<td>1.150</td>
<td>0.832</td>
<td>1.203</td>
</tr>
<tr>
<td>4.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.635</td>
<td>4.63</td>
<td>9.531</td>
<td>2.198</td>
<td>2.78</td>
<td>8.325</td>
<td>1.149</td>
<td>0.782</td>
<td>1.111</td>
</tr>
<tr>
<td>3.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.519</td>
<td>4.14</td>
<td>7.370</td>
<td>1.786</td>
<td>2.44</td>
<td>10.784</td>
<td>1.147</td>
<td>0.732</td>
<td>1.023</td>
</tr>
<tr>
<td>2.0</td>
<td>0.125</td>
<td>0</td>
<td>1.01</td>
<td>1.875</td>
<td>1.403</td>
<td>3.67</td>
<td>5.642</td>
<td>1.457</td>
<td>2.12</td>
<td>13.804</td>
<td>1.145</td>
<td>0.682</td>
<td>0.936</td>
</tr>
</tbody>
</table>

---

**Charts and Tables for use with the March 16, 1971 AISC Cold-Formed Specification**
### TABLE 1 (continued)

**CHANNEL WITH STIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size (D)</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>B</th>
<th>Area</th>
<th>Wgt. per Foot</th>
<th>Axis x-x</th>
<th>Axis y-y</th>
<th>Properties of Full Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>1.625</td>
<td>0.075</td>
<td>0.60</td>
<td>0.09375</td>
<td>0.594</td>
<td>2.02</td>
<td>1.443</td>
<td>0.722</td>
<td>1.56</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.458</td>
<td>1.59</td>
<td>1.153</td>
<td>0.578</td>
<td>1.57</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0.53</td>
<td>0.09375</td>
<td>0.277</td>
<td>1.28</td>
<td>0.806</td>
<td>0.467</td>
<td>1.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.038</td>
<td>0.60</td>
<td>0.09375</td>
<td>0.285</td>
<td>0.97</td>
<td>0.713</td>
<td>0.356</td>
<td>1.58</td>
</tr>
<tr>
<td>3.625</td>
<td>1.625</td>
<td>0.075</td>
<td>0.60</td>
<td>0.09375</td>
<td>0.566</td>
<td>1.93</td>
<td>1.146</td>
<td>0.631</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.445</td>
<td>1.51</td>
<td>0.915</td>
<td>0.505</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.357</td>
<td>1.21</td>
<td>0.744</td>
<td>0.410</td>
<td>1.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.038</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.271</td>
<td>0.922</td>
<td>0.567</td>
<td>0.314</td>
<td>1.45</td>
</tr>
<tr>
<td>3.5</td>
<td>2.00</td>
<td>0.155</td>
<td>0.70</td>
<td>0.1875</td>
<td>1.069</td>
<td>3.64</td>
<td>2.003</td>
<td>1.145</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.100</td>
<td>0.70</td>
<td>0.1875</td>
<td>0.847</td>
<td>2.88</td>
<td>1.620</td>
<td>0.926</td>
<td>1.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0.60</td>
<td>0.1875</td>
<td>0.613</td>
<td>2.08</td>
<td>1.222</td>
<td>0.698</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0.50</td>
<td>0.1875</td>
<td>0.483</td>
<td>1.64</td>
<td>0.979</td>
<td>0.589</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0.50</td>
<td>0.1875</td>
<td>0.389</td>
<td>1.22</td>
<td>0.795</td>
<td>0.454</td>
<td>1.43</td>
</tr>
<tr>
<td>3.0</td>
<td>1.75</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>0.742</td>
<td>2.52</td>
<td>1.017</td>
<td>0.678</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0.53</td>
<td>0.1875</td>
<td>0.628</td>
<td>1.79</td>
<td>0.767</td>
<td>0.572</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0.53</td>
<td>0.1875</td>
<td>0.426</td>
<td>1.45</td>
<td>0.628</td>
<td>0.418</td>
<td>1.21</td>
</tr>
<tr>
<td>2.5</td>
<td>1.625</td>
<td>0.075</td>
<td>0.60</td>
<td>0.09375</td>
<td>0.492</td>
<td>1.64</td>
<td>0.800</td>
<td>0.363</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.378</td>
<td>1.28</td>
<td>0.557</td>
<td>0.310</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.305</td>
<td>1.04</td>
<td>0.316</td>
<td>0.253</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.036</td>
<td>0.50</td>
<td>0.09375</td>
<td>0.231</td>
<td>0.786</td>
<td>0.242</td>
<td>0.193</td>
<td>1.02</td>
</tr>
</tbody>
</table>
**TABLE 2**

<table>
<thead>
<tr>
<th>Channel with Unstiffened Flanges</th>
<th>Properties of Full Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td></td>
</tr>
</tbody>
</table>

**Properties of Full Section**

<table>
<thead>
<tr>
<th>A'</th>
<th>b'</th>
<th>x</th>
<th>m</th>
<th>J</th>
<th>Cw</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- Charts and tables for use with the March 1986 Edition of the LRFD Cold-Formed Steel Specification.
<table>
<thead>
<tr>
<th>D</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>12</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>10</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>12</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>10</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>12</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>10</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>12</td>
<td>3.5</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
</tbody>
</table>

See notes on page V-25.
<table>
<thead>
<tr>
<th>Size</th>
<th>Wgt.</th>
<th>Area</th>
<th>Pol.</th>
<th>Foot</th>
<th>B</th>
<th>Ix</th>
<th>Sx</th>
<th>rx</th>
<th>Z-SECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.060</td>
<td>0.0864</td>
<td>0.376</td>
<td>0.2367</td>
<td>0.260</td>
<td>10.77</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0643</td>
<td>0.0806</td>
<td>0.464</td>
<td>0.0855</td>
<td>0.262</td>
<td>27.12</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0427</td>
<td>0.0388</td>
<td>0.420</td>
<td>0.0860</td>
<td>0.275</td>
<td>15.80</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2647</td>
<td>0.1840</td>
<td>0.480</td>
<td>0.8157</td>
<td>0.345</td>
<td>8.92</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 4**

See notes on page V-25
<table>
<thead>
<tr>
<th>Size</th>
<th>D</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>R</th>
<th>Area per Foot</th>
<th>Wgt. Axis x-x</th>
<th>Axis y-y</th>
<th>J</th>
<th>C_w</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.01</td>
<td>0.01</td>
<td>0.1875</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
<td>0.0241</td>
</tr>
<tr>
<td>7.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.1875</td>
<td>0.1875</td>
<td>0.06375</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
<td>0.0274</td>
</tr>
<tr>
<td>8.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.1875</td>
<td>0.1875</td>
<td>0.06375</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
<td>0.0298</td>
</tr>
<tr>
<td>9.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.1875</td>
<td>0.1875</td>
<td>0.06375</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
<td>0.0322</td>
</tr>
</tbody>
</table>

**TABLE 5**

**TWO CHANNELS WITH STIFFENED FLANGES BACK-TO-BACK**

**Properties of Full Section**

<table>
<thead>
<tr>
<th>D</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>R</th>
<th>Area per Foot</th>
<th>Wgt. Axis x-x</th>
<th>Axis y-y</th>
<th>J</th>
<th>C_w</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.1875</td>
<td>0.1875</td>
<td>0.06375</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
</tr>
<tr>
<td>12.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0.1875</td>
<td>0.1875</td>
<td>0.06375</td>
<td>0.060</td>
<td>0.72</td>
<td>0.06375</td>
<td>0.09375</td>
</tr>
</tbody>
</table>

See notes on page V-25
<table>
<thead>
<tr>
<th>Size</th>
<th>D (in)</th>
<th>B (in)</th>
<th>t (in)</th>
<th>R (in)</th>
<th>Area (in²)</th>
<th>Weight per Foot (Lb)</th>
<th>Properties of Full Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>l₁ (in)</td>
<td>S₁ (in²)</td>
</tr>
<tr>
<td>2.0</td>
<td>210</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.761</td>
<td>4.80</td>
<td>12.254</td>
</tr>
<tr>
<td>3.0</td>
<td>300</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>2.013</td>
<td>4.79</td>
<td>12.436</td>
</tr>
<tr>
<td>4.0</td>
<td>400</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.461</td>
<td>4.97</td>
<td>9.206</td>
</tr>
<tr>
<td>5.0</td>
<td>500</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.173</td>
<td>3.99</td>
<td>7.435</td>
</tr>
<tr>
<td>6.0</td>
<td>600</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>2.298</td>
<td>7.81</td>
<td>10.669</td>
</tr>
<tr>
<td>7.0</td>
<td>700</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.803</td>
<td>6.13</td>
<td>8.481</td>
</tr>
<tr>
<td>8.0</td>
<td>800</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.311</td>
<td>4.46</td>
<td>6.300</td>
</tr>
<tr>
<td>9.0</td>
<td>900</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.053</td>
<td>3.58</td>
<td>5.094</td>
</tr>
<tr>
<td>10.0</td>
<td>1000</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.845</td>
<td>2.87</td>
<td>4.110</td>
</tr>
<tr>
<td>12.0</td>
<td>1200</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.488</td>
<td>5.06</td>
<td>4.798</td>
</tr>
<tr>
<td>14.0</td>
<td>1400</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.093</td>
<td>3.69</td>
<td>3.585</td>
</tr>
<tr>
<td>16.0</td>
<td>1600</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.873</td>
<td>2.97</td>
<td>2.913</td>
</tr>
<tr>
<td>18.0</td>
<td>1800</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.732</td>
<td>2.46</td>
<td>1.846</td>
</tr>
<tr>
<td>20.0</td>
<td>2000</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.581</td>
<td>1.98</td>
<td>1.581</td>
</tr>
<tr>
<td>24.0</td>
<td>2400</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>1.015</td>
<td>3.45</td>
<td>1.281</td>
</tr>
<tr>
<td>28.0</td>
<td>2800</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.748</td>
<td>2.54</td>
<td>0.973</td>
</tr>
<tr>
<td>32.0</td>
<td>3200</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.603</td>
<td>2.05</td>
<td>0.794</td>
</tr>
<tr>
<td>36.0</td>
<td>3600</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.455</td>
<td>1.65</td>
<td>0.645</td>
</tr>
<tr>
<td>40.0</td>
<td>4000</td>
<td>225</td>
<td>0.105</td>
<td>0.1875</td>
<td>0.385</td>
<td>1.32</td>
<td>0.502</td>
</tr>
</tbody>
</table>

**2 CHANNELS WITH UNSTIFFENED FLANGES BACK-TO-BACK**

See notes on page V-25
### TABLE 7

**EQUAL LEG ANGLE WITH STIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size</th>
<th>D</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>R</th>
<th>Area</th>
<th>Wgt. per Foot</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>In.²</td>
<td>Lb.</td>
</tr>
<tr>
<td>4.0</td>
<td>4.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>1.10</td>
<td>1.278</td>
<td>4.345</td>
<td>2.594</td>
</tr>
<tr>
<td>3.0</td>
<td>3.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.962</td>
<td>1.271</td>
<td>4.345</td>
<td>1.083</td>
</tr>
<tr>
<td>2.5</td>
<td>2.5</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.797</td>
<td>2.498</td>
<td>0.828</td>
<td>0.407</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.632</td>
<td>2.148</td>
<td>0.500</td>
<td>0.302</td>
</tr>
</tbody>
</table>

### TABLE 8

**EQUAL LEG ANGLE WITH UNSTIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size</th>
<th>D</th>
<th>B</th>
<th>t</th>
<th>R</th>
<th>Area</th>
<th>Wgt. per Foot</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>In.²</td>
<td>Lb.</td>
</tr>
<tr>
<td>4.0</td>
<td>4.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>1.047</td>
<td>3.566</td>
<td>1.6847</td>
</tr>
<tr>
<td>3.0</td>
<td>3.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.777</td>
<td>2.642</td>
<td>0.7063</td>
</tr>
<tr>
<td>2.5</td>
<td>2.5</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.569</td>
<td>1.771</td>
<td>0.3985</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>0.135</td>
<td>0.1875</td>
<td>0.459</td>
<td>1.215</td>
<td>0.3965</td>
</tr>
</tbody>
</table>

**Properties of Full Section**

<table>
<thead>
<tr>
<th>Axis x-x and Axis y-y</th>
<th>r₁</th>
<th>J</th>
<th>Cₓ</th>
<th>rₓ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axis x-x and Axis y-y</td>
<td>L₁</td>
<td>rₓ</td>
<td>j</td>
<td>xₓ</td>
</tr>
</tbody>
</table>
rB~

TABLE 9

!

HAT SECTIONS

b

~d~

See notes on page V-25
Size

Sx

rx

Iv

Sy

ry

y

J

Cw

j

ro

Xo

In.

In.4

In. 3

In.

In.

In.4

In. 6

In.

In.

In.

23.2710
18.2608
10.7092
7.8514
3.1722
14.7525
11.5586
6.7247
4.9542
1.9756
1.6215
8.2408
6.4229
3.7063
2.7299
1.0707
0.8812
0.7221
3.7178
2.8447
1.6409
1.1788
0.4572
0.3704
0.3032
2.1885
1.6327
0.9563
0.6594
0.2611
0.2054
0.1650
1.1484
0.7719
0.3062
0.2368
0.0975
0.0743
0.7149
0.4537
0.1893
0.1406
0.0633
0.0455

6.456
6.421
4.468
4.451
2.352
5.208
5.166
3.602
3.569
1.889
1.881
3.971
3.918
2.744
2.691
1.431
1.418
1.412
2.763
2.691
1.911
1.832
0.996
0.972
0.956
2.184
2.095
1.515
1.413
0.793
0.759
0.738
1.896
1.772
1.014
0.973
0.565
0.532
1.540
1.392
0.831
0.780
0.484
0.442

3.461
3.346
3.863
3.758
4.458
2.878
2.767
3.188
3.059
3.623
3.566
2.290
2.185
2.507
2.360
2.787
2.726
2.686
1.686
1.592
1.814
1.680
1.970
1.911
1.861
1.371
1.287
1.457
1.334
1.555
1.500
1.453
0.873
0.776
0.978
0.931
1.080
1.039
0.722
0.636
0.791
O. 750
0.863
0.827

11.21
11.06
10.16
9.99
9.74
9.13
9.00
8.24
8.06
7.80
7.80
7.03
6.92
6.28
6.12
5.84
5.86
5.86
4.86
4.79
4.22
4.17
3.77
3.86
3.91
3.73
3.69
3.12
3.15
2.64
2.79
2.88
3.08
2.85
2.06
2.10
1.62
1.76
2.47
2.24
1.49
1.54
1.01
1.15

10.87
10.72
10.29
10.14
10.17
8.85
8.71
8.35
8.18
8.17
8.14
6.79
6.68
6.37
6.22
6.14
6.13
6.12
4.66
4.60
4.31
4.25
4.02
4.08
4.10
3.55
3.52
3.20
3.22
2.87
2.98
3.05
2.64
2.54
2.12
2.15
1.80
1.91
2.04
1.96
1.55
1.58
1.20
1.31

-7.95
-7.79
-8.54
-8.38
-9.27
-6.51
-6.37
-6.94
-6.78
-7.44
-7.42
-5.02
-4.92
-5.30
-5.16
-5.58
-5.59
-5.58
-3.41
-3.40
-3.54
-3.53
-3.62
-3.70
-3.73
-2.53
-2.57
-2.57
-2.66
-2.53
-2.68
-2.76
-1.65
-1.64
-1.70
-1.75
-1.55
-1.69
-1.19
-1.24
-1.17
-1.25
-0.953
-1.10

In.

In. 2

Lb.

In.4

In. 3

10.0

15.00

10.0

10.00

0.135
0.105
0.105
0.075
0.075
0.135
0.105
0.105
0.075
0.075
0.060
0.135
0.105
0.105
0.075
0.075
0.060
0.048
0.135
0.105
0.105
0.075
0.075
0.060
0.048
0.135
0.105
0.105
0.075
0.075
0.060
0.048
0.105
0.075
0.075
0.060
0.060
0.048
0.105
0.075
0.075
0.060
0.060
0.048

1.670
1.340
1.340
1.050
1.050
1.670
1.340
1.340
0.980
0.980
0.840
1.670
1.340
1.340
0.915
0.915
0.760
0.660
1.670
1.340
1.340
0.915
0.915
0.750
0.618
1.670
1.340
1.340
0.915
0.915
0.750
0.618
1.340
0.915
0.915
0.750
0.750
0.618
1.340
0.915
0.915
0.750
0.750
0.618

0.1875
0.1875
0.1875
0.09375
0.09375
0.1875
0.1875
0.1875
0.09375
0.09375
0.09375
0.1875
0.1875
0.1875
0.09375
0.09375
0.09375
0.09375
0.1875
0.1875
0.1875
0.09375
0.09375
0.09375
0.09375
0.1875
0.1875
0.1875
0.09375
0.09375
0.09375
0.09375
0.1875
0.09375
0.09375
0.09375
0.09375
0.09375
0.1875
0.09375
0.09375
0.09375
0.09375
0.09375

5.044
3.869
3.344
2.368
1.993
4.099
3.134
2.714
1.908
1.608
1.274
3.154
2.399
2.084
1.448
1.223
0.964
0.764
2.209
1.664
1.454
0.998
0.848
0.663
0.520
1.736
1.297
1.139
0.773
0.660
0.513
0.400
1.034
0.698
0.548
0.423
0.363
0.280
0.824
0.548
0.435
0.333
0.288
0.220

17.149
13.155
11.370
8.052
6.777
13.936
10.656
9.228
6.486
5.466
4.330
10.723
8.157
7.086
4.923
4.158
3.278
2.599
7.510
5.658
4.944
3.393
2.883
2.254
1.769
5.904
4.408
3.873
2.628
2.245
1. 744
1.361
3.516
2.373
1.863
1.438
1.234
0.953
2.802
1.863
1.480
1.132
0.979
0.749

67.50
50.20
43.60
30.20
24.10
36.30
26.80
23.30
15.80
12.60
9.82
16.40
12.10
10.40
6.91
5.47
4.24
3.32
5.42
3.96
3.39
2.23
1.75
1.34
1.03
2.47
1.80
1.53
1.01
0.784
0.599
0.459
0.670
0.432
0.328
0.249
0.193
0.147
0.303
0.198
0.147
0.112
0.0855
0.0656

10.3279
7.5430
7.1050
4.8420
4.3405
7.0912
5.1291
4.8340
3.2011
2.8710
2.2149
4.4300
3.1629
2.9828
1.8989
1.7042
1.2947
1.0019
2.3437
1.6444
1.5510
0.9602
0.8631
0.6428
0.4831
1.5155
1.0526
0.9921
0.6033
0.5423
0.3995
0.2970
0.5948
0.3531
0.3210
0.2333
0.2093
0.1534
0.3892
0.2293
0.2078
0.1500
0.1341
0.0975

6.0

9.00

6.0

6.00

6.0

3.00

4.0

6.00

4.0

4.00

4.0

2.00

3.0

4.50

3.0

3.00

3.0

1.50

2.0

4.00

2.0

2.00

2.0

1.00

1.5

3.00

1.5

1.50

1.5

0.75

,

Ix

In.

4.00

~d~

Wgt
per
Foot

In.

8.0

~

Area

In.

8.00

eN

R

In.

8.0

~

x

d

B

5.00
12.00

~

Bo

I

R

t

D

10.0
8.0

£1

x-I·+-·

TABLE 9

i

3.659 210.00
3.602 160.00
3.611. 66.80
3.572 46.90
3.474 11.00
2.977 111.00
2.926 83.60
2.928 35.20
2.879 24.30
2.796
5.74
2.777
4.51
2.283 49.70
2.243 36.80
2.236 15.70
2.185 10.50
2.116
2.51
2.097
1.94
2.084
1.53
1.567 16.90
1.543 12.00
1.527
5.31
1.494
3.35
1.437
0.841
1.423
0.626
1.409
0.476
1.192
8.28
1.179
5.69
1.159
2.62
1.140
1.54
1.089
0.415
1.081
0.296
1.071
0.218
0.805
3.72
0.787
2.19
0.774
0.563
0.768
0.400
0.728
0.116
0.725
0.0795
0.606
1.96
0.601
1.06
0.582
0.301
0.581
0.202
0.545
0.0674
0.546
0.0430

0.0306
2340.00
0.0142
1810.00
0.0123
685.00
0.00444
493.00
0.00374
98.40
0.0249
762.00
0.0115
597.00
0.00997
222.00
0.00358
161.00
0.00301
319.00
0.00153
25.80
0.0192
177.00
0.00882
140.00
0.00766
51.60
0.00271
37.70
0.00229
75.50
0.00116
60.40
0.000587
48.80
0.0134
22.60
0.00612
17.90
0.00534
6.89
0.00187
4.83
0.00159
1.15
0.000795
0.829
0.000400
0.640
0.0105
5.66
0.00476
4.20
0.00419
1.89
0.00145
1.14
0.00124
0.371
0.000615
0.235
0.000308
0.164
0.00380
1.13
0.00131
0.799
0.00103
0.180
0.000507
0.127
0.000435
0.0542
0.000215
0.0321
0.00303
0.312
0.00103
0.187
0.000816
0.0606
0.00039~
0.0376
0.000345
0.0210
0.000169
0.0122

(1
!:r'
Po'
'"'$

(""1-

lfJ

Po'

::::l
0..

~

0-

ro

lfJ

0'
'"'$
~
lfJ
(p

:E

;:t:
!:r'
(""1-

!:r'

(p

~

Po'

'"'$

~

!:r'

.....
.....
.....

~O'J
~
~

tr:l

0..
~.

o·
::::l

g,
(""1-

!:r'

(p

l'
~
~

t:l
(1

o

0:

~

3
(p

0..

r.n.
1'"1::j
(p
~

~
~

~

o·
::::l


## TABLE 10

### CHANNEL WITH STIFFENED FLANGES

<table>
<thead>
<tr>
<th>Size (In.)</th>
<th>( t ) (In.)</th>
<th>( d ) (In.)</th>
<th>( R )</th>
<th>Effective Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D = 12.0 )</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
<td>56.266, 55.387, 9.378, 9.134</td>
</tr>
<tr>
<td>10.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>43.231, 40.904, 7.139, 6.546</td>
</tr>
<tr>
<td>9.0</td>
<td>0.105</td>
<td>1.01</td>
<td>0.1875</td>
<td>36.525, 35.956, 7.305, 7.110</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>28.110, 26.513, 6.035, 6.036</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>20.917, 20.029, 4.620, 4.377</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>14.234, 13.551, 3.027, 2.897</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>19.371, 18.731, 4.843, 4.843</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>16.125, 15.726, 3.781, 3.613</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>10.335, 9.965, 2.516, 2.493</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>7.917, 7.397, 1.879, 1.867</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>6.567, 6.067, 1.540, 1.507</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>5.124, 4.604, 1.227, 1.197</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>4.843, 4.377, 1.043, 1.007</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>3.812, 3.362, 0.818, 0.774</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>2.922, 2.512, 0.600, 0.560</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>2.662, 2.282, 0.413, 0.380</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>2.343, 2.003, 0.303, 0.273</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>1.903, 1.603, 0.213, 0.183</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>1.683, 1.443, 0.143, 0.113</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>1.504, 1.254, 0.104, 0.084</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>1.324, 1.075, 0.074, 0.064</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>1.164, 1.014, 0.054, 0.044</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>1.044, 0.895, 0.034, 0.024</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>0.904, 0.755, 0.014, 0.014</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>0.784, 0.655, 0.004, 0.004</td>
</tr>
<tr>
<td>7.0</td>
<td>0.105</td>
<td>0.80</td>
<td>0.1875</td>
<td>0.694, 0.575, 0.000, 0.000</td>
</tr>
<tr>
<td>6.0</td>
<td>0.105</td>
<td>0.70</td>
<td>0.1875</td>
<td>0.624, 0.515, 0.000, 0.000</td>
</tr>
<tr>
<td>8.0</td>
<td>0.105</td>
<td>0.90</td>
<td>0.1875</td>
<td>0.554, 0.455, 0.000, 0.000</td>
</tr>
</tbody>
</table>

See notes on page V-25
### TABLE 11

**CHANNEL WITH UNSTIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size D In.</th>
<th>t In.</th>
<th>d In.</th>
<th>R In.</th>
<th>Effective Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( I_x, \text{ in.}^4 )</td>
</tr>
<tr>
<td>( F_y = 33\text{ksi} )</td>
<td>( F_y = 50\text{ksi} )</td>
<td>( F_y = 33\text{ksi} )</td>
<td>( F_y = 50\text{ksi} )</td>
<td></td>
</tr>
<tr>
<td>8.0</td>
<td>2.00</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>7.0</td>
<td>1.50</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>6.0</td>
<td>1.50</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>5.0</td>
<td>1.25</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>4.0</td>
<td>1.125</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>3.0</td>
<td>1.125</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td>2.0</td>
<td>1.125</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.069</td>
<td>0</td>
<td>0.09375</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
</tr>
</tbody>
</table>
### TABLE 12

**Z-SECTION WITH STIFFENED FLANGES**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size (In.)</th>
<th>t (In.)</th>
<th>d (In.)</th>
<th>R (In.)</th>
<th>Effective Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>B</td>
<td></td>
<td></td>
<td>( I_x, ) In.(^4 )</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>( F_y = 33 \text{ksi} )</td>
</tr>
<tr>
<td>12.0</td>
<td>3.50</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>10.0</td>
<td>3.50</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
</tr>
<tr>
<td>9.0</td>
<td>3.25</td>
<td>0.135</td>
<td>1.00</td>
<td>0.1875</td>
</tr>
<tr>
<td>8.0</td>
<td>3.00</td>
<td>0.135</td>
<td>0.90</td>
<td>0.1875</td>
</tr>
<tr>
<td>7.0</td>
<td>2.75</td>
<td>0.135</td>
<td>0.88</td>
<td>0.1875</td>
</tr>
<tr>
<td>6.0</td>
<td>2.50</td>
<td>0.135</td>
<td>0.82</td>
<td>0.1875</td>
</tr>
<tr>
<td>5.0</td>
<td>2.00</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
</tr>
<tr>
<td>4.0</td>
<td>2.00</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
</tr>
<tr>
<td>3.5</td>
<td>2.00</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
</tr>
<tr>
<td>3.0</td>
<td>1.75</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
</tr>
</tbody>
</table>

---

**Notes:**
- See notes on page V-25.
- Effective section properties are calculated for different yield strengths: \( F_y = 33 \text{ksi} \) and \( F_y = 50 \text{ksi} \).
- The table provides values for \( I_x, S_x \), and section properties at various thicknesses and sizes.
### TABLE 13

**Z-SECTION**  
WITH  
UNSTIFFENED FLANGES

See notes on page V-25

<table>
<thead>
<tr>
<th>Size</th>
<th>$t$</th>
<th>$d$</th>
<th>$R$</th>
<th>$I_{xx}$, In.$^4$</th>
<th>$S_{xx}$, In.$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>In.</td>
<td>In.</td>
<td>In.</td>
<td>$F_y = 33$ksi</td>
<td>$F_y = 50$ksi</td>
</tr>
<tr>
<td>8.0</td>
<td>2.00</td>
<td>0.136</td>
<td>0</td>
<td>0.1875</td>
<td>13.075</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>10.335</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>6.764</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>5.231</td>
</tr>
<tr>
<td>7.0</td>
<td>1.50</td>
<td>0.136</td>
<td>0</td>
<td>0.1875</td>
<td>7.840</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>6.218</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>4.600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>3.391</td>
</tr>
<tr>
<td>6.0</td>
<td>1.50</td>
<td>0.136</td>
<td>0</td>
<td>0.1875</td>
<td>5.334</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>4.240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>2.976</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>2.396</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>1.784</td>
</tr>
<tr>
<td>5.0</td>
<td>1.25</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>2.999</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>1.760</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>1.170</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>0.858</td>
</tr>
<tr>
<td>4.0</td>
<td>1.50</td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>1.286</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>0.971</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>0.758</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>0.588</td>
</tr>
<tr>
<td>3.0</td>
<td>1.50</td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>1.370</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>1.062</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>0.858</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>0.643</td>
</tr>
<tr>
<td>2.0</td>
<td>1.125</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>0.636</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>0.485</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>0.379</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>0.293</td>
</tr>
<tr>
<td>1.5</td>
<td>1.125</td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>0.240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075</td>
<td>0</td>
<td>0.09375</td>
<td>0.187</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>0</td>
<td>0.09375</td>
<td>0.146</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.048</td>
<td>0</td>
<td>0.09375</td>
<td>0.113</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.036</td>
<td>0</td>
<td>0.09375</td>
<td>0.067</td>
</tr>
</tbody>
</table>

Notes:
- See notes on page V-25 for detailed instructions and explanations.
### TABLE 14

#### TWO CHANNELS

**WITH STIFFENED FLANGES**

**BACK TO BACK**

See notes on page V-25

<table>
<thead>
<tr>
<th>Size D</th>
<th>B</th>
<th>t</th>
<th>d</th>
<th>R</th>
<th>( I_x, \text{ in.}^2 )</th>
<th>( S_x, \text{ in.}^1 )</th>
<th>( F_y = 33 \text{ ksi} )</th>
<th>( F_y = 50 \text{ ksi} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.0</td>
<td>7.0</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
<td>112.53</td>
<td>110.77</td>
<td>18.755</td>
<td>18.269</td>
</tr>
<tr>
<td>10.0</td>
<td>7.0</td>
<td>0.135</td>
<td>1.01</td>
<td>0.1875</td>
<td>73.051</td>
<td>71.912</td>
<td>14.610</td>
<td>14.221</td>
</tr>
<tr>
<td>9.0</td>
<td>6.5</td>
<td>0.135</td>
<td>1.00</td>
<td>0.1875</td>
<td>54.313</td>
<td>54.313</td>
<td>12.070</td>
<td>12.070</td>
</tr>
<tr>
<td>8.0</td>
<td>6.0</td>
<td>0.135</td>
<td>0.84</td>
<td>0.1875</td>
<td>41.834</td>
<td>40.058</td>
<td>9.245</td>
<td>8.614</td>
</tr>
<tr>
<td>7.0</td>
<td>5.5</td>
<td>0.135</td>
<td>0.88</td>
<td>0.1875</td>
<td>33.741</td>
<td>38.741</td>
<td>9.685</td>
<td>9.685</td>
</tr>
<tr>
<td>6.0</td>
<td>5.0</td>
<td>0.135</td>
<td>0.82</td>
<td>0.1875</td>
<td>28.671</td>
<td>27.102</td>
<td>9.054</td>
<td>8.749</td>
</tr>
<tr>
<td>5.0</td>
<td>4.0</td>
<td>0.135</td>
<td>0.82</td>
<td>0.1875</td>
<td>24.699</td>
<td>22.702</td>
<td>8.054</td>
<td>7.723</td>
</tr>
<tr>
<td>4.0</td>
<td>4.0</td>
<td>0.135</td>
<td>0.82</td>
<td>0.1875</td>
<td>20.871</td>
<td>19.831</td>
<td>7.054</td>
<td>6.882</td>
</tr>
<tr>
<td>3.5</td>
<td>4.0</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
<td>17.491</td>
<td>16.491</td>
<td>6.154</td>
<td>5.914</td>
</tr>
<tr>
<td>3.0</td>
<td>3.5</td>
<td>0.135</td>
<td>0.70</td>
<td>0.1875</td>
<td>14.491</td>
<td>13.491</td>
<td>5.254</td>
<td>4.994</td>
</tr>
</tbody>
</table>

Notes:
- \( t \) = Wall thickness
- \( d \) = Depth
- \( R \) = Flange width
- \( I_x \) = Moment of inertia about x-axis
- \( S_x \) = Section modulus about x-axis
- \( F_y \) = Yield strength

See page V-25 for additional notes.
### TABLE 15

#### TWO CHANNELS WITH UNSTIFFENED FLANGES BACK TO BACK

See notes on page V-25

<table>
<thead>
<tr>
<th>Size (in.)</th>
<th>t (in.)</th>
<th>d (in.)</th>
<th>R (in.)</th>
<th>Effective Section Properties</th>
<th>( I_x, \text{in.}^2 )</th>
<th>( S_y, \text{in.}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( F_y = 33\text{ksi} )</td>
<td>( F_y = 50\text{ksi} )</td>
<td>( F_y = 33\text{ksi} )</td>
</tr>
<tr>
<td>8.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>26.150</td>
<td>25.380</td>
<td>6.537</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>20.669</td>
<td>19.161</td>
<td>5.167</td>
</tr>
<tr>
<td>7.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>15.679</td>
<td>15.679</td>
<td>4.480</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>12.435</td>
<td>12.239</td>
<td>3.553</td>
</tr>
<tr>
<td>6.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>10.668</td>
<td>10.668</td>
<td>3.556</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>8.490</td>
<td>8.336</td>
<td>2.827</td>
</tr>
<tr>
<td>5.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>5.552</td>
<td>5.736</td>
<td>1.927</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>4.121</td>
<td>4.445</td>
<td>1.462</td>
</tr>
<tr>
<td>4.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>3.520</td>
<td>3.400</td>
<td>1.392</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>2.740</td>
<td>2.641</td>
<td>1.062</td>
</tr>
<tr>
<td>3.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>2.123</td>
<td>2.047</td>
<td>0.806</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>1.716</td>
<td>1.313</td>
<td>0.562</td>
</tr>
<tr>
<td>2.0</td>
<td>0.135</td>
<td>0</td>
<td>0.1875</td>
<td>1.176</td>
<td>1.086</td>
<td>0.373</td>
</tr>
<tr>
<td></td>
<td>0.105</td>
<td>0</td>
<td>0.1875</td>
<td>0.751</td>
<td>0.839</td>
<td>0.348</td>
</tr>
</tbody>
</table>
COMPUTER AIDS
FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE
LOAD AND
RESISTANCE FACTOR
DESIGN SPECIFICATION
FOR COLD-FORMED
STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part VI

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.

1st Printing—December 1991

Produced by William P. Reyman, New York

Copyright American Iron and Steel Institute 1991
PREFACE

This document, Part VI of the LRFD Cold-Formed Steel Design Manual, contains many useful design aids; flow charts for many sections of the Specification are included. These charts will be very helpful in getting all users familiar with the new Specification. The charts serve to direct the user to the appropriate sections. They will also be useful to programmers.

These Computer Aids should be used in conjunction with the other parts of the Design Manual, which include Commentary (Part II), Supplementary Information (Part III), Illustrative Examples (Part IV), Design Aids (Part V), and Test Procedures (Part VII).

American Iron and Steel Institute
December 1991
# Table of Contents

**Part VI**

**Computer Aids**

Based on the

**March 16, 1991 Edition of the**

**Load and Resistance Factor Design Specification**

**for Cold-Formed Steel Structural Members**

---

<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preface</td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Table of Contents</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>Flow Charts for Sections of the Specification</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B2.1</td>
<td>Uniformly Compressed Stiffened Elements</td>
<td>6</td>
</tr>
<tr>
<td>B2.2</td>
<td>Uniformly Compressed Stiffened Elements with Circular Holes</td>
<td>10</td>
</tr>
<tr>
<td>B2.3</td>
<td>Effective Widths of Webs and Stiffened Elements with Stress Gradient</td>
<td>11</td>
</tr>
<tr>
<td>B3.1</td>
<td>Uniformly Compressed Unstiffened Elements</td>
<td>13</td>
</tr>
<tr>
<td>B3.2</td>
<td>Unstiffened Elements and Edge Stiffeners with Stress Gradient</td>
<td>14</td>
</tr>
<tr>
<td>B4.1</td>
<td>Uniformly Compressed Elements with an Intermediate Stiffener</td>
<td>15</td>
</tr>
<tr>
<td>B4.2</td>
<td>Uniformly Compressed Elements with an Edge Stiffener</td>
<td>17</td>
</tr>
<tr>
<td>B5</td>
<td>Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener</td>
<td>19</td>
</tr>
<tr>
<td>B6.1</td>
<td>Transverse Stiffeners</td>
<td>21</td>
</tr>
<tr>
<td>B6.2</td>
<td>Shear Stiffeners</td>
<td>23</td>
</tr>
<tr>
<td>C3.1</td>
<td>Strength for Bending Only</td>
<td>26</td>
</tr>
<tr>
<td>C3.1.1</td>
<td>Nominal Section Strength</td>
<td>27</td>
</tr>
<tr>
<td>C3.1.2</td>
<td>Lateral Buckling Strength</td>
<td>30</td>
</tr>
<tr>
<td>C3.1.3</td>
<td>Beams Having One Flange Through-Fastened to Deck or Sheathing</td>
<td>35</td>
</tr>
<tr>
<td>C3.2</td>
<td>Strength for Shear Only</td>
<td>37</td>
</tr>
<tr>
<td>C3.3</td>
<td>Strength for Combined Bending and Shear</td>
<td>39</td>
</tr>
<tr>
<td>C3.4</td>
<td>Web Crippling Strength</td>
<td>41</td>
</tr>
<tr>
<td>C3.5</td>
<td>Combined Bending and Web Crippling Strength</td>
<td>46</td>
</tr>
<tr>
<td>C4</td>
<td>Concentrically Loaded Compression Members</td>
<td>47</td>
</tr>
<tr>
<td>C5</td>
<td>Combined Axial Load and Bending (Includes Moment Capacity of Flexural Members)</td>
<td>49</td>
</tr>
<tr>
<td>C6.1</td>
<td>Bending - Cylindrical Tubular Members</td>
<td>54</td>
</tr>
<tr>
<td>C6.2</td>
<td>Compression - Cylindrical Tubular Members</td>
<td>55</td>
</tr>
<tr>
<td>D4.1</td>
<td>Capacity of a Wall Stud</td>
<td>57</td>
</tr>
<tr>
<td>E2.2</td>
<td>Arc Spot Welds</td>
<td>61</td>
</tr>
<tr>
<td>E2.3</td>
<td>Arc Seam Welds</td>
<td>63</td>
</tr>
<tr>
<td>E2.4</td>
<td>Fillet Welds</td>
<td>64</td>
</tr>
<tr>
<td>E2.5</td>
<td>Flare Groove Welds</td>
<td>66</td>
</tr>
<tr>
<td>E3.1</td>
<td>Spacing and Edge Distance</td>
<td>68</td>
</tr>
<tr>
<td>E3.2</td>
<td>Tension in Connected Part</td>
<td>70</td>
</tr>
<tr>
<td>E3.3</td>
<td>Bearing</td>
<td>72</td>
</tr>
</tbody>
</table>
B2.1 Uniformly Compressed Stiffened Elements

Load Capacity Determination

Yes

For Inelastic Reserve Capacity

Yes

No

$f = F_y$

No

Initial Yield in Compression

Yes

No

$f = M_y/S$

or $f = M_c/S$

S = Section modulus to compression element

(continued on next page)
B2.1 Uniformly Compressed Stiffened Elements (continued)

Stiffened by a Web on Each Longitudinal Edge

No

Yes

k = 4

Determine k

See Sections B4.1 and B4.2

\( \lambda, \text{ Eq. B2.1-4} \)

\( \lambda \leq 0.673 \)

Yes

\( b = w \)

No

\( \rho, \text{ Eq. B2.1-3} \)

\( b = \rho w \)

(continued on next page)
B2.1 Uniformly Compressed Stiffened Elements (continued)

\[ f = \frac{M}{S} \]

\[ \lambda, \text{ Eq. B2.1-4} \]

\[ \lambda \leq 0.673 \]

Yes

Use Procedure I

No

Stiffened by Web on Each Long Edge

No

Yes

\[ \lambda_c, \text{ Eq. B2.1-10} \]

Section F or Rational Analysis

M = Service Moment

S = Section Modulus to Compression Element

(continued on next page)
B2.1 Uniformly Compressed Stiffened Elements (continued)

- \( \lambda < \lambda_c \)
  - Yes: \( \rho, \text{Eq. B2.1-3} \)
  - No:
    - \( \rho, \text{Eq. B2.1-8} \)
    - \( \rho, \text{Eq. B2.1-9} \)

- \( b_d = w \)
- \( b_d = \rho w \)
**B2.2 Uniformly Compressed Stiffened Elements with Circular Holes**

1. $w/t \leq 70$
   - Yes
   - $d_h/w \leq 0.50$
     - Yes
     - $0.5w < \text{Spacing}$
       - Yes
       - $3d_h < \text{Spacing}$
         - Yes
         - $\lambda \leq 0.673$
           - No
           - $\lambda \leq 0.673$
             - Yes
             - $b, \text{Eq. B2.2-1}$
             - No
             - $b, \text{Eq. B2.2-2}$
   - No
   - $b, \text{Eq. B2.2-1}$

- Section F
B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient

Load Capacity Determination

Yes

\( f_1 \) & \( f_2 \) calculated at Maximum Moment Capacity Using Effective Section Properties

No

\( f_1 \) & \( f_2 \) Calculated at Actual Moment Using Effective Section Properties

\( f_1 \) Compression
\( f_1 \) Tension

No

Yes

\( \psi = \frac{f_1}{f_2} \)

\( k \)

See Figure B2.3-1

(continued on next page)
B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient (continued)

- $b_e$, Effective Width
- $b_1$, Using Section B2.1 at Stress $f_1$, and $w = h$

- $b_i$, Eq. B2.3-1

- $\psi \leq -0.236$
  - No
  - $b_1$, Eq. B2.3-2
  - $b_2$, Eq. B2.3-3

- $b_1 + b_2 < h_c$
  - No
  - $h_c = \text{Width of Element Subject to Compression}$
  - Yes
  - Use $b_i$, & $b_2$ as Effective Widths

- Yes
  - Use $h_c$ as Effective Width
B3.1 Uniformly Compressed Unstiffened Elements

\[ k = 0.43 \]
\[ w, \text{Figure B3.1-1} \]

Flowchart:
- **Load Capacity Determination**
  - **Yes**
    - Determine Effective Width \( b \), Using Section B2.1
  - **No**
    - Use Procedure I in Section B2.1(b)
B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

- $k = 0.43$

- $f = \text{Maximum Compression Stress in Element}$

- Load Capacity Determination

  - Yes: Determining Effective Width, $b$, Using Section B2.1
  - No: Use Procedure I in Section B2.1(b)
B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

Load Capacity Determination

Yes

See Section B2.1 for Stress, f

See Section B4

\[ \frac{b_o}{t} \leq S \]

Yes

\[ I_a, \text{ Eq. B4.1-2} \]

No

\[ \frac{b_o}{t} < 3S \]

Yes

\[ I_a, \text{ Eq. B4.1-6} \]

k, Eq. B4.1-7

No

\[ f = \frac{M}{S_e} \]

M = Service Moment

\[ S_e = \text{Effective Section Modulus} \]

(continued on next page)
B4.1 Uniformly Compressed Elements with an Intermediate Stiffener (continued)

1. **b**, Eq. B4.1-3
   - **A_s**, Eq. B4.1-4

2. **Determine Effective Width, b, and Effective Stiffener Area, A_s', Using Section B2.1**
   - **A_s**, Eq. B4.1-8

3. **Determine Effective Width, b, and Effective Stiffener Area, A_s', Using Section B2.1**
   - **A_s**, Eq. B4.1-11
B4.2 Uniformly Compressed Elements with an Edge Stiffener

Load Capacity Determination

Yes

See Section B2.1 for Stress, $f$

No

$f = M/S$

$S = \text{Section modulus to compression element}$

$w/t \leq S/3$

No

$w/t < S$

No

Yes

Yes

$I_a$, Eq. B4.2-2

$I_a$, Eq. B4.2-6

$n = 1/2$

$n = 1/3$

$C_t$, Eq. B4.2-7

$C_t$, Eq. B4.2-8

(continued on next page)
B4.2 Uniformly Compressed Elements with an Edge Stiffener

Determine Effective Width, $b$, and Effective Stiffener Area, $A_s'$, Using Section B2.1

- $D/w \leq 0.8$
- $D/w \leq 0.25$

$k$, Eq. B4.2-9

$b$, Eq. B4.2-3

$d_s$, Eq. B4.2-4

$A_s$, Eq. B4.2-5

$k$, Eq. B4.2-10

$d_s$, Eq. B4.2-11

$A_s$, Eq. B4.2-12
Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

\( I_{\text{min}}, \text{Eq. B5-1} \)

\[ I_S = \text{Moment of Inertia of Stiffener} \]

\( I_S > I_{\text{min}} \)

No \( \rightarrow \) Stiffener is Inadequate

Yes

Effective Width, \( b \), Using Section B2.1

\( b < w \)

No

Yes

(continued on next page)
B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener (continued)

Stiffener Between Two Webs

Yes

Use the Two Stiffeners Closest to the Webs

No

Use the One Stiffener Closest to the Web

Disregard Stiffener

Use All the Stiffeners

\( t_s, \text{ Eq. B5-2} \)
B6.1 Transverse Stiffeners

Stiffened Elements

Yes

No

\( w/t_s > 1.28 \sqrt{E/F_{ys}} \)

No

Yes

No

\( w/t_s > 0.37 \sqrt{E/F_{ys}} \)

Yes

Section F

Interior Support or Concentrated Load

No

Yes

\( A_c, \text{ Eq. B6.1-2} \)

\( A_b, \text{ Eq. B6.1-4} \)

\( A_c, \text{ Eq. B6.1-3} \)

\( A_b, \text{ Eq. B6.1-5} \)

(continued on next page)
B6.1 Transverse Stiffeners (continued)

\[ P_{nl}, \text{Eq. B6.1-1} \]

\[ b_1, \text{Eq. B6.1-6} \]
\[ b_2, \text{Eq. B6.1-7} \]
\[ A_e = A_b \]

\[ P_{n2}, \text{Using} \]
\[ \text{Section C4(a)} \]

\[ P_{n1} > P_{n2} \]
\[ \text{No} \]

\[ \text{Yes} \]

\[ \text{Use } \Phi_c = 0.85 \]
\[ P_n = P_{n2} \]

\[ \text{Use } \Phi_c = 0.85 \]
\[ P_n = P_{nl} \]
B6.2 Shear Stiffeners

\[ \Phi_v V_n, \text{ Section C3.2} \]

\[ V_u \leq \Phi_v V_n \]

- Yes

\[ a/h \leq 3.0 \]

- No

\[ a/h \leq \left[ \frac{260}{(h/t)^2} \right] \]

- Yes

\[ I_s \geq I_{s min}, \text{ Eq. B6.2-1} \]

\[ I_s = \text{Moment of Inertia of Stiffener} \]

\[ I_s \geq I_{s min} \]

- No

\[ V_u = \text{Required Shear Strength} \]

Stiffener Spacing Inadequate

Section F

Stiffener Moment of Inertia Inadequate

(continued on next page)
B6.2 Shear Stiffeners (continued)

\( a/h \leq 1.0 \)

No

Yes

\( k_v, \text{Eq. B6.2-5} \)

\( k_v, \text{Eq. B6.2-6} \)

\( C_v, \text{Eq. B6.2-3} \)

\( C_v \leq 0.8 \)

No

Yes

\( C_v, \text{Eq. B6.2-4} \)

Pairs of Stiffeners

No

Yes

Single-Angle Stiffeners

No

(continued on next page)
B6.2 Shear Stiffeners (continued)

- **Yes**
  - D = 1.0
  - D = 1.8
  - D = 2.4

- **No**
  - Single-Plate Stiffeners
    - Yes
  - Section F

**A_{st}**, Eq. B6.2-2

**A_s \geq A_{st}**

- **No**
  - Stiffener Area Inadequate
- **Yes**
  - Stiffener is Adequate
C3.1 Strength for Bending Only

Beams Having One Flange Through-Fastened to Deck or Sheathing

Yes

No

$\phi_b M_{n1} = \phi_b M_n \text{ by Section C3.1.1}$

$\phi_b M_{n2} = \phi_b M_n \text{ by Section C3.1.2}$

$\phi_b M_{n1} < \phi_b M_{n2}$

No

Yes

$\phi_b M_n = \phi_b M_{n1}$

$\phi_b M_n = \phi_b M_{n2}$

$\phi_b M_n \text{ by Section C3.1.3}$
C3.1.1 Nominal Section Strength

\[ M_{n1} = M_n \text{ by Eq. C3.1.1-1} \]

- Procedure I
  - Yes
  - No
  - (Procedure II)

- Conditions 1-5 met [See (b)]
  - Yes
  - No

- Unstiffened Compression Elements
  - Yes
  - No

- Calculate Effective Widths Using Section B3.1
C3.1.1 Nominal Section Strength (continued)

Without Intermediate Stiffeners

- **Yes**
  - w/t ≤ \( \lambda_1 \)
    - **Yes** => \( C_Y = 3 \)
    - **No**
  - w/t < \( \lambda_2 \)
    - **Yes** => \( C_Y = 1 \)
    - **No** => \( C_Y = 3 - 2(w/t - \lambda_1)/(\lambda_2 - \lambda_1) \)

- **No**

Use Effective Widths for Calculating \( M_n \)

(continued on next page)
C3.1.1 Nominal Section Strength (continued)

Yes

\[ M_{n2} = 1.25M_{n1} \]

No

\[ M_{n2} > 1.25M_{n1} \]

Calculate \( M_n \) Using Maximum Compression Strain of \( C_y e_y \)

\( M_{n2} = \text{Calculated} \ M_n \)

\( M_n = M_{n1} \)

Stiffened or Partially Stiffened Compression Flanges

Yes

\( \Phi_b = 0.95 \)

No

\( \Phi_b = 0.90 \)

\( \Phi_b M_n \)

(continued on next page)
C3.1.2 Lateral Buckling Strength

Singly-, Doubly-, and Point-Symmetric Sections

Yes

\[ \sigma_{ex}, \text{ Eq. C3.1.2-7} \]
\[ \sigma_{ey}, \text{ Eq. C3.1.2-8} \]
\[ \sigma_t, \text{ Eq. C3.1.2-9} \]

No

Section F

(continued on next page)
C3.1.2 Lateral Buckling Strength (continued)

No

Bending About Symmetric Axis

Yes

No

\[ C_{TF} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) \]

Yes

\[ C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3 \left( \frac{M_1}{M_2} \right)^2 \]

Compression on Shear Center Side

No

Yes

\[ C_s = +1 \]

\[ C_s = -1 \]

\[ M_e, \text{ Eq. C3.1.2-6} \]

\[ C_b \leq 2.3 \]

No

Yes

\[ C_b = 2.3 \]

\[ M_e, \text{ Eq. C3.1.2-5} \]

(continued on next page)
C3.1.2 Lateral Buckling Strength (continued)

My, Eq. C3.1.2-4

\[ M_c > 0.5 M_y \]

No

Yes

\[ M_c, \text{ Eq. C3.1.2-2} \]

\[ M_c, \text{ Eq. C3.1.2-3} \]

\[ M_n, \text{ Eq. C3.1.2-1} \]

Use \( \Phi_b = 0.90 \)
\[ \Phi_b M_n \]

(continued on next page)
C3.1.2 Lateral Buckling Strength (continued)

I or Z-sections Bent about the Centroidal Axis Perpendicular to the Web (x-axis)

Yes

Doubly Symmetric I

Yes

No

No


$M_e$, Eq. C3.1.2-15

$M_e$, Eq. C3.1.2-16

$M_y$, Eq. C3.1.2-4

(continued on next page)
C3.1.2 Lateral Buckling Strength (continued)

\[ M_c \leq 0.56M_y \]

No

Yes

\[ M_c < 2.78M_y \]

No

Yes

\[ M_c, \text{ Eq. C3.1.2-14} \]

\[ M_c, \text{ Eq. C3.1.2-13} \]

\[ M_c, \text{ Eq. C3.1.2-12} \]

\[ M_n, \text{ Eq. C3.1.2-1} \]

Use \( \phi_b = 0.90 \)

\( \phi_b M_n \)

(continued on next page)
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

C- or Z-Sections

Limitations (1) Through (14) Satisfied

Simple Span

(continued on next page)
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

- C-Sections: No → R = 0.40
- C-Sections: Yes → R = 0.50
- C-Sections: No → R = 0.60
- C-Sections: Yes → R = 0.70

\[ \Phi_b = 0.90 \]
\[ M_n, \text{ Eq. C3.1.3-1} \]

\[ \Phi_b M_n \]

Section F or Rational Analysis
### C3.2 Strength for Shear Only

1. **Unreinforced Web**
   - **No**
   - **Yes**

2. **a/h > 1.0**
   - **No**
   - **Yes**

3. **k_v, Eq. C3.2-5**
   - **No**
   - **Yes**

4. **k_v, Eq. C3.2-4**
   - **No**
   - **Yes**

5. **k_v = 5.34**

6. **h/t ≤ \(\sqrt{E_k v / F_y}\)**
   - **No**
   - **Yes**

7. **h/t ≤ 1.415\(\sqrt{E_k v / F_y}\)**
   - **No**
   - **Yes**

(continued on next page)
C3.2 Strength for Shear Only (continued)

\[ \Phi_v = 0.90 \]
\[ V_n, \text{ Eq. C3.2-3} \]

\[ \Phi_v = 0.90 \]
\[ V_n, \text{ Eq. C3.2-2} \]

\[ \Phi_n = 1.0 \]
\[ V_n, \text{ Eq. C3.2-1} \]

\[ \Phi_v V_n \]
C3.3 Strength for Combined Bending and Shear

\[ \Phi_b M_n \] by Section C3.1
\[ \Phi_b M_{nxo} \] by Section C3.1.1
\[ \Phi_v V_n \] by Section C3.2

Yes

Transverse Stiffeners

No

\[ M_u \leq \Phi_b M_n \]
\[ V_u \leq \Phi_v V_n \]

Yes

\[ M_u / \Phi_b M_{nxo} > 0.5 \]
\[ V_u / \Phi_v V_n > 0.7 \]

Yes

Eq. C3.3-2 Satisfied

No

Eq. C3.3-1 Satisfied

(continued on next page)
C3.3 Strength for Combined Bending and Shear (continued)

- Section is Satisfactory
- Section is not Satisfactory
C3.4 Web Crippling Strength

\[ \text{h/t} \leq 200 \]
\[ \frac{R}{t} \leq 6 \text{ for beams} \]
\[ \frac{R}{t} \leq 7 \text{ for deck} \]
\[ \frac{N}{t} \leq 210 \]
\[ \frac{N}{h} \leq 3.5 \]

No \rightarrow \text{Section F}

Yes \rightarrow \text{Single Unreinforced Webs}

\[ \Phi_w = 0.75 \]

No \rightarrow \text{To A}

(continued on next page)
C3.4 Web Crippling Strength (continued)

Opposing Loads
Spaced > 1.5h

Yes

End Reaction

No

Stiffened Flanges

No

Yes

End Reaction

No

Yes

(continued on next page)
C3.4 Web Crippling Strength (continued)

\[
P_n \quad \text{Eq. C3.4-1}
\]

\[
P_n \quad \text{Eq. C3.4-2}
\]

\[
P_n \quad \text{Eq. C3.4-4}
\]

\[
P_n \quad \text{Eq. C3.4-6}
\]

\[
P_n \quad \text{Eq. C3.4-8}
\]

\[
B
\]

\[
\Phi_w P_n
\]

(continued on next page)
C3.4 Web Crippling Strength (continued)

\[ \Phi_w = 0.80 \]

Opposing Loads
Spaced > 1.5h

End Reaction

(continued on next page)
C3.4 Web Crippling Strength (continued)

\[ P_n, \text{ Eq. C3.4-3} \]
\[ P_n, \text{ Eq. C3.4-5} \]
\[ P_n, \text{ Eq. C3.4-7} \]
\[ P_n, \text{ Eq. C3.4-9} \]

To B
C3.5 Combined Bending and Web Crippling Strength

\[ \Phi_b M_{nxo} \text{ by Section C3.1.1} \]
\[ \Phi_w P_n \text{ by Section C3.4} \]

Shape with Single Web

Yes

Eq. C3.5-1 Satisfied

Yes

Section is Satisfactory

No

Eq. C3.5-2 Satisfied

Yes

Section is Satisfactory

No

No
C4 Concentrically Loaded Compression Members

Not Subject to T/F Buckling

No

Doubly & Singly Symmetric

No

\[ \sigma_t \text{ & } \sigma_{ex} \]
by Section C3.1.2(a)

Yes

Conservative Estimate

No

\[ F_{e1} = F_e \text{ by Section C4.1} \]
\[ F_{e2} = F_e \text{ by Section C4.2} \]

Yes

(continued on next page)
C4 Concentrically Loaded Compression Members (continued)

- **Fe by Rational Analysis**
  - **Fe**, Eq. C4.1-1

- **Fe**, Eq. C4.2-2
  - **Fe = F_e1**
    - **Fe = F_e2**

  - **Fe > F_y/2**
    - **No**
    - **F_n**, Eq. C4-2
    - **F_n**, Eq. C4-3
      - Calculate **A_e**
          - at **F_n**

        - **Φ_c = 0.85**
          - **P_n**, Eq. C4-1

          - **Φ_cP_n**
C5 Combined Axial Load and Bending

\[ \Phi_c P_n \text{ by Section C4} \]

If \( \frac{P_u}{\Phi_c P_n} > 0.15 \)

No

Yes

\[ \Phi_c P_{no} \text{ by Section C4 using } F_n = F_y \]
\[ \Phi_b M_{nx} & \Phi_b M_{ny} \text{ by Section C3} \]
\[ P_E, \text{ Eq. C5-5} \]
\[ \text{Magnification Factor, Eq. C5-4} \]

Frame
Subject to
Sidesway

No

Yes

(continued next page)
C5 Combined Axial Load and Bending (continued)

1. Not Subject to Transverse Loads
   - Yes
   - No

2. Ends Restrained
   - Yes
   - No

3. Eq. C5-3 Satisfied
   - Yes
   - No

4. Eq. C5-1 & Eq. C5-2 Satisfied
   - Yes
   - No

5. Section is Unsatisfactory

(continued on next page)
Moment Capacity of Flexural Members

Assume:
1. Maximum Compressive Bending Stress
2. Position of Neutral Axis

Edge Stiffener

Yes

\[ A_s & \quad d_s \]
by Section B4.2

Effective Edge Stiffener Length and Area

Intermediate Stiffeners

No

Yes

(continued on next page)
Effective Stiffeners & Thickness by Section B5

$A_s$ & $b$ by Section B4.1

$b$ by Section B2.1

$b$ by Sections B3.1 & B2.1

$b_1$ & $b_2$, by Section B2.3

Effective Web Depth

Calculate Effective Section Properties

Calculate Actual Compressive Bending Stress

Effective Stiffener Area and Flange Area

Effective Flange Width

(continued on next page)
Moment Capacity of Flexural Members (continued)

- Assume Stress & N.A. = Actual Values
  - Yes: $\Phi_b M_n$ by Section C3.1
  - No: Revise Compressive Stress and Location of Neutral Axis
C6.1 Bending - Cylindrical Tubular Members

- If $D/t \leq 0.441E/F_y$ then Yes, go to $M_n$, Eq. C6.1-1.
- If $D/t \leq 0.070E/F_y$ then No, go to $D/t \leq 0.319E/F_y$.
  - If $D/t \leq 0.319E/F_y$ then No, go to Section F.
  - If $D/t \leq 0.319E/F_y$ then Yes, go to $M_n$, Eq. C6.1-2.
- If $D/t \leq 0.070E/F_y$ then Yes, go to $M_n$, Eq. C6.1-1.

- $\Phi_b = 0.95$
- $\Phi_b M_n$
C6.2 Compression - Cylindrical Tubular Members

D/t ≤ 0.441E/F_y

Yes

F_e by Section C4.1

F_e > F_y/2

Yes

F_n, Eq. C6.2-2
A_e, Eq. C6.2-3

No

F_n = F_e
A_e = A

No

Section F

(continued on next page)
C6.2 Compression - Cylindrical Tubular Members (continued)

\[ \Phi_c = 0.85 \]

\[ P_n, \text{ Eq. C6.2-1} \]

\[ \Phi_c P_n \]
D4.1 Capacity of a Wall Stud

Determine $F_n$ by Section C4 with $K_L$ Equal to Twice the Distance Between Fasteners

- **No**
  - Doubly-Symmetric I-Section
  - Z-Section
  - Singly Symmetric Channel or C-Section

- **Yes**
  - $\sigma_{CR} =$ Smaller value from Eq. D4.1-6
  - $\sigma_{CR} =$ Smaller value from Eq. D4.1-7

(continued on next page)
\[
F_n = \sigma_{CR}
\]

B

Smallest \( F_n \)

\( \Phi_c = 0.85 \)

\( P_n \), Eq. D4.1-1

\( \Phi_c P_n \)

Section F

(continued on next page)
D4.1 Capacity of a Wall Stud (continued)

Assume $F_n$

I-Section

No

Yes

$C_1$, $E_j$

Eq. D4.1-20

Z-Section

No

Yes

$C_1$, $E_j$

Eq. D4.1-18

Eq. D4.1-19

C-Section

No

Yes

$C_1$, $E_j$

Eq. D4.1-16

Eq. D4.1-17

$\gamma$, Eq. D4.1-15

(continued on next page)
D4.1 Capacity of a Wall Stud (continued)

\[ \gamma \approx \bar{\gamma} \]

No

Yes

\( F_n = \text{Assumed } F_n \)

To B

Section F

(continued on next page)
E2.2 Arc Spot Welds

\[ \Phi = 0.50 \]
\[ P_{n1}, \text{Eq. E2.2-4} \]

\[ \Phi = 0.60 \]
\[ P_{n1}, \text{Eq. E2.2-2} \]

\[ \Phi = 0.50 \]
\[ P_{n1}, \text{Eq. E2.2-3} \]

\[ \Phi = 0.60 \]
\[ P_{n2}, \text{Eq. E2.2-1} \]

\[ \Phi P_{n1} \& \Phi P_{n2} \]

\[ \Phi P_{n} = \text{Smaller Value of } \Phi P_{n1} \text{ and } \Phi P_{n2} \]

(continued on next page)
E2.2 Arc Spot Welds (continued)

\[ \begin{align*}
\tau_{\text{min}} & \geq d \\
F_{xx} & \geq 60 \text{ ksi} \\
F_{uu} & \leq 60 \text{ ksi} \\
t & \geq 0.028 \text{ in.}
\end{align*} \]

No \quad \rightarrow \quad \text{Section F}

Yes

\[ \Phi = 0.65 \]

\[ P_{n}, \text{ Eq. E2.2-7} \]

\[ \Phi P_{n} \]
E2.3 Arc Seam Welds

\[
\Phi = 0.60
\]

\[P_{n1}, \text{Eq. E2.3-1}\]

\[
\Phi = 0.60
\]

\[P_{n2}, \text{Eq. E2.3-2}\]

\[\Phi P_{n1} \& \Phi P_{n2}\]

\[\Phi P_{n} = \text{Smaller of } \Phi P_{n1} \text{ and } \Phi P_{n2}\]
E2.4 Fillet Welds

Longitudinal Loading

Yes

L/t = < 25

Yes

\( \Phi = 0.60 \)

\( P_{n1}, \text{ Eq. E2.4-1} \)

No

\( \Phi = 0.55 \)

\( P_{n1}, \text{ Eq. E2.4-2} \)

No

\( \Phi P_{n1} \)

\( t > 0.150^\circ \)

Yes

\( \Phi = 0.60 \)

\( P_{n2}, \text{ Eq. E2.4-4} \)

No

\( \Phi P_{n2} \)

(continued on next page)
E2.4 Fillet Welds (continued)

\[ \Phi P_n = \Phi P_{n1} \]

\[ \Phi P_n = \text{Smaller of } \Phi P_{n1} \text{ and } \Phi P_{n2} \]
E2.5 Flare Groove Welds

Flare Bevel Groove Welds Transverse Loading

No

Flare Groove Welds Longitudinal Loading

No

Section F

Yes

Philip = 0.55

P_{nl1}, Eq. E2.5-1

\( \ell \leq l_w < 2t \)

Lip Height < L

No

\( l_w \geq 2t \)

Lip Height \geq L

No

Yes

Yes

\( \Phi = 0.55 \)

P_{nl1}, Eq. E2.5-2

\( \Phi = 0.55 \)

P_{nl1}, Eq. E2.5-3

(continued on next page)
E2.5 Flare Groove Welds (continued)

Yes

\[ t > 0.15'' \]

No

\[ \Phi P_{n1} \]

\[ \Phi P_n = \Phi P_{n1} \]

\[ \Phi = 0.60 \]

\[ P_{n2}, \text{ Eq. E2.5-4} \]

\[ \Phi P_{n2} \]

\[ \Phi P_n = \text{Smaller of } \Phi P_{n1} \text{ and } \Phi P_{n2} \]
E3.1 Spacing and Edge Distance

1. In Line of Stress
   - Yes: $P_n^*$, Eq. E3.1-1
   - No: Oversized or Slotted Holes

2. Oversized or Slotted Holes
   - Yes: *Clear Distance Between Edges of Holes $\geq 2d$*
     - Edge of Hole to End of Member $\geq d$
   - No: Center of Hole to End or Boundary $\geq 1-1/2 \ d$

3. $F_u/F_y \geq 1.15$
   - Yes: $\Phi = 0.70$
   - No: $\Phi = 0.60$

(continued on next page)
E3.1 Spacing and Edge Distance  (continued)

If \( \Phi_P \) is:

- Yes, then calculate \( e_{\text{min2}} = \lfloor e - (d_h/2) \rfloor \)
- No, then calculate \( e_{\text{min1}} = 3d \)

Oversized and Slotted Holes
E3.2 Tension in Connected Part

- \( t \geq 3/16 \)  
  - Yes: Use AISC
  - No: Washers Under Both Bolt Head and Nut
    - Yes: \( P_{n1}, \text{Eq. E3.2-1} \)
    - No: \( P_{n1}, \text{Eq. E3.2-2} \)
      - \( \Phi = 0.65 \), Double Shear
        - \( = 0.55 \), Single Shear

(continued on next page)
E3.2 Tension in Connected Part (continued)

\[ \Phi P_{n1} \& \Phi t P_{n2} \]

\[ \Phi P_n = \text{Smaller of} \]
\[ \Phi P_{n1} \text{ and } \Phi t P_{n2} \]
E3.3 Bearing

Washers Under Both Bolt Head and Nut

- No → To A
- Yes → t ≥ 3/16

- Yes → Use AISC
- No → t ≥ 0.024

- No → Section F
- Yes → (continued on next page)
E3.3 Bearing (continued)

- Inside Sheet of Double Shear Connection
  - Yes
  - No

- $F_u/F_{sy} \geq 1.15$
  - Yes
  - No
    - $\Phi = 0.55$
      - $P_n = 3.33 F_u dt$
    - $\Phi = 0.65$
      - $P_n = 3.00 F_u dt$
    - $\Phi = 0.60$
      - $P_n = 3.00 F_u dt$

- $\Phi P_n$

(continued on next page)
E3.3 Bearing (continued)

A

$t \geq 3/16$

Yes: Use AISC

No

$t \geq 0.036$

No: Section F

Yes

Inside Sheet of Double Shear Connection

No

Yes

(continued on next page)
E3.3 Bearing (continued)

- \( \frac{F_u}{F_{sy}} \geq 1.15 \) 
  - Yes: \( \Phi = 0.70 \) \( P_n = 3.00F_u dt \) 
  - No: To B

- \( \frac{F_u}{F_{sy}} \geq 1.15 \) 
  - Yes: \( \Phi = 0.65 \) \( P_n = 2.22F_u dt \) 
  - No: To B
TEST PROCEDURES
FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE
LOAD AND
RESISTANCE FACTOR
DESIGN SPECIFICATION
FOR COLD-FORMED
STEEL STRUCTURAL
MEMBERS

LRFD Cold-Formed Steel Design Manual-Part VII

AMERICAN IRON AND STEEL INSTITUTE
1101 17th STREET, NW
WASHINGTON, DC 20036-4700
This publication is for general information only. The information in it should not be used without first securing competent advice with respect to its suitability for any given application. The publication of the information is not intended as a representation or warranty on the part of American Iron and Steel Institute—or any other person named herein—that the information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of the information assumes all liability arising from such use.
PREFACE


American Iron and Steel Institute
December 1991
Test Procedures for use with the March 16, 1991 Edition of the LRFD Cold-Formed Specification
# TABLE OF CONTENTS

## PART VII

**TEST PROCEDURES FOR USE WITH THE**

**MARCH 16, 1991, EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS**

**PREFACE** ................................................................. 3

**ROTATIONAL-LATERAL STIFFNESS TEST METHOD FOR BEAM-TO-PANEL ASSEMBLIES**

1. Scope .................................................................. 6
2. Description of Terms ............................................ 6
3. Materials ............................................................ 8
4. Test Specimens ................................................... 8
5. Test Setup .......................................................... 9
6. Test Procedures .................................................. 11
7. Number of Tests .................................................. 11
8. Test Evaluation Procedure ..................................... 12
9. Test Report ......................................................... 13
10. Alternate Rotational-Lateral Stiffness Test .................... 14

**STUB-COLUMN TEST METHOD FOR EFFECTIVE AREA OF COLD-FORMED STEEL COLUMNS**

1. Scope ................................................................. 16
2. Applicable Documents ............................................. 16
3. Terminology ......................................................... 17
4. Significance ......................................................... 18
5. Apparatus ............................................................ 19
6. Test Units ............................................................ 19
7. Stub-Column Specimens ......................................... 20
8. Stub-Column Procedure ......................................... 22
9. Calculations ......................................................... 23
10. Report ............................................................... 25
11. Precision ........................................................... 25
    References ........................................................ 25
    Appendix A - Use of Axial Shortening Measurements in Design .................................................. 26
    Appendix B - Parametric Studies and Figures .......... 27

**STANDARD METHODS FOR DETERMINATION OF UNIFORM AND LOCAL DUCTILITY**

1. Scope ................................................................. 28
2. Referenced Documents ............................................ 28
3. Symbols ............................................................ 28
4. Test Procedure .................................................... 28
5. Alternate Test Procedure ......................................... 29
1. Scope

1.1 The purpose of this test is to determine the rotational-lateral stiffness of beam-to-panel assemblies. This test method is used primarily in determining the strength of beams connected to panels as part of a structural assembly. The unattached "free" flange of the beam is restrained from lateral displacements and twisting by the bending stiffness of the beam elements, the connection between the "attached" flange of the beam and the panel, and the bending stiffness of the panel.

1.2 This test method applies to structural subassemblies consisting of panel, beam, and joint components, or of the joint between a wall, floor, ceiling, or roof panel and the supporting beam (purlin, girt, joist, stud).

1.3 This test method is also used to establish a limit of the displacements for avoiding joint failure.

1.4 The combined stiffness of the assembly determined by this method, $K$, consists of: (a) the lateral stiffness of the beam, $K_a$, which is a function of the geometry of the beam and geometric details of the beam-to-panel connection, (b) the local stiffness of the joint components in the immediate vicinity of the connection, $K_p$, which is affected by the type of fasteners, the fastener spacing used, and the geometry of the elements connected, and (c) the bending stiffness of the panel, $K_c$, which is a function of the moment of inertia of the panel, the beam spacing, and the beam location (edge vs interior). The latter stiffness shall be taken into account by theoretical analysis or by using the alternate test procedure described in Section 10.

1.5 For specific geometric conditions the design engineer may require duplicate testing using a new specimen with the beam orientation, or the force direction, reversed.

2. Description of Terms

2.1 Subassembly — A subassembly is a representative portion of a larger structural assembly consisting of a wall, floor, ceiling, or roof panel with one beam connected to the panel either continuously or at regular intervals (Figure 1).

2.2 Panel — The panel used in the subassembly may be made of any structural material, for example: aluminum, reinforced concrete, fiberboard, gypsum board, plastic, plywood, steel, etc. (Figure 1).

2.3 Beam — A beam may have an open or a closed cross section. One flange of the beam is connected to the panel, and is called the "attached" flange. The other is the "unattached" flange (Figure 1).
2.4 Joint or Connection—A joint or connection includes the local area around a mechanical fastener, weld, or adhesively bonded area that connects the beam with the panel. The local area also includes filler material such as insulation located between the panel and the beam flange.

2.5 Lateral Load—The total lateral load, $P$ (in kips), is applied to the unattached flange of the beam (Figure 2) in a plane parallel to that of the original panel position.

2.6 Lateral Deflection—The lateral deflection (Figure 2) is the lateral displacement, $D$ (in inches), of the unattached flange due to the lateral load, $P$. 

---

Figure 1 Wall, Floor, Ceiling, or Roof Assembly

![Figure 1](image1)

(a) Loading Diagram

![Figure 2](image2)

(b) Deflected Subassembly

Figure 2 Loaded and Deflected Subassembly
2.7 Rotational-Lateral Stiffness—The rotational-lateral stiffness, \( K \), is equal to the total lateral load applied on the unattached flange of the test beam, divided by the length dimension of the beam, \( L_{ab} \) (Figure 3b), and divided by the lateral deflection of the unattached flange of the beam at that load level. Thus, the units of \( K \) are: kips of lateral load per inch of beam length per inch of deflection, or kips/in./in.

3. Materials

3.1 Components of the test specimen(s) shall be measured, and the component suppliers shall be identified.

3.2 Physical and material properties of the panel and beam shall be determined according to the latest edition of Specification ASTM E370 or other applicable standards.

4. Test Specimens

4.1 The overall panel width, \( W \) (Figure 3), of the specimen shall be such that the dial-gage support and the specimen support are each separated from the beam by a distance, \( W_i \), not less than the largest of the following distances: (a) 1.5 times the overall panel depth \( P_D \), (b) the overall width of the attached beam flange \( W_F \), and (c) the fastener spacing along the flange of the beam, \( F_s \). For ribbed panels, \( W_i \) shall also exceed two times the width of the attached flat of the panel.
4.2 The clamped width of the specimen, \( W_c \), shall be at least equal to two times the panel depth, but not less than 2 inches.

4.3 The end dimension, \( W_e \), shall be long enough to conveniently attach a dial gage or an extensometer to the end of the panel.

4.4 The minimum overall panel width shall be equal to:

\[
W = W_e + 2W_i + W_r + W_c
\]  

4.5 The minimum beam and panel length, \( L_{B_i} \), of the test specimen shall not be less than the largest of (a) two times the maximum connector spacing, \( F_s \), used in actual field installations, or (b) the nominal coverage width of the panel. The specimen shall contain at least two fasteners in each line of connections along the beam.

4.6 Each specimen shall be assembled under the supervision of a representative of the testing laboratory, either at the manufacturer's facilities or at the testing laboratory.

4.7 Each specimen shall be assembled from new material; i.e., materials not used in previous test specimens, and in accordance with manufacturer's specifications.

4.8 The fabrication and field installation procedures specified for the overall assembly, and the tools used, shall also be used in the specimen construction as much as possible.

4.9 Drilled or punched pilot holes in the panels or beams shall be the same as those used in field installations.

5. Test Setup

5.1 The test specimens may be tested in a horizontal or vertical position (Figure 3 and Figure 4, respectively). The zero-load readings of the deflection-measuring device(s) shall be recorded.
5.2 The clamped end of the panel shall be the only support of the test specimen.

5.3 When the test specimen panel is a hollow-core, corrugated, or trapezoidal panel, voids of the clamped regions shall be filled with filler materials such as wood, gypsum, or similar filler materials to ensure that the clamped overall depth of the panel is reasonably maintained. For foam-filled sandwich panels, if necessary, the filler material over the distance $W_c$ may be replaced with wood, gypsum, or similar filler materials.

5.4 Loads applied to the unattached flange shall be introduced as close as possible to the extreme fiber of the beam, or at the intersection of the outer faces of the unattached flange and the web.

5.5 If the beam does not have a flat face perpendicular to the panel at the locations where the load is to be applied and the lateral displacement is to be measured, brackets are to be mechanically attached to the beam web to provide a flat surface. Figure 5 shows a typical application of a load bracket and/or dial gage bracket. The attachment of either bracket shall be accomplished such that the bracket does not stiffen the beam, or reduce its distortion.

5.6 The total lateral load applied, $P$, shall be distributed over several locations, if necessary, to reduce variations in the lateral deflection along the length of the unattached flange.

5.7 The load application shall be accomplished by chain or wire, and the necessary precautions shall be taken to ensure that the direction of the applied load remains essentially parallel to the original plane of the panel (Figure 5).
5.8 One or more dial gages or displacement transducers shall be used to measure the lateral displacements during loading. The gages shall be arranged symmetrically about the mid-width point, and have graduations at not greater than 0.001-inch intervals.

6. Test Procedures

6.1 The dial-gage height, \( H_D \), and load height, \( H_L \), as shown in Figure 3, shall be arranged such as to equal as close as possible the overall beam depth, \( H \). Prior to loading the test specimen, the dimensions \( H_D \) and \( H_L \), and the dial-gage readings shall be recorded.

6.2 No preload is to be used. The load shall be applied in a direction which is critical for the intended use of the results.

6.3 The applied load shall be increased in five or more equal increments to the maximum expected value, in order to produce deflection increments of not more than 5 percent of the beam depth.

6.4 If the specimen includes fiberglass insulation or other non-metallic elements in the joint between panel and beam, the load shall be held at each increment for 5 minutes before reading the lateral movement.

6.5 After each load increment is added, and the deflection has stabilized, the load and lateral movement of the unattached flange shall be measured and recorded.

6.6 A test shall be terminated at failure (fastener pullout, fastener failure, panel buckling, panel failure, beam failure, etc.) and the mode of failure recorded, unless the design engineer has determined that the application of the rotational-lateral stiffness, \( K \), occurs at lower load or displacement levels and that the test may be terminated earlier.

7. Number of Tests

7.1 The minimum number of tests for one set of parameters shall be three. For parametric studies using multiple values of one or more parameters a smaller number of tests may be used.

7.2 If used as part of a series of at least three tests, one test is sufficient for a specific condition of an all-metallic mechanically-fastened specimen using the same basic components, but using unique geometrical or physical-property differences such as fastener spacings, different beam or panel yield strengths, etc.

7.3 Three tests are required for any specific condition of welded or adhesively-bonded specimens, or for specimens using non-metallic materials.

7.4 When the rotational-lateral stiffness for three or more panel or beam thicknesses with otherwise identical parameters is to be determined, at least two specimens each with the minimum and the maximum thickness shall be tested. For a ratio of maximum-to-minimum thicknesses greater than 2.5, additional specimens with intermediate thicknesses must be tested. One test of every thickness may be used in accordance with Section 7.2.

7.5 When the rotational-lateral stiffness for a range of screw spacings is to be determined, the minimum number of specimens shall be as follows: For a ratio of maximum-to-minimum screw spacings equal to or less than 2, at least two specimens each with the minimum and the maximum screw spacing shall be tested. For a range of five or more different screw spacings, or for a ratio of maximum-to-minimum screw spacings greater than 2, additional specimens with intermediate spacings must be tested. One test of every screw spacing may be used in accordance with Section 7.2.

7.6 Where the rotational-lateral stiffness for a range of other panel parameters—such as
yield or ultimate strength, changes in geometry, etc.—are to be determined, a number of

tests similar to the requirements under Sections 7.2 through 7.5 shall be performed.

7.7 For unsymmetric or staggered fastener arrays and/or beams unsymmetric about a

plane parallel to the web, duplicate tests may be required by the design engineer using new

specimens with the beam orientation, or the force direction, reversed.

8. Test Evaluation Procedure

8.1 Typical load-displacement curves (P vs. D) obtained from the tests are as shown in

Figure 6. For multiple tests of one set of test parameters, the curve resulting in the lowest

value of $K_t$, as defined in Section 8.2, shall be used for the test evaluation procedure.*

---

*The test stiffness, $K_t$, includes the stiffness effects of the beam, $K_n$, and the beam-to-panel connection, $K_w$, but

excludes the effects of the bending stiffness of the panel, $K_v$, and follows the relationship $K_t = (1/K_n + 1/K_v)^{-1}$. 

---

Figure 6 Typical Load-displacement Curves
8.2 The test stiffness, $K_t$, at any load level is determined by

$$K_t = \frac{P}{D/L_B}$$  \hspace{1cm} (2)

8.3 The nominal test stiffness, $K_N$, shall be determined by

$$K_N = \frac{P_N}{D_N/L_B}$$  \hspace{1cm} (3)

where $P_N$ and $D_N$ shall be determined for a point, $N$, such that either $P_N$ shall be equal to 0.8 times the ultimate load, $P_u$, for load-displacement curves as shown in Figure 6(a), or the displacement $D_N$ shall be equal to 0.8 times the ultimate displacement, $D_u$, for load-displacement curves as shown in Figure 6(b), or by a tangent drawn from the origin to the P-D curve as shown in Figure 6(c), resulting in $P_N \leq 0.8P_u$ and $D_N \leq 0.8D_u$.

8.4 When the design engineer specifies in advance a desired maximum lateral displacement limit of $D_{NL}$, the test may be discontinued when $D_{NL}$ is reached, and $K_N$ may be determined from $P_N$ at $D_{NL}$, as long as the limits under Section 8.3 are observed and $D_{NL}$ is not exceeded in actual design applications.

8.5 Where either $H_D$ or $H_L$ are not equal to the overall beam height, $H$, $K_t$ and $K_N$ shall be corrected by the factor $H_D H_L/H^2$.

8.6 In addition, $K_t$ and $K_N$ shall be adjusted by the stiffness contributions of the panel, $K_p$, derived from the linear-elastic displacement analysis representing the actual design applications, unless such an analysis shows that these contributions are insignificant. Alternately, the panel stiffness may be included by using the alternate test method under Section 10.

8.7 For subassemblies such as shown in Figure 2, the applied lateral test loads cause a bending moment distribution in the panel similar to that shown in Figure 7, and a lateral displacement of the unattached flange of the beam, $D_c$, equal to

$$D_c = \frac{PH_L^2 W_S}{12EI}$$  \hspace{1cm} (4)

where $W_S$ is the width of the subassembly (Figure 2 and Figure 7), $E$ is the modulus of elasticity of the panel material, and $I$ is the effective moment of inertia of the panel cross section (obtained from deflection determination calculations for cold-formed metal deck panels).

The panel stiffness is equal to

$$K_p = \frac{1}{D_c}$$  \hspace{1cm} (5)

8.8 The overall rotational-lateral stiffness of the subassembly shall be determined as

$$K = \left(\frac{1}{K_t} + \frac{1}{K_p}\right)^{-1}$$  \hspace{1cm} (7)

8.9 When tests covering ranges of parameters (thickness, yield strengths, screw spacings, etc.) are conducted according to Section 7, a linear interpolation may be used to determine intermediate $K$ values.

9. Test Report

9.1 The test report shall consist of a description of all specimen components, including drawings defining the actual and nominal geometry, material specifications, material properties test results describing the actual physical properties of each component, and the sources of supply. Differences between the actual and the nominal dimensions and material properties shall be noted in the report.
9.2 In addition, the test report shall contain a sketch or photograph of the test setup, the latest calibration date and accuracy of the equipment used, the signature of the person responsible for the tests, and a tabulation of all raw and evaluated test data.

9.3 All graphs resulting from the test evaluation procedure shall be included in the test report.

9.4 A summary statement, or tabulation, shall be included in the summary of the report to define the actual and nominal rotational-lateral stiffness derived from the tests conducted, including all limitations.

10. Alternate Rotational-Lateral Stiffness Test*

10.1 To include the panel-stiffness contribution in the test, rather than by linear-elastic analysis, the design engineer may request a test specimen and setup as shown in Figure 8 and Figure 9, respectively.

10.2 The test specimens shall be as described under Section 4 except as follows.

10.2.1 The minimum overall panel width of the specimen, \( W \) (Figure 8), shall be

\[
W = W_p + W_i + W_c
\]  

(6)

10.2.2 The minimum end dimension, \( W_p \), shall equal the width of the attached beam flange plus 4 inches to allow the development of local deformation patterns around the fasteners as they would develop in a real structure.

*This method is conservative as compared to the basic methods which analytically account for the stiffness of the panel.
10.2.3 For specimens representing interior-beam subassemblies, as shown in Figures 1 and 2, the dimension $W_t$ of the test specimen (Figure 8) shall be equal to $\frac{1}{12}$ of the subassembly width, $W_s$ (Figures 1 and 2), to assure that the overall rotational-lateral stiffness contribution of the test-specimen panel is the same as that of the subassembly.

10.2.4 For other subassembly conditions, $W_t$ shall be determined to represent the actual conditions.

10.3 The test-setup shall be as described under Section 5 except as follows.

10.3.1 The clamped support as shown in Figures 8 and 9 shall be sufficiently rigid to minimize the rotation and translation of the test specimen at the support.

10.3.2 The lateral-displacement measuring device shall be located on a support fixed relative to the clamped support of the test panel, as shown in Figure 9.
10.4 Test procedures shall be the same as described under Section 6.

10.5 The number of tests shall be determined as described in Section 7.

10.6 The test-evaluation procedure shall follow the underlying principles used to develop Section 8. The test stiffness at any load level shall be determined according to Equation 2 and the nominal test stiffness shall be determined according to Equation 3. No further adjustments are needed.

10.7 For other interior-beam spacings, for exterior-beam conditions, or for other geometrical conditions, the measured displacements shall be adjusted by a linear-elastic analysis to represent the actual field conditions, unless such an analysis shows that these displacements and their effect on K are insignificant.

STUB-COLUMN TEST METHOD(1)
FOR
EFFECTIVE AREA OF COLD-FORMED STEEL COLUMNS

1. Scope

1.1 This test method covers the determination of the effective cross-sectional area of cold-formed steel columns. It primarily considers the effects of local buckling and residual stresses and applies to solid or perforated columns that have holes (or hole patterns) in the flat and/or curved elements of the cross section (1).

1.2 The effective area is used to determine the allowable axial loads of cold-formed column sections in accordance with the AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members, hereafter called AISI Specification.

1.3 The effective area is a variable section property of columns. It reflects the effects of local buckling in relatively thin area elements caused by axial stresses, or loads. When the axial load is zero, the effective area is equal to the gross cross-sectional area; however, when an axial load is applied, the effective area may be less than the gross area. In such a case, the effective area will reduce with increasing load.

1.4 Local buckling reduces the axial load-carrying capacity that would otherwise be limited only by general yielding or overall column buckling. The amount of the reduction depends on the width-to-thickness ratio of the flat elements of the column cross section, the yield strength of the steel sheet from which the column is formed, and the size and frequency of holes or hole patterns, if present.

2. Applicable Documents

2.1 ASTM Standards:
   - A370—Tensile Test Method For Steel Sheets
   - E4—Verification of Testing Machines

2.2 AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members, 1991 Edition.

(1)This test and evaluation method will be proposed to the appropriate ASTM Committee for review and adoption.
(2)Numbers in parentheses refer to references at the end of this test method.
3. Terminology

3.1 ASTM Definitions Standards:
E6—Definitions of Terms Relating to Methods of Mechanical Testing.
E380—Standard for Metric Practice.

3.2 Description of terms specific to this standard:
Elements = Straight or curved portions of the cross section of a column or stub column.
Local Buckling = The local buckling mode of a flat element of a column cross section, which influences the overall column-buckling behavior.
Overall Buckling = Buckling of a column as a function of its overall length.
Stub-Column = An axial compression member of the same cross section and material as the column for which the strength needs to be determined, but of sufficiently short length to preclude overall column buckling, if possible.

3.3 Symbols:

\( A \) = the gross cross-sectional area of a column without holes or perforations, or the minimum gross cross-sectional area of a column with holes or perforations.

\( A_s \) = the average of all gross cross-sectional areas of the stub columns in a test unit, or the average of gross cross-sectional areas of a stub column.

\( A_e \) = the effective cross-sectional area of a stub column at a load less than the ultimate test load, or the effective area of a full-length column.

\( A_{ei} \) = the effective cross-sectional area of a stub column at load \( P_i \).

\( A_{eu} \) = the nominal effective cross-sectional area at ultimate load adjusted to the nominal thickness and the minimum specified yield strength.

\( A_{eua} \) = the average effective cross-sectional area of a test unit of stub columns at the ultimate axial load.

\( A_{eu1} \) = the effective cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

\( A_{eu2} \) = the effective cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

\( A_1 \) = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

\( A_2 \) = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

\( D \) = the axial shortening of a stub column at load \( P \).

\( D_i \) = the axial shortening of a stub column at load \( P_i \).

\( D_u \) = the axial shortening of a stub column at load \( P_u \).

\( f \) = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, \( A_e \).

\( f_i \) = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, \( A_{ei} \) at load \( P_i \).
f_o = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e, above which the section is not fully effective.

F_n = the nominal ultimate stress, assumed to be uniformly distributed over the effective cross section of a column as calculated from Section C4 of the AISI Specification, at which flexural, torsional, torsional-flexural, or local buckling, and/or yielding, may occur.

F_u = the ultimate stress, assumed to be uniformly distributed, at which local failure occurs in a tested stub column.

F_y = the minimum specified elastic limit or yield stress of column or stub-column material.

F_ya = the average elastic limit or yield stress of the sheet steel for a given test unit.

F_yi = the individual elastic limit or yield stress of the sheet-steel specimens in a test unit.

i = load-displacement-reading number for a particular stub-column test (load displacement D_i at load P_i).

j = total number of load-displacement readings taken for a particular stub-column test.

L = the length of the stub-column test specimen.

L_p = the pitch of a repeating pattern of perforations along the longitudinal column axis.

n = the ratio of the effective cross-sectional area at the ultimate load to the full cross-sectional area, A_e/A.

P = the applied axial compression force (column load).

P_i = the applied load at load-increment i.

P_n = the nominal failure load of a column.

P_u = the ultimate stub-column load at which local failure occurs.

P_ws = the average of all ultimate stub-column loads within a test unit.

r = the minimum radius of gyration of the cross-sectional area, A.

t = the nominal base-steel thickness exclusive of coating.

t_a = the average of all base-steel thicknesses within a test unit, exclusive of coating.

W = the greatest overall width of the cross section including corner(s).

4. Significance

4.1 This test method provides requirements for testing, and equations to determine, the effective area of a cold-formed column section at ultimate load, A_e, and the load- or stress-dependent effective area, A_e. These properties are used in the AISI Specification to determine the ultimate and less-than-ultimate column strengths. The ultimate column strength, P_u, is the product of the minimum specified yield stress, F_y, or the buckling stress F_n, and the corresponding effective cross-sectional area at that stress, A_e. At an applied column strength of P less than P_u, the corresponding effective cross-sectional area shall be A_e.
4.2 The test method also provides a means to observe, measure, and account for local buckling deformations when the appearance of a column section under stress must be determined.

4.3 An inherent assumption of the test method is that true stub-column behavior (which considers local buckling effects only) is achieved when overall column-buckling effects are eliminated. For this condition the ultimate test load on a stub column, \( P_u' \), equals the product of the effective cross-sectional area at ultimate load, \( A_{eu}' \), times the stress that causes local buckling, or times the yield stress of the virgin steel sheet. In case overall buckling cannot be avoided because of geometrical constraints, the critical column-buckling stress must be used.

4.4 The determination of \( A_e \) may be conducted by either one of the two following methods:

(1) The basic, more simple, and conservative method:
This method is embodied in the main part of this document and is based on the measured test loads of stub columns and their measured and tested physical and mechanical properties.

(2) An alternate and less conservative method:
This method is based on the shortening of stub columns which occurs during testing. Also, this evaluation method requires more calculations. The results of this method lead to more accurate results for \( A_{eu}' \), and to higher allowable axial loads at lower-than-ultimate stress levels. The evaluation procedure for this method is described in Appendix A.

5. Apparatus

5.1 The tests shall be conducted on a testing machine that complies with the requirements of ASTM E4.

5.2 Linear displacement devices for measuring lateral displacements shall have a 0.001-inch least-reading capability.

5.3 Measuring devices for determination of the actual geometry of a test specimen shall have a 0.001-inch least-reading capability.

5.4 If axial shortening is recorded, the measuring device shall have a 0.0001-inch least-reading capability.

6. Test Unit

6.1 A test unit shall include a minimum of three identical stub-column specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed steel section with the same specified geometrical, physical, and chemical properties. The specimens may be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If stub-column specimens are taken from different production runs, at least two corresponding sheet-type specimens must be taken and tested from each production run.

6.4 The stub-column test specimens shall be used to determine:

(1) The actual geometry of each specimen.

(2) The ultimate stub-column test load.
(3) Axial shortenings at each load level if the alternate test-evaluation method described in Appendix A is used.

(4) Lateral displacements of the specimen at locations of interest (if desired).

6.5 The tensile test specimens shall be used to define the yield stress of each stub-column specimen according to the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

6.7 If the average area, thickness, or yield strength of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit is considered to be non-representative of the column section, and further evaluations of the effective area are considered to be invalid.

7. Stub-Column Specimens

The stub-column specimens shall meet length and end-flatness requirements as follows, depending on whether or not unconnected or welded endplates are used.

7.1 Stub-Column Length—The length requirements of the stub-column test specimen, L, as shown in Figures I and 2, are that it be (1) sufficiently short to eliminate overall column buckling effects, and (2) sufficiently long to minimize the end effects during loading, which means that its center portion be representative of the repetitive hole pattern in the full column.

7.1.1 To eliminate overall column-buckling effects, the stub-column length shall not exceed twenty times the minimum radius of gyration, r, of the cross section, A, except where necessary to meet the requirements of Sections 7.1.2 through 7.1.5.

7.1.2 For unperforated columns (Figure 1a) the stub-column length shall not be less than three times the greatest overall width of the cross section, W.

7.1.3 For perforated columns in which the pitch (gage length) of the perforation pattern, L_p, for a single hole or a group of holes, is smaller than, or equal to, the greatest overall width, W, of the cross section (Figures 1b and 1g), or for a single hole pattern with a gage length larger than the greatest overall width (Figure 1c), the specimen length shall not be less than three times the greatest overall width of the cross section. W. For widely spaced hole patterns (Figure 1c) the significant hole or hole pattern shall be located at or near the midlength of the stub column.

7.1.4 For perforated columns in which the pitch of the perforation pattern, L_p, is greater than the widest side, W, of the cross section (Figures 1d, 1e, 1f, and 1h), the specimen length shall not be less than three times the pitch of the perforation pattern.

7.1.5 For perforated sections in which the specimen end planes must pass through the normal perforation pattern (Figure 1i), a special section (Figure 1j) may be fabricated to obtain full cross-sectional surfaces at the specimen ends.

7.2 Stub-Column End Surface Preparation—The end planes of the stub-column test specimens shall be carefully cut to a flatness tolerance of plus or minus 0.002 inches. When the required flatness can be achieved, welding of the stub-column ends to the endplates is not required. However, when this flatness cannot be achieved, steel endplates shall be continuously welded to both ends of the specimen so that there shall be no gap between the ends of the stub column and the endplates.

7.3 Stub-Column Specimen Source—Stub-column test specimens may be cut from the commercially fabricated column product. Alternatively, stub columns may be specially
Figure 1 Hypothetical Perforation Patterns And Suggested Stub Column Lengths

NOTES: (1) Perforations shown are in a flat portion of a member with width W
(2) $L = \text{Length of Stub Column}$
(3) $L_p = \text{Pitch Length of Perforation Pattern}$
fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product; however, subsequent proof tests using specimens from commercially produced columns are recommended.

7.4 Tensile Specimen Source—Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the stub-column specimens have been taken. If perforations are large and frequent in all flats of the formed section, the tensile specimens may be taken from the sheet or coil material used for the fabrication of the stub-column specimens. The tensile specimens shall not be taken from parts of a previously tested stub column.

7.5 Endplate Requirements—Steel endplates shall be at least 0.5 inch thick and have a flatness tolerance of plus or minus 0.002 inches.

8. Stub-Column Test Procedure

8.1 Vertical alignment of the stub column is essential to ensure that the applied load is uniformly distributed over the specimen end surfaces. Care should also be taken to center the specimen on the axis of the test machine.

8.1.1 Steel endplates shall be used to transfer the test loads uniformly into the stub columns (Figure 2).
8.1.2 A ½-inch-thick layer of grout, similar to gypsum-based concrete capping compound used for fast setting, shall be placed between the stub-column endplates and the machine heads to facilitate aligning the test specimen (Figure 2).

8.2 When an axial compression load is applied to the test specimen as a result of grout expansion during curing, or if a small preload is purposely applied to ensure proper contact between the stub-column endplates and the machine heads, the load shall be treated as part of the applied test load.

8.3 The load increments applied during the test shall not exceed 10 percent of the estimated ultimate test load.

8.4 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 kips per square inch of cross-sectional area per minute.

8.5 When axial shortening values are recorded, the following procedures shall be required:

(1) The change in the vertical distance between the inside surfaces of the endplates (Figure 2) shall be measured to the nearest 0.0001-inch at each load increment for each specimen.

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

9. Calculations

9.1 For a given test unit, all individual ultimate loads, \( P_u \), derived from the stub-column tests shall be used to calculate the average ultimate load, \( P_{uav} \). Similarly, all individual yield strengths, \( F_{y,i} \), derived from the tensile tests of the same unit shall be used to calculate the average yield stress of the same test unit, \( F_{yav} \).

9.2 The effective areas \( A_{eua} \), \( A_{eu} \), and \( A_e \) shall be calculated as specified in Sections 9.3 through 9.6; however, the final value of these effective areas shall not exceed that of the minimum gross cross-sectional area, \( A \).

9.3 For tests in which the length of the stub column does not exceed twenty times the minimum radius of gyration of the cross section, \( r \), the average effective area at the ultimate load, \( A_{eua} \), for a given test unit shall be calculated as

\[
A_{eua} = \frac{P_{uav}}{F_{yav}}
\]

9.4 For tests in which the length of the stub column exceeds twenty times the minimum radius of gyration of the cross section, the average effective area at the ultimate load shall be determined by iteration of the following equations:

\[
A_{eua} = A_a - \left( A_a - \frac{P_{uav}}{F_a} \right) / \left( \frac{F_a}{F_{yav}} \right)^n
\]

where \( A_a \) is the average minimum gross area of the stub columns in the test unit, and \( F_a \) is the flexural or torsional-flexural buckling stress derived from Section C4 of the AISI Specification with \( K = 0.5 \) (using the average cross-sectional properties of the test unit). The exponent \( n \) is determined as follows:

\[
n = \frac{A_{eua}}{A_a}
\]

Assuming an initial value for \( n \) equal to less than 1.0, \( A_{eua} \) can be calculated from the first equation. Using this \( A_{eua} \) in the second equation will provide a new value for \( n \). Repeating this process will lead to convergence of the above equations and an acceptable value of \( A_{eua} \) for one specific test unit.
9.5 The value of $A_{eu}$ for a specific test unit shall be adjusted to $A_{eu'}$, which is the effective cross-sectional area of a column at ultimate load with a nominal cross section of $A$ and a specified minimum yield strength of $F_y$. The adjustment shall be performed in one or two steps as follows.

9.5.1 If the average area of the stub columns in the test unit, $A_a$, or the average base steel thickness, $t_a$, are different from the nominal area or thickness, respectively, the effective cross-sectional area at ultimate load shall be calculated as follows:

\[ A_{eu} = A_{eu}(A/A_a) \]

or

\[ A_{eu} = A_{eu}(t/t_a) \]

9.5.2 If the average yield strength of all stub columns in a test unit, $F_{ya}$, is different from the nominal yield strength, $F_y$, the effective cross-sectional area at ultimate load shall be the lower of the two values calculated as follows:

\[ A_{eu} = A[1 - (1-A_{eu}/A)(F_y/F_{ya})] \]

or

\[ A_{eu} = A_{eu}(F_{ya}/F_y)^{0.4} \]

9.5.3. If the average area and the minimum specified yield strength are different from the nominal values of a test unit, $A_{eu}$ derived from the equation in Section 9.5.1 shall be used as $A_{eu}$ in the equations of Section 9.5.2, which will lead to an acceptable value of $A_{eu'}$.

9.6 The effective area at any working stress level, $A_e$, may be determined by

\[ A_e = A - (A - A_{eu})(f/F_y)^n \]

9.7 For a series of sections, such as in a parameter study during which only one parameter (thickness, depth, width, yield strength, etc.) is changed, interpolations between test units, or extrapolations beyond test units, shall be acceptable as described in Appendix B.

9.8 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.
10. Report

10.1 Documentation—The report shall include a complete record of the sources and locations of all stub-column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each stub-column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) longitudinal yield strength, (4) end preparation procedure, (5) applicable material specification, and (6) test and evaluation procedure used.

10.3 The determination of the selected stub-column length shall be fully documented with appropriate calculations.

10.4 A description of the test setup—including the endplates, the grout layer used for alignment, and the instrumentation used to measure lateral displacements and axial shortening—shall be included.

10.5 The report shall include the load increments, rate of loading, and intermediate and ultimate loads for each stub column tested.

10.6 The report shall include complete calculations and results of the effective area, $A_{eu}$, for each test unit and calculations of $A_e$, if requested.

11. Precision

11.1 The following criteria shall be used to judge the acceptability of the test results.

11.1.1 Repeatability—Individual stub-column test results shall be considered suspect if they differ by more than 10 percent from the mean value for a test unit with at least three specimens.

11.1.2 Reproducability—The results of tests on stub-columns conducted at two or more laboratories should agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield strength.

REFERENCES

(1) T. Pekoz, "Development of a Unified Approach to the Design of Cold-Formed Steel Members, Committee of Sheet Steel Producers, American Iron and Steel Institute, 1000 16th Street, NW, Washington, DC 20036, 1986."
APPENDIX A

Use Of Axial Shortening Measurements In Design

A-1 Axial shortening measurements as part of thin-walled cold-formed steel stub-column tests may be used as an alternative method of determining the effective area of a column, $A_e$, at a certain design load or stress. This method provides a more accurate and less conservative alternative to design engineers to determine the effective area of a column section, $A_e$.

A-2 The calculations by this method shall be made separately for each stub-column specimen within a test unit. This shall result in a total of $j$ calculations as a result of a total of $j$ load-displacement tests for each test unit.

A-3 For a given specimen the effective area at ultimate load, $A_{eu}$, shall be calculated from Section 9.3 or 9.4 letting $A_{eu} = A_{en}$, $A_e = A$, $F_y = F_y'$, and $P_{eu} = P_u$.

A-3.1 Calculations at each load-displacement reading, $i$, shall be conducted according to the following procedure; however, at zero load, the effective area, $A_e$, shall be equal to the minimum gross cross-sectional area, $A$. This provides results for the effective area at each load point:

1. Starting with the lowest load-displacement reading, the effective area, $A_i$, and the assumed uniformly distributed stress $f_i$, shall be calculated for each reading, $i$, from:

\[ A_i = \frac{P_i D_i}{F_y D_i} \]
\[ f_i = F_y D_i/D_0 \]

where $D_i$ an $D_0$ are the axial shortening at loads $P_i$ and $P_u$, respectively.

2. If $A_i$ calculated is greater than $A$, $A_i$ shall be set equal to $A$.

3. If $A_i$ calculated is less than $A$, $A_i$ shall be as calculated, and $f_i$, the stress above which the section is not fully effective, shall be set equal to $f_i$, as calculated for the previous load-displacement reading.

A.3.2 For specimens within a test unit, the lowest $A_i$ values shall be used for further evaluations.

A-4 For any load that causes a stress $f$ higher than $f_y$, an exponential equation may be developed as follows:

\[ A_i = A[1-(1-A_{eu}/A) (f-f_y)/(F_y-f_y)]^b \]

\[ \frac{\sum_{i=1}^j (X_i Y_i) - (a) \sum_{i=1}^j (X_i)}{\sum_{i=1}^j (X_i)^2} \]

where

\[ b = \frac{\sum_{i=1}^j (Y_i)}{\sum_{i=1}^j (X_i)^2} \]

and

\[ X = \ln[(f-f_y)/(F_y-f_y)] \]
\[ Y_i = \ln(1-A_{eu}/A) \]
\[ a = \ln(1-A_{eu}/A) \]

and ln designates the natural logarithm.

A-5 If the effective areas for a section with specified dimensions and minimum yield strength are desired, which are different from the tested specimens, the $A_{eu}$ and $A_{ei}$ values calculated under Section A-3 shall be normalized to the specified parameters according to Section 9.5 before the curve-fitting procedure of Section A-4 is employed.

A-6 All calculations pertaining to this procedure shall be included in the report, as discussed in Section 10.
APPENDIX B

Parametric Studies

B-1 For parametric studies intended to develop the effective area for a series of sections with the same basic cross section (either C, U, H, or any other shape) and the same hole pattern, but with one or more changing parameters, the required number of test units may be less than the sum of all sections with different geometries and yield strengths.

B-1.1 For a series of sections with three different values for one parameter only (dimension or nominal yield strength), at least two test units shall be chosen to include the minimum and the maximum value of the changing parameter. For the third value, $A_{eu}$ may be interpolated according to Section B-2.

B-1.2 If more than three different values for one parameter are included in a series of sections, additional units with intermediate values shall be tested such that the ratio of the changing values in adjacent units is not greater than 1.5 or be less than 0.67. For intermediate values of the changing parameter, $A_{eu}$ may be interpolated according to Section B-2.

B-1.3 For a series of sections with the same basic cross section that includes different values for several parameters (dimensions and/or yield strength), an appropriate factorial of test units shall be established by the responsible professional engineer in accordance with the guidelines for changes in an individual parameter, and in compliance with responsible code authorities. Interpolations and extrapolations may be made as mutually agreeable, following the general guidelines set forth in Section B-2 for changes of one parameter only.

B-1.4 For a section that falls outside a series of tested members with the same basic cross section, $A_{eu}$ may be extrapolated provided the changing parameter does not exceed a value of 20 percent below or above the respective minimum or maximum values tested in the series.

B-2 Interpolations and extrapolations are allowed as part of a parametric study, and as defined under B-1.

B-2.1 For a section with a thickness different from the thicknesses tested, but with identical overall nominal cross-sectional dimensions and minimum specified yield strength, $A_{eu}$ for a thickness $t$ and an area $A$ may be calculated provided $t$ does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, $A_{eu}$ may be determined by interpolation or extrapolation from the results of the nearest two test units with thicknesses $t_1$ and $t_2$, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2 - A_{eu1}/A_1)(t_1 - t)/(t_1 - t_2)]$$

where $A_1$ and $A_2$ are the minimum gross cross-sectional areas, and $A_{eu1}$ and $A_{eu2}$ are the nominal effective cross-sectional areas for Test Units 1 and 2, respectively.

B-2.2. For a section with a yield strength different from the yield strengths tested, but with identical cross-sectional dimensions, $A_{eu}$ for a yield strength $F_y$ may be calculated provided $F_y$ does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, $A_{eu}$ may be determined by interpolation or extrapolation from the results of the nearest two test units with yield strengths $F_{y1}$ and $F_{y2}$, and with effective areas $A_{eu1}$ and $A_{eu2}$, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2 - A_{eu1}/A_1)(F_{y1} - F_y)/(F_{y1} - F_{y2})]$$
1. Scope

This method covers the determination of uniform and local ductility from a tension test. Its primary use is as an alternative method of determining if a steel has adequate ductility as defined in the AISI Specification. It is based on the method suggested by Dhalla and Winter.

2. Referenced Documents


AISI Specification for the Design of Cold-Formed Steel Structural Members, 1986 Specification with the December 11, 1989 Addendum.


3. Symbols

\( e_3 \) = linear elongation, in., in 3-in. gage length

\( e_{3e} \) = linear elongation, in., in 2-in gage length not containing 1-in. length of fractured portion

\( e_u \) = linear elongation, in., at ultimate load in standard tension coupon test

\( \varepsilon_3 \) = percent elongation in 3-in. gage length

\( \varepsilon_{3e} \) = percent elongation in 2-in. gage length not containing 1-in. length of fractured portion

\( \varepsilon_f \) = percent elongation at fracture in 2-in. gage length of standard tension coupon

\( \varepsilon_u \) = percent elongation at ultimate load in standard tension coupon test

\( \varepsilon_{\text{uniform}} \) = uniform percent elongation

\( \varepsilon_{\text{local}} \) = local percent elongation in 1/2 in. gage length

\( \varepsilon_{1/2} \) = percent elongation in 1/2 in. gage length

4. Test Procedure

4.1 Prepare a tension coupon according to ASTM Standard A370 except that the central length of 1/2 in. (12.7 mm) uniform width of the coupon should be at least \( 3\frac{1}{2} \) in. (88.9 mm) long.

4.2 Scribe gage lines at 1/2-in. (12.7 mm) intervals along the entire length of the coupon.

4.3 After completion of the coupon test, measure the following two permanent plastic deformations: (a) the linear elongation in a 3-in. (76.2 mm) gage length, \( e_3 \), such that the fractured portion is included (preferably near the middle third of this 3-in. gage length); and (b) the linear elongation in a 1-in. (25.4 mm) gage length containing the fracture.

4.4 Subtract the latter from the former. This difference gives the linear elongation, \( e_{3e} \), in a 2-in. (50.8 mm) gage length not containing the 1-in. length of the fractured portion.

4.5 From the two preceding elongation measurements, \( e_3 \) and \( e_{3e} \), calculate the percentage elongations \( \varepsilon_3 = (e_3/3) \times 100 \), and \( \varepsilon_{3e} = (e_{3e}/2) \times 100 \). From these percentage elongations, the uniform and local ductility parameters are obtained as follows.
4.6 Since the fractured portion which includes local elongation is eliminated from $\epsilon_{3e}$, it is a measure of the uniform ductility of the material. Thus

$$\epsilon_{\text{uniform}} = \epsilon_{3e} \quad (1)$$

4.7 The local elongation is determined over a small length which includes the fractured portion. For simplicity, this length is here assumed to be 1/2 in. (12.7 mm) which is large enough to include the necked portion of most thicknesses and type of sheet steels used, and is small enough to give valid comparison for different types of steels. Thus

$$\epsilon_{\text{local}} = \epsilon_{1/2} = 6(\epsilon_3 - \epsilon_{3e}) + \epsilon_{3e} \quad (2)$$

in which $6$ = the multiplication factor which converts the local elongation $(\epsilon_3 - \epsilon_{3e})$ measured in 3 in. (76.2 mm) to local elongation in 1/2 in. (12.7 mm) gage length.

5. Alternate Test Procedure

5.1 Prepare a standard tension coupon according to ASTM A370 with a standard 2-in. (50.8 mm) gage length.

5.2 The strain at the tensile strength, i.e., percentage strain $\epsilon_u$ at the peak of the stress-strain curve, is a measure of uniform ductility, because up to this strain no necking or local elongation has taken place. Therefore, to obtain the uniform ductility the stress-strain curve is plotted at least up to the maximum load or the linear elongation, $\epsilon_u$, at maximum load is measured directly, so that $\epsilon_u = (\epsilon_u/2) \times 100$.

5.3 To obtain a measure of the local ductility it is necessary to measure the percentage strain at fracture $\epsilon_f$, also in a 2-in. gage length. However, the strain which occurs after the maximum load has been passed (descending branch) is the necking strain, and is localized at the eventual fracture zone, thus $(\epsilon_f - \epsilon_u)$ is the local percentage elongation referred to in a 2-in. (50.4 mm) gage length. The following equation converts this $(\epsilon_f - \epsilon_u)$ into the percentage elongation in a 1/2 in. (12.7 mm) gage length:

$$\epsilon_{\text{local}} = \epsilon_{1/2} = \epsilon_u + 4(\epsilon_f - \epsilon_u) \quad (3)$$

in which $4$ = the multiplication factor to convert a 2-in. gage length local elongation to a 1/2 in. gage length.