North American Specification for the Design of Cold-Formed Steel Structural Members

2001 EDITION

WITH INCORPORATION OF ERRATA NO. 1

Approved in Canada by the
Canadian Standards Association
CSA S136-01

Approved in Mexico by CANACERO
The material contained herein has been developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, the Canadian Standards Association Technical Committee on Cold-Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. The organizations and the Committees have made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committees acknowledge and are grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the Commentary on the Specification.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that future editions of this specification will update this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all resulting liability arising therefrom.
PREFACE

This is the first edition of the North American Specification for the Design of Cold-Formed Steel Structural Members; as its name implies, it is intended for use throughout Canada, Mexico, and the United States. This Specification supersedes the previous editions of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute and the previous editions of CSA Standard S136, Cold Formed Steel Structural Members, published by the Canadian Standards Association.

The Specification was developed by a joint effort of the American Iron and Steel Institute’s Committee on Specifications, the Canadian Standards Association’s Technical Committee on Cold Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. This effort was coordinated through the North American Specification Committee, which was made up of three members each from the AISI Committee on Specifications, CSA’s S136 Committee, and CANACERO.

Since the Specification is intended for use in Canada, Mexico, and the United States, it was necessary to develop a format that would allow for requirements particular to each country. This resulted in a main document, Chapters A through G, that is intended for use in all three countries, and three country-specific appendices (A to C). Appendix A is for use in the United States, Appendix B is for use in Canada, and Appendix C is for use in Mexico. A symbol \( A,B,C \) is used in the main document to point out that additional provisions are provided in the corresponding appendices indicated by the letters.

This Specification provides an integrated treatment of Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD). This is accomplished by including the appropriate resistance factors (\( \phi \)) for use with LRFD and LSD, and the appropriate factors of safety (\( \Omega \)) for use with ASD. It should be noted that LSD is limited to Canada and LRFD and ASD are limited to Mexico and the United States.

The Specification also contains some terminology that is defined differently in Canada, the United States, and Mexico. These differences are set out in Section A1.2, “Terms”.

The Specification provides well-defined procedures for the design of load-carrying cold-formed steel members in buildings, as well as other applications provided that proper allowances are made for dynamic effects. The provisions reflect the results of continuing research to develop new and improved information on the structural behavior of cold-formed steel members. The success of these efforts is evident in the wide acceptance of the predecessor documents to this Specification.

The AISI and CSA consensus committees responsible for developing these provisions provide a balanced forum, with representatives from steel producers, fabricators, users, educators, researchers, and building code regulators. They are composed of engineers with a wide range of experience and high professional standing from throughout Canada, Mexico, and the United States. AISI, CANACERO, and CSA acknowledge the continuing dedication by the members of the specifications committees and their subcommittees. The membership of these committees follows this Preface.
Because this is the first edition of the *North American Cold-Formed Steel Specification*, no attempt will be made here to list provisions that represent changes to the documents that it supersedes. Such changes are numerous and are distributed throughout.

Users of the *Specification* are encouraged to offer comments and suggestions for improvement.

American Iron and Steel Institute  
Canadian Standards Association  
Camara Nacional de la Industria del Hierro y del Acero  

December 2001
North American Specification Committee

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# North American Cold-Formed Steel Specification

## APPENDIX B: PROVISIONS APPLICABLE TO CANADA

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| b<sub>1</sub>, b<sub>2</sub> | Effective widths of transverse stiffeners                                 | C3.6.1  |

| C | For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange | A7.2 |
| C | Coefficient                                                               | C3.4.1  |
| C | Bearing factor                                                            | E3.3.1  |
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| C<sub>m</sub> | End moment coefficient in interaction formula                            | C5.2.1, C5.2.2 |
| C<sub>ms</sub> | Coefficient for lateral bracing of Z-section                             | D3.2.1  |
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<tr>
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<tr>
<td>$\phi_v$</td>
<td>Resistance factor for shear strength</td>
<td>C3.2.1, C3.3.2</td>
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<td>Resistance factor for web crippling strength</td>
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<td>f_2/f_1</td>
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<td>$\psi$</td>
<td>Load combination factor</td>
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# SYMBOLS AND DEFINITIONS

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<th>Section</th>
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<td>$\Omega_t$</td>
<td>Factor of safety for tension member</td>
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</tr>
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<td>$\Omega_v$</td>
<td>Factor of safety for shear strength</td>
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<tr>
<td>$\Omega_w$</td>
<td>Factor of safety for web crippling strength</td>
<td>C3.4.1, C3.5.1</td>
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NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

A1. Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

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 in. (25.4 mm) in thickness and used for load-carrying purposes in buildings. It shall be permitted to be used for structures other than buildings provided appropriate allowances are made for dynamic effects.

This Specification includes Symbols and Definitions, Chapters A through G, and Appendices A through C which shall apply as follows:

- Appendix A shall apply only in the United States,
- Appendix B shall apply only in Canada, and
- Appendix C shall apply only in Mexico

This Specification includes design provisions for Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD) and Limit States Design (LSD). These design methods shall apply as follows:

- The use of ASD and LRFD shall be limited to the United States and Mexico, and
- The use of LSD shall be limited to, and is mandatory in Canada

The nominal strength [nominal resistance]* and stiffness of cold-formed steel elements, members, assemblies, connections, and details shall be determined in accordance with the provisions in Chapters B through G and Appendices A through C of the Specification. Where the composition or configuration of such components is such that calculation of strength [resistance] and/or stiffness cannot be made in accordance with those provisions, structural performance shall be established from either of the following:

(a) Determine design strength [factored resistance] or stiffness by tests, undertaken and evaluated in accordance with Chapter F.
(b) Determine design strength [factored resistance] or stiffness by rational engineering analysis based on appropriate theory, related testing if data is available, and engineering judgment. Specifically, the design strength [factored resistance] shall be determined from the calculated nominal strength [resistance] by applying the following factors of safety or resistance factors:

<table>
<thead>
<tr>
<th>Members</th>
<th>Connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>USA and Mexico</td>
<td>USA and Mexico</td>
</tr>
<tr>
<td>Canada</td>
<td>Canada</td>
</tr>
<tr>
<td>Omega (ASD)</td>
<td>Omega (ASD)</td>
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<tr>
<td>2.00</td>
<td>2.50</td>
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<tr>
<td>0.80</td>
<td>0.65</td>
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<tr>
<td>0.75</td>
<td>0.60</td>
</tr>
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</table>
Note:
* Bracketed terms are equivalent terms that apply particularly to LSD.
** Symbol $\text{AC}$ is used to point out that additional provisions are provided in the appendices as indicated by the letters.

### A1.2 Terms

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

**General Terms**

*Cold-Formed Steel Structural Members.* Shapes 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.

*Confirmatory Test.* Test made, when desired, on members, connections, and assemblies designed according to the provisions of Chapters A through G of this Specification or its specific references, in order to compare actual versus calculated performance.

*Cross-Sectional Area:*

- **Effective Area.** Effective area, $A_e$, calculated using the effective widths of component elements in accordance with Chapter B. It can be a gross area or a net area, as applicable, if the effective widths of all component elements, determined in accordance with Chapter B, are equal to the actual flat widths.
- **Full, Unreduced Area.** Full, unreduced area, $A$, calculated without reducing the widths of component elements to their effective widths. It can be an unreduced gross area or an unreduced net area, as applicable.
- **Gross Area.** Gross area, $A_g$, without deductions for holes, openings, and cutouts.
- **Net Area.** Net area, $A_n$, equal to gross area less the area of holes, openings, and cutouts.

*Distortional Buckling.* A mode of buckling involving change in cross-sectional shape, excluding local buckling.

*Doubly Symmetric Section.* A section symmetric about two orthogonal axes through its centroid.

*Effective Design Width.* Flat width of an element reduced for design purposes, also known simply as the effective width.

*Flange of a Section in Bending.* Flat width of flange including any intermediate stiffeners plus adjoining corners.

*Flat Width.* Width of an element exclusive of corners measured along its plane.

*Flat-Width-to-Thickness Ratio (Flat Width Ratio).* Flat width of an element measured along its plane, divided by its thickness.
Girt. Horizontal structural member which supports wall panel and is subjected to principally bending under applied loads.

Local Buckling. Buckling of elements only within a section, where the line junctions between elements remain straight and angles between elements do not change.

Master Coil. One continuous, weld-free coil as produced by a hot mill, cold mill, metallic coating line or paint line and identifiable by unique coil number. This coil may be cut into smaller coils or slit into narrower coils; however, all of these smaller and/or narrower finished coils could be said to have come from the same master coil if they are traceable to the original master coil number.

Multiple-Stiffened Element. Element stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress.

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

Point-Symmetric Section. Section symmetrical about a point (centroid) such as a Z-section having equal flanges.

Purlin. Horizontal structural member which supports roof deck and is subjected to principally bending under applied loads.

Rational Engineering Analysis. Analysis based on theory that is appropriate for the situation, any available test data that is relevant, and sound engineering judgment.

Singly-Symmetric Section. Section symmetric about only one axis through its centroid.

Specified Minimum Yield Point. Lower limit of yield point in a test specified to qualify a lot of steel for use in a cold-formed steel structural member designed at that yield point.

Stiffened or Partially Stiffened Compression Elements. 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.

SS. ASTM designation for certain sheet steels intended for structural applications.

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

Sub-Element of a Multiple Stiffened Element. Portion of a multiple stiffened element between adjacent intermediate stiffeners, between web and intermediate stiffener, or between edge and intermediate stiffener.

Tensile Strength. Maximum stress reached in a tension test.

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

Torsional-Flexural Buckling. Buckling mode in which compression members bend and twist simultaneously without change in cross sectional shape.

Unstiffened Compression Elements. Flat compression element stiffened at only one edge parallel to the direction of stress.
Unsymmetric Section. Section not symmetric either about an axis or a point.

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

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

Web. In a member subjected to flexure, the portion of the section that is joined to two flanges, or that is joined to only one flange provided it crosses the neutral axis.

Yield Point. Yield point, $F_y$ or $F_{sy}$, as used in this Specification shall mean yield point or yield strength.

**ASD and LRFD Terms (USA and Mexico):**

ASD (Allowable Stress Design, herein referred as Allowable Strength Design). A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that the allowable stress, allowable force or allowable moment is not exceeded by the required allowable strength of the component determined by the load effects of all appropriate combinations of nominal loads.

Allowable Design Strength. Allowable strength, $R_n/\Omega$, (force, moment, as appropriate), provided by the structural component.

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

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 combinations of factored loads.

Nominal loads. The magnitudes of the loads specified by the applicable code not including load factors.

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined in accordance with this Specification using specified material strengths and dimensions.

Required Allowable Strength. Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the nominal loads for ASD (using all appropriate load combinations).

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

Resistance. See the definition of Nominal Strength.

Resistance Factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

**LSD Terms (Canada):**

Limit States. Those conditions in which a structural member ceases to fulfill the function for which it was designed. Those states concerning safety are called the ultimate limit states. The ultimate limit state for strength is
the maximum load-carrying capacity. Limit states that restrict the intended use of a member for reasons other than safety, such as deflection and vibration, are called serviceability limit states.

**Limit States Design (LSD).** 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.

**Factored Load.** Product of a specified load and appropriate load factor.

**Factored Resistance.** Product of nominal resistance and appropriate resistance factor.

**Nominal Resistance.** The capacity of a structure or component to resist the effects of loads, determined in accordance with this *Specification* using specified material strengths and dimensions.

**Resistance Factor.** A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

**Specified loads.** The magnitudes of the loads specified by the applicable code not including load factors.

### 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. The unit systems considered in those sections are U.S. customary units (force in kilopounds and length in inches), SI units (force in Newtons and length in millimeters) and MKS units (force in kilograms and length in centimeters).

### A2 Material

#### A2.1 Applicable Steels

This *Specification* requires the use of steels intended for structural applications as defined in general by the specifications of the American Society for Testing and Materials listed below. Such steels are identified in many ASTM specifications for sheet material as SS.

ASTM A36/A36M, Carbon Structural Steel

ASTM A242/A242M, High-Strength Low-Alloy Structural Steel

ASTM A283/A283M, Low and Intermediate Tensile Strength Carbon Steel Plates

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

ASTM A529/A529M, High-Strength Carbon-Manganese Steel of Structural Quality

ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Structural Steel

ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. (100 mm) Thick
ASTM A606, Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 40 (275), 50 (340), 60 (410), 70 (480) and 80 (550)), Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), and 50 Class 1 (340 Class 1)), Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process
ASTM A847, Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance
ASTM A875/A875M (SS Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 3; HSLAS Types A and B, Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process
ASTM A1003/A1003M, Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members
ASTM A1008/A1008M (SS Grades 25 (170), 30 (205), 33 (230) Types 1 and 2, and 40 (275) Types 1 and 2; HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), and 70 (480); HSLAS-F Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
ASTM A1011/A1011M (SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, 40 (275), 45 (310), 50 (340), and 55 (380); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), 65 (450), and 70 (480); HSLAS-F Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability

A2.2 Other Steels

The provisions of this section are given in Section A2.2 of the Appendices.

A2.3 Ductility

Steels not listed in Section A2.1 and used for structural members and connections in accordance with Section A2.2 shall comply with one of the following ductility requirements:

A2.3.1 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 (50 mm) gage length or 7 percent for an eight-inch (200 mm) 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 in. (12.7 mm) gage length across the fracture shall not be less than 20 percent, (2) uniform elongation outside the fracture shall not be less than 3 percent. 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 accordance with Sections C3.1.1(a), C3.1.2, C3.1.3, and C3.1.4. For purlins and girts subject to combined axial load and bending moment (Section C5), $\frac{\Omega CP}{P_n}$ shall not exceed 0.15 for ASD, $\frac{P_u}{\phi CP_n}$ shall not exceed 0.15 for LRFD and $\frac{P_f}{\phi CP_n}$ shall not exceed 0.15 for LSD.

**A2.3.2** Steels conforming to ASTM A653/A653M SS Grade 80 (550), A1008/A1008M SS Grade 80 (550), A792/A792M Grade 80 (550), A875/A875M SS Grade 80 (550) and other steels which do not meet the provisions of Section A2.3.1 shall be permitted for multiple-web configurations such as roofing, siding and floor decking provided that:

1. the yield point, $F_{y'}$, used for determining nominal strength [resistance] in Chapters B, C, and D is taken as 75 percent of the specified minimum yield point or 60 ksi (410 MPa or 4220 kg/cm²), whichever is less, and
2. the tensile strength, $F_{u'}$, used for determining nominal strength [resistance] in Chapter E is taken as 75 percent of the specified minimum tensile strength or 62 ksi (427 MPa or 4360 kg/cm²), whichever is less.

Alternatively, the suitability of such steels for any multi-web configuration shall be demonstrated by load tests according to the provisions of Section F1. Design strengths [factored resistances] based on these tests shall not exceed the design strengths [factored resistances] calculated according to Chapters B through G, using the specified minimum yield point, $F_{y'}$, and the specified minimum tensile strength, $F_{u'}$.

Exception: For multiple-web configurations, a reduced yield point, $R_bF_{y'}$, shall be permitted for determining the nominal flexural strength [moment resistance] in Section C3.1.1(a), for which the reduction factor, $R_b$, shall be determined as follows:

(a) Stiffened and Partially Stiffened Compression Flanges

For $w/t \leq 0.067E/F_Y$

$R_b = 1.0$

For $0.067E/F_Y < w/t < 0.974E/F_Y$

$R_b = 1-0.26[wF_Y/(tE) – 0.067]^{0.4}$  \hspace{1cm} (Eq. A2.3.2-1)

For $0.974E/F_Y \leq w/t \leq 500$

$R_b = 0.75$

(b) Unstiffened Compression Flanges

For $w/t \leq 0.0173E/F_Y$

$R_b = 1.0$

For $0.0173E/F_Y < w/t \leq 60$

A2.3.2 Steels conforming to ASTM A653/A653M SS Grade 80 (550), A1008/A1008M SS Grade 80 (550), A792/A792M Grade 80 (550), A875/A875M SS Grade 80 (550) and other steels which do not meet the provisions of Section A2.3.1 shall be permitted for multiple-web configurations such as roofing, siding and floor decking provided that:

1. the yield point, $F_{y'}$, used for determining nominal strength [resistance] in Chapters B, C, and D is taken as 75 percent of the specified minimum yield point or 60 ksi (410 MPa or 4220 kg/cm²), whichever is less, and
2. the tensile strength, $F_{u'}$, used for determining nominal strength [resistance] in Chapter E is taken as 75 percent of the specified minimum tensile strength or 62 ksi (427 MPa or 4360 kg/cm²), whichever is less.

Alternatively, the suitability of such steels for any multi-web configuration shall be demonstrated by load tests according to the provisions of Section F1. Design strengths [factored resistances] based on these tests shall not exceed the design strengths [factored resistances] calculated according to Chapters B through G, using the specified minimum yield point, $F_{y'}$, and the specified minimum tensile strength, $F_{u'}$.

Exception: For multiple-web configurations, a reduced yield point, $R_bF_{y'}$, shall be permitted for determining the nominal flexural strength [moment resistance] in Section C3.1.1(a), for which the reduction factor, $R_b$, shall be determined as follows:

(a) Stiffened and Partially Stiffened Compression Flanges

For $w/t \leq 0.067E/F_Y$

$R_b = 1.0$

For $0.067E/F_Y < w/t < 0.974E/F_Y$

$R_b = 1-0.26[wF_Y/(tE) – 0.067]^{0.4}$  \hspace{1cm} (Eq. A2.3.2-1)

For $0.974E/F_Y \leq w/t \leq 500$

$R_b = 0.75$

(b) Unstiffened Compression Flanges

For $w/t \leq 0.0173E/F_Y$

$R_b = 1.0$

For $0.0173E/F_Y < w/t \leq 60$
\[ R_b = 1.079 - 0.6 \sqrt{w \cdot F_y / (t \cdot E)} \]  
\( (Eq. A2.3.2-2) \)

where
- \( E \) = Modulus of elasticity
- \( F_y \) = Yield point as specified in Section A7.1 ≤ 80 ksi (550 MPa, or 5620 kg/cm²)
- \( t \) = Thickness of section
- \( w \) = Flat width of compression flange

The above Exception does not apply to the use of steel deck for composite slabs, for which the steel deck acts as the tensile reinforcement of the slab.

**A2.4 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.

**A3 Loads**

Loads and load combinations shall be as stipulated by the applicable country specific provisions, Section A3 of Appendix A, B, or C.

**A4 Allowable Strength Design**

**A4.1 Design Basis**

Design under this Section of the Specification shall be based on Allowable Strength Design (ASD) principles. All provisions of this Specification, except for those in Sections A5 and A6 and in Chapters C and F designated for LRFD and LSD, shall apply.

**A4.1.1 ASD Requirements**

A design satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required allowable strength, determined on the basis of the nominal loads, for all applicable load combinations.

The design shall be performed in accordance with Equation (A4.1.1-1):

\[ R \leq \frac{R_n}{\Omega} \]  
\( (Eq. A4.1.1-1) \)

where
- \( R \) = Required allowable strength
- \( R_n \) = Nominal strength specified in Chapters B through G
- \( \Omega \) = Factor of safety specified in Chapters B through G
- \( R_n/\Omega \) = Allowable design strength
A4.1.2 Load Combinations for ASD

Load combinations for ASD shall be as stipulated by Section A4.1.2 of Appendix A or C.

A5 Load and Resistance Factor Design

A5.1 Design Basis

Design under this Section of the Specification shall be based on Load and Resistance Factor Design (LRFD) principles. All provisions of this Specification, except for those in Sections A4 and A5 and in Chapters C and F designated for ASD and LSD, shall apply.

A5.1.1 LRFD Requirements

A design satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the nominal loads, multiplied by the appropriate load factors, for all applicable load combinations.

The design shall be performed in accordance with Equation (A5.1.1-1):

\[ R_u \leq \phi R_n \]  

(Eq. A5.1.1-1)

where

- \( R_u \) = Required strength
- \( R_n \) = Nominal strength specified in Chapters B through G
- \( \phi \) = Resistance factor specified in Chapters B through G
- \( \phi R_n \) = Design strength

A5.1.2 Load Factors and Load Combinations for LRFD

Load factors and load combinations for LRFD shall be as stipulated by Section A5.1.2 of Appendix A or C.

A6 Limit States Design

A6.1 Design Basis

Design under this Section of the Specification shall be based on Limit States Design (LSD) principles. All provisions of this Specification, except for those in Sections A4 and A5 and Chapters C and F designated for ASD and LRFD, shall apply.

A6.1.1 LSD Requirements

Structural members and their connections shall be designed to have resistance such that the factored resistance equals or exceeds the effect of factored loads. The design shall be performed in accordance with Equation (A6.1.1-1):

\[ \phi R_n \geq R_f \]  

(Eq. A6.1.1-1)
where
\[ R_f = \text{Effect of factored loads} \]
\[ R_n = \text{Nominal resistance specified in Chapters B through G} \]
\[ \phi = \text{Resistance factor specified in Chapters B through G} \]
\[ \phi R_n = \text{Factored resistance} \]

**A6.1.2 Load Factors and Load Combinations for LSD**

Load factors and load combinations for LSD shall be as stipulated by Section A6.1.2 of Appendix B.

**A7  Yield Point and Strength Increase from Cold Work of Forming**

**A7.1 Yield Point**

The yield point used in design, \( F_y \), shall not exceed the specified minimum yield point of steels as listed in Section A2.1 or A2.3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A7.2.

**A7.2 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 C2, 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 \( \rho \) for strength determination is unity as calculated according to Section B2 for each of the component elements of the section, the design yield point, \( F_ya \), 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:

   \[ F_ya = CF_{yc} + (1 - C) F_yf \]  

   \( \text{Eq. A7.2-1} \)

Where

- \( F_{yc} = \text{Average yield point of full unreduced section of compression members or full flange sections of flexural members} \)
- \( C = \text{For compression members, ratio of total corner cross-sectional area to total cross-sectional area of full section; for flexural members, ratio of total corner cross-sectional area of controlling flange to full cross-sectional area of controlling flange} \)
- \( F_yf = \text{Weighted average tensile yield point of flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made} \)
\[ F_{yc} = \frac{B_c F_{yv}}{(R/t)^m}, \text{ tensile yield point of corners. This equation is applicable only when } F_{uv}/F_{yv} \geq 1.2, R/t \leq 7, \text{ and the included angle } \leq 120^\circ \]  
(Eq. A7.2-2)

\[ B_c = 3.69 \left( \frac{F_{uv}}{F_{yv}} \right) - 0.819 \left( \frac{F_{uv}}{F_{yv}} \right)^2 - 1.79 \]  
(Eq. A7.2-3)

\[ m = 0.192 \left( \frac{F_{uv}}{F_{yv}} \right) - 0.068 \]  
(Eq. A7.2-4)

\[ R = \text{Inside bend radius} \]

\[ F_{yv} = \text{Tensile yield point of virgin steel specified by Section A2 or established in accordance with Section F3.3} \]

\[ F_{uv} = \text{Tensile strength of virgin steel specified by Section A2 or established in accordance with Section F3.3} \]

(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.

(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.

### A8 Serviceability

A structure shall be designed to perform its required functions during its expected life. Serviceability limits shall be chosen based on the intended function of the structure, and shall be evaluated using realistic loads and load combinations.

### A9 Referenced Documents

The following documents are referenced in this Specification. Refer to Section A9a of Appendix A, B, or C for documents applicable to the corresponding country.


2. American Society for Testing and Materials (ASTM), 100 Barr Harbor Drive, West Conshohocken, Pennsylvania 19428-2959:
   - ASTM A36/A36M-00a, Carbon Structural Steel
   - ASTM A194/A194M-00b, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
   - ASTM A242/A242M-00a, High-Strength Low-Alloy Structural Steel
   - ASTM A283/A283M-00, Low and Intermediate Tensile Strength Carbon Steel Plates
   - ASTM A307-00, Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength
   - ASTM A325-00, Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
   - ASTM A325M-00, High Strength Bolts for Structural Steel Joints [Metric]
ASTM A354-00a, Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
ASTM A370-97a, Standard Test Methods and Definitions for Mechanical Testing of Steel Products
ASTM A449-00, Quenched and Tempered Steel Bolts and Studs
ASTM A490-00, Heat-Treated Steel Structural Bolts, 150ksi Minimum Tensile Strength
ASTM A490M-00, High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]
ASTM A500-99, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A529/A529M-00, High-Strength Carbon-Manganese Steel of Structural Quality
ASTM A563-00, Carbon and Alloy Steel Nuts
ASTM A563M-00, Carbon and Alloy Steel Nuts [Metric]
ASTM A572/A572M-00a, High-Strength Low-Alloy Columbium-Vanadium Structural Steel
ASTM A588/A588M-00a, High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick
ASTM A606-98, Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
ASTM A653/A653M-00, Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
ASTM A792/A792M-99, Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process
ASTM A847-99a, Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance
ASTM A875/A875M-99, Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process
ASTM A1003/A1003M-00, Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members
ASTM A1008/A1008M-00, Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
ASTM A1011/A1011M-00, Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
ASTM F436-00, Hardened Steel Washers
ASTM F436M-00, Hardened Steel Washers [Metric]
ASTM F844-00, Washers, Steel, Plain (Flat), Unhardened for General Use
ASTM F959-99a, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners
ASTM F959M-99a, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]
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
- Any other kind of stiffener
  - i) when \( I_s < I_a \)
  - ii) when \( I_s \geq I_a \)

\[ w/t = \left( \frac{60}{E} \right) \left( \frac{t}{d} \right) \]

where
- \( I_s = \) Actual moment of inertia of full stiffener about its own centroidal axis parallel to element to be stiffened
- \( I_a = \) Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.

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

- 500

(3) Unstiffened compression element

- 60

It shall be noted that 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 [factored resistance], without affecting the ability of the member to develop the required strength [factored strength].

Stiffened elements having w/t ratios larger than 500 can be used with adequate design strength [factored resistance] to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design equations 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 equation applies to compression and tension flanges, either stiffened or unstiffened:

\[ w_f = \sqrt{\frac{0.061tdE}{f_{av}}} \left( \frac{4}{100c_f / d} \right) \]  

(Eq. B1.1-1)
where

\( w_f \) = Width of flange projecting beyond web;
or half of distance between webs for box- or U-type beams
\( t \) = Flange thickness
\( d \) = Depth of beam
\( c_f \) = Amount of curling displacement
\( f_{av} \) = Average stress in 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 beam has a span of less than 30\( w_f \) (\( w_f \) as defined below) and it carries one concentrated load, or several loads spaced farther apart than 2\( w_f \), the effective design width of any flange, whether in tension or compression, shall be limited by the values in Table B1.1(c).

<table>
<thead>
<tr>
<th>L/wf</th>
<th>Ratio b/w</th>
<th>L/wf</th>
<th>Ratio b/w</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 distance between inflection points for continuous beams; or twice the length for cantilever beams.

\( w_f \) = Width of flange projection beyond web for I-beam and similar sections; or half distance between webs for 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 Ratios

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 C3.6.1:
   (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 plane of web
\(t\) = 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) Strength Determination

The effective width, \(b\), shall be determined from the following equations:

\[
\begin{align*}
\text{\(b = w\) when } & \lambda \leq 0.673 & \text{(Eq. B2.1-1)} \\
\text{\(b = \rho w\) when } & \lambda > 0.673 & \text{(Eq. B2.1-2)}
\end{align*}
\]

where
\(w\) = Flat width as shown in Figure B2.1-1
\(\rho = \frac{(1 - 0.22/\lambda)}{\lambda}\) \(\text{(Eq. B2.1-3)}\)
\(\lambda\) is a slenderness factor determined as follows:

\[
\lambda = \sqrt{\frac{f}{F_{\text{cr}}}} \quad \text{(Eq. B2.1-4)}
\]

\[
F_{\text{cr}} = k \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{w} \right)^2 
\quad \text{(Eq. B2.1-5)}
\]

where
\(t\) = Thickness of uniformly compressed stiffened element
\(\mu\) = Poisson’s ratio of steel, and
\(f\) = Stress in compression element computed as follows:
For flexural members:
   (1) If Procedure I of Section C3.1.1 is used:
       When the initial yielding is in compression in the element
       considered, \(f = F_y\).
       When 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, \(f\) is the stress in the
       element considered at \(M_n\) determined on the basis of the
       effective section.
   (3) If Section C3.1.2.1 is used, \(f\) is the stress \(F_c\) as described in that
       Section in determining \(S_c\).
For compression members, \( f \) is taken equal to \( F_n \) as determined in Section C4 or D4.1 as applicable.

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

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

\[ k = 4 \text{ for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.} \]

\[ b_d = \begin{cases} 
  w & \text{when } \lambda \leq 0.673 \\
  \rho w & \text{when } \lambda > 0.673 
\end{cases} \quad (Eq. B2.1-6) \]

where

\[ w = \text{Flat width} \]

\[ \rho = \text{Reduction factor determined by either of the following two procedures:} \]

1. **Procedure I.**
   
   A low estimate of the effective width can be obtained from Eqs. B2.1-3 and B2.1-4 except that \( f_d \) 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:

\[ \rho = \begin{cases} 
  1 & \text{when } \lambda \leq 0.673 \\
  (1.358 - 0.461/\lambda)/\lambda & \text{when } 0.673 < \lambda < \lambda_c \\
  (0.41 + 0.59 \sqrt{F_y / f_d} - 0.22/\lambda)/\lambda & \text{when } \lambda \geq \lambda_c 
\end{cases} \quad (Eq. B2.1-8) \]

\[ \rho \text{ shall not exceed 1.0 for all cases.} \]

where

\[ \lambda_c = 0.256 + 0.328 (w/t) \sqrt{F_y / E} \quad (Eq. B2.1-11) \]

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) **Strength Determination**

The effective width, $b$, shall be determined as follows:

For $0.50 \geq \frac{d_h}{w} \geq 0$, and $\frac{w}{t} \leq 70$ and

the distance between centers of holes $\geq 0.50w$ and $\geq 3d_h$,

\[
b = w - d_h \quad \text{when} \quad \lambda \leq 0.673 \quad \text{(Eq. B2.2-1)}
\]

\[
b = \frac{w}{\lambda} \left[ 1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w} \right] \quad \text{when} \quad \lambda > 0.673 \quad \text{(Eq. B2.2-2)}
\]

$b$ 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) **Serviceability Determination**

The effective width, $b_d$, used in determining serviceability shall be equal to $b$ calculated in accordance with Procedure I of Section B2.1(b), except that $f_d$ is substituted for $f$, where $f_d$ is the computed compressive stress in the element being considered.

**B2.3 Webs and other Stiffened Elements under Stress Gradient**

The following notation is used in this section:

- $b_1$  = Effective width, dimension defined in Figure B2.3-1
- $b_2$  = Effective width, dimension defined in Figure B2.3-1
- $b_e$  = Effective width $b$ determined in accordance with Section B2.1 with $f_1$ substituted for $f$ and with $k$ determined as given in this section
- $b_o$  = Out-to-out width of the compression flange as defined in Figure B2.3-2
- $f_1, f_2$ = Stresses shown in Figure B2.3-1 calculated on the basis of effective section. Where $f_1$ and $f_2$ are both compression, $f_1 \geq f_2$
- $h_o$  = Out-to-out depth of web as defined in Figure B2.3-2
- $k$  = Plate buckling coefficient
- $\psi = |f_2/f_1| \ (\text{absolute value}) \quad \text{(Eq. B2.3-1)}$

(a) **Strength Determination**

(i) For webs under stress gradient ($f_1$ in compression and $f_2$ in tension as shown in Figure B2.3-1)

\[
k = 4 + 2(1 + \psi)^3 + 2(1 + \psi) \quad \text{(Eq. B2.3-2)}
\]
For $h_o/b_o \leq 4$

\[ b_1 = \frac{b_e}{3 + \psi} \quad (Eq. B2.3-3) \]

\[ b_2 = \frac{b_e}{2} \quad \text{when } \psi > 0.236 \quad (Eq. B2.3-4) \]

\[ b_2 = b_e - b_1 \quad \text{when } \psi \leq 0.236 \quad (Eq. B2.3-5) \]

In addition, $b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.

For $h_o/b_o > 4$

\[ b_1 = \frac{b_e}{3 + \psi} \quad (Eq. B2.3-6) \]

\[ b_2 = \frac{b_e}{1 + \psi} - b_1 \quad (Eq. B2.3-7) \]

(ii) For other stiffened elements under stress gradient ($f_1$ and $f_2$ in compression as shown in Figure B2.3-1)

(a) Webs under Stress Gradient

(b) Other Stiffened Elements under Stress Gradient

**Figure B2.3-1 Webs and other Stiffened Elements under Stress Gradient**
\[
k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad (Eq. \ B2.3-8)
\]
\[
b_1 = b_e/(3 - \psi) \quad (Eq. \ B2.3-9)
\]
\[
b_2 = b_e - b_1 \quad (Eq. \ B2.3-10)
\]

(b) Serviceability Determination

The effective widths used in determining serviceability shall be calculated in accordance with Section B2.3(a) except that \(f_{d1}\) and \(f_{d2}\) are substituted for \(f_1\) and \(f_2\), where \(f_{d1}\) and \(f_{d2}\) are the computed stresses \(f_1\) and \(f_2\) based on the effective section at the load for which serviceability is determined.

![Figure B2.3-2 Out-to-Out Dimensions of Webs and Stiffened Elements under Stress Gradient](image)

B2.4 C-Section Webs with Holes under Stress Gradient

These provisions shall be applicable within the following limits:

1. \(d_0/h \leq 0.7\)
2. \(h/t \leq 200\)
3. Holes centered at mid-depth of web
4. Clear distance between holes \(\geq 18\) in. (457 mm)
5. Non-circular holes, corner radii \(\geq 2t\)
6. Non-circular holes, \(d_0 \leq 2.5\) in. (64 mm) and \(b \leq 4.5\) in. (114 mm)
7. Circular holes, diameter \(\leq 6\) in. (152 mm)
8. \(d_0 > 9/16\) in. (14 mm)

(a) Strength Determination

When \(d_0/h < 0.38\), the effective widths, \(b_1\) and \(b_2\), shall be determined by Section B2.3(a) by assuming no hole exists in the web.

When \(d_0/h \geq 0.38\), the effective width shall be determined by Section B3.1(a) assuming the compression portion of the web consists of an unstiffened element adjacent to the hole with \(f = f_1\) as shown in Figure B2.3-1.

(b) Serviceability Determination

The effective widths shall be determined by Section B2.3(b) by
assuming no hole exists in the web.

where

\[ \begin{align*}
    d_0 & = \text{Depth of web hole} \\
    b & = \text{Length of web hole} \\
    b_1, b_2 & = \text{Effective widths defined by Figure B2.3-1} \\
    h & = \text{Depth of flat portion of web measured along plane of web} \\
    \text{Other variables are defined in B2.3.}
\end{align*} \]

### B3 Effective Widths of Unstiffened Elements

#### B3.1 Uniformly Compressed Unstiffened Elements

(a) **Strength Determination**

The effective width, \( b \), shall be determined in accordance with Section B2.1(a), except that \( k \) shall be taken as 0.43 and \( w \) as defined in Figure B3.1-1.

(b) **Serviceability Determination**

The effective width, \( b_d \), used in determining serviceability shall be calculated in accordance with Procedure I of Section B2.1(b), except that \( f_d \) is substituted for \( f \) and \( k = 0.43 \).

![Figure B3.1-1 Unstiffened Element with Uniform Compression](image)

#### B3.2 Unstiffened Elements and Edge Stiffeners under Stress Gradient

(a) **Strength Determination**

The effective width, \( b \), shall be determined in accordance with Section B2.1(a) with \( f = f_3 \) as in Figure B4-2 in the element and \( k = 0.43 \).

(b) **Serviceability Determination**

The effective width, \( b_{d_3} \), used in determining serviceability shall be calculated in accordance with Procedure I of Section B2.1(b), except that \( f_{d_3} \) is substituted for \( f \) and \( k = 0.43 \), where \( f_{d_3} \) = computed stress \( f_3 \) as shown in Figure B4-2. Calculations are based on the effective section at the load for which the serviceability is determined.
**B4 Effective Widths of Elements with One Intermediate Stiffener or an Edge Stiffener**

The following notation is used in this section.

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

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

\[ b_o = \text{Dimension defined in Figure B4-1} \]

\[ d, w, D = \text{Dimensions defined in Figure B4-2} \]

\[ d_s = \text{Reduced effective width of stiffener as specified in this section.} \]

\[ d_s' = \text{Effective width of stiffener calculated according to Section B3.2} \]

\[ A_s = \text{Reduced area of stiffener as specified in this section.} \]

\[ I_a = \text{Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.} \]

\[ I_s, A'_s = \text{Moment of inertia of full section of stiffener about its own centroidal axis parallel to element to be stiffened, and effective area of stiffener, respectively. For edge stiffeners, the round corner between stiffener and element to be stiffened shall not be considered as a part of the stiffener.} \]

For the stiffener shown in Figure B4-2:

\[ I_s = (d^3 t \sin^2 \theta)/12 \quad (Eq. \ B4-2) \]

\[ A'_s = d'_s t \quad (Eq. \ B4-3) \]

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

(a) **Strength Determination**

For \( b_o/t \leq S \)

\[ I_a = 0 \quad \text{(no intermediate stiffener required)} \]

\[ b = w \quad (Eq. \ B4.1-1) \]

\[ A_s = A'_s \quad (Eq. \ B4.1-2) \]

For \( b_o/t > S \)

\[ A_s = A'_s(R_l) \quad (Eq. \ B4.1-3) \]

\[ n = \left[ 0.583 - \frac{b_o}{t} \frac{125}{I_a} \right] \geq \frac{1}{3} \quad (Eq. \ B4.1-4) \]

\[ k = 3(R_l)^n + 1 \quad (Eq. \ B4.1-5) \]

\[ R_l = I_s/I_a \leq 1 \quad (Eq. \ B4.1-6) \]
where

i) For $S < b_o / t < 3S$
\[ I_a = t^4 \left[ 50 \frac{b_o}{t} / S - 50 \right] \quad (Eq. B4.1-7) \]

ii) For $b_o / t \geq 3S$
\[ I_a = t^4 \left[ 128 \frac{b_o}{t} / S - 285 \right] \quad (Eq. B4.1-8) \]

The effective width, $b$, is calculated in accordance with Section B2.1(a).

(b) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be calculated as in Section B4.1(a), except that $f_d$ is substituted for $f$.

---

**Figure B4-1 Elements with One Intermediate Stiffener**

**B4.2 Uniformly Compressed Elements with an Edge Stiffener**

(a) Strength Determination

For $w / t \leq 0.328S$:
\[
\begin{align*}
I_a & = 0 \quad \text{(no edge stiffener needed)} \\
 b & = w \\
 b_1 & = b_2 = w / 2 \quad \text{(see Fig. B4-2)} \\
 d_s & = d_s' \quad \text{for simple lip stiffener} \\
 A_s & = A_s' \quad \text{for other stiffener shapes} \\
\end{align*}
\]

$\quad (Eq. B4.2-1)$

$\quad (Eq. B4.2-2)$

$\quad (Eq. B4.2-3)$

$\quad (Eq. B4.2-4)$

For $w / t > 0.328S$
\[
\begin{align*}
 b_1 & = b / 2 (R_l) \quad \text{(see Fig. B4-2)} \\
 b_2 & = b - b_1 \quad \text{(see Fig. B4-2)} \\
 d_s & = d_s' (R_l) \quad \text{for simple lip stiffener} \\
 A_s & = A_s' (R_l) \quad \text{for other stiffener shapes} \\
\end{align*}
\]

$\quad (Eq. B4.2-5)$

$\quad (Eq. B4.2-6)$

$\quad (Eq. B4.2-7)$

$\quad (Eq. B4.2-8)$
where

\[ S = \text{Term defined in Eq. B4-1}. \]

\[ (R_i) = \frac{I_s}{I_a} \leq 1 \quad \text{(Eq. B4.2-9)} \]

\[ I_a = 399t^4 \left[ \frac{w}{S} - 0.328 \right]^3 \leq t^4 \left[ 115 \frac{w}{S} + 5 \right] \quad \text{(Eq. B4.2-10)} \]

\[ n = \left[ 0.582 - \frac{w}{4S} \right] \geq \frac{1}{3} \quad \text{(Eq. B4.2-11)} \]

The effective width, \( b \), shall be calculated in accordance with Section B2.1 with \( k \) as given in Table B4.2.

**Table B4.2 Determination of Plate Buckling Coefficient \( k \)**

<table>
<thead>
<tr>
<th>Simple Lip Edge Stiffener (140° ≥ ( \theta ) ≥ 40°)</th>
<th>Other Edge Stiffener Shapes</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D/w \leq 0.25 )</td>
<td>( 0.25 &lt; D/w \leq 0.8 )</td>
</tr>
<tr>
<td>( (4.82 - \frac{5D}{w})(R_1)^n + 0.43 \leq 4 )</td>
<td>( 3.57(R_1)^n + 0.43 \leq 4 )</td>
</tr>
</tbody>
</table>

\[ D, d = \text{Actual stiffener dimensions} \]

**Figure B4-2 Elements with Simple Lip Edge Stiffener**
(b) Serviceability Determination

The effective width, \( b_{\text{d}} \), used in determining serviceability shall be calculated as in Section B4.2(a), except that \( f_d \) is substituted for \( f \).

B5 Effective Widths of Stiffened Elements with Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffeners

B5.1 Effective Widths of Uniformly Compressed Stiffened Elements with Multiple Intermediate Stiffeners

The following notation is used in this section.

- \( A_g \): Gross area of element including stiffeners
- \( A_s \): Gross area of stiffener
- \( b_e \): Effective width of element, located at centroid of element including stiffeners, see Figure B5.1-2.
- \( b_p \): Largest sub-element flat width, see Figure B5.1-1.
- \( b_o \): Total flat width of stiffened element, see Figure B5.1-1.
- \( c_i \): Horizontal distance from edge of element to centerline(s) of stiffener(s), see Figure B5.1-1.
- \( F_{\text{cr}} \): Plate elastic buckling stress
- \( f \): Uniform compressive stress acting on flat element
- \( h \): Width of elements adjoining stiffened element (e.g., depth of web in hat section with multiple intermediate stiffeners in compression flange is equal to \( h \); if adjoining elements have different widths, use smallest one.)
- \( I_{sp} \): Moment of inertia of stiffener about centerline of flat portion of element. The radii which connect the stiffener to the flat may be included.
- \( k \): Plate buckling coefficient of element
- \( k_d \): Plate buckling coefficient for distortional buckling.
- \( k_{\text{loc}} \): Plate buckling coefficient for local sub-element buckling.
- \( L_{\text{br}} \): Unsupported length between brace points or other restraints which restrict distortional buckling of element.
- \( R \): Modification factor for distortional plate buckling coefficient
- \( n \): Number of stiffeners in element
- \( t \): Element thickness
- \( i \): Index for stiffener “\( i \)”
- \( \lambda \): Slenderness factor
- \( \rho \): Reduction factor

The effective width shall be determined as follows:

\[
 b_e = \rho \left( \frac{A_g}{t} \right) \quad \text{(Eq. B5.1-1)}
\]

\[
 \rho = 1 \quad \text{when } \lambda \leq 0.673 \quad \text{(Eq. B5.1-2)}
\]
\[ \rho = \frac{(1 - 0.22/\lambda)}{\lambda} \quad \text{when } \lambda > 0.673 \quad (Eq. B5.1-3) \]
\[ \lambda = \frac{f}{\sqrt{F_{cr}}} \quad (Eq. B5.1-4) \]
\[ F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left( \frac{t}{b_o} \right)^2 \quad (Eq. B5.1-5) \]

The plate buckling coefficient, \( k \), shall be determined from the minimum of \( R k_d \) and \( k_{loc} \) as determined from section B5.1.1 or B5.1.2, as appropriate.
\[ k = \text{the minimum of } R k_d \text{ and } k_{loc} \quad (Eq. B5.1-6) \]
\[ R = 2 \quad \text{when } b_o/h < 1 \quad (Eq. B5.1-7) \]
\[ R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \quad \text{when } b_o/h \geq 1 \quad (Eq. B5.1-8) \]

**B5.1.1 Specific Case: ‘n’ Identical Stiffeners, Equally Spaced**

(a) **Strength Determination**

\[ k_{loc} = \frac{Q}{o} \quad (Eq. B5.1.1-1) \]
\[ k_d = \frac{(1 + \beta^2)^2 + \gamma(1 + n)}{\beta^2(1 + \delta(n + 1))} \quad (Eq. B5.1.1-2) \]
\[ \beta = (1 + \gamma(n + 1))^{1/4} \quad (Eq. B5.1.1-3) \]

If \( L_{br} < \beta b_o \) then \( L_{br}/b_o \) shall be permitted to be substituted for \( \beta \) to account for increased capacity due to bracing.
\[ \gamma = \frac{10.92I_{sp}}{b_o t^3} \quad (Eq. B5.1.1-4) \]
\[ \delta = \frac{A_s}{b_o t} \quad (Eq. B5.1.1-5) \]

(b) **Serviceability Determination**

The effective width, \( b_d \), used in determining serviceability shall be calculated as in Section B5.1.1(a), except that \( f_d \) shall be substituted for \( f \), where \( f_d \) is the computed compressive stress in the element being considered based on the effective section at the load for which serviceability is determined.

**B5.1.2 General Case: Arbitrary Stiffener Size, Location and Number**

(a) **Strength Determination**

\[ k_{loc} = 4 \left( \frac{b_o}{b_p} \right)^2 \quad (Eq. B5.1.2-1) \]
\[ k_d = \frac{(1 + \beta^2)^2 + 2 \sum_{i=1}^{n} \gamma_i \omega_i}{\beta^2 \left( 1 + 2 \sum_{i=1}^{n} \delta_i \omega_i \right)} \]  

(Eq. B5.1.2-2)

\[ \beta = \left( 0.658 \lambda^2 \frac{\mu}{c} \right) F_y \]  

(Eq. B5.1.2-3)

If \( L_{br} < \beta b_o \) then \( L_{br}/b_o \) shall be permitted to be substituted for \( \beta \) to account for increased capacity due to bracing.

\[ \gamma_i = \frac{10.92(l_{sp})_i}{b_o t^3} \]  

(Eq. B5.1.2-4)

\[ \omega_i = \sin^2 \left( \pi \frac{c_i}{b_o} \right) \]  

(Eq. B5.1.2-5)

\[ \delta_i = \frac{(A_s)_i}{b_o t} \]  

(Eq. B5.1.2-6)

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated as in Section B5.1.2(a), except that \( f_d \) shall be substituted for \( f \), where \( f_d \) is the computed compressive stress in the element being considered based on the effective section at the load for which serviceability is determined.
B5.2 Edge Stiffened Elements with Intermediate Stiffeners

(a) Strength Determination

The effective width, \( b_e \), shall be determined as follows:

If \( \frac{b_o}{t} \leq 0.328S \), the element is fully effective and no local buckling reduction is required.

If \( \frac{b_o}{t} > 0.328S \), then the plate buckling coefficient, \( k \), shall be determined from the provisions of Section B4.2, but with \( b_o \) replacing \( w \) in all expressions.

If \( k \) calculated from Section B4.2 is less than 4.0 (\( k < 4 \)), the intermediate stiffener(s) shall be ignored and the provisions of Section B4.2 be followed for calculation of the effective width.

If \( k \) calculated from Section B4.2 is equal to 4.0 (\( k = 4 \)), the effective width of the edge stiffened element shall be calculated from the provisions of Section B5.1, with the following exception:

\[ R \] calculated from equations B5.1-7 and B5.1-8 must be less than or equal to 1.

where

\( b_o = \) Total flat width of edge stiffened element

Other variables are defined in Section B4 and B5.1.

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated as in Section B5.2(a), except that \( f_d \) shall be substituted for \( f \) and \( f_1 \), where \( f_d \) is the computed compressive stress in the element being considered.
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

The provisions of this section are given in Section C2 of the Appendices.

C3 Flexural Members

C3.1 Bending

The nominal flexural strength [moment resistance], $M_n$, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, C3.1.3, C3.1.4, and C3.1.5, where applicable.

The provisions of this Section do not consider torsional effects, such as those resulting from loads that do not pass through the shear center of the cross section. See Section D3 for the design of lateral bracing required to restrain lateral bending or twisting.

C3.1.1 Nominal Section Strength [Resistance]

The nominal flexural strength [moment resistance], $M_n$, shall be 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.

For sections with stiffened or partially stiffened compression flanges:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b(ASD)$</td>
<td>$\phi_b(LRFD)$</td>
</tr>
<tr>
<td>1.67</td>
<td>0.95</td>
</tr>
</tbody>
</table>

For sections with unstiffened compression flanges:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b(ASD)$</td>
<td>$\phi_b(LRFD)$</td>
</tr>
<tr>
<td>1.67</td>
<td>0.90</td>
</tr>
</tbody>
</table>

(a) Procedure I - Based on Initiation of Yielding

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

$$M_n = S_e F_y$$

(Eq. C3.1.1-1)
where
\[ F_y = \text{Design yield point as determined in Section A7.1} \]
\[ S_e = \text{Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at } F_y \]

(\textit{b}) \textit{Procedure II - Based on Inelastic Reserve Capacity} 

The inelastic flexural reserve capacity is permitted to 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 work of 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 \( 0.35F_y \times \text{the web area, } ht \), for ASD, and \( 0.6F_yht \) for LRFD and LSD.
5. The angle between any web and the vertical does not exceed 30 degrees.

The nominal flexural strength [moment resistance], \( M_n \), shall not exceed either \( 1.25S_eF_y \) 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
\[ C_y = \begin{cases} 3 & \text{for } w/t \leq \lambda_1 \\ \left( \frac{\Omega_b M}{M_{nxo}} \right)^2 + \left( \frac{\Omega_v V}{V_n} \right)^2 & \leq 1.0 \\ 1 & \text{for } w/t > \lambda_2 \end{cases} \]

where
\[ \lambda_1 = \frac{1.11}{\sqrt{\frac{F_y}{E}}} \quad (\text{Eq. C3.1.1-2}) \]
\[ \lambda_2 = \frac{1.28}{\sqrt{\frac{F_y}{E}}} \quad (\text{Eq. C3.1.1-3}) \]

(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-Torsional Buckling Strength [Resistance]

#### C3.1.2.1 Lateral-Torsional Buckling Strength [Resistance] of Open Cross Section Members

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.

For laterally unbraced segments of singly-, doubly-, and point-symmetric sections subject to lateral-torsional buckling, the nominal flexural strength [moment resistance], $M_n$, shall be calculated as follows:

$$M_n = S_c F_c$$  \[Eq. C3.1.2.1-1\]

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b$(ASD)</td>
<td>$\phi_b$(LRFD)</td>
</tr>
<tr>
<td>1.67</td>
<td>0.90</td>
</tr>
</tbody>
</table>

where

- $S_c$ = Elastic section modulus of effective section calculated relative to extreme compression fiber at $F_c$
- $F_c$ is determined as follows:
  - For $F_e \geq 2.78F_y$, $F_c = F_y$  \[Eq. C3.1.2.1-2\]
  - For $2.78F_y > F_e > 0.56F_y$, $F_c = \frac{10}{9}F_y \left(1 - \frac{10F_y}{36F_e}\right)$  \[Eq. C3.1.2.1-3\]
  - For $F_e \leq 0.56F_y$, $F_c = F_e$  \[Eq. C3.1.2.1-4\]

where

- $F_e$ = Elastic critical lateral-torsional buckling stress calculated according to (a) or (b)
(a) For singly-, doubly-, and point-symmetric sections:

\[ F_e = \frac{C_b r_o \sigma_t}{S_f} \sqrt{\frac{\sigma_{ey}}{\sigma_{ex}}} \]  

for bending about the symmetry axis.  \( (Eq. \ C3.1.2.1-5) \)

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 \( F_e \).  X-axis of Z-sections is the centroidal axis perpendicular to the web.

Alternatively, \( F_e \) can be calculated using the equation given in (b) for doubly-symmetric I-sections, singly-symmetric C-sections, or point-symmetric Z-sections.

For singly-symmetric sections bending about the centroidal axis perpendicular to the axis of symmetry:

\[ F_e = \frac{C_s A \sigma_{ex}}{C T F S_f} \left[ j + C_s \sqrt{j^2 + r_o^2 \left( \frac{\sigma_t}{\sigma_{ex}} \right)} \right] \]  

\( (Eq. \ C3.1.2.1-6) \)

\( C_s = +1 \) for moment causing compression on shear center side of centroid

\( C_s = -1 \) for moment causing tension on shear center side of centroid

\[ \sigma_{ex} = \frac{\pi^2 E}{(K_x L_x/r_x)^2} \]  

\( (Eq. \ C3.1.2.1-7) \)

\[ \sigma_{ey} = \frac{\pi^2 E}{(K_y L_y/r_y)^2} \]  

\( (Eq. \ C3.1.2.1-8) \)

\[ \sigma_t = \frac{1}{A r_o^2} \left[ GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] \]  

\( (Eq. \ C3.1.2.1-9) \)

\( A = \) Full unreduced cross-sectional area

\( S_f = \) Elastic section modulus of full unreduced section relative to extreme compression fiber

\[ C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3 M_A + 4 M_B + 3 M_C} \]  

\( (Eq. \ C3.1.2.1-10) \)

where:

\( M_{\max} = \) Absolute value of maximum moment in unbraced segment

\( M_A = \) Absolute value of moment at quarter point of unbraced segment

\( M_B = \) Absolute value of moment at centerline of unbraced segment

\( M_C = \) Absolute value of moment at three-quarter point of unbraced segment

\( C_b \) is permitted to be conservatively taken as unity for all cases.  For cantilevers or overhangs where the free end is unbraced, \( C_b \) shall be taken as unity.
\( E \) = Modulus of elasticity
\( C_{TF} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) \)  
(Eq. C3.1.2.1-11)

where

\( M_1 \) is the smaller and \( M_2 \) the larger bending moment at the ends of the unbraced length in the plane of bending, 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, \( C_{TF} \) shall be taken as unity.

\( r_o \) = Polar radius of gyration of cross section about shear center
\( = \sqrt{r_x^2 + r_y^2 + x_o^2} \)  
(Eq. C3.1.2.1-12)

\( r_x, r_y \) = Radii of gyration of cross section about centroidal principal axes

\( G \) = Shear modulus

\( K_x, K_y, K_t \) = Effective length factors for bending about x- and y-axes, and for twisting

\( L_x, L_y, L_t \) = Unbraced length of member for bending about x- and y-axes, and for twisting

\( x_o \) = Distance from shear center to centroid along principal x-axis, taken as negative

\( J \) = Saint-Venant torsion constant of cross section

\( C_w \) = Torsional warping constant of cross section

\( j = \frac{1}{2I_y} \left[ \int_A x^3 dA + \int_A xy^2 dA \right] - x_o \)  
(Eq. C3.1.2.1-13)

(b) For I-sections, singly-symmetric C-sections, or Z-sections bent about the centroidal axis perpendicular to the web (x-axis), the following equations are permitted to be used in lieu of (a) to calculate \( F_e \):

\[ F_e = \frac{C_b \pi^2 E I_{yc}}{S_f (K_y L_y)^2} \] for doubly-symmetric I-sections

\[ = \frac{C_b \pi^2 E I_{yc}}{2S_f (K_y L_y)^2} \] for point-symmetric Z-sections  
(Eq. C3.1.2.1-14)

where

\( d \) = Depth of section

\( I_{yc} \) = Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web, using full unreduced section

Other terms are defined in (a).
C3.1.2.2 Lateral-Torsional Buckling Strength [Resistance] of Closed Box Members

For closed box members, the nominal flexural strength [moment resistance], $M_n$, shall be determined as follows:

If the laterally unbraced length of the member is less than or equal to $L_u$, the nominal flexural strength [moment resistance] shall be determined by using Section C3.1.1.

where

$$L_u = \frac{0.36 C_b \pi}{F_y S_f} \sqrt{\frac{E G J}{I_y}}$$  \hspace{1cm} (Eq. C3.1.2.2-1)

If the laterally unbraced length of a member is larger than $L_u$, the nominal flexural strength [moment resistance] shall be determined in accordance with C3.1.2.1, where the critical lateral buckling stress, $F_e$, is calculated as follows:

$$F_e = \frac{C_b \pi}{K_y L_y S_f} \sqrt{\frac{E G J}{I_y}}$$  \hspace{1cm} (Eq. C3.1.2.2-2)

where

$I_y$ = Moment of inertia of full unreduced section about centroidal axis parallel to web

$J$ = Torsional constant of box section

Other variables are defined in Section C3.1.2.1.

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 nominal flexural strength [moment resistance], $M_n$, of a C- 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 calculated as follows:

$$M_n = R S_e F_y$$  \hspace{1cm} (Eq. C3.1.3-1)

where $R$ is obtained from Table C3.1.3-1 for simple span C- or Z-sections, and

$R \begin{cases} 0.60 & \text{for continuous span C-sections} \\ 0.70 & \text{for continuous span Z-sections} \end{cases}$

$S_e$ 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 in. (292 mm)
2. Member flanges shall have edge stiffeners
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
(7) Member span length shall be no greater than 33 feet (10 m)
(8) For continuous span systems, the longest member span length shall not be more than 20% greater than the shortest span length
(9) Both flanges shall be prevented from moving laterally at the supports
(10) Roof or wall panels shall be steel sheets with 50 ksi (340 MPa or 3520 kg/cm\(^2\)) minimum yield point, and a minimum of 0.018 in. (0.46 mm) base metal thickness, having a minimum rib depth of 1-1/4 in. (32 mm), spaced a maximum of 12 in. (305 mm) 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 in. (152 mm) 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. (4.76 mm) rivets, having washers 1/2 in. (12.7 mm) diameter
(13) Fasteners shall not be standoff type screws
(14) Fasteners shall be spaced not greater than 12 in. (305 mm) on centers and placed near the center of the beam flange, and adjacent to the panel high rib
(15) The design yield point of the member shall not exceed 60 ksi (410 MPa or 4220 kg/cm\(^2\))

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

<table>
<thead>
<tr>
<th>Depth Range, in. (mm)</th>
<th>Profile</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d \leq 6.5) (165)</td>
<td>C or Z</td>
<td>0.70</td>
</tr>
<tr>
<td>6.5 (165) &lt; (d \leq 8.5) (216)</td>
<td>C or Z</td>
<td>0.65</td>
</tr>
<tr>
<td>8.5 (216) &lt; (d \leq 11.5) (292)</td>
<td>Z</td>
<td>0.50</td>
</tr>
<tr>
<td>8.5 (216) &lt; (d \leq 11.5) (292)</td>
<td>C</td>
<td>0.40</td>
</tr>
</tbody>
</table>

For simple span members, R shall be reduced for the effects of compressed insulation between the sheeting and the member. The
reduction shall be calculated by multiplying R from Table C3.1.3-1 by the following correction factor, r:

\[ r = 1.00 - 0.01 t_i \quad \text{when } t_i \text{ is in inches} \quad (Eq. C3.1.3-2) \]

\[ r = 1.00 - 0.0004 t_i \quad \text{when } t_i \text{ is in millimeters} \quad (Eq. C3.1.3-3) \]

\( t_i = \) Thickness of uncompressed glass fiber blanket insulation

**C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System**

The provisions of this section are given in Section C3.1.4 of the Appendices.

**C3.1.5 Strength [Resistance] of Standing Seam Roof Panel Systems**

When results of tests on standing seam roof panel systems conducted according to ASTM E1592-95 are to be evaluated, the “Standard Procedures for Panel and Anchor Structural Tests” as published by AISI shall be followed. Strength [Resistance] under uplift loading shall be evaluated by this procedure.

When the number of physical test assemblies is 3 or more, factors of safety and resistance factors shall be determined in accordance with the procedures of Section F1.1(b) with the following definition for the variables:

\( \beta_o = \) Target reliability index

- 2.0 for USA and Mexico and 2.5 for Canada for panel flexural limits
- 2.5 for USA and Mexico and 3.0 for Canada for anchor limits

\( F_m = \) Mean value of the fabrication factor

- 1.0

\( M_m = \) Mean value of the material factor

- 1.1

\( V_M = \) Coefficient of variation of the material factor

- 0.08 for anchor failure mode
- 0.10 for other failure modes

\( V_F = \) Coefficient of variation of the fabrication factor

- 0.05

\( V_Q = \) Coefficient of variation of the load effect

- 0.21

\( V_P = \) Actual calculated coefficient of variation of the test results, without limit

\( n = \) Number of anchors in the test assembly with same tributary area (for anchor failure), or number of panels with identical spans and loading to the failed span (for non-anchor failures)

When the number of physical test assemblies is less than 3, a factor of safety, \( \Omega \), of 2.0 and a resistance factor, \( \phi \), of 0.8 (LRFD) and 0.70 (LSD) shall be used.
C3.2 Shear

C3.2.1 Shear Strength [Resistance] of Webs without Holes

The nominal shear strength [resistance], $V_n$, shall be calculated as follows:

$$V_n = A_w F_v$$  \hspace{1cm} (Eq. C3.2.1-1)

(a) For $h/t \leq \sqrt[6]{\frac{E_k}{F_y}}$

$$F_v = 0.60 F_y$$  \hspace{1cm} (Eq. C3.2.1-2)

(b) For $\sqrt[6]{\frac{E_k}{F_y}} < h/t \leq 1.51 \sqrt[6]{\frac{E_k}{F_y}}$

$$F_v = \frac{0.60 \sqrt{E_k F_y}}{(h/t)}$$  \hspace{1cm} (Eq. C3.2.1-3)

(c) For $h/t > 1.51 \sqrt[6]{\frac{E_k}{F_y}}$

$$F_v = \frac{\pi^2 E_k}{12(1-\mu^2)(h/t)^2} = 0.904 \frac{E_k}{(h/t)^2}$$  \hspace{1cm} (Eq. C3.2.1-4)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_v$(ASD)</td>
<td>$\phi_v$(LRFD)</td>
</tr>
<tr>
<td>1.60</td>
<td>0.95</td>
</tr>
</tbody>
</table>

where

- $A_w$ = Area of web element = $ht$
- $E$ = Modulus of elasticity of steel
- $F_v$ = Nominal shear stress
- $V_n$ = Nominal shear strength [resistance]
- $t$ = Web thickness
- $h$ = Depth of flat portion of web measured along plane of web
- $\mu$ = Poisson’s ratio = 0.3
- $k_v$ = Shear buckling coefficient determined as follows:
  1. For unreinforced webs, $k_v = 5.34$
  2. For webs with transverse stiffeners satisfying the requirements of Section C3.6
     when $a/h \leq 1.0$
     $$k_v = 4.00 + \frac{5.34}{(a/h)^2}$$  \hspace{1cm} (Eq. C3.2.1-5)
     when $a/h > 1.0$
     $$k_v = 5.34 + \frac{4.00}{(a/h)^2}$$  \hspace{1cm} (Eq. C3.2.1-6)

where

- $a$ = Shear panel length of unreinforced web element
- = Clear distance between transverse stiffeners of 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.

**C3.2.2 Shear Strength [Resistance] of C-Section Webs with Holes**

These provisions shall be applicable within the following limits:
1. \( \frac{d_0}{h} \leq 0.7 \)
2. \( \frac{h}{t} \leq 200 \)
3. Holes centered at mid-depth of web
4. Clear distance between holes \( \geq 18 \text{ in. (457 mm)} \)
5. Non-circular holes, corner radii \( \geq 2t \)
6. Non-circular holes, \( d_0 \leq 2.5 \text{ in. (64 mm)} \) and \( b \leq 4.5 \text{ in. (114 mm)} \)
7. Circular holes, diameter \( \leq 6 \text{ in. (152 mm)} \)
8. \( d_0 > 9/16 \text{ in. (14 mm)} \)

The nominal shear strength [resistance], \( V_n \), determined by Section C3.2.1 shall be multiplied by \( q_s \):

When \( c/t \geq 54 \)
\[
q_s = 1.0 \quad (\text{Eq. C3.2.2-1})
\]

When \( 5 \leq c/t < 54 \)
\[
q_s = c/(54t) \quad (\text{Eq. C3.2.2-2})
\]

where
\[
c = h/2 - d_0/2.83 \quad \text{for circular holes} \quad (\text{Eq. C3.2.2-3})
\]
\[
c = h/2 - d_0/2 \quad \text{for non-circular holes} \quad (\text{Eq. C3.2.2-4})
\]
\[
d_0 = \text{Depth of web hole}
\]
\[
b = \text{Length of web hole}
\]
\[
h = \text{Depth of flat portion of web measured along plane of web}
\]

**C3.3 Combined Bending and Shear**

**C3.3.1 ASD Method**

For beams subjected to combined bending and shear, the required allowable flexural strength, \( M \), and required allowable shear strength, \( V \), shall not exceed \( M_n/\Omega_b \) and \( V_n/\Omega_v \), respectively.

For beams with unreinforced webs, the required allowable flexural strength, \( M \), and required allowable shear strength, \( V \), shall also satisfy the following interaction equation:

\[
\left( \frac{\Omega_bM}{M_{nxo}} \right)^2 + \left( \frac{\Omega_vV}{V_n} \right)^2 \leq 1.0 \quad (\text{Eq. C3.3.1-1})
\]

For beams with transverse web stiffeners, when \( \Omega_bM/M_{nxo} > 0.5 \) and \( \Omega_vV/V_n > 0.7 \), \( M \) and \( V \) shall also satisfy the following interaction equation:

\[
0.6 \left( \frac{\Omega_bM}{M_{nxo}} \right) + \left( \frac{\Omega_vV}{V_n} \right) \leq 1.3 \quad (\text{Eq. C3.3.1-2})
\]
where:

- \( \Omega_b \) = Factor of safety for bending (See Section C3.1.1)
- \( \Omega_v \) = Factor of safety for shear (See Section C3.2)
- \( M_n \) = Nominal flexural strength when bending alone is considered
- \( M_{n\text{xo}} \) = Nominal flexural strength about centroidal x-axis determined in accordance with Section C3.1.1
- \( V_n \) = Nominal shear strength when shear alone is considered

### C3.3.2 LRFD and LSD Methods

For beams subjected to combined bending and shear, the required flexural strength [factored moment], \( \bar{M} \), and the required shear strength [factored shear], \( \bar{V} \), shall not exceed \( \phi_b M_n \) and \( \phi_v V_n \), respectively.

For beams with unreinforced webs, the required flexural strength [factored moment], \( \bar{M} \), and the required shear strength [factored shear], \( \bar{V} \), shall also satisfy the following interaction equation:

\[
\left( \frac{\bar{M}}{\phi_b M_{n\text{xo}}} \right)^2 + \left( \frac{\bar{V}}{\phi_v V_n} \right)^2 \leq 1.0
\]  

(Eq. C3.3.2-1)

For beams with transverse web stiffeners, when \( \bar{M} / (\phi_b M_{n\text{xo}}) > 0.5 \) and \( \bar{V} / (\phi_v V_n) > 0.7 \), \( \bar{M} \) and \( \bar{V} \) shall also satisfy the following interaction equation:

\[
0.6 \left( \frac{\bar{M}}{\phi_b M_{n\text{xo}}} \right) + \left( \frac{\bar{V}}{\phi_v V_n} \right) \leq 1.3
\]  

(Eq. C3.3.2-2)

where:

- \( \phi_b \) = Resistance factor for bending (See Section C3.1.1)
- \( \phi_v \) = Resistance factor for shear (See Section C3.2)
- \( M_n \) = Nominal flexural strength [moment resistance] when bending alone is considered
- \( M_{n\text{xo}} \) = Nominal flexural strength [moment resistance] about centroidal x-axis determined in accordance with Section C3.1.1
- \( \bar{M} \) = Required flexural strength [factored moment]
  - \( \bar{M} = M_u \) (LRFD)
  - \( \bar{M} = M_f \) (LSD)
- \( V_n \) = Nominal shear strength [resistance] when shear alone is considered
- \( \bar{V} \) = Required shear strength [factored shear]
  - \( \bar{V} = V_u \) (LRFD)
  - \( \bar{V} = V_f \) (LSD)
C3.4 Web Crippling

C3.4.1 Web Crippling Strength [Resistance] of Webs without Holes

The nominal web crippling strength [resistance], \( P_n \), shall be determined as follows:

\[
P_n = C t^2 F_y \sin \theta \left( 1 - C_R \sqrt{\frac{R}{t}} \right) \left( 1 + C_N \sqrt{\frac{N}{t}} \right) \left( 1 - C_h \sqrt{\frac{h}{t}} \right)
\]

(Eq. C3.4.1-1)

where:

- \( P_n \) = Nominal web crippling strength [resistance]
- \( C \) = Coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5
- \( C_h \) = Web slenderness coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5
- \( C_N \) = Bearing length coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5
- \( C_R \) = Inside bend radius coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4 or C3.4.1-5
- \( F_y \) = Design yield point as determined in Section A7.1
- \( h \) = Flat dimension of web measured in plane of web
- \( N \) = Bearing length [3/4 in. (19 mm) minimum]
- \( R \) = Inside bend radius
- \( t \) = Web thickness
- \( \theta \) = Angle between plane of web and plane of bearing surface, \( 45^\circ \leq \theta \leq 90^\circ \)

Webs of members in bending for which \( h/t \) is greater than 200 shall be provided with adequate means of transmitting concentrated loads or reactions directly into the web(s).

\( P_n \) represents the nominal strength [resistance] for load or reaction for one solid web connecting top and bottom flanges. For webs consisting of two or more such sheets, \( P_n \) shall be calculated for each individual sheet and the results added to obtain the nominal strength for the full section.

One-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is greater than 1.5h.

Two-flange loading or reaction occurs when the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or less than 1.5h.

End loading or reaction occurs when the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h.

Interior loading or reaction occurs when the distance from the edge of the bearing to the end of the member is greater than 1.5h, except as otherwise noted herein.

The factors of safety and resistance factors are provided in the Tables C3.4.1-1 to C3.4.1-5.
### TABLE C3.4.1-1
**BUILT-UP SECTIONS**

<table>
<thead>
<tr>
<th>Support and Flange Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>CR</th>
<th>CN</th>
<th>C_h</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>20</td>
<td>0.15</td>
<td>0.05</td>
<td>0.003</td>
<td>1.65</td>
<td>0.90</td>
</tr>
<tr>
<td>Unfastened</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>20.5</td>
<td>0.17</td>
<td>0.11</td>
<td>0.001</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>15.5</td>
<td>0.09</td>
<td>0.08</td>
<td>0.04</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>36</td>
<td>0.14</td>
<td>0.08</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>20.5</td>
<td>0.17</td>
<td>0.11</td>
<td>0.001</td>
<td>1.75</td>
<td>0.85</td>
</tr>
</tbody>
</table>

**Notes:**
This Table applies to I-beams made from two channels connected back to back. See Section C3.4.1 of *Commentary* for explanation.

The above coefficients apply when h/t ≤ 200, N/t ≤ 210, N/h ≤ 1.0 and θ = 90°.

### TABLE C3.4.1-2
**SINGLE WEB CHANNEL AND C-SECTIONS**

<table>
<thead>
<tr>
<th>Support and Flange Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>CR</th>
<th>CN</th>
<th>C_h</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>7.5</td>
<td>0.08</td>
<td>0.12</td>
<td>0.048</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>20</td>
<td>0.10</td>
<td>0.08</td>
<td>0.031</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Unfastened</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>13</td>
<td>0.32</td>
<td>0.05</td>
<td>0.04</td>
<td>1.65</td>
</tr>
<tr>
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<td></td>
<td>Interior</td>
<td>24</td>
<td>0.52</td>
<td>0.15</td>
<td>0.001</td>
<td>1.90</td>
<td>0.80</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
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<td>0.60</td>
<td>0.03</td>
<td>1.80</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.32</td>
<td>0.10</td>
<td>0.01</td>
<td>1.80</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>2</td>
<td>0.11</td>
<td>0.37</td>
<td>0.01</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.47</td>
<td>0.25</td>
<td>0.04</td>
<td>1.90</td>
<td>0.80</td>
</tr>
</tbody>
</table>
Note:

1. The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90^\circ$.
2. For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least 2.5$h$. For unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least 1.5$h$.

### TABLE C3.4.1-3
**SINGLE WEB Z-SECTIONS**

<table>
<thead>
<tr>
<th>Support and Flange Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>$C_R$</th>
<th>$C_N$</th>
<th>$C_h$</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>Stiffened or Partially</td>
<td>One-Flange Loading or Reaction</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td>Support</td>
<td>Interior</td>
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<td>0.05</td>
<td>0.16</td>
<td>0.052</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>Partially Stiffened Flanges</td>
<td>Two-Flange Loading or Reaction</td>
<td>24</td>
<td>0.07</td>
<td>0.07</td>
<td>0.04</td>
<td>1.85</td>
<td>0.80</td>
<td>0.70</td>
</tr>
<tr>
<td>Unfastened</td>
<td>End</td>
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<td>0.09</td>
<td>0.02</td>
<td>0.001</td>
<td>1.80</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>Stiffened or Partially</td>
<td>One-Flange Loading or Reaction</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td>Support</td>
<td>Interior</td>
<td>13</td>
<td>0.32</td>
<td>0.05</td>
<td>0.04</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td>Partially Stiffened Flanges</td>
<td>Two-Flange Loading or Reaction</td>
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<td>0.15</td>
<td>0.001</td>
<td>1.90</td>
<td>0.80</td>
<td>0.65</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>End</td>
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<td>0.60</td>
<td>0.03</td>
<td>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<td>Loading or Reaction</td>
<td>Interior</td>
<td>13</td>
<td>0.32</td>
<td>0.10</td>
<td>0.01</td>
<td>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>2</td>
<td>0.11</td>
<td>0.37</td>
<td>0.01</td>
<td>2.00</td>
<td>0.75</td>
<td>0.65</td>
</tr>
<tr>
<td>Unfastened</td>
<td>Interior</td>
<td>13</td>
<td>0.47</td>
<td>0.25</td>
<td>0.04</td>
<td>1.90</td>
<td>0.80</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Note:

1. The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90^\circ$.
2. For interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least 2.5$h$. For unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least 1.5$h$.  

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### TABLE C3.4.1-4
**SINGLE HAT SECTIONS**

<table>
<thead>
<tr>
<th>Support Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>CR</th>
<th>CN</th>
<th>Ch</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ASD Ω&lt;sub&gt;w&lt;/sub&gt;</td>
<td>LRFD φ&lt;sub&gt;w&lt;/sub&gt;</td>
<td></td>
</tr>
<tr>
<td>Fastened to Support</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.25</td>
<td>0.68</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>17</td>
<td>0.13</td>
<td>0.13</td>
<td>0.04</td>
<td>1.90</td>
<td>0.80</td>
</tr>
<tr>
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<td>Two-Flange Loading or Reaction</td>
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<td>0.03</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>10</td>
<td>0.14</td>
<td>0.22</td>
<td>0.02</td>
<td>1.80</td>
<td>0.85</td>
</tr>
<tr>
<td>Unfastened</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.25</td>
<td>0.68</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>17</td>
<td>0.13</td>
<td>0.13</td>
<td>0.04</td>
<td>1.70</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**Note:**
The above coefficients apply when h/t ≤ 200, N/t ≤ 200, N/h ≤ 2 and θ = 90°.

### TABLE C3.4.1-5
**MULTI-WEB DECK SECTIONS**

<table>
<thead>
<tr>
<th>Support Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>CR</th>
<th>CN</th>
<th>Ch</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ASD Ω&lt;sub&gt;w&lt;/sub&gt;</td>
<td>LRFD φ&lt;sub&gt;w&lt;/sub&gt;</td>
<td></td>
</tr>
<tr>
<td>Fastened to Support</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>3</td>
<td>0.08</td>
<td>0.70</td>
<td>0.055</td>
<td>2.25</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>8</td>
<td>0.10</td>
<td>0.17</td>
<td>0.004</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>9</td>
<td>0.12</td>
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<td>0.040</td>
<td>1.80</td>
<td>0.85</td>
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<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>10</td>
<td>0.11</td>
<td>0.21</td>
<td>0.020</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Unfastened</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>3</td>
<td>0.08</td>
<td>0.70</td>
<td>0.055</td>
<td>2.25</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>8</td>
<td>0.10</td>
<td>0.17</td>
<td>0.004</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>6</td>
<td>0.16</td>
<td>0.15</td>
<td>0.050</td>
<td>1.65</td>
<td>0.90</td>
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<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>17</td>
<td>0.10</td>
<td>0.10</td>
<td>0.046</td>
<td>1.65</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**Notes:**
(1) The above coefficients apply when h/t ≤ 200, N/t ≤ 210, N/h ≤ 3.
(2) 45° ≤ θ ≤ 90°°

**C3.4.2 Web Crippling Strength [Resistance] of C-Section Webs with Holes**

When a web hole is within the bearing length, a bearing stiffener shall be used.

For beam webs with holes, the web crippling strength [resistance] shall be computed by using Section C3.4.1 multiplied by the reduction factor, R<sub>CR</sub>, given in this section.
These provisions shall be applicable within the following limits:

1. \( \frac{d_0}{h} \leq 0.7 \)
2. \( h/t \leq 200 \)
3. Hole centered at mid-depth of web
4. Clear distance between holes \( \geq 18 \text{ in. (457 mm)} \)
5. Distance between end of member and edge of hole \( \geq d \)
6. Non-circular holes, corner radii \( \geq 2t \)
7. Non-circular holes, \( d_0 \leq 2.5 \text{ in. (64 mm)} \) and \( b \leq 4.5 \text{ in. (114 mm)} \)
8. Circular holes, diameters \( \leq 6 \text{ in. (152 mm)} \)
9. \( d_0 > 9/16 \text{ in. (14 mm)} \)

For end-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) when a web hole is not within the bearing length:

\[
R_c = 0.1 h x 0.83 h d_0 0.083 x/h \leq 1.0 \]

\[ N \geq 1 \text{ in. (25 mm)} \]  

\[ (Eq. \ C3.4.2-1) \]

For interior-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) when any portion of a web hole is not within the bearing length:

\[
R_c = 0.90 - 0.047 d_0 / h + 0.053 x/h \leq 1.0 \]

\[ N \geq 3 \text{ in. (76 mm)} \]  

\[ (Eq. \ C3.4.2-2) \]

where

- \( b = \) Length of web hole
- \( d = \) Depth of cross section
- \( d_0 = \) Depth of web hole
- \( h = \) Depth of flat portion of web measured along plane of web
- \( x = \) Nearest distance between web hole and edge of bearing
- \( N = \) Bearing length

**C3.5 Combined Bending and Web Crippling**

**C3.5.1 ASD Method**

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:

\[
1.2 \left( \frac{\Omega_w P}{P_n} \right) + \left( \frac{\Omega_b M}{M_{nXO}} \right) \leq 1.5 \]

\[ (Eq. \ C3.5.1-1) \]

Exception: At the interior supports of continuous spans, the above equation 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 in. (254 mm).

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections 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 C-section);

\[
1.1 \left( \frac{\Omega_w P}{P_n} \right) + \left( \frac{\Omega_b M}{M_{n\times o}} \right) \leq 1.5 \quad (Eq. C3.5.1-2)
\]

Exception: In lieu of equation C3.5.1-2, when \( h/t \leq 2.33/\sqrt{F_y/E} \) and \( \lambda \leq 0.673 \), it shall be permitted to determine the allowable concentrated load or reaction by using \( \frac{P_n}{\Omega_w} \) from Section C3.4.

In the above equations:
- \( \Omega_b \) = Factor of safety for bending (See Section C3.1.1)
- \( \Omega_w \) = Factor of safety for web crippling (See Section C3.4)
- \( P \) = Required allowable strength for concentrated load or reaction in the presence of bending moment
- \( P_n \) = Nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section C3.4
- \( M \) = Required allowable flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction
- \( M_{n\times o} \) = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1.1
- \( w \) = Flat width of beam flange which contacts bearing plate
- \( t \) = Thickness of web or flange
- \( \lambda \) = Slenderness factor given by Section B2.1

(c) For the support point of two nested Z-shapes:

\[
\frac{M}{M_{n\circ}} + 0.85 \frac{P}{P_n} \leq 1.65 \quad (Eq. C3.5.1-3)
\]

In addition, the moment, \( M \), and the concentrated load or reaction, \( P \), shall satisfy \( M \leq M_{n\circ}/\Omega_b \), and \( P \leq P_n/\Omega_w \).

where
- \( M \) = Required allowable flexural strength at section under consideration
- \( M_{n\circ} \) = Nominal flexural strength for nested Z-sections, i.e. sum of two sections evaluated individually, determined in accordance with Section C3.1.1
- \( P \) = Required allowable strength for concentrated load or reaction in presence of bending moment
- \( P_n \) = Nominal web crippling strength assuming single web interior one-flange loading for nested Z-sections, i.e., sum of two webs evaluated individually
- \( \Omega \) = Factor of safety for combined bending and web crippling = 1.75

The above equation is valid for shapes that meet the following limits:
- \( h/t \leq 150 \)
N/t ≤ 140
F_y ≤ 70 ksi (480 MPa or 4910 kg/cm^2)
R/t ≤ 5.5

The following conditions shall also be satisfied:

1) The ends of each section shall be connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.
2) The combined section shall be connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.
3) The webs of the two sections shall be in contact.
4) The ratio of the thicker to the thinner part shall not exceed 1.3.

C3.5.2 LRFD and LSD Methods

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:

\[
1.07 \left( \frac{P}{\phi WP_n} \right) + \left( \frac{M}{\phi b Mn_{x0}} \right) \leq 1.42 \quad (Eq. C3.5.2-1)
\]

Exception: At the interior supports of continuous spans, the above equation 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 in. (254 mm).

(b) For shapes having multiple unreinforced webs such as I-sections made of two C-sections 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 C-section):

\[
0.82 \left( \frac{P}{\phi WP_n} \right) + \left( \frac{M}{\phi b Mn_{x0}} \right) \leq 1.32 \quad (Eq. C3.5.2-2)
\]

Exception: In lieu of equation C3.5.2-2, when h/t ≤ 2.33/\sqrt{F_y/E} and \lambda ≤ 0.673, it shall be permitted to determine the design strength for a concentrated load or reaction by using \phi WP_n from Section C3.4.

In the above equations:

\phi_b = Resistance factor for bending (See Section C3.1.1)
\phi_w = Resistance factor for web crippling (See Section C3.4)
P = Required strength for concentrated load or reaction [factored concentrated load or reaction] in presence of bending moment
\[ P = P_u \text{ (LRFD)} \]
\[ \bar{P} = P_f \text{ (LSD)} \]

\( P_n \) = Nominal strength [resistance] for concentrated load or reaction in absence of bending moment determined in accordance with Section C3.4

\( \bar{M} \) = Required flexural strength [factored moment] at, or immediately adjacent to, the point of application of the concentrated load or reaction \( \bar{P} \)

\[ \bar{M} = M_u \text{ (LRFD)} \]
\[ \bar{M} = M_f \text{ (LSD)} \]

\( M_{n_{xo}} \) = Nominal flexural strength [moment resistance] about centroidal x-axis determined in accordance with Section C3.1.1

\( w \) = Flat width of beam flange which contacts bearing plate

\( t \) = Thickness of web or flange

\( \lambda \) = Slenderness factor given by Section B2.1

(c) For two nested Z-shapes

\[ \frac{\bar{M}}{M_{n_{xo}}} + 0.85 \frac{\bar{P}}{P_n} \leq 1.65\phi \]  \hspace{1cm} (Eq. C3.5.2-3)

In addition, the moment, \( \bar{M} \), and the concentrated load or reaction, \( \bar{P} \), shall satisfy \( \bar{M} \leq \phi \bar{M}_{n_{xo}} \), and \( \bar{P} \leq \phi w P_n \),

where

\( \bar{M} \) = Required flexural strength [factored moment] at section under consideration

\[ \bar{M} = M_u \text{ (LRFD)} \]
\[ \bar{M} = M_f \text{ (LSD)} \]

\( M_{n_{xo}} \) = Nominal flexural strength for nested Z-sections, i.e., sum of two sections evaluated individually, determined in accordance with Section C3.1.1

\( \bar{P} \) = Required strength for concentrated load or reaction [factored concentrated load or reaction] in presence of bending moment

\[ \bar{P} = P_u \text{ (LRFD)} \]
\[ \bar{P} = P_f \text{ (LSD)} \]

\( P_n \) = Nominal web crippling strength [resistance] assuming single web interior one-flange loading for nested Z-sections, i.e., sum of two webs evaluated individually

\( \phi \) = 0.90 (LRFD)
\( \phi \) = 0.80 (LSD)

The above equation is valid for shapes that meet the following limits:

\[ h/t \leq 150 \]
\[ N/t \leq 140 \]
The following conditions shall also be satisfied:

1. The ends of each section shall be connected to the other section by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the web.
2. The combined section shall be connected to the support by a minimum of two 1/2 in. (12.7 mm) diameter A307 bolts through the flanges.
3. The webs of the two sections shall be in contact.
4. The ratio of the thicker to the thinner part shall not exceed 1.3.

### C3.6 Stiffeners

#### C3.6.1 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. For concentrated loads or reactions the nominal strength [resistance] equals \( P_n \), where \( P_n \) is the smaller value given by (a) and (b) as follows:

(a) \( P_n = F_{wy} A_c \)  
(b) \( P_n = \text{Nominal axial strength [resistance]} \) evaluated according to Section C4(a), with \( A_e \) replaced by \( A_b \)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega_c(ASD) )</td>
<td>( \phi_c(LRFD) )</td>
</tr>
<tr>
<td>2.00</td>
<td>0.85</td>
</tr>
</tbody>
</table>

where

\[ A_c = 18t^2 + A_{sr} \] for transverse stiffeners at interior support and under concentrated load  
\[ A_c = 10t^2 + A_{sr} \] for transverse stiffeners at end support  
\[ F_{wy} = \text{Lower value of } F_y \text{ for beam web, or } F_{ys} \text{ for stiffener section} \]

\[ A_b = b_1 t + A_{sr} \] for transverse stiffeners at interior support and under concentrated load  
\[ A_b = b_2 t + A_{sr} \] for transverse stiffeners at end support  
\[ A_s = \text{Cross sectional area of transverse stiffeners} \]

\[ b_1 = 25t \left[ 0.0024 \left( \frac{L_{st}}{t} \right) + 0.72 \right] \leq 25t \]  
\[ b_2 = 12t \left[ 0.0044 \left( \frac{L_{st}}{t} \right) + 0.83 \right] \leq 12t \]

\( L_{st} \) = Length of transverse stiffener  
\( t \) = Base thickness of beam web
The w/t_s ratio for the stiffened and unstiffened elements of transverse stiffeners shall not exceed $1.28 \sqrt{E/F_{ys}}$ and $0.42 \sqrt{E/F_{ys}}$, respectively, where $F_{ys}$ is the yield point, and $t_s$ is the thickness of the stiffener steel.

### C3.6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be based on the nominal shear strength [resistance], $V_n$, 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

$$I_{s_{\text{min}}} = 5h^3[h/a - 0.7(a/h)] \geq (h/50)^4 \quad (\text{Eq. C3.6.2-1})$$

The gross area of shear stiffeners shall not be less than

$$A_{st} = \frac{1-C_v}{2} \left[ h - \frac{(a/h)^2}{(a/h) + \sqrt{1+(a/h)^2}} \right] YDht \quad (\text{Eq. C3.6.2-2})$$

where

$$C_v = \begin{cases} 
1.53E_{k,v} & \text{when } C_v \leq 0.8 \\
F_y(h/t)^2 & \\
1.11 \frac{E_{k,v}}{h/t} & \text{when } C_v > 0.8 \\
F_y & \\
\end{cases} \quad (\text{Eq. C3.6.2-3})$$

$$k_v = \begin{cases} 
4.00 + \frac{5.34}{(a/h)^2} & \text{when } a/h \leq 1.0 \\
4.00 & \text{when } a/h > 1.0 \\
\end{cases} \quad (\text{Eq. C3.6.2-5})$$

$a =$Distance between transverse stiffeners

$Y =$Yield point of web steel

$D =$ Yield point of stiffener steel

$D = 1.0$ 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

### C3.6.3 Non-Conforming Stiffeners

The design strength [factored resistance] of members with transverse stiffeners that do not meet the requirements of Section C3.6.1 or C3.6.2, such as stamped or rolled-in transverse stiffeners, shall be determined by tests in accordance with Chapter F or rational engineering analysis in accordance with A1.1(b).
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 nominal axial strength [compressive resistance], \( P_n \), shall be calculated as follows:

\[
P_n = A_e F_n \tag{Eq. C4-1}
\]

<table>
<thead>
<tr>
<th></th>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega_c(ASD) )</td>
<td>1.80</td>
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</tr>
<tr>
<td>( \phi_c(LRFD) )</td>
<td></td>
<td>0.80</td>
</tr>
</tbody>
</table>

where

\( A_e \) = Effective area calculated at stress \( F_n \). For sections with circular holes, \( A_e \) shall be determined according to Section B2.2(a), 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 \( \lambda_c \leq 1.5 \)

\[
F_n = \left( 0.658 \lambda_c^2 \right) F_y \tag{Eq. C4-2}
\]

For \( \lambda_c > 1.5 \)

\[
F_n = \left( \frac{0.877}{\lambda_c^2} \right) F_y \tag{Eq. C4-3}
\]

where

\[
\lambda_c = \sqrt{\frac{F_y}{F_e}} \tag{Eq. C4-4}
\]

\( F_e \) = The least of the elastic flexural, torsional and torsional-flexural buckling stress determined according to Sections C4.1 through C4.4.

(b) Concentrically loaded angle sections shall be designed for an additional bending moment as specified in the definitions of \( M_x \) and \( M_y \) (ASD) or \( \bar{M}_x \) and \( \bar{M}_y \) (LRFD or LSD) in Section C5.2.

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, \( F_{e_c} \), shall be determined as follows:

\[
F_{e_c} = \frac{\pi^2 E}{(KL/r)^2} \tag{Eq. C4.1-1}
\]
where

- \( E \) = Modulus of elasticity
- \( K \) = Effective length factor
- \( L \) = Laterally unbraced length of member
- \( r \) = Radius of gyration of full unreduced cross section about axis of buckling

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 shall be permitted to 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.

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

For singly-symmetric sections subject to 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 Section C3.1.2.1:

\[
\beta = 1 - \left( \frac{x_o}{r_o} \right)^2
\]

(Eq. C4.2-3)

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

For doubly-symmetric sections subject to torsional buckling, \( F_e \) shall be taken as the smaller of \( F_e \) calculated according to Section C4.1 and \( F_e = \sigma_t \) where \( \sigma_t \) is defined in Section C3.1.2.1.

For singly-symmetric unstiffened angle sections for which the effective area \( (A_e) \) at stress \( F_y \) is equal to the full unreduced cross-sectional area \( (A) \), \( F_e \) shall be computed using Eq. C4.1-1 where \( r \) is the least radius of gyration.

### C4.3 Point-Symmetric Sections

For point-symmetric sections, \( F_e \) shall be taken as the lesser of \( \sigma_t \) as defined in Section C3.1.2.1 and \( F_e \) as calculated in Section C4.1 using the
minor principal axis of the section.

C4.4 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 shall be permitted to be tested in accordance with Chapter F.

C4.5 Built-Up Members

For compression members composed of two sections in contact, the nominal axial strength [compressive resistance] shall be determined in accordance with Section C4(a) subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, \( KL/r \) is replaced by \( (KL/r)_m \) determined as follows:

\[
(KL/r)_m = \sqrt{(KL/r)_o^2 + \left(\frac{a}{r_i}\right)^2}
\]

(Eq. C4.5-1)

where:
- \((KL/r)_o\) = Overall slenderness ratio of entire section about built-up member axis
- \(a\) = Intermediate fastener or spot weld spacing
- \(r_i\) = Minimum radius of gyration of full unreduced cross-sectional area of an individual shape in a built-up member

Other symbols are defined in C4.1.

In addition, the fastener strength [resistance] and spacing shall satisfy the following:

1. The intermediate fastener or spot weld spacing, \(a\), shall be limited such that \(a/r_i\) does not exceed one half the governing slenderness ratio of the built-up member.
2. The ends of a built-up compression member shall be connected by a weld having a length not less than the maximum width of the member or by connectors spaced longitudinally not more than 4 diameters apart for a distance equal to 1.5 times the maximum width of the member.
3. Each discrete connector shall be capable of transmitting a longitudinal shear force of 2.5% of the total force (unfactored force for ASD and factored force for LRFD and LSD) in the built-up member.

C4.6 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions are applicable to C- or Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to deck or sheathing with through fasteners.

The nominal axial strength [resistance] of simple span or continuous C-
or Z-sections shall be calculated as follows:

(a) For weak axis nominal strength [resistance]

\[ P_n = C_1 C_2 C_3 A E / 29500 \text{ kips (Newtons)} \]  

(eq. C4.6-1)

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>( \Omega (ASD) )</td>
<td>1.80</td>
<td>0.85</td>
</tr>
<tr>
<td>( \phi (LRFD) )</td>
<td>0.85</td>
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</tr>
</tbody>
</table>

where:

\[ C_1 = (0.79x + 0.54) \]  

(eq. C4.6-2)

\[ C_2 = (1.17 \alpha t + 0.93) \]  

(eq. C4.6-3)

\[ C_3 = \alpha (2.5b - 1.63d) + 22.8 \]  

(eq. C4.6-4)

For Z-sections:

\[ x = \text{The fastener distance from the outside web edge divided by the} \]  

\[ \text{flange width, as shown in Figure C4.6.} \]

For C-sections:

\[ x = \text{the flange width minus the fastener distance from the outside web} \]  

\[ \text{edge divided by the flange width, as shown in Figure C4.6.} \]

\[ t = \text{C- or Z-section thickness} \]

\[ b = \text{C- or Z-section flange width} \]

\[ d = \text{C- or Z-section depth} \]

\[ A = \text{Full unreduced cross-sectional area of C- or Z-section} \]

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

\[ = 29,500 \text{ ksi for U.S. customary units} \]

\[ = 203,000 \text{ MPa for SI units} \]

\[ = 2,070,000 \text{ kg/cm}^2 \text{ for MKS units} \]

\[ \alpha = \text{Coefficient for conversion of units} \]

\[ = 1 \text{ when } t, b, \text{ and } d \text{ are in inches} \]

\[ = 0.0394 \text{ when } t, b, \text{ and } d \text{ are in mm} \]

\[ = 0.394 \text{ when } t, b, \text{ and } d \text{ are in cm} \]

Eq. C4.6-1 shall be limited to roof and wall systems meeting the following conditions:

1. \( t \leq 0.125 \text{ in. (3.22 mm)} \)
2. \( 6 \text{ in. (152 mm)} \leq d \leq 12 \text{ in. (305 mm)} \)
3. Flanges are edge stiffened compression elements
4. \( 70 \leq d/t \leq 170 \)
5. \( 2.8 \leq d/b \leq 5 \)
6. \( 16 \leq \text{flange flat width }/ t \leq 50 \)
7. Both flanges are prevented from moving laterally at the supports
8. Steel roof or steel wall panels with fasteners spaced 12 in. (305 mm) on center or less and having a minimum rotational lateral stiffness of 0.0015 k/in./in. (10,300 N/m/m) (fastener at mid-flange width for stiffness determination) as determined by the AISI test procedure*
9. C- and Z-sections having a minimum yield point of 33 ksi (230 MPa or 2320 kg/cm²)
10. Span length not exceeding 33 feet (10 m)
(b) For strong axis nominal strength [resistance], the equations contained in Sections C4 and C4.1 of the Specification shall be used.

\[
\begin{align*}
\text{For Z-Section } x &= \frac{a}{b} & (\text{Eq. C4.6-5}) \\
\text{For C-Section } x &= \frac{b-a}{b} & (\text{Eq. C4.6-6})
\end{align*}
\]

Note:
* Further information on the test procedure should be obtained from "Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies", AISI Cold-Formed Steel Design Manual, Part VIII.

C5 Combined Axial Load and Bending

C5.1 Combined Tensile Axial Load and Bending

C5.1.1 ASD Method

The required allowable strengths \( T, M_x, \) and \( M_y \) shall satisfy the following interaction equations:

\[
\frac{\Omega_b M_X}{M_{nx}} + \frac{\Omega_b M_Y}{M_{ny}} + \frac{\Omega_t T}{T_n} \leq 1.0
\]  

\( (\text{Eq. C5.1.1-1}) \)

and

\[
\frac{\Omega_b M_X}{M_{nx}} - \frac{\Omega_b M_Y}{M_{ny}} - \frac{\Omega_t T}{T_n} \leq 1.0
\]  

\( (\text{Eq. C5.1.1-2}) \)

where

\begin{align*}
T &= \text{Required allowable tensile axial strength} \\
M_X, M_Y &= \text{Required allowable flexural strengths with respect to centroidal axes of section} \\
T_n &= \text{Nominal tensile axial strength determined in accordance with Section C2} \\
M_{nx}, M_{ny} &= \text{Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1} \\
M_{nxr}, M_{nyt} &= S_{ft} F_Y \\
S_{ft} &= \text{Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis}
\end{align*}
\( \Omega_b = 1.67 \) for bending strength (Section C3.1.1) or for laterally unbraced beams (Section C3.1.2)

\( \Omega_t = 1.67 \)

**C5.1.2 LRFD and LSD Methods**

The required strengths [factored tension and moments] \( \bar{T}, \bar{M}_x, \) and \( \bar{M}_y \) shall satisfy the following interaction equations:

\[
\frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} + \frac{\bar{T}}{\phi_t T_n} \leq 1.0 \quad (Eq. C5.1.2-1)
\]

\[
\frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} - \frac{\bar{T}}{\phi_t T_n} \leq 1.0 \quad (Eq. C5.1.2-2)
\]

where

- \( \bar{T} \quad \) Required tensile axial strength [factored tension]
  - \( \bar{T} = T_u \) (LRFD)
  - \( \bar{T} = T_f \) (LSD)
- \( \bar{M}_x, \bar{M}_y \quad \) Required flexural strengths [factored moments] with respect to centroidal axes.
  - \( \bar{M}_x = M_{ux} \) \( \bar{M}_y = M_{uy} \) (LRFD)
  - \( \bar{M}_x = M_{fx} \) \( \bar{M}_y = M_{fy} \) (LSD)
- \( T_n \quad \) Nominal axial strength determined in accordance with Section C2
- \( M_{nx}, M_{ny} \quad \) Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1
- \( M_{nxt}, M_{nyt} \quad \) \( S_{ft}F_y \)
- \( S_{ft} \quad \) Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis
- \( \phi_b \quad \) For bending strength [resistance] (Section C3.1.1), \( \phi_b = 0.90 \) or 0.95 (LRFD) and 0.90 (LSD). For laterally unbraced beams (Section C3.1.2), \( \phi_b = 0.90 \) (LRFD and LSD)
- \( \phi_t \quad \) \( 0.95 \) (LRFD)
  - \( 0.90 \) (LSD)

**C5.2 Combined Compressive Axial Load and Bending**

**C5.2.1 ASD Method**

The required allowable strengths \( P, M_x, \) and \( M_y \) shall satisfy the following interaction equations. In addition, each individual ratio in Eqs. C5.2.1-1 to C5.2.1-3 shall not exceed unity.
\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_m x M_x}{M_{nx} \alpha_x} + \frac{\Omega_b C_m y M_y}{M_{ny} \alpha_y} \leq 1.0
\]  
(Eq. C5.2.1-1)

\[
\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0
\]  
(Eq. C5.2.1-2)

When \( \Omega_c P / P_n \leq 0.15 \), the following equation shall be permitted to be used in lieu of the above two equations:

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0
\]  
(Eq. C5.2.1-3)

where

- \( P \) = Required allowable compressive axial strength
- \( M_x, M_y \) = Required allowable flexural strengths with respect to centroidal axes of effective section determined for required compressive axial strength alone. For singly-symmetric unstiffened angle sections with unreduced effective area, \( M_y \) shall be permitted to be taken as the required flexural strength only. For other angle sections or singly-symmetric unstiffened angles for which the effective area \( (A_e) \) at stress \( F_y \) is less than the full unreduced cross-sectional area \( (A) \), \( M_y \) shall be taken either as the required flexural strength or the required flexural strength plus PL/1000, whichever results in a lower permissible value of \( P \).
- \( P_n \) = Nominal axial strength determined in accordance with Section C4 and C6
- \( P_{no} \) = Nominal axial strength determined in accordance with Section C4 and C6, with \( F_n = F_y \)
- \( M_{nx}, M_{ny} \) = Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1

\[
\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}}
\]  
(Eq. C5.2.1-4)

\[
\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}}
\]  
(Eq. C5.2.1-5)

\[
P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2}
\]  
(Eq. C5.2.1-6)

\[
P_{Ey} = \frac{\pi^2 E I_y}{(K_y L_y)^2}
\]  
(Eq. C5.2.1-7)

\( \Omega_b \) = 1.67 for bending strength (Section C3.1.1) or for laterally unbraced beams (Section C3.1.2)

\( \Omega_c \) = 1.80
Moment of inertia of full unreduced cross section about x-axis
Moment of inertia of full unreduced cross section about y-axis
Unbraced length for bending about x-axis
Unbraced length for bending about y-axis
Effective length factor for buckling about x-axis
Effective length factor for buckling about y-axis
Coefficients whose values shall be determined 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. C5.2.1-8)
   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 \( C_m \) shall be permitted to be determined by rational analysis. However, in lieu of such analysis, the following values shall be permitted to be used:
   (a) for members whose ends are restrained, \( C_m = 0.85 \)
   (b) for members whose ends are unrestrained, \( C_m = 1.0 \)

**C5.2.2 LRFD and LSD Methods**

The required strengths [factored axial force and moment] \( P, \overline{M}_x, \) and \( \overline{M}_y \) shall satisfy the following interaction equations. In addition, each individual ratio in Eqs. C5.2.2-1 to C5.2.2-3 shall not exceed unity.

\[
\frac{P}{\phi_c P_n} + \frac{C_{mx} \overline{M}_x}{\phi_b M_{nx} \alpha_x} + \frac{C_{my} \overline{M}_y}{\phi_b M_{ny} \alpha_y} \leq 1.0 \quad \text{(Eq. C5.2.2-1)}
\]

\[
\frac{P}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0 \quad \text{(Eq. C5.2.2-2)}
\]
When \( \bar{P} / \phi_c P_n \leq 0.15 \), the following equation shall be permitted to be used in lieu of the above two equations:

\[
\frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}}{\phi_b M_{nx}} + \frac{\bar{M}}{\phi_b M_{ny}} \leq 1.0
\]

(Eq. C5.2.2-3)

where

\( \bar{P} \) = Required compressive axial strength [factored compressive force]

\[
\bar{P} = P_u \text{ (LRFD)}
\]

\[
\bar{P} = P_f \text{ (LSD)}
\]

\( \bar{M}_x, \bar{M}_y \) = Required flexural strengths [factored moments] with respect to centroidal axes of effective section determined for required compressive axial strength [factored axial force] alone. For singly-symmetric unstiffened angle sections with unreduced effective area, \( \bar{M}_y \) shall be permitted to be taken as the required flexural strength [factored moment] only. For other angle sections or singly-symmetric unstiffened angles for which the effective area \( (A_e) \) at stress \( F_y \) is less than the full unreduced cross-sectional area \( (A) \), \( \bar{M}_y \), shall be taken either as the required flexural strength [factored moment] or the required flexural strength [factored moment] plus \( (\bar{P})L/1000 \), whichever results in a lower permissible value of \( \bar{P} \).

\[
\bar{M}_x = M_{ux} \quad \bar{M}_y = M_{uy} \text{ (LRFD)}
\]

\[
\bar{M}_x = M_{fx} \quad \bar{M}_y = M_{fy} \text{ (LSD)}
\]

\( P_n \) = Nominal axial strength [axial resistance] determined in accordance with Section C4 and C6

\( P_{no} \) = Nominal axial strength [axial resistance] determined in accordance with Section C4 and C6, with \( F_n = F_y \)

\( M_{nx}, M_{ny} \) = Nominal flexural strengths [moment resistances] about centroidal axes determined in accordance with Section C3.1

\[
\alpha_x = 1 - \frac{\bar{P}}{\bar{P}_{Ex}}
\]

(Eq. C5.2.2-4)

\[
\alpha_y = 1 - \frac{\bar{P}}{\bar{P}_{Ey}}
\]

(Eq. C5.2.2-5)

\[
\bar{P}_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2}
\]

(Eq. C5.2.2-6)
\( P_{Ey} = \frac{\pi^2 E I_y}{(K_y L_y)^2} \)  \( (Eq. \ C5.2.2-7) \)

\( \phi_b \) = For bending strength [resistance] (Section C3.1.1), \( \phi_b = 0.90 \) or 0.95 (LRFD) and 0.90 (LSD). For laterally unbraced beams (Section C3.1.2), \( \phi_b = 0.90 \) (LRFD and LSD)

\( \phi_c = 0.85 \) (LRFD)
\( \phi_c = 0.80 \) (LSD)

\( I_x \) = Moment of inertia of full unreduced cross section about x-axis

\( I_y \) = Moment of inertia of full unreduced cross section about y-axis

\( L_x \) = Unbraced length for bending about x-axis

\( L_y \) = Unbraced length for bending about y-axis

\( K_x \) = Effective length factor for buckling about x-axis

\( K_y \) = Effective length factor for buckling about y-axis

\( C_{mx}, C_{my} \) = Coefficients whose values shall be determined 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
   \( C_m = 0.6 - 0.4 (M_1/M_2) \)  \( (Eq. \ C5.2.2-8) \)

   \textit{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 \( C_m \) shall be permitted to be determined by rational analysis. However, in lieu of such analysis, the following values shall be permitted to 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 Closed Cylindrical Tubular Members

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

C6.1 Bending

For flexural members, the nominal flexural strength [moment resistance], \( M_n \), shall be calculated as follows:

\[
M_n = F_c S_f
\]  \hspace{1cm} (Eq. C6.1-1)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega_b(ASD) )</td>
<td>( \phi_b(LRFD) )</td>
</tr>
<tr>
<td>1.67</td>
<td>0.95</td>
</tr>
</tbody>
</table>

For \( D/t \leq 0.0714 \  E/F_y \)

\[ F_c = 1.25 \ F_y \] \hspace{1cm} (Eq. C6.1-2)

For \( 0.0714 \ E/F_y < D/t \leq 0.318 \ E/F_y \)

\[
F_c = \left[ 0.970 + 0.020 \left( \frac{E/F_y}{D/t} \right) \right] F_y
\]  \hspace{1cm} (Eq. C6.1-3)

For \( 0.318 \ E/F_y < D/t \leq 0.441 \ E/F_y \)

\[ F_c = 0.328 E/(D/t) \] \hspace{1cm} (Eq. C6.1-4)

where

- \( D \) = Outside diameter of cylindrical tube
- \( t \) = Thickness
- \( F_c \) = Critical flexural buckling stress
- \( S_f \) = Elastic section modulus of full unreduced cross section relative to extreme compression fiber

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 nominal axial strength [compressive resistance], \( P_n \), shall be calculated as follows:

\[
P_n = F_n A_e
\]  \hspace{1cm} (Eq. C6.2-1)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega_c(ASD) )</td>
<td>( \phi_c(LRFD) )</td>
</tr>
<tr>
<td>1.80</td>
<td>0.85</td>
</tr>
</tbody>
</table>

\( F_n \) is determined as follows:

For \( \lambda_c \leq 1.5 \)
\( F_n = \left(0.658\lambda_c^2\right)F_y \)  \( \text{(Eq. C6.2-2)} \)

For \( \lambda_c > 1.5 \)
\[ F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \]  \( \text{(Eq. C6.2-3)} \)

where
\[ \lambda_c = \sqrt{\frac{F_y}{F_e}} \]  \( \text{(Eq. C6.2-4)} \)

In the above equations:
- \( F_e \) = Elastic flexural buckling stress determined according to Section C4.1
- \( A_e = A_o + R(A - A_o) \)  \( \text{(Eq. C6.2–5)} \)
- \( R = F_y/(2F_e) \leq 1.0 \)  \( \text{(Eq. C6.2–6)} \)
- \( A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right]A \leq A \text{ for } \frac{D}{t} \leq 0.441\frac{E}{F_y} \)  \( \text{(Eq. C6.2-7)} \)

\( A \) = Area of full unreduced cross section
\( D \) = Outside diameter of cylindrical tube
\( t \) = Thickness

**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 C-Sections

(a) For compression members:
Refer to Section C4.5.

(b) For flexural members:
The maximum permissible longitudinal spacing of welds or other connectors, \( s_{\text{max}} \), joining two C-sections to form an I-section shall be:

\[
 s_{\text{max}} = \frac{L}{6} \leq \frac{2gT_s}{m q}
\]  

(Eq. D1.1-1)

where

- \( L \) = Span of beam
- \( T_s \) = Design strength [factored resistance] of connection in tension (Chapter E)
- \( g \) = Vertical distance between two rows of connections nearest to top and bottom flanges
- \( q \) = Design load on beam for spacing of connectors (Use nominal loads for ASD, factored loads for LRFD and LSD. For methods of determination, see below)
- \( m \) = Distance from shear center of one C-section to mid-plane of web.

The load, \( q \), is obtained by dividing the 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 uniformly distributed load, based on nominal loads for ASD, factored loads for LRFD and LSD. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, \( s \), the required design strength [factored resistance] of the welds or connections closest to the load or reaction is

\[
 T_s = \frac{P_s m}{2g}
\]  

(Eq. D1.1-2)

where \( P_s \) is a concentrated load or reaction based on nominal loads for ASD, factored loads for LRFD and LSD.

The allowable maximum spacing of connections, \( s_{\text{max}} \), depends upon the intensity of the 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 shall be permitted to 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 [factored resistance] per connection specified elsewhere herein; nor

(b) \[1.16t \sqrt{E/f_c},\] where \(t\) is the thickness of the cover plate or sheet, and \(f_c\) is the stress at nominal load [specified 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

\[1.11t \sqrt{E/F_y}\] if \(w/t < 0.50 \sqrt{E/F_y}\), or \[1.33t \sqrt{E/F_y}\] if \(w/t \geq 0.50 \sqrt{E/F_y}\), 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 1/2 in. (12.7 mm). 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 C-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of C-sections 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. When 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 C-sections and Z-sections designed according to Section C3.1.1, and having deck or sheathing fastened to the top flanges (through fastened or standing seam systems), 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 shall satisfy 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, a restraint system shall be provided to resist the down-slope component of the total gravity load.

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 [resistance] of the purlin assembly is equal to or greater than its nominal flexural strength [moment resistance], instead of that required by Chapter F.

(a) C-Sections

For roof systems using C-sections for purlins with all compression flanges facing in the same direction, a system possessing restraint force, $P_L$, in addition to resisting other loading, shall be provided:

$$P_L = (0.05\alpha \cos \theta - \sin \theta)W$$  \hspace{1cm} (Eq. D.3.2.1-1)

where

- $W =$ Total vertical load (nominal load for ASD, factored load for LRFD and LSD) supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force $P_L$ shall be divided equally between all braces.
- $\alpha =$ +1 for purlin facing upward direction, and
- -1 for purlin facing down slope direction.
- $\theta =$ Angle between vertical and plane of web of C-section, degrees.

A positive value for the force, $P_L$, means that restraint is required to prevent movement of the purlin flanges in the upward roof slope direction, and a negative value means that restraint is required to prevent movement of purlin flanges in the downward slope direction.

(b) Z-Sections

For roof systems 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:
   \[ PL = 0.5 \left[ \frac{0.220b^{1.50}}{d^{0.72}t^{0.90}L^{0.60}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-2)} \)

2. Single-Span System with Third-Point Restraints:
   \[ PL = 0.5 \left[ \frac{0.474b^{1.22}}{d^{0.57}t^{0.89}L^{0.33}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-3)} \)

3. Single-Span System with Midspan Restraint:
   \[ PL = \left[ \frac{0.224b^{1.32}}{d^{0.65}t^{0.83}L^{0.50}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-4)} \)

4. Multiple-Span System with Restraints at the Supports:
   \[ PL = C_{tr} \left[ \frac{0.053b^{1.88}L^{0.13}}{n_p d^{1.07}t^{0.94}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-5)} \)

with
   \( C_{tr} = 0.63 \) for braces at end supports of multiple-span systems
   \( C_{tr} = 0.87 \) for braces at the first interior supports
   \( C_{tr} = 0.81 \) for all other braces

5. Multiple-Span System with Third-Point Restraints:
   \[ PL = C_{th} \left[ \frac{0.181b^{1.15}L^{0.25}}{n_p d^{1.11}t^{0.29}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-6)} \)

with
   \( C_{th} = 0.57 \) for outer braces in exterior spans
   \( C_{th} = 0.48 \) for all other braces

6. Multiple-Span System with Midspan Restraints:
   \[ PL = C_{ms} \left[ \frac{0.116b^{1.32}L^{0.18}}{n_p d^{0.70}t^{0.50}} \cos \theta - \sin \theta \right] W \]  
   \( \text{(Eq. D3.2.1-7)} \)

with
   \( C_{ms} = 1.05 \) for braces in exterior spans
   \( C_{ms} = 0.90 \) for all other braces

where
   \( b = \) Flange width
   \( d = \) Depth of section
   \( t = \) Thickness
   \( L = \) Span length
   \( \theta = \) Angle between vertical and plane of web of Z-section, degrees
   \( n_p = \) Number of parallel purlin lines
\[ W = \text{Total vertical load supported by purlin lines between adjacent supports (Use nominal loads for ASD, factored loads for LRFD and LSD)} \]

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 shall be determined by taking 1.1 times the force found from Equations D3.2.1-2 through D3.2.1-7, with \( n_p = 4 \). For systems having more than twenty purlin lines, the brace force shall be determined from Equations D3.2.1-2 through D3.2.1-7, with \( n_p = 20 \) and \( W \) based on the total number of purlins.

**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 design load (nominal loads for ASD, factored loads for LRFD and LSD) within a distance 0.5\( a \) each side of the brace.

(b) For concentrated loads, \( P_L = 1.0K' \) times each design concentrated load within a distance 0.3\( a \) each side of the brace, plus \( 1.4K' (1-x/a) \) times each design concentrated load located farther than 0.3\( a \) but not farther than 1.0\( a \) from the brace. The design concentrated load is the nominal load for ASD or the factored load for LRFD and LSD.

In the above equations:

*For C-sections:*
\[
K' = \frac{m}{d} \quad \text{(Eq. D3.2.2-1)}
\]

where
- \( m \) = Distance from shear center to mid-plane of web
- \( d \) = Depth of C-section

The brace force, \( P_L \), shall be applied to both flanges in opposite directions in order to resist the twist caused by the load.

*For Z-sections:*
\[
K' = \frac{I_{xy}}{2I_x} \quad \text{(Eq. D3.2.2-2)}
\]

where
- \( I_{xy} \) = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to web
- \( I_x \) = Moment of inertia of full unreduced section about centroidal axis perpendicular to web

The brace force, \( P_L \), shall be applied to both flanges in the same direction in order to constrain bending of the section about the axis perpendicular to its web.

For C-sections and Z-sections:
- \( x \) = Distance from concentrated load to brace
- \( a \) = Distance between center line of braces
When braces are provided, they shall be attached in such a manner to effectively restrain the section against lateral deflection of both flanges at the ends and at any intermediate brace points.

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 torsional rotation and lateral displacement, no additional braces shall be required except those required for strength according to Section C3.1.2.

**D4 Wall Studs and Wall Stud Assemblies**

Wall studs shall be designed either on the basis of an all steel system in accordance with Chapter C or on the basis of sheathing in accordance with Section D4.1 through D4.3. Both solid and perforated webs shall be permitted. Both ends of the stud shall be connected to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis.

(a) **All Steel Design**

Wall stud assemblies using an all steel design shall be designed neglecting the structural contribution of the attached sheathings and shall comply with the requirements of Chapter C. For compression members with circular web perforations, see Section B2.2, and for non-circular web perforations, the effective area shall be determined as follows:

The effective area, \( A_e \), at a stress \( F_n \), shall be determined in accordance with Chapter B, assuming the web to consist of two unstiffened elements, one on each side of the perforation, or the effective area, \( A_e \), shall be determined from stub-column tests.

When \( A_e \) is determined in accordance with Chapter B, the following limitations related to the size and spacing of perforations and the depth of the stud shall apply:

1. The center-to-center spacing of web perforations shall not be less than 24 in. (610 mm).
2. The maximum width of web perforations shall be the lesser of 0.5 times the depth, \( d \), of the section or 2-1/2 in. (63.5 mm).
3. The length of web perforations shall not exceed 4-1/2 in. (114 mm).
4. The section depth-to-thickness ratio, \( d/t \), shall not be less than 20.
5. The distance between the end of the stud and the near edge of a perforation shall not be less than 10 in. (254 mm).

(b) **Sheathing Braced Design**

Wall stud assemblies using a sheathing braced design shall be designed in accordance with Sections D4.1 through D4.3 and in addition shall comply with the following requirements:

In the case of perforated webs, the effective area, \( A_{ew} \), shall be determined as in (a) above.
Sheathing shall be attached to both sides of the stud and connected to the bottom and top horizontal members of the wall to provide lateral and torsional support to the stud in the plane of the wall.

Sheathing shall conform to the limitations specified under Table D4. Additional bracing shall be provided during construction, if required.

**D4.1 Compression**

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal axial strength [compressive resistance], $P_n$, shall be calculated as follows:

$$P_n = A_e F_n$$  \hspace{1cm} \text{Eq. D4.1-1}

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_c(ASD)$</td>
<td>$\phi_c(LRFD)$</td>
</tr>
<tr>
<td>1.80</td>
<td>0.85</td>
</tr>
</tbody>
</table>

where

- $A_e = \text{Effective area determined at } F_n$
- $F_n = \text{Lowest value determined by three conditions (a), (b), and (c) given below. The equations provided in these three conditions are applicable within the following limits:}$
  - Yield point, $F_y \leq 50$ ksi (340 MPa or 3520 kg/cm²)
  - Section depth, $d \leq 6.0$ in. (152 mm)
  - Section thickness, $t \leq 0.075$ in. (1.91 mm)
  - Overall length, $L \leq 16$ ft. (4.88 m)
  - Stud spacing, 12 in. (305 mm) minimum; 24 in. (610 mm) maximum
  - Fastener spacing, 6 in. (152 mm) \( s \leq 12 \) in. (305 mm)

(a) 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.

(b) To prevent flexural and/or torsional overall column buckling, $F_n$ shall be calculated in accordance with Section C4 with $F_e$ 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 C-Sections**

   $$\sigma_{CR} = \sigma_{ey} + Q_{a}$$  \hspace{1cm} \text{Eq. D4.1-2}

   $$\sigma_{CR} = \frac{1}{2\beta} \left[ (\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\sigma_{ex}\sigma_{tQ}} \right]$$  \hspace{1cm} \text{Eq. D4.1-3}

2. **Z-Sections**

   $$\sigma_{CR} = \sigma_{t} + Q_{t}$$  \hspace{1cm} \text{Eq. D4.1-4}

   $$\sigma_{CR} = \frac{1}{2} \left[ (\sigma_{ex} + \sigma_{ey} + Q_{a}) - \sqrt{(\sigma_{ex} + \sigma_{ey} + Q_{a})^2 - 4(\sigma_{ex}\sigma_{ey} + \sigma_{ex}Q_{a} - \sigma_{ey}^2)} \right]$$  \hspace{1cm} \text{Eq. D4.1-5}
(3) I-Sections (doubly-symmetric)

\[ \sigma_{CR} = \sigma_{ey} + Q_a \]  \hspace{1cm} (Eq. D4.1-6)

\[ \sigma_{CR} = \sigma_{ex} \]  \hspace{1cm} (Eq. D4.1-7)

In the above equations:

\[ \sigma_{ex} = \dfrac{\pi^2 E}{(L/r_x)^2} \]  \hspace{1cm} (Eq. D4.1-8)

\[ \sigma_{exy} = \dfrac{\pi^2 EI_{xy}}{(AL^2)} \]  \hspace{1cm} (Eq. D4.1-9)

\[ \sigma_{ey} = \dfrac{\pi^2 E}{(L/r_y)^2} \]  \hspace{1cm} (Eq. D4.1-10)

\[ \sigma_t = \dfrac{1}{Ar_o^2} \left[ GJ + \dfrac{\pi^2 ECw}{L^2} \right] \]  \hspace{1cm} (Eq. D4.1-11)

\[ \sigma_{tQ} = \sigma_t + Q_t \]  \hspace{1cm} (Eq. D4.1-12)

\[ Q = Q_o (2 - s/s') \]  \hspace{1cm} (Eq. D4.1-13)

where

\[ s \] = Fastener spacing, in. (mm)

\[ s' = 12 \text{ in. (305 mm)} \]

\[ Q_o = \text{See Table D4} \]

\[ Q_a = \dfrac{Q}{A} \]  \hspace{1cm} (Eq. D4.1-14)

\[ A \] = Area of full unreduced cross section

\[ L \] = Length of stud

\[ Q_t = \dfrac{(Qd^2)}{(4Ar_o^2)} \]  \hspace{1cm} (Eq. D4.1-15)

\[ d \] = Depth of section

\[ I_{xy} \] = Product of inertia

Other variables are defined in Section C3.1.2.1.

(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, \( \bar{\gamma} \). The shear strain, \( \gamma \), shall be determined as follows:

\[ \gamma = \dfrac{\pi}{L} [C_1 + (E_1 d/2)] \]  \hspace{1cm} (Eq. D4.1-16)

where

\[ C_1 \text{ and } E_1 \] are the absolute values of \( C_1 \) and \( E_1 \) specified below for each section type:

(1) Singly-Symmetric C-sections

\[ C_1 = \dfrac{F_n C_o}{(\sigma_{ey} - F_n + Q_a)} \]  \hspace{1cm} (Eq. D4.1-17)

\[ E_1 = \dfrac{F_n[(\sigma_{ex} - F_n)(\sigma_{ex} - F_n)(\sigma_{ey} - F_n)(\sigma_{ey} - F_n)]}{(\sigma_{ex} - F_n)(\sigma_{ex} - F_n)(\sigma_{ex} - F_n)(\sigma_{ex} - F_n)} \]  \hspace{1cm} (Eq. D4.1-18)
(2) Z-Sections

\[ C_1 = \frac{F_n [C_0 (\sigma_{ex} - F_n) - D_0 \sigma_{exy}]}{(\sigma_{ey} - F_n + \bar{Q}_o) (\sigma_{ex} - F_n) - \sigma_{exy}^2} \]  
\[ E_1 = \frac{(F_n E_o)}{(\sigma_{tQ} - F_n)} \]  
\[ \text{(Eq. D4.1-19)} \]

(3) I-Sections

\[ C_1 = \frac{(F_n C_o)}{(\sigma_{ey} - F_n + \bar{Q}_a)} \]  
\[ E_1 = 0 \]

where

\[ \kappa_o = \text{Distance from shear center to centroid along principal x-axis, (absolute value)} \]

\[ C_o, E_o, \text{ and } D_o \text{ are initial column imperfections which shall be assumed to be at least:} \]

\[ C_o = L/350 \text{ in direction parallel to the wall} \]  
\[ D_o = L/700 \text{ in direction perpendicular to the wall} \]  
\[ E_o = L/(d \times 10,000), \text{ rad., measure of initial twist of stud from initial, ideal, unbuckled shape} \]  
\[ \text{(Eq. D4.1-22)} \]

\[ \text{Other symbols are defined in Sections C3.1.2.1 and D4.1(b).} \]

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

\[ E' = 4EF_n (F_y - F_n)/F_y^2 \]  
\[ G' = G (E' / E) \]  
\[ \text{(Eq. D4.1-25)} \]

Sheathing parameters \( \bar{Q}_o \) and \( \bar{\gamma} \) shall be permitted to 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>
<thead>
<tr>
<th>TABLE D4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheathing Parameters(1)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sheathing(2)</th>
<th>( \bar{Q}_o )</th>
<th>( \bar{\gamma} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8 in. (9.5 mm) to 5/8 in. (15.9 mm) thick gypsum</td>
<td>24.0</td>
<td>107.0</td>
</tr>
<tr>
<td>Lignocellulosic board</td>
<td>12.0</td>
<td>53.4</td>
</tr>
<tr>
<td>Fiberboard (regular or impregnated)</td>
<td>7.2</td>
<td>32.0</td>
</tr>
<tr>
<td>Fiberboard (heavy impregnated)</td>
<td>14.4</td>
<td>64.1</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.

(2) All sheathing is 1/2 in. (12.7 mm) thick except as noted.

For other types of sheathing, \( \bar{Q}_o \) and \( \bar{\gamma} \) shall be permitted to be determined conservatively from representative small-specimen tests as...
described by published documented methods (see Commentary).

**D4.2 Bending**

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal flexural strengths [moment resistances] are $M_{nxo}$ and $M_{nyo}$ where:

For sections with stiffened or partially stiffened compression flanges:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b(ASD)$</td>
<td>$\phi_b(LRFD)$</td>
</tr>
<tr>
<td>1.67</td>
<td>0.95</td>
</tr>
</tbody>
</table>

For sections with unstiffened compression flanges:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_b(ASD)$</td>
<td>$\phi_b(LRFD)$</td>
</tr>
<tr>
<td>1.67</td>
<td>0.90</td>
</tr>
</tbody>
</table>

$m_{nxo}$ and $m_{nyo}$ = Nominal flexural strengths [moment resistances] about centroidal axes determined in accordance with Section C3.1.1

**D4.3 Combined Axial Load and Bending**

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

$P_n = $ Nominal axial strength [resistance] determined according to Section D4.1

$M_{nx}$ and $M_{ny}$ in Equations C5.2.1-1, C5.2.1-2 and C5.2.1-3 for ASD or C5.2.2-1, C5.2.2-2 and C5.2.2-3 for LRFD or LSD shall be replaced by nominal flexural strengths [moment resistances], $M_{nxo}$ and $M_{nyo}$ respectively.

**D5 Floor, Roof or Wall Steel Diaphragm Construction**

The nominal in-plane diaphragm shear strength [resistance], $S_n$ shall be established by calculation or test.

$\Omega_d = $ As specified in Table D5 (ASD)

$\phi_d = $ As specified in Table D5 (LRFD and LSD)
### TABLE D5
Factors of Safety and Resistance Factors for Diaphragms

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
<th>Diaphragm Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega_d$ (ASD)</td>
<td>$\phi_d$ (LRFD)</td>
<td>$\phi_d$ (LSD)</td>
</tr>
<tr>
<td>2.65</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>3.0</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>2.35</td>
<td>0.55</td>
<td>0.50</td>
</tr>
<tr>
<td>2.5</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>2.0</td>
<td>0.65</td>
<td>0.50</td>
</tr>
<tr>
<td>2.45</td>
<td>0.65</td>
<td>0.50</td>
</tr>
</tbody>
</table>
E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum design forces [factored forces] acting on the connected members. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following 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. (4.57 mm) or less. For the design of welded connections in which the thickness of the thinnest connected part is greater than 0.18 in. (4.57 mm), refer to the specifications or standards stipulated in the corresponding Section E2a of Appendix A, B or C.

Welds shall follow the requirements of the weld standards also stipulated in Section E2a of Appendix A, B, or C.

E2.1 Groove Welds in Butt Joints

The nominal strength [resistance], $P_n$, 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

$$P_n = L t_e F_y$$  \hspace{1cm} (Eq. E2.1-1)

(b) Shear on the effective area, the smaller of either Eq. E2.1-2 or E2.1-3

$$P_n = L t_e 0.6 F_{xx}$$  \hspace{1cm} (Eq. E2.1-2)

$$P_n = L t_e F_y / \sqrt{3}$$  \hspace{1cm} (Eq. E2.1-3)

where

- $P_n$ = Nominal strength [resistance] of groove weld
- $F_{xx}$ = Tensile strength of electrode classification
- $F_y$ = Yield point of lowest strength base steel
- $L$ = Length of weld
- $t_e$ = Effective throat dimension of 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 exceeds 0.15 in. (3.81 mm) in thickness, nor through a combination of steel sheets having a total thickness over 0.15 in. (3.81 mm).

Weld washers, shown in Figures E2.2-1 and E2.2-2, shall be used when the thickness of the sheet is less than 0.028 in. (0.711 mm). Weld washers shall have a thickness between 0.05 (1.27 mm) and 0.08 in. (2.03 mm) with a minimum prepunched hole of 3/8 in. (9.53 mm) diameter.

Arc spot welds shall be specified by minimum effective diameter of fused area, $d_e$. Minimum allowable effective diameter is 3/8 in. (9.5 mm).

E2.2.1 Shear

The nominal shear strength [resistance], $P_n$, of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

$$(a) \quad P_n = \frac{\pi d_e^2}{4} 0.75 F_{xx} \quad (Eq. \ E2.2.1-1)$$
USA and Mexico  |  Canada  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
<td>φ(LSD)</td>
</tr>
<tr>
<td>2.55</td>
<td>0.60</td>
<td>0.50</td>
</tr>
</tbody>
</table>

(b) For \( (d_a/t) \leq 0.815 \sqrt{\left( \frac{E}{F_u} \right)} \)

\[
P_n = 2.20 t d_a F_u \quad (Eq. E2.2.1-2)
\]

USA and Mexico  |  Canada  
<table>
<thead>
<tr>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
<td>φ(LSD)</td>
</tr>
<tr>
<td>2.20</td>
<td>0.70</td>
<td>0.60</td>
</tr>
</tbody>
</table>

For \( 0.815 \sqrt{\left( \frac{E}{F_u} \right)} < \frac{(d_a/t)}{\leq 1.397 \sqrt{\left( \frac{E}{F_u} \right)}} \)

\[
P_n = 0.280 \left[ 1 + 5.59 \sqrt{\left( \frac{E}{F_u} \right)} \right] t d_a F_u \quad (Eq. E2.2.1-3)
\]

USA and Mexico  |  Canada  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
<td>φ(LSD)</td>
</tr>
<tr>
<td>2.80</td>
<td>0.55</td>
<td>0.45</td>
</tr>
</tbody>
</table>

For \( (d_a/t) \geq 1.397 \sqrt{\left( \frac{E}{F_u} \right)} \)

\[
P_n = 1.40 t d_a F_u \quad (Eq. E2.2.1-4)
\]

USA and Mexico  |  Canada  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
<td>φ(LSD)</td>
</tr>
<tr>
<td>3.05</td>
<td>0.50</td>
<td>0.40</td>
</tr>
</tbody>
</table>

where

- \( P_n \) = Nominal shear strength [resistance] of arc spot weld
- \( d \) = Visible diameter of outer surface of arc spot weld
- \( d_a \) = Average diameter of arc spot weld at mid-thickness of \( t \) where \( d_a = (d - t) \) for single sheet or multiple sheets not more than four lapped sheets over a supporting member
- \( d_e \) = Effective diameter of fused area at plane of maximum shear transfer
  \[ = 0.7d - 1.5t \leq 0.55d \quad (Eq. E2.2.1-5) \]
- \( t \) = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
- \( F_{xx} \) = Tensile strength of electrode classification
- \( F_u \) = Tensile strength as specified in Section A2.1, A2.2 or A2.3.2

Note: See Figures E2.2.1-1 and E2.2.1-2 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:
emin = \frac{P}{F_{uy} t} \quad \text{For ASD} \quad (Eq. E2.2.1-6a)

emin = \frac{P_u}{\phi F_{uy} t} \quad \text{For LRFD} \quad (Eq. E2.2.1-6b)

emin = \frac{P_f}{\phi F_{uy} t} \quad \text{For LSD} \quad (Eq. E2.2.1-6c)

When \( \frac{F_u}{F_{sy}} \geq 1.08 \)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega ) (ASD)</td>
<td>( \phi ) (LRFD)</td>
</tr>
<tr>
<td>2.20</td>
<td>0.70</td>
</tr>
</tbody>
</table>

When \( \frac{F_u}{F_{sy}} < 1.08 \)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega ) (ASD)</td>
<td>( \phi ) (LRFD)</td>
</tr>
<tr>
<td>2.55</td>
<td>0.60</td>
</tr>
</tbody>
</table>

where

- \( P \) = Required strength (nominal force) transmitted by weld (ASD)
- \( P_u \) = Required strength (factored force) transmitted by weld (LRFD)
- \( P_f \) = Shear force due to factored loads transmitted by weld (LSD)
- \( t \) = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
- \( F_{sy} \) = Yield point as specified in Sections A2.1, A2.2 or A2.3.2
Note: See Figures E2.2.1-3 and E2.2.1-4 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 1.5\(d\). In no case shall the clear distance between welds and the end of member be less than 1.0\(d\).

**E2.2.2 Tension**

The uplift nominal tensile strength [resistance], \(P_n\), of each concentrically loaded arc spot weld connecting sheets and supporting member, shall be computed as the smaller of either:

\[
P_n = \frac{\pi d^2}{4} F_{xx} \quad (Eq. \ E2.2.2-1)
\]

or

\[
P_n = 0.8(F_u/F_y)^2tdaF_u \quad (Eq. \ E2.2.2-2)
\]
For panel and deck applications:

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
</tr>
<tr>
<td>2.50</td>
<td>0.60</td>
</tr>
<tr>
<td>3.00</td>
<td>0.50</td>
</tr>
</tbody>
</table>

For all other applications

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ω(ASD)</td>
<td>φ(LRFD)</td>
</tr>
<tr>
<td>3.00</td>
<td>0.50</td>
</tr>
</tbody>
</table>

The following limitations shall apply:
- \( t_d F_u \leq 3 \text{ kips (13.34 kN)} \)
- \( e_{\min} \geq d \)
- \( F_{xx} \geq 60 \text{ ksi (410 MPa or 4220 kg/cm}^2) \)
- \( F_u \leq 82 \text{ ksi (565 MPa or 5770 kg/cm}^2) \) (of connecting sheets)
- \( F_{xx} > F_u \)

where all other parameters are as defined in Section E2.2.1

For eccentrically loaded arc spot welds subjected to an uplift tension load, the nominal tensile strength [resistance] shall be taken as 50 percent of the above value.

For connections having multiple sheets, the strength [resistance] shall be determined by using the sum of the sheet thicknesses as given by Equation E2.2.2-2.

At the side lap connection within a deck system, the nominal tensile strength [resistance] of the weld connection shall be 70 percent of the above values.

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 shall be permitted to be used providing the particular welding procedure used for making those welds is followed.

**E2.3 Arc Seam Welds**

Arc seam welds (Figure E2.3-1) covered by this Specification apply only to the following joints:
- (a) Sheet to thicker supporting member in the flat position.
- (b) Sheet to sheet in the horizontal or flat position.

The nominal shear strength [resistance], \( P_n \), of arc seam welds shall be determined by using the smaller of either:

\[
P_n = \left[ \frac{\pi d_e^2}{4} + L d_e \right] 0.75 F_{xx} \quad (Eq. E2.3-1)
\]

\[
P_n = 2.5 t F_u (0.25 L + 0.96 d_a) \quad (Eq. E2.3-2)
\]
where

\[ P_n = \text{Nominal shear strength [resistance] of arc seam weld} \]
\[ d = \text{Width of arc seam weld} \]
\[ L = \text{Length of seam weld not including circular ends} \]
   (For computation purposes, L shall not exceed 3d)
\[ d_a = \text{Average width of seam weld} \]
   \[= (d - t) \text{ for single or double sheets} \quad (Eq. E2.3-3) \]
\[ d_e = \text{Effective width of seam weld at fused surfaces} \]
   \[= 0.7d - 1.5t \quad (Eq. E2.3-4) \]

and \( F_{w} \), \( F_{xx} \), and \( t \) are defined in Section E2.2.1. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2.1. See Figure E2.3-2.

### Table: Connections and Joints - USA and Mexico vs. Canada

<table>
<thead>
<tr>
<th></th>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Omega ) (ASD)</td>
<td>2.55</td>
<td>0.60</td>
</tr>
<tr>
<td>( \phi ) (LRFD)</td>
<td>0.60</td>
<td>0.50</td>
</tr>
<tr>
<td>( \phi ) (LSD)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure E2.3-1** Arc Seam Welds - Sheet to Supporting Member in Flat Position

**Figure E2.3-2** Edge Distances for Arc Seam Welds

### 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 nominal shear strength [resistance], $P_n$, of a fillet weld shall be determined as follows:

(a) For longitudinal loading:

For $L/t < 25$:

$$P_n = (1 - \frac{0.01L}{t})LtF_u$$  \hspace{1cm} (Eq. E2.4-1)

For $L/t \geq 25$:

$$P_n = 0.75 tLF_u$$  \hspace{1cm} (Eq. E2.4-2)

(b) For transverse loading:

$$P_n = tLF_u$$  \hspace{1cm} (Eq. E2.4-3)

where $t =$ Least value of $t_1$ or $t_2$, as shown in Figures E2.4-1 and E2.4-2

In addition, for $t > 0.10$ in. (2.54 mm), the nominal strength [resistance] determined above shall not exceed the following value of $P_n$:

$$P_n = 0.75 twLF_{xx}$$  \hspace{1cm} (Eq. E2.4-4)

where

- $P_n =$ Nominal strength [resistance] of fillet weld
- $L =$ Length of fillet weld
- $tw =$ Effective throat = $0.707 w_1$ or $0.707 w_2$, whichever is smaller. A larger effective throat shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of $tw$.  

---

Figure E2.4-1 Fillet Welds – Lap Joint

Figure E2.4-2 Fillet Welds – T Joint
w₁ and w₂ = leg of weld (see Figures E2.4-1 and E2.4-2). \( w₁ \leq t₁ \) in lap joints.

\( F_u \) and \( F_{XX} \) are defined in Section E2.2.1.

**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 nominal shear strength [resistance], \( P_n \), of a flare groove weld shall be determined as follows:

\[
P_n = 0.833tLF_u
\]

(Eq. E2.5-1)

USA and Mexico Canada

<table>
<thead>
<tr>
<th>( \Omega \text{(ASD)} )</th>
<th>( \phi \text{(LRFD)} )</th>
<th>( \phi \text{(LSD)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.55</td>
<td>0.60</td>
<td>0.50</td>
</tr>
</tbody>
</table>

![Figure E2.5-1 Flare-Bevel Groove Weld](image1)

![Figure E2.5-2 Shear in Flare Bevel Groove Weld](image2)
(b) For flare groove welds, longitudinal loading (see Figures E2.5-2 through E2.5-7):

(1) For $t \leq t_w < 2t$ or if the lip height, $h$, is less than weld length, $L$:

$$P_n = 0.75t_L F_u$$

(Eq. E2.5-2)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega (ASD)$</td>
<td>$\phi (LRFD)$</td>
</tr>
<tr>
<td>2.80</td>
<td>0.55</td>
</tr>
</tbody>
</table>

(2) For $t_w \geq 2t$ and the lip height, $h$, is equal to or greater than weld length $L$:

$$P_n = 1.50t_L F_u$$

(Eq. E2.5-3)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega (ASD)$</td>
<td>$\phi (LRFD)$</td>
</tr>
<tr>
<td>2.80</td>
<td>0.55</td>
</tr>
</tbody>
</table>

In addition, for $t > 0.10$ in. (2.54 mm), the nominal strength [resistance] determined above shall not exceed the following value of $P_n$:

$$P_n = 0.75t_w L F_{xx}$$

(Eq. E2.5-4)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega (ASD)$</td>
<td>$\phi (LRFD)$</td>
</tr>
<tr>
<td>2.55</td>
<td>0.60</td>
</tr>
</tbody>
</table>

where

- $P_n$ = Limiting nominal strength [resistance] of weld
- $h$ = Height of lip
- $L$ = Length of weld
- $t_w$ = Effective throat of flare groove weld filled flush to surface (See Figures E2.5-4 and E2.5-5):
  - For flare bevel groove weld = $5/16$R
  - For flare V-groove weld = $1/2R$ ($3/8$R when $R > 1/2$ in. (12.7mm))
  - Effective throat of flare groove weld not filled flush to surface = 0.707$tw_1$ or 0.707$tw_2$, whichever is smaller. (See Figures E2.5-6 and E2.5-7.)
  - A larger effective throat than those above shall be permitted if...
measurement shows that the welding procedure to be used consistently yields a larger value of \( t_w \).

\[ R = \text{Radius of outside bend surface.} \]

\[ w_1 \text{ and } w_2 = \text{Leg of weld (see Figures E2.5-6 and E2.5-7).} \]

\( F_u \text{ and } F_{xx} \) are defined in Section E2.2.1.

---

**E2.6 Resistance Welds**

The nominal shear strength [resistance], \( P_n \), of spot welds shall be determined as follows:

When \( t \) is in inches and \( P_n \) is in kips:

For \( 0.01 \text{ in.} \leq t < 0.14 \text{ in.} \):

\[ P_n = 144t^{1.47} \]  

\( (Eq. \ E2.6-1) \)

For \( 0.14 \text{ in.} \leq t \leq 0.18 \text{ in.} \):

\[ P_n = 43.4t + 1.93 \]  

\( (Eq. \ E2.6-2) \)

When \( t \) is in millimeters and \( P_n \) is in kN:
For $0.25 \text{mm} \leq t < 3.56 \text{mm}$:
\[
P_n = 5.51t^{1.47}
\]  
(Eq. E2.6-3)

For $3.56 \text{mm} \leq t \leq 4.57 \text{mm}$:
\[
P_n = 7.6t + 8.57
\]  
(Eq. E2.6-4)

When $t$ is in centimeters and $P_n$ is in kg:

For $0.025 \text{cm} \leq t < 0.356 \text{cm}$:
\[
P_n = 16600t^{1.47}
\]  
(Eq. E2.6-5)

For $0.356 \text{cm} \leq t \leq 0.457 \text{cm}$:
\[
P_n = 7750t + 875
\]  
(Eq. E2.6-6)

where $t$ = Thickness of thinnest outside sheet.

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega$ (ASD)</td>
<td>$\phi$ (LRFD)</td>
</tr>
<tr>
<td>2.35</td>
<td>0.65</td>
</tr>
</tbody>
</table>

**E2.7 Fracture in Net Section of Members other than Flat Sheets (Shear Lag)**

The nominal tensile strength [resistance] of a welded member shall be determined in accordance with Section C2. For fracture and/or yielding in the effective net section of the connected part, the nominal tensile strength [resistance], $P_n$, shall be determined as follows:

\[
P_n = A_e F_u
\]  
(Eq. E2.7-1)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega$ (ASD)</td>
<td>$\phi$ (LRFD)</td>
</tr>
<tr>
<td>2.50</td>
<td>0.60</td>
</tr>
</tbody>
</table>

$F_u$ = Tensile strength of the connected part as specified in Section A2.1 or A2.3.2

$A_e = AU$, effective net area with $U$ defined as follows:

When the load is transmitted only by transverse welds:
\[
A = \text{Area of directly connected elements}
\]
\[
U = 1.0
\]

When the load is transmitted only by longitudinal welds or by longitudinal welds in combination with transverse welds:
\[
A = \text{Gross area of member, } A_g
\]
\[
U = 1.0 \text{ for members when load is transmitted directly to all of the cross sectional elements. Otherwise the reduction coefficient } U \text{ is determined as follows:}
\]

(a) For angle members:
\[
U = 1.0 - 1.20 \frac{\bar{x}}{L} < 0.9
\]  
(Eq. E2.7-2)

but $U$ shall not be less than 0.4.

(b) For channel members
\[
U = 1.0 - 0.36 \left( \frac{E_k}{E} \right) \frac{\bar{x}}{L} < 0.9
\]  
(Eq. E2.7-3)

but $U$ shall not be less than 0.5.

$\bar{x}$ = Distance from shear plane to centroid of cross section
L = Length of longitudinal weld

**E3 Bolted Connections**

The following design criteria and the requirements stipulated in Section E3a of Appendix A, B, and C govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm), refer to the specifications and standards stipulated in Section E3a of Appendix A, B, or C.

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

- ASTM A194/A194M, Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service
- ASTM A307(Type A), Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength
- ASTM A325, Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
- ASTM A325M, High Strength Bolts for Structural Steel Joints [Metric]
- ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 in.)
- ASTM A449, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 1/2 in.)
- ASTM A490, Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength
- ASTM A490M, High Strength Steel bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]
- ASTM A563, Carbon and Alloy Steel Nuts
- ASTM A563M, Carbon and Alloy Steel Nuts [Metric]
- ASTM F436, Hardened Steel Washers
- ASTM F436M, Hardened Steel Washers [Metric]
- ASTM F844, Washers, Steel, Plain (Flat), Unhardened for General Use
- ASTM F959, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners
- ASTM F959M, Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

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

Bolts shall be installed and tightened to achieve satisfactory performance of the connections.

**E3.1 Shear, Spacing and Edge Distance**

The provisions of this section are given in Section E3.1 of the Appendices.
### E3.2 Fracture in Net Section (Shear Lag)

The provisions of this section are given in Section E3.2 of the Appendices.

### E3.3 Bearing

The design bearing strength [factored resistance] of bolted connections shall be determined according to Sections E3.3.1 and E3.3.2. For conditions not shown, the design bearing strength [factored resistance] of bolted connections shall be determined by tests.

#### E3.3.1 Strength [Resistance] without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the nominal bearing strength [resistance], $P_n$, of the connected sheet for each loaded bolt shall be determined as follows:

$$P_n = m_f C d t F_u$$

(Eq. E3.3.1-1)

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Omega$(ASD)</td>
<td>$\phi$(LRFD)</td>
</tr>
<tr>
<td>2.50</td>
<td>0.60</td>
</tr>
</tbody>
</table>

where

- $C$ = Bearing factor, which shall be determined according to Table E3.3.1-1
- $d$ = Nominal bolt diameter
- $t$ = Uncoated sheet thickness
- $F_u$ = Tensile strength of sheet as defined in Section A2.1 or A2.2
- $m_f$ = Modification factor for type of bearing connection, which shall be determined according to Table E3.3.1-2

#### Table E3.3.1-1

<table>
<thead>
<tr>
<th>Thickness of Connected Part, $t$, in. (mm)</th>
<th>Ratio of Fastener Diameter to Member Thickness, $d/t$</th>
<th>$C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.024 \leq t &lt; 0.1875$ (0.61 $\leq t &lt; 4.76$)</td>
<td>$d/t &lt; 10$</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>$10 \leq d/t \leq 22$</td>
<td>4.0 - 0.1($d/t$)</td>
</tr>
<tr>
<td></td>
<td>$d/t &gt; 22$</td>
<td>1.8</td>
</tr>
</tbody>
</table>
Table E3.3.1-2  
Modification Factor, $m_f$, for Type of Bearing Connection

<table>
<thead>
<tr>
<th>Type of Bearing Connection</th>
<th>$m_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Shear and Outside Sheets of Double Shear Connection with Washers under Both Bolt Head and Nut</td>
<td>1.00</td>
</tr>
<tr>
<td>Single Shear and Outside Sheets of Double Shear Connection without Washers under Both Bolt Head and Nut, Or with only One Washer</td>
<td>0.75</td>
</tr>
<tr>
<td>Inside Sheet of Double Shear Connection with or without Washers</td>
<td>1.33</td>
</tr>
</tbody>
</table>

### E3.3.2 Strength [Resistance] with Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the nominal bearing strength [resistance], $P_n$, shall also be limited by the following values:

$$P_n = (4.64\alpha t + 1.53)dtF_u$$  
*(Eq. E3.3.2-1)*

<table>
<thead>
<tr>
<th>USA and Mexico</th>
<th>Canada</th>
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<tbody>
<tr>
<td>$\Omega (ASD)$</td>
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<tr>
<td>2.22</td>
<td>0.65</td>
</tr>
</tbody>
</table>

where

- $\alpha = $ Coefficient for conversion of units
- $= 1$ for US customary units (with $t$ in inches)
- $= 0.0394$ for SI units (with $t$ in mm)
- $= 0.394$ for MKS units (with $t$ in cm)

Other symbols are defined in Section E3.3.1.

### E3.4 Shear and Tension in Bolts

The provisions under this section are provided in Section E3.4 of the Appendices.

### E4 Screw Connections

All E4 requirements shall apply to screws with $0.08 \text{ in.} \leq d \leq 0.25 \text{ in.}$ ($2.03 \text{ mm} \leq d \leq 6.35 \text{ mm}$). The screws shall be thread-forming or thread-cutting, with or without a self-drilling point. Screws shall be installed and tightened in accordance with the manufacturer’s recommendations.

The nominal screw connection strengths [resistances] shall also be limited by Section C2.

For diaphragm applications, Section D5 shall be used.

The following factor of safety or resistance factor shall be used for the subsections of Chapter E4.
Alternatively, design values for a particular application shall be permitted to be based on tests, with the factor of safety, $\Omega$, and the resistance factor, $\phi$, determined according to Chapter F.

The following notation applies to this section:
- $d$ = Nominal screw diameter
- $d_w$ = Larger of screw head diameter or washer diameter
- $P_{ns}$ = Nominal shear strength [resistance] per screw
- $P_{ss}$ = Nominal shear strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing
- $P_{nt}$ = Nominal tension strength [resistance] per screw
- $P_{not}$ = Nominal pull-out strength [resistance] per screw
- $P_{nov}$ = Nominal pull-over strength [resistance] per screw
- $P_{ts}$ = Nominal tension strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing
- $t_1$ = Thickness of member in contact with screw head
- $t_2$ = Thickness of member not in contact with screw head
- $t_c$ = Lesser of depth of penetration and thickness $t_2$
- $F_{u1}$ = Tensile strength of member in contact with screw head
- $F_{u2}$ = Tensile strength of member not in contact with screw head

### E4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

### E4.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge of any part shall not be less than 1.5d. If the end distance is parallel to the force on the fastener, the nominal shear strength per screw, $P_{ns}$, shall be limited by Section E4.3.2.

### E4.3 Shear

#### E4.3.1 Connection Shear Limited by Tilting and Bearing

The nominal shear strength [resistance] per screw, $P_{ns}$, shall be determined as follows:

For $t_2/t_1 \leq 1.0$, $P_{ns}$ shall be taken as the smallest of

- $P_{ns} = 4.2 (t_2^2 d)^{1/2} F_{u2}$  
  \[Eq. E4.3.1-1\]
- $P_{ns} = 2.7 t_1 d F_{u1}$  
  \[Eq. E4.3.1-2\]
- $P_{ns} = 2.7 t_2 d F_{u2}$  
  \[Eq. E4.3.1-3\]
For $t_2/t_1 \geq 2.5$, $P_{ns}$ shall be taken as the smaller of
\[ P_{ns} = 2.7 \ t_1 \ d \ F_{u1} \]  
\[ P_{ns} = 2.7 \ t_2 \ d \ F_{u2} \]  
(Eq. E4.3.1-4)
(Eq. E4.3.1-5)

For $1.0 < t_2/t_1 < 2.5$, $P_{ns}$ shall be determined by linear interpolation between the above two cases.

**E4.3.2 Connection Shear Limited by End Distance**

The provisions of this section are given in Section E4.3.2 of the Appendices.

**E4.3.3 Shear in Screws**

The nominal shear strength [resistance] of the screw shall be calculated as follows:
\[ P_{ns} = 0.8P_{ss} \]  
(Eq. E4.3.3-1)

**E4.4 Tension**

For screws which carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter $d_w$ not less than $5/16$ in. (7.94 mm). Washers shall be at least 0.050 in. (1.27 mm) thick.

**E4.4.1 Pull-Out**

The nominal pull-out strength [resistance], $P_{not}$, shall be calculated as follows:
\[ P_{not} = 0.85 \ t_c \ d \ F_{u2} \]  
(Eq. E4.4.1-1)

**E4.4.2 Pull-Over**

The nominal pull-over strength [resistance], $P_{nov}$, shall be calculated as follows:
\[ P_{nov} = 1.5 \ t_1 \ d_w \ F_{u1} \]  
where $d_w$ shall be taken not larger than 1/2 in. (12.7 mm).

(Eq. E4.4.2-1)

**E4.4.3 Tension in Screws**

The nominal tension strength [resistance], $P_{nt}$, per screw shall be calculated as follows:
\[ P_{nt} = 0.8 \ P_{ts} \]  
(Eq. E4.4.3-1)

**E5 Rupture**

The provisions provided under this section are given in Section E5 of the Appendices.
E6 Connections to Other Materials

E6.1 Bearing

Proper provisions shall be made to transfer bearing forces from steel components covered by the Specification to adjacent structural components made of other materials.

E6.2 Tension

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

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

E6.3 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 [resistance] on the steel components shall not exceed that allowed by this Specification. The design shear strength [resistance] on the fasteners and other material shall not be exceeded. Embedment 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. Refer to Section D5.

F1 Tests for Determining Structural Performance

F1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance which is required to be established by tests shall be evaluated in accordance with the following performance procedure:
(a) 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 three identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent. If such deviation from the average value exceeds 15 percent, more tests of the same kind shall be made until the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent, or until at least three additional tests have been made. No test result shall be eliminated unless a rationale for its exclusion can be given. The average value of all tests made shall then be regarded as the nominal strength [resistance], $R_n$, for the series of the tests. $R_n$ and the coefficient of variation $V_P$ of the test results shall be determined by statistical analysis.

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

$$\sum \gamma_i Q_i \leq \phi R_n \quad \text{for LRFD} \quad \text{(Eq. F1.1-1a)}$$

$$\phi R_n \geq \sum \gamma_i Q_i \quad \text{for LSD} \quad \text{(Eq. F1.1-1b)}$$

where

$\Sigma \gamma_i Q_i =$ Required strength [effect of factored loads] based on the most critical load combination determined in accordance with Section A5.1.2. $\gamma_i$ and $Q_i$ are load factors and load effects, respectively.

$R_n =$ Average value of all test results

$\phi =$ Resistance factor

$$= C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{\frac{V_M^2 + V_F^2 + C_P V_F^2 + V_Q^2}{Q}}} \quad \text{(Eq. F1.1-2)}$$

$C_\phi =$ Calibration coefficient

= 1.52 for the United States and Mexico

= 1.42 for Canada

$M_m =$ Mean value of material factor, $M$, listed in Table F1 for type of component involved

$F_m =$ Mean value of fabrication factor, $F$, listed in Table F1 for type of component involved

$P_m =$ Mean value of professional factor, $P$, for tested component

= 1.0
\( \beta_0 \) = Target reliability index
- 2.5 for structural members and 3.5 for connections for the United States and Mexico
- 3.0 for structural members and 4.0 for connections for Canada

\( V_M \) = Coefficient of variation of material factor listed in Table F1 for type of component involved

\( V_F \) = Coefficient of variation of fabrication factor listed in Table F1 for type of component involved

\( C_P \) = Correction factor
- \((1+1/n)m/(m-2)\) for \( n \geq 4 \), and 5.7 for \( n = 3 \)  
(Eq. F1.1-3)

\( V_P \) = Coefficient of variation of test results, but not less than 6.5%

\( m \) = Degrees of freedom
- \( n-1 \)

\( n \) = Number of tests

\( V_Q \) = Coefficient of variation of load effect
- 0.21

\( e \) = Natural logarithmic base
- 2.718...

Note:
* For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced, \( \phi \) shall be determined with the following coefficients:
  - For the United States and Mexico, \( C_\phi = 1.6, \beta_0 = 1.5 \) and \( V_Q = 0.43 \).
  - For Canada, \( C_\phi = 1.42, \beta_0 = 3.0 \) and \( V_Q = 0.21 \).

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 A2.1, 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. F1.1-1, except that the resistance factor \( \phi \) is taken as unity and that the load factor for dead load is 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.
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-Torsional 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>
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<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>
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<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>
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<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Structural Members Not Listed Above</td>
<td>1.00</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>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Shear Strength of Welds</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.15</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>Arc Seam Welds</td>
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<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>
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<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Fillet Welds</td>
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<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>
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<tr>
<td>Flare Groove Welds</td>
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<td></td>
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<tr>
<td>Shear Strength of Welds</td>
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<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><strong>Bolted Connections</strong></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>
<tr>
<td><strong>Screw Connections</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum Spacing and Edge Distance</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Tension Strength on Net Section</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Bearing Strength</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Connections Not Listed Above</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.15</td>
</tr>
</tbody>
</table>
F1.2 Allowable Strength Design

Where the composition or configuration of elements, assemblies, connections or details of cold-formed steel structural members are such that calculation of their strength cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with Section F1.1, except as modified in this section for allowable strength design.

The allowable design strength shall be calculated as:

\[ R = \frac{R_n}{\Omega} \]  \hspace{1cm} (Eq. F1.2-1)

where
\[ R_n = \text{Average value of all test results} \]
\[ \Omega = \text{Factor of safety to be computed as follows:} \]
\[ \Omega = \frac{1.6}{\phi} \]  \hspace{1cm} (Eq. F1.2-2)

in which \( \phi \) is evaluated in accordance with Section F1.1.

The required allowable strength shall be determined from nominal loads and load combinations as described in Section A4.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies for which the nominal strength [resistance] can be computed according to this Specification or its specific references, confirmatory tests shall be permitted to be made to demonstrate the strength is not less than the nominal strength [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 A7.2 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.

(d) For acceptance and control purposes, one full section test shall be made from each master coil.

(e) At the option of the manufacturer, either tension or compression tests shall be permitted to 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 A7.2 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 A2.1 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 A7.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each master coil for the establishment of the representative values of the virgin tensile yield point and tensile strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.
G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

This design procedure shall apply to cold-formed steel members and connections subject to cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure (fatigue).

G1 General

When cyclic loading is a design consideration, the provisions of this Chapter apply to stresses calculated on the basis of unfactored loads. The maximum permitted tensile stress due to unfactored loads is 0.6 $F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the unfactored live load. In the case of a stress reversal, the stress range shall be computed as the sum of the absolute values of maximum repeated tensile and compressive stresses or the sum of the absolute values of maximum shearing stresses of opposite direction at the point of probable crack initiation.

The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. Therefore, evaluation of fatigue resistance is not required for wind load applications in buildings. If the live load stress range is less than the threshold stress range, $F_{TH}$, given in Table G1, evaluation of fatigue resistance is also not required.

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant $C_f$</th>
<th>Threshold $F_{TH}$ ksi (MPa) [kg/cm^2]</th>
<th>Reference Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-received base metal and components with as-rolled surfaces, including sheared edges and cold-formed corners.</td>
<td>I</td>
<td>3.2x10^{10}</td>
<td>25 (172) [1760]</td>
<td>G1-1</td>
</tr>
<tr>
<td>As-received base metal and weld metal in members connected by continuous longitudinal welds.</td>
<td>II</td>
<td>1.0x10^{10}</td>
<td>15 (103) [1050]</td>
<td>G1-2</td>
</tr>
<tr>
<td>Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than and equal to 2 in. (50.8 mm). Bolt and screw connections and spot welds.</td>
<td>III</td>
<td>3.2x10^{9}</td>
<td>16 (110) [1120]</td>
<td>G1-3, G1-4</td>
</tr>
<tr>
<td>Longitudinal fillet welded attachments greater than 2 in. (50.8 mm) parallel to the direction of the applied stress, and intermittent welds parallel to the direction of the applied force.</td>
<td>IV</td>
<td>1.0x10^{9}</td>
<td>9 (62) [633]</td>
<td>G1-4</td>
</tr>
</tbody>
</table>
Evaluation of fatigue resistance is not required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Chapter is applicable to structures with suitable corrosion protection or subject only to non-aggressive atmospheres.

The cyclic load resistance determined by the provisions of this Chapter is applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall provide, either complete details including weld sizes, or shall specify the planned cycle life and the maximum range of moments, shears, and reactions for the connections.

G2 Calculation of Maximum Stresses and Stress Ranges

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if applicable.

In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of applied load.

For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially stressed angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

G3 Design Stress Range

The range of stress at service loads shall not exceed the design stress range computed using Equation G3-1.

For all stress categories,

\[
F_{SR} = \left( \alpha C_f/N \right)^{0.333} \geq F_{TH} \quad (Eq. \ G3-1)
\]

where

- \( F_{SR} \) = Design stress range
- \( C_f \) = Constant from Table G1
- \( N \) = Number of stress range fluctuations in design life
- \( F_{TH} \) = Threshold fatigue stress range, maximum stress range for indefinite design life from Table G1
- \( \alpha \) = Coefficient for conversion of units
G4  Bolts and Threaded Parts

For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation G3-1. The factor $C_f$ shall be taken as $22 \times 10^8$. The threshold stress, $F_{TH}$, shall be taken as 7 ksi (48 MPa or 492 kg/cm²).

For not-fully-tightened high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation G3-1. The factor $C_f$ shall be taken as $3.9 \times 10^8$. The threshold stress, $F_{TH}$, shall be taken as 7 ksi (48 MPa or 492 kg/cm²). The net tensile area is given by Equation G4-1.

$$A_t = \frac{\pi}{4} \left[d_b - (0.9743/n)\right]^2 \quad (Eq. G4-1)$$

For SI or MKS units:

$$A_t = \frac{\pi}{4} \left[d_b - (0.9382P)\right]^2 \quad (Eq. G4-1a)$$
where:

\[ A_t = \text{Net tensile area} \]
\[ d_b = \text{Nominal diameter (body or shank diameter)} \]
\[ n = \text{Number of threads per inch} \]
\[ P = \text{Pitch (mm per thread for SI units and cm per thread for MKS units)} \]

**G5 Special Fabrication Requirements**

Backing bars in welded connections that are parallel to the stress field are permitted to remain in place, and if used, shall be continuous.

Backing bars that are perpendicular to the stress field, if used, shall be removed and the joint back gouged and welded.

Flame cut edges subject to cyclic stress ranges shall have a surface roughness not to exceed 1,000 \( \mu \text{in.} \) (25 \( \mu \text{m} \)), where the reference standard is ASME B46.1.

Re-entrant corners at cuts, copes and weld access holes shall form a radius of not less than 3/8 in. (9.53 mm), by pre-drilling or sub-punching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal contour to provide a radiused transition, free of notches, with a surface roughness not to exceed 1,000 \( \mu \text{in.} \) (25 \( \mu \text{m} \)), where the reference standard is ASME B46.1 or other equivalent standards shall be referenced.

![Typical Plate](image)

(a) Transverse Welds, Category III  
(b) Longitudinal Welds  
For Category III, \( L \leq 2 \text{ in.} \) (50.8 mm)  
For Category IV, 2 in. (50.8 mm) < \( L \leq 4 \text{ in.} \) (102 mm)

**Figure G1-3 Typical Attachments for Categories III and IV**

For transverse butt joints in regions of high tensile stress, weld tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member. Exception: Weld tabs are not required for sheet material if the welding procedures used result in smooth, flush edges.
Figure G1-4 Typical Attachments for Category III

(c) Arc Spot or Plug Weld

(c) Screws
Appendix A:
Provisions Applicable to
the United States

2001 EDITION
PREFACE TO APPENDIX A:

Appendix A provides specification provisions that are only applicable to the United States. Included are items of a broad nature such as provisions for the design method to be used, ASD or LRFD, and provisions to use ASCE 7 for loads and load combinations where there is not an applicable building code. Reference documents that are not used by all three countries are listed here as well.

Also included in Appendix A are technical items where full agreement between the three countries was not reached. Such items included certain provisions pertaining to the design of

- beams (C and Z sections) for standing seam roofs,
- bolted connections, and
- tension members

Efforts will be made to minimize these differences in future editions of the Specification.
APPENDIX A: PROVISIONS APPLICABLE TO THE UNITED STATES

This Appendix provides design provisions or supplements to Chapters A through G that are only applicable to the United States. A section number ending with a letter indicates that the provisions herein supplement the corresponding section in Chapters A through G of the Specification. A section number not ending with a letter indicates that the section gives the entire design provision.

A1.1a Scope and Limits of Applicability

Designs shall be made according to the provisions for Load and Resistance Factor Design, or to the provisions for Allowable Strength Design. Where allowed, both methods are equally acceptable although they may or may not produce identical designs. However, the two methods shall not be mixed in designing the various cold-formed steel components of a structure.

A2.2 Other Steels

The listing in Section A2.1 does not exclude the use of steel up to and including one in. (25.4 mm) 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 A2.3.

A3 Loads

A3.1 Nominal Loads

The nominal loads shall be as stipulated by the applicable building code under which the structure is designed or as dictated by the conditions involved. In the absence of a building code, the nominal loads shall be those stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

A4.1.2 Load Combinations for ASD

The structure and its components shall be designed so that allowable design strengths equal or exceed the effects of the nominal loads and load combinations as stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

The combined effects of two or more loads, excluding dead load, shall be permitted to be multiplied by 0.75. The combined load used in design shall not be less than the sum of the effects of dead load and any
single load that produces the largest effect. The above 0.75 load reduction shall not be used where similar load reductions are permitted by the applicable building code or ASCE 7.

Exception: When evaluating diaphragms using the provisions of Section D5, no decrease in forces is permitted for load combinations including wind or earthquake loads.

**A5.1.2 Load Factors and Load Combinations for LRFD**

The structure and its components shall be designed so that design strengths equal or exceed the effects of the factored nominal loads and load combinations stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

**A9a Referenced Documents**

The following documents are referenced in Appendix A:

1. American Society of Civil Engineers, ASCE 7-98, “Minimum Design Loads in Buildings and Other Structures,” American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191

**C2 Tension Members**

For axially loaded tension members, the nominal tensile strength, $T_n$, shall be the smallest value obtained according to the limit states of (a) yielding in the gross section, (b) fracture in the net section away from connections, and (c) fracture in the effective net section at the connection:
(a) For yielding:
\[ T_n = A_g F_y \]
\[ \Omega_t = 1.67 \quad (ASD) \]
\[ \phi_t = 0.90 \quad (LRFD) \]

(b) For fracture away from connection:
\[ T_n = A_n F_u \]
\[ \Omega_t = 2.00 \quad (ASD) \]
\[ \phi_t = 0.75 \quad (LRFD) \]

where
- \( T_n \) = Nominal strength of member when loaded in tension
- \( A_g \) = Gross area of cross section
- \( A_n \) = Net area of cross section
- \( F_y \) = Design yield point as determined in Section A7.1
- \( F_u \) = Tensile strength as specified in Section A2.1 or A2.3.2

(c) For fracture at connection:
The nominal tensile strength shall also be limited by Sections E2.7, E3, and E5 for tension members using welded connections, bolted connections, and screw connections.

**C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System**

The nominal flexural strength, \( M_n \), of a C- or Z-section, loaded in a plane parallel to the web with the top flange supporting a standing seam roof system shall be determined using discrete point bracing and the provisions of Section C3.1.2.1 or shall be calculated as follows:
\[ M_n = R S_e F_y \]
\[ \Omega_b = 1.67 \quad (ASD) \]
\[ \phi_b = 0.90 \quad (LRFD) \]

where
- \( R \) = Reduction factor determined by the "Base Test Method for Purlins Supporting a Standing Seam Roof System" of Part VIII of the AISI Cold-Formed Steel Design Manual.
- \( S_e \) and \( F_y \) are defined in Section C3.1.1.

**E2a Welded Connections**

For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in. (4.57 mm), refer to the AISC “Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design”, or the “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 in. (4.57 mm) or less in thickness shall be made in accordance with the AWS D1.3 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 E2a.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1 or AWS C1.3.

### TABLE E2a

<table>
<thead>
<tr>
<th>Welding Position</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>Connection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sheet to Sheet</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
</tr>
<tr>
<td>Sheet to Supporting Member</td>
<td></td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>V</td>
<td>V</td>
<td>V</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
<td></td>
</tr>
</tbody>
</table>

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

### E3a Bolted Connections

In addition to the design criteria given in Section E3 of the Specification, the following design requirements shall also be followed for bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm), refer to AISC “Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design”, or the “Load and Resistance Factor Design Specification for Structural Steel Buildings”.

The holes for bolts shall not exceed the sizes specified in Table E3a, except that larger holes may be used in column base details or structural systems connected to concrete walls.

### TABLE E3a

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d (in.)</th>
<th>Standard Hole Diameter, d_h (in.)</th>
<th>Oversized Hole Diameter, d_h (in.)</th>
<th>Short-Slotted Hole Dimensions in.</th>
<th>Long-Slotted Hole Dimensions in.</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>
TABLE E3a
Maximum Size of Bolt Holes, millimeters

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d (mm)</th>
<th>Standard Hole Diameter, d_h (mm)</th>
<th>Oversized Hole Diameter, d_h (mm)</th>
<th>Short-Slotted Hole Dimensions (mm)</th>
<th>Long-Slotted Hole Dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 12.7</td>
<td>d + 0.8</td>
<td>d + 1.6</td>
<td>(d + 0.8) by (d + 6.4)</td>
<td>(d + 0.8) by (2^{1/2}d)</td>
</tr>
<tr>
<td>≥ 12.7</td>
<td>d + 1.6</td>
<td>d + 3.2</td>
<td>(d + 1.6) by (d + 6.4)</td>
<td>(d + 1.6) 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 slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Chapter F.

**E3.1 Shear, Spacing and Edge Distance**

The nominal shear strength, \( P_n \), of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated as follows:

\[
P_n = teF_u \tag{Eq. E3.1-1}
\]

(a) When \( F_u/F_{sy} \geq 1.08 \):

\[
\Omega = 2.00 \quad \text{(ASD)}
\]

\[
\phi = 0.70 \quad \text{(LRFD)}
\]

(b) When \( F_u/F_{sy} < 1.08 \):

\[
\Omega = 2.22 \quad \text{(ASD)}
\]

\[
\phi = 0.60 \quad \text{(LRFD)}
\]

where

\( P_n \) = Nominal strength per bolt

\( e \) = Distance measured in line of force from center of a standard hole to nearest edge of adjacent hole or to end of connected part

\( t \) = Thickness of thinnest connected part

\( F_u \) = Tensile strength of connected part as specified in Section A2.1, A2.2 or A2.3.2

\( F_{sy} \) = Yield point of connected part as specified in Section A2.1, A2.2 or A2.3.2

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench. 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 E3a. In no case shall the clear distance 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 Fracture in Net Section (Shear Lag)**

The nominal tensile strength of a bolted member shall be determined in accordance with Section C2. For fracture in the effective net section of the connected part, the nominal tensile strength [$\text{resistance}$], $P_n$, shall be determined as follows:

1. For flat sheet connections not having staggered hole patterns:
   
   \[ P_n = A_n F_t \]  
   \[ \text{(Eq. E3.2-1)} \]
   
   (a) When washers are provided under both the bolt head and the nut:
   
   For a single bolt, or a single row of bolts perpendicular to the force
   \[ F_t = (0.1 + 3d/s) F_u \leq F_u \]  
   \[ \text{(Eq. E3.2-2)} \]
   
   For multiple bolts in the line parallel to the force
   \[ F_t = F_u \]  
   \[ \text{(Eq. E3.2-3)} \]
   
   For double shear:
   \[ \Omega = 2.00 \quad \text{(ASD)} \]
   \[ \phi = 0.65 \quad \text{(LRFD)} \]
   
   For single shear:
   \[ \Omega = 2.22 \quad \text{(ASD)} \]
   \[ \phi = 0.55 \quad \text{(LRFD)} \]

   (b) When either washers are not provided under the bolt head and the nut, or only one washer is provided under either the bolt head or the nut:
   
   For a single bolt, or a single row of bolts perpendicular to the force
   \[ F_t = (2.5d/s) F_u \leq F_u \]  
   \[ \text{(Eq. E3.2-4)} \]
   
   For multiple bolts in the line parallel to the force
   \[ F_t = F_u \]  
   \[ \text{(Eq. E3.2-5)} \]
   
   \[ \Omega = 2.22 \quad \text{(ASD)} \]
   \[ \phi = 0.65 \quad \text{(LRFD)} \]
   
   where
   \[ A_n = \text{Net area of connected part} \]
   \[ s = \text{Sheet width divided by number of bolt holes in cross section being analyzed (when evaluating } F_t) \]
   \[ F_u = \text{Tensile strength of connected part as specified in Section A2.1, A2.2 or A2.3.2} \]
   \[ d = \text{Nominal bolt diameter} \]

2. For flat sheet connections having staggered hole patterns:
   
   \[ P_n = A_n F_t \]  
   \[ \text{(Eq. E3.2-6)} \]
\[ \Omega = 2.22 \quad \text{(ASD)} \]
\[ \phi = 0.65 \quad \text{(LRFD)} \]

where

- \( F_t \) is determined in accordance with Eqs. E3.2-2 to E3.2-5.
- \( A_n = 0.90 [A_g - nbd_h t + (\sum s'^2/4g)t] \) \quad (Eq. E3.2-7)
- \( A_g = \text{Gross area of member} \)
- \( s' = \text{Longitudinal center-to-center spacing of any two consecutive holes} \)
- \( g = \text{Transverse center-to-center spacing between fastener gage lines} \)
- \( n_b = \text{Number of bolt holes in the cross section being analyzed} \)
- \( d_h = \text{Diameter of a standard hole} \)
- \( t \) is defined in Section E3.1.

(3) For other than flat sheet:

\[ P_n = A_e F_u \] \quad (Eq. E3.2-8)
\[ \Omega = 2.22 \quad \text{(ASD)} \]
\[ \phi = 0.65 \quad \text{(LRFD)} \]

where

- \( F_u = \text{Tensile strength of the connected part as specified in Section A2.1, A2.2 or A2.3.2} \)
- \( A_e = A_n U, \text{effective net area with } U \text{ defined as follows:} \)
- \( U = 1.0 \text{ for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, the reduction coefficient } U \text{ is determined as follows:} \)
  (a) For angle members having two or more bolts in the line of force
  \[ U = 1.0 - 1.20 \frac{x}{L} < 0.9 \] \quad (Eq. E3.2-9)
  but \( U \) shall not be less than 0.4.
  (b) For Channel members having two or more bolts in the line of force
  \[ U = 1.0 - 0.36 \frac{x}{L} < 0.9 \] \quad (Eq. E3.2-10)
  but \( U \) shall not be less than 0.5.
  \( x = \text{Distance from shear plane to centroid of the cross section} \)
  \( L = \text{Length of the connection} \)
  \( A_n = \text{Net area of connected part} \)

### E3.4 Shear and Tension in Bolts

The nominal bolt strength, \( P_n \), resulting from shear, tension or a combination of shear and tension shall be calculated as follows:

\[ P_n = A_b F_n \] \quad (Eq. E3.4-1)

where

- \( A_b = \text{Gross cross-sectional area of bolt} \)

When bolts are subject to shear or tension:

- \( F_n \) is given by \( F_nv \) or \( F_{nt} \) in Table E3.4-1.
- \( \Omega \) and \( \phi \) are given in Table E3.4-1.
The pullover strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved, see Section E6.2. When bolts are subject to a combination of shear and tension:

For ASD

- $F_n$ is given by $F'_{nt}$ in Table E3.4-2 or E3.4-4 (SI units)
- $\Omega$ is given in Table E3.4-2 or E3.4-4 (SI units)

For LRFD

- $F_n$ is given by $F'_{nt}$ in Table E3.4-3 or E3.4-5 (SI units)
- $\phi$ is given in Table E3.4-3 or E3.4-5 (SI units)
### TABLE E3.4-1
Nominal Tensile and Shear Strengths for Bolts

<table>
<thead>
<tr>
<th></th>
<th>Tensile Strength</th>
<th>Shear Strength*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Factor of Safety</td>
<td>Resistance Factor</td>
</tr>
<tr>
<td></td>
<td>Ω (ASD)</td>
<td>φ (LRFD)</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4 in. (6.4 mm) ≤ d</td>
<td>2.25</td>
<td>0.75</td>
</tr>
<tr>
<td>&lt; 1/2 in. (12.7 mm)</td>
<td>(279)</td>
<td></td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d ≥ 1/2 in (12.7 mm)</td>
<td>2.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(310)</td>
<td></td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A325 bolts, when threads are excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4 in. (6.4 mm) ≤ d</td>
<td>101.0</td>
<td>2.0</td>
</tr>
<tr>
<td>&lt; 1/2 in. (12.7 mm), when threads are not excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4 in. (6.4 mm) ≤ d</td>
<td>101.0</td>
<td>2.0</td>
</tr>
<tr>
<td>&lt; 1/2 in. (12.7 mm), when threads are excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4 in. (6.4 mm) ≤ d</td>
<td>81.0</td>
<td></td>
</tr>
<tr>
<td>&lt; 1/2 in. (12.7 mm), when threads are not excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/4 in. (6.4 mm) ≤ d</td>
<td>81.0</td>
<td></td>
</tr>
<tr>
<td>&lt; 1/2 in. (12.7 mm), when threads are excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are excluded from shear planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Applies to bolts in holes as limited by Table E3a. 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 Chapter F.
### TABLE E3.4-2 (ASD)
Nominal Tensile Stress, $F'_{nt}$ (ksi), for Bolts
Subjected 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>
<th>Factor of Safety $\Omega$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>110 - 3.6f$_V$ ≤ 90</td>
<td>110 - 2.8f$_V$ ≤ 90</td>
<td>2.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>122 - 3.6f$_V$ ≤ 101</td>
<td>122 - 2.8f$_V$ ≤ 101</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>100 - 3.6f$_V$ ≤ 81</td>
<td>100 - 2.8f$_V$ ≤ 81</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>136 - 3.6f$_V$ ≤ 112.5</td>
<td>136 - 2.8f$_V$ ≤ 112.5</td>
<td></td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>When 1/4 in. ≤ d &lt; 1/2 in.</td>
<td>52 - 4f$_V$ ≤ 40.5</td>
<td>52 - 4f$_V$ ≤ 40.5</td>
<td>2.25</td>
</tr>
<tr>
<td>When d ≥ 1/2 in.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The shear stress, f$_V$, shall also satisfy Table E3.4-1.

### TABLE E3.4-3 (LRFD)
Nominal Tensile Stress, $F'_{nt}$ (ksi), for Bolts
Subjected 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>
<th>Resistance Factor $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>113 - 2.4f$_V$ ≤ 90</td>
<td>113 - 1.9f$_V$ ≤ 90</td>
<td>0.75</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>127 - 2.4f$_V$ ≤ 101</td>
<td>127 - 1.9f$_V$ ≤ 101</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>101 - 2.4f$_V$ ≤ 81</td>
<td>101 - 1.9f$_V$ ≤ 81</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>141 - 2.4f$_V$ ≤ 112.5</td>
<td>141 - 1.9f$_V$ ≤ 112.5</td>
<td></td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></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>47 - 2.4f$_V$ ≤ 40.5</td>
<td>0.75</td>
</tr>
<tr>
<td>When d ≥ 1/2 in.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The shear stress, f$_V$, shall also satisfy Table E3.4-1.
### TABLE E3.4-4 (ASD)
Nominal Tensile Stress, $F'_{nt}$ (MPa), for Bolts Subjected 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>
<th>Factor of Safety $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>758 – 3.6$f_v$ ≤ 621</td>
<td>758 – 2.8$f_v$ ≤ 621</td>
<td>2.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>841 – 3.6$f_v$ ≤ 696</td>
<td>841 – 2.8$f_v$ ≤ 696</td>
<td>2.0</td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>690 – 3.6$f_v$ ≤ 558</td>
<td>690 – 2.8$f_v$ ≤ 558</td>
<td>2.0</td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>938 – 3.6$f_v$ ≤ 776</td>
<td>938 – 2.8$f_v$ ≤ 776</td>
<td>2.0</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td>359 - 4$f_v$ ≤ 279</td>
<td>2.25</td>
</tr>
<tr>
<td>When 6.4 mm ≤ d &lt; 12.7 mm</td>
<td></td>
<td>403 - 4$f_v$ ≤ 310</td>
<td></td>
</tr>
<tr>
<td>When d ≥ 12.7 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The shear stress, $f_v$, shall also satisfy Table E3.4-1.

### TABLE E3.4-5 (LRFD)
Nominal Tensile Stress, $F'_{nt}$ (MPa), for Bolts Subjected 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>
<th>Resistance Factor $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>779 – 2.4$f_v$ ≤ 621</td>
<td>779 – 1.9$f_v$ ≤ 621</td>
<td>0.75</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>876 - 2.4$f_v$ ≤ 696</td>
<td>876 - 1.9$f_v$ ≤ 696</td>
<td>0.75</td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>696 - 2.4$f_v$ ≤ 558</td>
<td>696 - 1.9$f_v$ ≤ 558</td>
<td>0.75</td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>972 - 2.4$f_v$ ≤ 776</td>
<td>972 - 1.9$f_v$ ≤ 776</td>
<td>0.75</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td>324 – 2.4$f_v$ ≤ 279</td>
<td>0.75</td>
</tr>
<tr>
<td>When 6.4 mm ≤ d &lt; 12.7 mm</td>
<td></td>
<td>359 – 2.4$f_v$ ≤ 310</td>
<td>0.75</td>
</tr>
<tr>
<td>When d ≥ 12.7 mm</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The shear stress, $f_v$, shall also satisfy Table E3.4-1.
E4.3.2 Connection Shear Limited by End Distance

The nominal shear strength per screw, \( P_{ns} \) shall not exceed that calculated as follows when the distance to an end of the connected part is parallel to the line of the applied force.

\[
P_{ns} = t e F_u \tag{Eq. E4.3.2-1}
\]

\[\Omega = 3.00 \quad \text{(ASD)}\]
\[\phi = 0.50 \quad \text{(LRFD)}\]

where

\( t \) = Thickness of part in which end distance is measured
\( e \) = Distance measured in line of force from center of a standard hole to nearest end of connected part.
\( F_u \) = Tensile strength of part in which end distance is measured.

E5 Rupture

E5.1 Shear Rupture

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the nominal shear strength, \( V_n \) shall be calculated as follows:

\[
V_n = 0.6 F_u A_{wn} \tag{Eq. E5.1-1}
\]

\[\Omega = 2.00 \quad \text{(ASD)}\]
\[\phi = 0.75 \quad \text{(LRFD)}\]

where

\( A_{wn} = (h_{wc} - n d_h) t \) \tag{Eq. E5.1-2}
\( h_{wc} \) = Coped flat web depth
\( n \) = Number of holes in critical plane
\( d_h \) = Hole diameter
\( F_u \) = Tensile strength of connected part as specified in Section A2.1 or A2.2
\( t \) = Thickness of coped web

E5.2 Tension Rupture

The nominal tensile rupture strength along a path in the affected elements of connected members shall be determined by Section E2.7 or E3.2 for welded or bolted connections, respectively.

E5.3 Block Shear Rupture

The nominal block shear rupture strength, \( R_n \) shall be determined as follows:

(a) When \( F_u A_{nt} \geq 0.6 F_u A_{nv} \)
\[
R_n = 0.6 F_y A_{gv} + F_u A_{nt} \tag{Eq. E5.3-1}
\]

(b) When \( F_u A_{nt} < 0.6 F_u A_{nv} \)
\[
R_n = 0.6 F_u A_{nv} + F_y A_{gt} \tag{Eq. E5.3-2}
\]
For bolted connections:
\[ \Omega = 2.22 \quad \text{(ASD)} \]
\[ \phi = 0.65 \quad \text{(LRFD)} \]

For welded connections:
\[ \Omega = 2.50 \quad \text{(ASD)} \]
\[ \phi = 0.60 \quad \text{(LRFD)} \]

where
- \( A_{gv} \) = Gross area subject to shear
- \( A_{gt} \) = Gross area subject to tension
- \( A_{nv} \) = Net area subject to shear
- \( A_{nt} \) = Net area subject to tension
Appendix B:

Provisions Applicable to

Canada

2001 EDITION
PREFACE TO APPENDIX B:

Appendix B provides specification provisions that are applicable only to Canada. Included are items of a general nature such as specific reference documents and provisions on loads and load combinations in accordance with the *National Building Code of Canada*.

While this document is referred to as a “Specification”, in Canada it is considered a “Standard”.

Also included in Appendix B are technical items where full agreement between the three countries was not reached. The most noteworthy of these items are:

- Beams (C and Z sections) for standing seam roofs,
- Bolted connections, and
- Tension members

Efforts will be made to minimize these differences in future editions of the *Specification*. 
APPENDIX B: PROVISIONS APPLICABLE TO CANADA

The material contained in this Appendix provides design provisions and supplements that, in addition to those in Chapters A through G, are mandatory for use in Canada. A section number ending with the letter “a” indicates that the provisions herein supplement the corresponding section in Chapters A through G of the Specification. A section number not ending with the letter “a” indicates that the section presents the entire design provision.

A1.2a Terms

The following additional definitions apply in Appendix B:

Importance factor (γ) - a factor applied to the factored loads, other than dead load, to take into account the consequences of collapse as related to the use and occupancy of the structure.

Load factor (α) - a factor applied to a specified load that, for the limit states under consideration, takes into account the variability in magnitude of the load, the loading patterns, and the analysis of their effects.

Load combination factor (ψ) - a factor applied to factored loads, other than dead load, to take into account the reduced probability of a number of loads from different sources acting simultaneously.

A2.1a Applicable Steels

These steels are in addition to those listed in Section A2.1:
CSA Standards G40.20-98/G40.21-98, General Requirements for Rolled or Welded Structural Quality Steel/Quality Steel.

A2.2 Other Steels

A2.2.1 Other Structural Quality Steels

For structural quality steels not listed in Section A2.1, Fy and Fu shall be the specified minimum values as given in the material Standard or published material Specification. These steels shall also meet the requirements of Section A2.3.

A2.2.2 Other Steels

For steels not covered by Section A2.1 of the Specification and A2.2.1 of this Appendix, tensile tests shall be conducted in accordance with Section F3. Fy and Fu shall be 0.8 times the yield strength and 0.8 times the tensile strength determined from the tests. These steels shall also meet the requirements of Section A2.3.

A2.4a Delivered Minimum Thickness

For hot-dipped metallic-coated material, the actual base steel thickness shall be taken as the measured coated thickness minus the coating allowance given in Table B-A2.4-1, and for pre-finished material, the organic coating
thickness shall also be subtracted. For electroplated material, the coating allowance shall be taken as zero.

Where more restrictive thickness tolerances are given in recognized product standards, or are part of a specific design or application, they shall take precedence.

### A3 Loads

The resistance factors adopted in this Specification are correlated with the loads and load factors for buildings specified in the National Building Code of Canada. For other cases, load factors shall be established in such a way that, in conjunction with the resistance factors used in this Specification, the required level of reliability is maintained.

#### A3.1 Specified Loads

The following loads, forces, and effects shall be considered in the design of cold-formed steel structural members and their connections:

- **D** dead loads, including the mass of the member and all permanent materials of construction, partitions, and permanent equipment, multiplied by the acceleration due to gravity to convert mass (kg) to force (N)
- **E** live load due to earthquake
- **L** live loads, including loads due to intended use and occupancy of the building, movable equipment, snow, ice, rain, soil, hydrostatic pressure, or impact
- **T** loads due to contraction or expansion caused by temperature changes
- **W** live load due to wind

---

**Table B-A2.4-1**

<table>
<thead>
<tr>
<th>Coating Designation</th>
<th>Thickness Allowance (mm)</th>
<th>Coating Designation</th>
<th>Thickness Allowance (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ZF001</td>
<td>0</td>
<td>A01</td>
<td>0</td>
</tr>
<tr>
<td>ZF075</td>
<td>0</td>
<td>A25</td>
<td>0</td>
</tr>
<tr>
<td>Z001</td>
<td>0</td>
<td>G01</td>
<td></td>
</tr>
<tr>
<td>Z180</td>
<td>0.025</td>
<td>G60</td>
<td>0.0010</td>
</tr>
<tr>
<td>Z275</td>
<td>0.040</td>
<td>G90</td>
<td>0.0015</td>
</tr>
<tr>
<td>Z350</td>
<td>0.050</td>
<td>G115</td>
<td>0.0020</td>
</tr>
<tr>
<td>Z450</td>
<td>0.065</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Z600</td>
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</tr>
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<td>Z700</td>
<td>0.100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AZM150</td>
<td>0.040</td>
<td>AZ50</td>
<td>0.0015</td>
</tr>
<tr>
<td>AZM165</td>
<td>0.045</td>
<td>AZ55</td>
<td>0.0018</td>
</tr>
<tr>
<td>AZM180</td>
<td>0.050</td>
<td>AZ60</td>
<td>0.0020</td>
</tr>
<tr>
<td>AZM210</td>
<td>0.055</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
A3.2 Temperature Effects

If it can be shown by engineering principles or if it is known from experience that neglect of some or all of the effects due to temperature, T, does not adversely affect structural safety or serviceability, these effects need not be considered in the calculations.

A6.1.2 Load Factors and Load Combinations for LSD

The effect of factored loads, in force units, is the structural effect due to specified loads multiplied by load factors, $\alpha$, defined in Section A6.1.2.1; a load combination factor, $\psi$, defined in Section A6.1.2.2; and an importance factor, $\gamma$, defined in Section A6.1.2.3. The combination of factored loads shall be taken as

$$\alpha_D D + \gamma(\alpha_L L + \alpha_W W + \alpha_T T) \quad (Eq. \ A6.1.2-1)$$

and

$$\alpha_D D + \gamma(\alpha_L L + \alpha_E E) \quad (Eq. \ A6.1.2-2)$$

A6.1.2.1 Load Factors ($\alpha$)

The load factors, $\alpha$, shall be taken as follows:

$\alpha_D = 1.25$, except in cases where the dead load resists overturning, uplift, or reversal of load effect, $\alpha_D = 0.85$, and in cases where the dead load is in combination with earthquake load, $\alpha_D = 1.0$

$\alpha_E = 1.0$

$\alpha_L = 1.50$, except when the live load is in combination with earthquake load, in which case $\alpha_L = 1.0$ for storage and assembly loads, and $\alpha_L = 0.5$ for all other live loads including snow

$\alpha_T = 1.25$

$\alpha_W = 1.50$

A6.1.2.2 Load Combination Factor ($\psi$)

The load combination factor, $\psi$, shall be taken as follows:

(a) when only one of L, W, or T act, $\psi = 1.00$;
(b) when two of L, W, or T act, $\psi = 0.70$; and
(c) when all of L, W, and T act, $\psi = 0.60$.

The most unfavourable effect shall be determined by considering L, W, and T acting alone with $\psi = 1.00$ or in combination with $\psi = 0.70$ or 0.60.

A6.1.2.3 Importance Factor ($\gamma$)

Unless otherwise specified, the importance factor, $\gamma$, shall be taken as follows:

(a) not less than 1.00 for all buildings except as noted in Item (b); and
(b) not less than 0.80 for
(i) farm buildings having low human occupancy, which is defined as an occupant load of not more than one person per 40 m\(^2\) of floor area during normal use; and
(ii) buildings for which it can be shown that collapse is not likely to cause injury or other serious consequences.

**A9a Reference Documents**

This Appendix refers to the following publications, and where such reference is made, it shall be to the edition listed below including all amendments published thereto:

1. **CSA Standards:**
   - CAN/CSA S16-02, *Limit States Design of Steel Structures*
   - W47.1-92 (R2001), *Certification of Companies for Fusion Welding of Steel Structures*
   - W55.3-1965 (R1998), *Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings*
   - W59-1989 (R2001), *Welded Steel Construction (Metal Arc Welding)*

2. **National Research Council of Canada:**

**C2 Tension Members**

The nominal tensile resistance, \(T_n\), shall be the lesser of the values determined in Sections C2.1 and C2.2 of this Appendix. The nominal tensile resistance shall also be limited by Sections E2.7 of the *Specification*, E3.2 of this Appendix and E3.3 of the *Specification* for tension members using welded, bolted and screw connections.

**C2.1 Yielding of Gross Section**

The nominal tensile resistance, \(T_n\), due to yielding of the gross section shall be determined as follows:

\[
T_n = A_g F_y \quad \phi_t = 0.90
\]

(Eq. C2.1-1)

where

- \(A_g\) = Gross area of cross section
- \(F_y\) = Yield point defined in Section A7.1

**C2.2 Fracture of Net Section**

The nominal tensile resistance, \(T_n\), due to fracture of the net section shall be determined as follows:

\[
T_n = A_n F_u \quad \phi_u = 0.75
\]

(Eq. C2.2-1)

where
\[ A_n = L_c t \]

= Critical net area of connected part

\[ F_u = \text{Tensile strength as specified in Section A2} \]

\[ L_c = \text{Summation of critical path lengths of each segment along a potential failure path of minimum strength.} \]

\[ L_c \text{ shall be determined as follows:} \]

(a) For failure normal to force due to direct tension:

\[ L_c = L_t \quad \text{not involving stagger} \quad (Eq. \ C2.2-2) \]

\[ L_c = 0.9L_s \quad \text{involving stagger} \quad (Eq. \ C2.2-3) \]

(b) For failure parallel to force due to shear:

\[ L_c = 0.6L_v \quad (Eq. \ C2.2-4) \]

(c) For failure due to block tear-out at end of member:

\[ L_c = L_t + 0.6L_v \quad \text{not involving stagger} \quad (Eq. \ C2.2-5) \]

\[ L_c = 0.9(L_t + L_s + 0.6L_v) \quad \text{involving stagger} \quad (Eq. \ C2.2-6) \]

where

\[ L_t = \text{Net failure path length normal to force due to direct tension} \]

\[ L_s = \text{Net failure path length inclined to force (including } \frac{s^2}{4g} \text{ allowance for staggered holes)} \]

\[ L_v = \text{Net failure path length parallel to force (i.e., in shear)} \]

\[ s = \text{Pitch, spacing of fastener parallel to force} \]

\[ g = \text{Gauge, spacing of fastener perpendicular to force} \]

\[ t = \text{Base steel thickness} \]

**C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System**

For this type of member, discrete bracing is required in accordance with Section D3.2.3 of this Appendix.

**D3a Lateral Bracing**

Structural members and assemblies shall be adequately braced to prevent collapse and to maintain their integrity during the anticipated service life of the structure. Care shall be taken to ensure that the bracing of the entire structural system is complete, particularly when there is interdependence between walls, floors, or roofs acting as diaphragms.

Erection diagrams shall show the details of the essential bracing requirements, including any details necessary to assure the effectiveness of the bracing or bracing system.

The spacing of braces shall not be greater than the unbraced length assumed in the design of the member or component being braced.

**D3.1a Symmetrical Beams and Columns**

The provisions of Sections D3.1.1 and D3.1.2 of this Appendix apply to symmetric sections in compression or bending in which the applied load does
not induce twist.

**D3.1.1 Discrete Bracing**

The factored resistance of braces shall be at least 2% of either the factored compressive force in a compressive member at the braced location or the factored compressive force in the compressive flange of a member in bending. When more than one brace acts at a common location and the nature of the braces is such that combined action is possible, the bracing force may be shared proportionately. The slenderness ratio of compressive braces shall not exceed 200.

**D3.1.2 Bracing by Deck, Slab, or Sheathing**

The factored resistance of the attachments along the entire length of the braced member shall be at least 5% of either the maximum factored compressive force in a compressive member or the maximum factored compressive force in the compressive flange of a member in bending.

**D3.2a C-Section and Z-Section Beams**

The provisions of Sections D3.2.3, D3.2.4 and D3.2.5 of this Appendix apply to members in bending in which the applied load in the plane of the web induces twist. Braces shall be designed to avoid local crippling at the points of attachment to the member.

**D3.2.3 Discrete Bracing**

Braces shall be connected so as to effectively restrain both flanges of the section 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 the intermediate braces. Fewer braces may be used if this approach can be shown to be acceptable by rational analysis, testing, or Section C3.1.3 of the Specification, taking into account the effects of both lateral and torsional displacements.

If fewer braces are used (when shown to be acceptable by rational analysis or testing), those sections used as purlins with "floating"-type roof sheathings that allow for expansion and contraction independent of the purlins, shall have a minimum of one brace per bay for spans \( \leq 7 \) m and two braces per bay for spans > 7 m.

If one-third or more of the total load on the member 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.

**D3.2.4 One Flange Braced by Deck, Slab, or Sheathing**

The factored resistance of the attachment of the continuous deck, slab, or sheathing shall be in accordance with Section D3.1.2 of this
Appendix. Discrete bracing shall be provided to restrain the flange that is not braced by the deck, slab, or sheathing. The spacing of discrete bracing shall be in accordance with Section D3.2.3 of this Appendix.

**D3.2.5 Both Flanges Braced by Deck, Slab, or Sheathing**

The factored resistance of the attachment shall be as given by Section D3.1.2 of this Appendix.

**E2a  Welded Connections**

Arc welding shall be performed by a fabricator or erector certified in accordance with CSA Standard W47.1. Resistance welding shall be performed by a fabricator or erector certified in accordance with CSA Standard W55.3.

Where each connected part is over 4.57 mm in base steel thickness, welding shall conform to CSA Standard W59. Where at least one of the connected parts is between 0.70 and 4.57 mm in base steel thickness, welding shall conform to the requirements contained herein and shall be performed in accordance with the applicable requirements of CSA Standard W59. Except as provided for in Section E2.2, where at least one of the connected parts is less than 0.70 mm in base steel thickness, welds shall be considered to have no structural value unless a value is substantiated by appropriate tests.

The resistance in tension or compression of butt welds shall be the same as prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the joint.

**E2.2a  Arc Spot Welds**

Arc spot welds (circular in shape) covered by this Specification are for welding sheet steel to thicker supporting members in the flat position. The weld is formed by melting through the steel sheet to fuse with the underlying supporting member, whose thickness at the weld location shall be at least 2.5 times the steel sheet thickness (aggregate sheet thickness in the case of multiple plies). The materials to be joined shall be of weldable quality and the electrodes to be used shall be suited to the materials, the welding method, and the ambient conditions during welding.

The following maximum and minimum sheet thicknesses shall apply:

- (a) maximum single sheet thickness shall be 2.0 mm;
- (b) minimum sheet thickness shall be 0.70 mm; and
- (c) maximum aggregate sheet thickness of double sheets shall be 2.5 mm.

**E2.3a  Arc Seam Welds**

The information of Section E2.2a also applies to arc seam welds that are oval in shape.
E3a  Bolted Connections

In addition to the design criteria given in Section E3 of the Specification, the design requirements given in Sections E3.1 and E3.2 of this Appendix shall also be followed for bolted connections where the thickness of the thinnest connected part is 4.76 mm or less, there are no gaps between connected parts, and fasteners are installed with sufficient tightness to achieve satisfactory performance of the connection under anticipated service conditions. Refer to CSA Standard S16 for the design of mechanically fastened connections in which the thickness of all connected parts exceeds 4.76 mm.

Unless otherwise specified, circular holes for bolts shall not be greater than the nominal bolt diameter, \(d\), plus 1 mm for bolt sizes up to 13 mm and plus 2 mm for bolt sizes over 13 mm.

E3.1 Shear, Spacing and Edge Distance

The nominal shear resistance per bolt as affected by spacing and edge distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix.

The center-to-center distance between fasteners shall not be less than 2.5\(d\), and the distance from the center of a fastener to an edge or end shall not be less than 1.5\(d\), where \(d\) = nominal diameter of fastener.

E3.2 Fracture in Net Section (Shear Lag)

The nominal tensile resistance of a tension member other than a flat sheet, \(P_{n}\), shall be determined as follows:

\[
P_{n} = A_{e} F_{u} \phi
\]

\[
\phi = 0.55
\]

where

\(F_{u}\) = Tensile strength of connected part as specified in Section A2

\(A_{e}\) = \(A_{n}U\), effective net area with reduction coefficient, \(U\)

\(U\) = 1.0 for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, \(U\) shall be determined as follows:

a) For angle members having two or more bolts in the line of force

\[
U = 1.0 - 1.2 \frac{x}{L} < 0.9 \quad (Eq. E3.2-2)
\]

\(U\) need not be less than 0.4.

b) For channel members having two or more bolts in the line of force

\[
U = 1.0 - 0.36 \frac{x}{L} < 0.9 \quad (Eq. E3.2-3)
\]

\(U\) need not be less than 0.5.

\(x\) = Distance from shear plane to centroid of cross section

\(L\) = Length of connection

\(A_{n}\) = Net area of connected part
**E3.3a Bearing**

When the thickness of connected steels is equal to or larger than 4.76 mm, the requirements of CSA Standard S16 shall be followed for connection design.

**E3.4 Shear and Tension in Bolts**

For ASTM A 307 bolts less than or equal to 12.7 mm in diameter, refer to Tables E3.4-1 and E3.4-5 of this Appendix. For all other bolts, refer to CSA Standard S16.

The nominal bolt resistance, $P_n$, resulting from shear, tension, or a combination of shear and tension shall be calculated as follows:

$$P_n = A_b F_n \quad (Eq. \ E3.4-1)$$

where

- $A_b =$ Gross cross-sectional area of bolt
- $F_n =$ Nominal tensile or shear stress

i) When bolts are subjected to shear or tension

$F_n$ is given by $F_{nt}$ or $F_{nv}$ in Table E3.4-1, as well as the $\phi$ values

ii) When bolts are subjected to a combination of shear and tension

$F_n$ is given by $F'_{nt}$ in Table E3.4-5, as well as the $\phi$ value

The pull-over resistance of the connected sheet at the bolt head, nut, or washer shall be considered where bolt tension is involved. See Section E6.2 of the Specification.

### TABLE E3.4-1

Nominal Tensile and Shear Stresses for Bolts

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Nominal Tensile Stress, $F_{nt}$ (MPa)</th>
<th>Resistance Factor, $\phi$</th>
<th>Nominal Shear Stress, $F_{nv}$ (MPa)</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 Bolts, Grade A</td>
<td>279</td>
<td>0.65</td>
<td>165</td>
<td>0.55</td>
</tr>
<tr>
<td>6.4 mm ≤ $d$ &lt; 12.7 mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TABLE E3.4-5

Nominal Tensile Stress for Bolts

Subjected to the Combination of Shear and Tension

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Nominal Tensile Stress, $F'_{nt}$ (MPa)</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 Bolts, Grade A</td>
<td>$324 - 2.4f_V \leq 279$</td>
<td>0.65</td>
</tr>
<tr>
<td>When 6.4 mm ≤ $d$ &lt; 12.7 mm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The actual shear stress, $f_V$, shall also satisfy Table E3.4-1 of this Appendix.
E4.3.2 Connection Shear Limited by End Distance

The nominal shear resistance per screw as affected by end distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix. For spacing requirements, see Section E3.1 of this Appendix.

E5 Rupture

Shear rupture, tension rupture, and block shear rupture shall be determined in accordance with the requirements of Section C2.2 of this Appendix.
Appendix C:

Provisions Applicable to

Mexico

2001 EDITION
APPENDIX C: PROVISIONS APPLICABLE TO MEXICO

This Appendix provides design provisions or supplements to Chapters A through G that are only applicable to Mexico. A section number ending with a letter indicates that the provisions herein supplement the corresponding section in Chapters A through G of the Specification. A section number not ending with a letter indicates that the section gives the entire design provision.

A1.1a Country Specific Scope and Limits of Applicability

Designs shall be made according to the provisions for Load and Resistance Factor Design, or to the provisions for Allowable Strength Design. Where allowed, both methods are equally acceptable although they may or may not produce identical designs. However, the two methods shall not be mixed in designing the various cold-formed steel components of a structure.

A2.2 Other Steels

The listing in Section A2.1 does not exclude the use of steel up to and including one in. (25.4 mm) 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 A2.3.

A3 Loads

A3.1 Nominal Loads

The nominal loads shall be as stipulated by the applicable building code under which the structure is designed or as dictated by the conditions involved. In the absence of a building code, the nominal loads shall be those stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

A4.1.2 Load Combinations for ASD

The structure and its components shall be designed so that allowable design strengths equal or exceed the effects of the nominal loads and load combinations as stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

The combined effects of two or more loads, excluding dead load, shall be permitted to be multiplied by 0.75. The combined load used in design shall not be less than the sum of the effects of dead load and any single load that produces the largest effect. The above 0.75 load reduction
shall not be used where similar load reductions are permitted by the applicable building code or ASCE 7.

Exception: When evaluating diaphragms using the provisions of Section D5, no decrease in forces is permitted for load combinations including wind or earthquake loads.

A5.1.2 Load Factors and Load Combinations for LRFD

The structure and its components shall be designed so that design strengths equal or exceed the effects of the factored nominal loads and load combinations stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7.

A9a Referenced Documents

The following documents are referenced in Appendix C:

1. American Society of Civil Engineers, ASCE 7-98, “Minimum Design Loads in Buildings and Other Structures,” American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191


9. Reglamento de Construcciones para el Distrito Federal 1999 or last Edition)
C2 Tension Members

For axially loaded tension members, the nominal tensile strength, \( T_n \), shall be the smallest value obtained according to the limit states of (a) yielding in the gross section, (b) fracture in the net section away from connections, and (c) fracture in the effective net section at the connection:

(a) For yielding:
\[
T_n = A_g F_y \quad \text{(Eq. C2-1)}
\]
\[
\Omega_t = 1.67 \quad \text{(ASD)}
\]
\[
\phi_t = 0.90 \quad \text{(LRFD)}
\]

(b) For fracture away from the connection:
\[
T_n = A_n F_u \quad \text{(Eq. C2-2)}
\]
\[
\Omega_t = 2.00 \quad \text{(ASD)}
\]
\[
\phi_t = 0.75 \quad \text{(LRFD)}
\]

where
\( T_n \) = Nominal strength of member when loaded in tension
\( A_g \) = Gross area of cross section
\( A_n \) = Net area of cross section
\( F_y \) = Design yield point as determined in Section A7.1
\( F_u \) = Tensile strength as specified in Section A2.1 or A2.3.2

(c) For fracture at the connection:
The nominal tensile strength shall also be limited by Sections E2.7, E3, and E5 for tension members using welded connections, bolted connections, and screw connections.

C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System

The nominal flexural strength, \( M_n \), of a C- or Z-section, loaded in a plane parallel to the web with the top flange supporting a standing seam roof system shall be determined using discrete point bracing and the provisions of Section C3.1.2.1 or shall be calculated as follows:
\[
M_n = R S_e F_y \quad \text{(Eq. C3.1.4-1)}
\]
\[
\Omega_b = 1.67 \quad \text{(ASD)}
\]
\[
\phi_b = 0.90 \quad \text{(LRFD)}
\]

where
\( R \) = Reduction factor determined by the "Base Test Method for Purlins Supporting a Standing Seam Roof System" of Part VIII of the AISI Cold-Formed Steel Design Manual.
\( S_e \) and \( F_y \) are defined in Section C3.1.1.

E2a Welded Connections

For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in. (4.57 mm), refer to the AISC/IMCA "Specification for

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 in. (4.57 mm) or less in thickness shall be made in accordance with the AWS D1.3 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 E2a.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1 or AWS C1.3.

### TABLE E2a

<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 Sheet</td>
<td>F</td>
<td>—</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>—</td>
<td>H</td>
<td>H</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td></td>
<td>V</td>
<td>—</td>
<td>—</td>
<td>V</td>
<td>V</td>
<td>V</td>
</tr>
<tr>
<td></td>
<td>OH</td>
<td>—</td>
<td>—</td>
<td>OH</td>
<td>OH</td>
<td>OH</td>
</tr>
<tr>
<td>Sheet to Supporting Member</td>
<td>—</td>
<td>F</td>
<td>F</td>
<td>H</td>
<td>H</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>V</td>
<td>V</td>
<td>—</td>
</tr>
<tr>
<td></td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>OH</td>
<td>OH</td>
<td>—</td>
</tr>
</tbody>
</table>

(\(F = \) Flat, \(H = \) horizontal, \(V = \) vertical, \(OH = \) overhead)

### E3a Bolted Connections

In addition to the design criteria given in Section E3 of the Specification, the following design requirements shall also be followed for bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than \(\frac{3}{16}\) in. (4.76 mm). For bolted connections in which the thickness of the thinnest connected part is equal to or greater than \(\frac{3}{16}\) in. (4.76 mm), refer to AISC “Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design”, or the “Load and Resistance Factor Design Specification for Structural Steel Buildings” or the Manual de Construcción en Acero del Instituto Mexicano del la Construcción en Acero, A.C. (IMCA)-Last Edition.

The holes for bolts shall not exceed the sizes specified in Table E3a, except that larger holes may be used in column base details or structural systems connected to concrete walls.

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 slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Chapter F.

**TABLE E3a**

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d mm</th>
<th>Standard Hole Diameter, d_h mm</th>
<th>Oversized Hole Diameter, d_h mm</th>
<th>Short-Slotted Hole Dimensions mm</th>
<th>Long-Slotted Hole Dimensions mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 12.7</td>
<td>d + 0.8</td>
<td>d + 1.6</td>
<td>(d + 0.8) by (d + 6.4)</td>
<td>(d + 0.8) by (2 1/2 d)</td>
</tr>
<tr>
<td>≥ 12.7</td>
<td>d + 1.6</td>
<td>d + 3.2</td>
<td>(d + 1.6) by (d + 6.4)</td>
<td>(d + 1.6) by (2 1/2 d)</td>
</tr>
</tbody>
</table>

**E3.1 Shear, Spacing and Edge Distance**

The nominal shear strength, P_n, of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated as follows:

\[ P_n = t e F_u \]  \hspace{1cm} (Eq. E3.1-1)

(a) When \( F_u/F_{sy} \geq 1.08 \):

\[ \Omega = 2.00 \] (ASD)

\[ \phi = 0.70 \] (LRFD)

(b) When \( F_u/F_{sy} < 1.08 \):

\[ \Omega = 2.22 \] (ASD)

\[ \phi = 0.60 \] (LRFD)

where

\( P_n \) = Nominal strength per bolt
\( e \) = Distance measured in line of force from center of a standard hole to nearest edge of adjacent hole or to end of connected part
\( t \) = Thickness of thinnest connected part
\( F_u \) = Tensile strength of connected part as specified in Section A2.1, A2.2 or A2.3.2
\( F_{sy} \) = Yield point of connected part as specified in Section A2.1, A2.2 or A2.3.2

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench. 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 E3a. In no case shall the clear distance 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 Fracture in Net Section (Shear Lag)**

The nominal tensile strength of a bolted member shall be determined in accordance with Section C2. For fracture in the effective net section of the connected part, the nominal tensile strength, \(P_n\), shall be determined as follows:

1. For flat sheet connections not having staggered hole patterns:
   \[ P_n = A_n F_t \]  
   \[(Eq. E3.2-1)\]
   (a) When washers are provided under both the bolt head and the nut:
   - For a single bolt, or a single row of bolts perpendicular to the force
     \[ F_t = (0.1 + 3d/s) F_u \leq F_u \]  
     \[(Eq. E3.2-2)\]
   - For multiple bolts in the line parallel to the force
     \[ F_t = F_u \]  
     \[(Eq. E3.2-3)\]
   For double shear:
   \[ \Omega = 2.00 \quad (ASD) \]
   \[ \phi = 0.65 \quad (LRFD) \]
   For single shear:
   \[ \Omega = 2.22 \quad (ASD) \]
   \[ \phi = 0.55 \quad (LRFD) \]
   (b) When either washers are not provided under the bolt head and the nut, or only one washer is provided under either the bolt head or the nut:
   - For a single bolt, or a single row of bolts perpendicular to the force
     \[ F_t = (2.5d/s) F_u \leq F_u \]  
     \[(Eq. E3.2-4)\]
   - For multiple bolts in the line parallel to the force
     \[ F_t = F_u \]  
     \[(Eq. E3.2-5)\]
   \[ \Omega = 2.22 \quad (ASD) \]
   \[ \phi = 0.65 \quad (LRFD) \]
   where
   \(A_n\) = Net area of connected part
   \(s\) = Sheet width divided by number of bolt holes in cross section being analyzed (when evaluating \(F_t\))
   \(F_u\) = Tensile strength of connected part as specified in Section A2.1, A2.2 or A2.3.2
   \(d\) = Nominal bolt diameter

2. For flat sheet connections having staggered hole patterns:
   \[ P_n = A_n F_t \]  
   \[(Eq. E3.2-6)\]
\[ \Omega = 2.22 \quad \text{(ASD)} \]
\[ \phi = 0.65 \quad \text{(LRFD)} \]

where

- \( F_t \) is determined in accordance with Eqs. E3.2-2 to E3.2-5.
- \( A_n = 0.90 \left[ A_g - n_b d_h t + \left( \sum s'/4g \right) t \right] \quad \text{(Eq. E3.2-7)} \]
- \( \Omega = 2.22 \quad \text{(ASD)} \)
- \( \phi = 0.65 \quad \text{(LRFD)} \)

\( A_g = \) Gross area of member

\( s' = \) Longitudinal center-to-center spacing of any two consecutive holes

\( g = \) Transverse center-to-center spacing between fastener gage lines

\( n_b = \) Number of bolt holes in the cross section being analyzed

\( d_h = \) Diameter of a standard hole

\( t \) is defined in Section E3.1.

(3) For other than flat sheet:

\[ P_n = A_e F_u \quad \text{(Eq. E3.2-8)} \]

\( A_e = A_n U \), effective net area with \( U \) defined as follows:

\( U = 1.0 \) for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, the reduction coefficient \( U \) is determined as follows:

(a) For angle members having two or more bolts in the line of force

\[ U = 1.0 - 1.20 \frac{x}{L} < 0.9 \quad \text{(Eq. E3.2-9)} \]

but \( U \) shall not be less than 0.4.

(b) For Channel members having two or more bolts in the line of force

\[ U = 1.0 - 0.36 \frac{x}{L} < 0.9 \quad \text{(Eq. E3.2-10)} \]

but \( U \) shall not be less than 0.5.

\( x = \) Distance from shear plane to centroid of the cross section

\( L = \) Length of the connection

\( A_n = \) Net area of connected part

### E3.4 Shear and Tension in Bolts

The nominal bolt strength, \( P_n \), resulting from shear, tension or a combination of shear and tension shall be calculated as follows:

\[ P_n = A_b F_n \quad \text{(Eq. E3.4-1)} \]

where

- \( A_b = \) Gross cross-sectional area of bolt

When bolts are subject to shear or tension:

- \( F_n \) is given by \( F_{nv} \) or \( F_{nt} \) in Table E3.4-1.
- \( \Omega \) and \( \phi \) are given in Table E3.4-1.
The pullover strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved, see Section E6.2. When bolts are subject to a combination of shear and tension:

For ASD

- $F_n$ is given by $F'_{nt}$ in E3.4-2.
- $\Omega$ is given in Table E3.4-2.

For LRFD

- $F_n$ is given by $F'_{nt}$ in Table E3.4-3.
- $\phi$ is given in Table E3.4-3.
<table>
<thead>
<tr>
<th>Bolt Type and Grade</th>
<th>Diameters (cm)</th>
<th>Tensile Strength</th>
<th>Shear Strength</th>
<th>Tensile Strength</th>
<th>Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Factor of Safety</td>
<td>Resistance Factor</td>
<td>Nominal Stress</td>
<td>Factor of Safety</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \Omega ) (ASD)</td>
<td>( \phi ) (LRFD)</td>
<td>( F_{nt} ) kg/cm(^2)</td>
<td>( \Omega ) (ASD)</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td>0.64 cm ≤ d &lt; 1.27 cm</td>
<td>2.25</td>
<td>0.75</td>
<td>2850</td>
<td>2.4</td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td>d ≥ 1.27 cm</td>
<td>2.25</td>
<td></td>
<td>3160</td>
<td></td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td>2.0</td>
<td>6330</td>
<td></td>
<td>7100</td>
<td></td>
</tr>
<tr>
<td>A325 bolts, when threads are excluded from shear planes</td>
<td></td>
<td>6330</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>0.64 cm ≤ d &lt; 1.27 cm, when threads are not excluded from shear planes</td>
<td></td>
<td></td>
<td>7100</td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>0.64 cm ≤ d &lt; 1.27 cm, when threads are excluded from shear planes</td>
<td></td>
<td></td>
<td>7100</td>
<td></td>
</tr>
<tr>
<td>A449 Bolt</td>
<td>0.64 cm ≤ d &lt; 1.27 cm, when threads are not excluded from shear planes</td>
<td></td>
<td></td>
<td>5700</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>0.64 cm ≤ d &lt; 1.27 cm, when threads are excluded from shear planes</td>
<td></td>
<td></td>
<td>5700</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td></td>
<td>7910</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are excluded from shear planes</td>
<td></td>
<td>7910</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Applies to bolts in holes as limited by Table E3a. 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 Chapter F.
### TABLE E3.4-2 (ASD)
Nominal Tensile Stress, $F'_{nt}$ (kg/cm$^2$), for Bolts Subjected 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>
<th>Factor of Safety $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>7730 – 3.6$f_v$ ≤ 6330</td>
<td>7730 – 2.8$f_v$ ≤ 6330</td>
<td>2.0</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>8580 – 3.6$f_v$ ≤ 7100</td>
<td>8580 – 2.8$f_v$ ≤ 7100</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>7030 – 3.6$f_v$ ≤ 5700</td>
<td>7030 – 2.8$f_v$ ≤ 5700</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>9560 – 3.6$f_v$ ≤ 7910</td>
<td>9560 – 2.8$f_v$ ≤ 7910</td>
<td></td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td>3660 - 4$f_v$ ≤ 2850</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>When 0.64 cm ≤ d &lt; 1.27 cm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>When d ≥ 1.27 cm</td>
<td>4110 - 4$f_v$ ≤ 3160</td>
<td></td>
</tr>
</tbody>
</table>

The shear stress, $f_v$, shall also satisfy Table E3.4-1.

### TABLE E3.4-3 (LRFD)
Nominal Tensile Stress, $F'_{nt}$ (kg/cm$^2$), for Bolts Subjected 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>
<th>Resistance Factor $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A325 Bolts</td>
<td>7950 – 2.4$f_v$ ≤ 6330</td>
<td>7950 – 1.9$f_v$ ≤ 6330</td>
<td>0.75</td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>8930 – 2.4$f_v$ ≤ 7100</td>
<td>8930 – 1.9$f_v$ ≤ 7100</td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>7100 – 2.4$f_v$ ≤ 5700</td>
<td>7100 – 1.9$f_v$ ≤ 5700</td>
<td></td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>9910 – 2.4$f_v$ ≤ 7910</td>
<td>9910 – 1.9$f_v$ ≤ 7910</td>
<td></td>
</tr>
<tr>
<td>A307 Bolts, Grade A</td>
<td></td>
<td>3300 – 2.4$f_v$ ≤ 2850</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>When 0.64 cm ≤ d &lt; 1.27 cm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>When d ≥ 1.27 cm</td>
<td>3660 – 2.4$f_v$ ≤ 3160</td>
<td></td>
</tr>
</tbody>
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The shear stress, $f_v$, shall also satisfy Table E3.4-1.
E4.3.2 Connection Shear Limited by End Distance

The nominal shear strength per screw, \( P_{ns} \) shall not exceed that calculated as follows when the distance to an end of the connected part is parallel to the line of the applied force.

\[
P_{ns} = t e F_u \quad (Eq. E4.3.2-1)
\]

\[\Omega = 3.00 \quad (ASD)\]
\[\phi = 0.50 \quad (LRFD)\]

where
\[t = \text{Thickness of part in which end distance is measured}\]
\[e = \text{Distance measured in line of force from center of standard hole to nearest end of connected part}\]
\[F_u = \text{Tensile strength of part in which end distance is measured}\]

E5 Rupture

E5.1 Shear Rupture

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the nominal shear strength, \( V_n \), shall be calculated as follows:

\[
V_n = 0.6 F_u A_{wn} \quad (Eq. E5.1-1)
\]

\[\Omega = 2.00 \quad (ASD)\]
\[\phi = 0.75 \quad (LRFD)\]

where
\[A_{wn} = (h_{wc} - n d_h) t \quad (Eq. E5.1-2)\]
\[h_{wc} = \text{Coped flat web depth}\]
\[n = \text{Number of holes in critical plane}\]
\[d_h = \text{Hole diameter}\]
\[F_u = \text{Tensile strength of connected part as specified in Section A2.1 or A2.2}\]
\[t = \text{Thickness of coped web}\]

E5.2 Tension Rupture

The nominal tensile rupture strength along a path in the affected elements of connected members shall be determined by Section E2.7 or E3.2 for welded or bolted connections, respectively.

E5.3 Block Shear Rupture

The nominal block shear rupture strength, \( R_n \), shall be determined as follows:

(a) When \( F_u A_{nt} \geq 0.6 F_u A_{nv} \)
\[
R_n = 0.6 F_y A_{gv} + F_u A_{nt} \quad (Eq. E5.3-1)
\]

(b) When \( F_u A_{nt} < 0.6 F_u A_{nv} \)
\[ R_n = 0.6F_uA_{nv} + F_yA_{gt} \]  
(Eq. E5.3-2)

For bolted connections:
\[ \Omega = 2.22 \quad (ASD) \]
\[ \phi = 0.65 \quad (LRFD) \]

For welded connections:
\[ \Omega = 2.50 \quad (ASD) \]
\[ \phi = 0.60 \quad (LRFD) \]

where
- \( A_{gv} \) = Gross area subject to shear
- \( A_{gt} \) = Gross area subject to tension
- \( A_{nv} \) = Net area subject to shear
- \( A_{nt} \) = Net area subject to tension
The material contained herein has been developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, the Canadian Standard Association Technical Committee on Cold-Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. The organizations and the Committees have made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committees acknowledge and are grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in the Commentary on the Specification.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that future editions of this Specification will update this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all resulting liability arising therefrom.

1st Printing – May 2002

Produced by American Iron and Steel Institute

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PREFACE

This document provides a commentary on the 2001 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members*. It was based on the Commentary on the 1996 edition of the AISI Specification with necessary additions and revisions. This *Commentary* should be used in combination with the AISI *Cold-Formed Steel Design Manual* to be published in 2003.

The purpose of the *Commentary* includes: (a) to provide a record of the reasoning behind, and justification for the various provisions of the *North American Specification* by cross-referencing the published supporting research data and to discuss the changes made in the current *Specification*; (b) to offer a brief but coherent presentation of the characteristics and performance of cold-formed steel structures to structural engineers and other interested individuals; (c) to furnish the background material for a study of cold-formed steel design methods to educators and students; and (d) to provide the needed information to those who will be responsible for future revisions of the *Specification*. The readers who wish to have more complete information, or who may have questions which are not answered by the abbreviated presentation of this *Commentary*, should refer to the original research publications.

The assistance and close cooperation of the North American Specification Committee under the Chairmanship of Professor Reinhold M. Schuster and the AISI Committee on Specifications under the Chairmanship of Mr. Roger L. Brockenbrough and the Vice Chairmanship of Mr. Jay W. Larson are gratefully acknowledged. Special thanks are extended to Professor Wei-Wen Yu for revising the draft of this *Commentary*. The Institute is very grateful to members of the Editorial Subcommittee and all members of the AISI Committee on Specifications for their careful review of the document and their valuable comments and suggestions. The background materials provided by various subcommittees are appreciated.

American Iron and Steel Institute
December 2001
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INTRODUCTION

Cold-formed steel members have been used economically for building construction and other applications (Winter, 1959a, 1959b; Yu, 2000). These types of sections are cold-formed from steel sheet, strip, plate or flat bar in roll-forming machines or by press brake or bending operations. The thicknesses of steel sheets or strip generally used for cold-formed steel structural members range from 0.0147 in. (0.373 mm) to about 1/4 in. (6.35 mm). Steel plates and bars as thick as 1 in. (25.4 mm) can be cold-formed successfully into structural shapes.

In general, cold-formed steel structural members can offer several advantages for building construction (Winter, 1970; Yu, 2000): (1) light members can be manufactured for relatively light loads and/or short spans, (2) unusual sectional configurations can be produced economically by cold-forming operations and consequently favorable strength-to-weight ratios can be obtained, (3) load-carrying panels and decks can provide useful surfaces for floor, roof and wall construction, and in some cases they can also provide enclosed cells for electrical and other conduits, and (4) panels and decks not only withstand loads normal to their surfaces, but they can also act as shear diaphragms to resist forces in their own planes if they are adequately interconnected to each other and to supporting members.

The use of cold-formed steel members in building construction began in about the 1850s. However, in North America such steel members were not widely used in buildings until the publication of the first edition of the American Iron and Steel Institute (AISI) Specification in 1946 (AISI, 1946). This first design standard was primarily based on the research work sponsored by AISI at Cornell University since 1939. It was revised subsequently by the AISI Committee in 1956, 1960, 1962, 1968, 1980, and 1986 to reflect the technical developments and the results of continuing research. In 1991, AISI published the first edition of the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members (AISI, 1991). Both allowable stress design (ASD) and load and resistance factor design (LRFD) specifications were combined into a single document in 1996. In Canada, the Canadian Standards Association (CSA) published its first edition of Design of Light Gauge Steel Structural Members in 1963 based on the 1962 edition of the AISI Specification. Subsequent editions were published in 1974, 1984, 1989 and 1994. The Canadian Standard for Cold-Formed Steel Structural Members (CSA, 1994) was based on the Limit States Design (LSD) method.

In Mexico, cold-formed steel structural members have also been designed on the basis of AISI Specifications. The 1962 edition of the AISI Design Manual (AISI, 1962) was translated to Spanish in 1965 (Camara, 1965).

The first edition of the unified North American Specification was prepared and issued in 2001. It is applicable to the United States, Canada, and Mexico for the design of cold-formed steel structural members. This edition of the Specification was developed on the basis of the 1996 AISI Specification with the 1999 Supplement (AISI, 1996, 1999), the 1994 CSA Standard (CSA, 1994), and subsequent developments. In this new North American Specification, the ASD and LRFD methods are used in the United States and Mexico, while the LSD method is used in Canada. For the ASD method, the term “Allowable Stress Design” was
renamed to “Allowable Strength Design” to clarify the nature of this design method.

In addition to the issuance of the design specification, AISI also published the first edition of the Design Manual in 1949 (AISI, 1949). This allowable stress design manual was revised later in 1956, 1961, 1962, 1968, 1977, 1983, and 1986. In 1991, the LRFD Design Manual was published for using the load and resistance factor design criteria. The AISI 1996 Cold-Formed Design Manual was prepared for the combined AISI ASD and LRFD Specifications.

During the period from 1958 through 1983, AISI published Commentaries on several editions of the AISI design specification, which were prepared by Professor George Winter of Cornell University in 1958, 1961, 1962, and 1970. From 1983, the format used for the AISI Commentary has been changed in that the same section numbers are used in the Commentary as in the Specification. The Commentary on the 1996 AISI Specification was prepared by Professor Wei-Wen Yu of the University of Missouri-Rolla (Yu, 1996). The current edition of the Commentary (AISI, 2001) was updated based on the Commentary on the 1996 AISI Specification. It contains Chapters A through G, and Appendices A through C, where commentary on provisions that are only applicable to a specific country is included in the corresponding Appendix.

As in previous editions of the Commentary, this document contains a brief presentation of the characteristics and the performance of cold-formed steel members. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the specification. A cross-reference is provided between various design provisions and the published research data.

In this Commentary, the individual sections, equations, figures, and tables are identified by the same notation as in the Specification and the material is presented in the same sequence. Bracketed terms used in the Commentary are equivalent terms that apply particularly to the LSD method in Canada.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence in their fields.

A. GENERAL PROVISIONS

A1  Limits of Applicability and Terms

A1.1  Scope and Limits of Applicability

The cross-sectional configurations, manufacturing processes and fabrication practices of cold-formed steel structural members differ in several respects from that of hot-rolled steel shapes. For cold-formed steel sections, the forming process is performed at, or near, room temperature by the use of bending brakes, press brakes, or roll-forming machines. Some of the significant differences between cold-formed sections and hot-rolled shapes are (1) absence of the residual stresses caused by uneven cooling due to hot-rolling, (2) lack of corner fillets, (3) presence of increased yield strength with decreased proportional limit and ductility resulting from cold-forming, (4) presence of cold-reducing stresses when cold-rolled steel stock has not been finally annealed, (5) prevalence of elements having large width-to-thickness
ratios, (6) rounded corners, and (7) stress-strain curves can be either sharp-yielding type or gradual-yielding type.

The *Specification* is applicable only to cold-formed sections not more than one inch (25.4 mm) in thickness. Research conducted at the University of Missouri-Rolla (Yu, Liu, and McKinney, 1973b and 1974) has verified the applicability of the specification’s provisions for such cases.

In view of the fact that most of the design provisions have been developed on the basis of the experimental work subject to static loading, the *Specification* is intended for the design of cold-formed steel structural members to be used for load-carrying purposes in buildings. For structures other than buildings, appropriate allowances should be made for dynamic effects.

Because of the diverse forms in which cold-formed steel structural members can be used, it is not possible to cover all design configurations by the design rules presented in the *Specification*. For those special cases where the design strength [factored resistance]* and/or stiffness cannot be so determined, it can be established either by (a) testing and evaluation in accord with the provisions of Chapter F, or (b) rational engineering analysis. Prior to 2001, the only option in such cases was testing. However, in 2001, in recognition of the fact that this was not always practical or necessary, the rational engineering analysis option was added. It is essential that such analysis be based on theory that is appropriate for the situation, any available test data that is relevant, and sound engineering judgment. These provisions must not be used to circumvent the intent of the *Specification*. Where the provisions of Chapters B through G of the *Specification* and Appendices A through C apply, those provisions must be used and cannot be avoided by testing or rational analysis.

**Note:**

* Bracketed terms are equivalent terms that apply particularly to LSD.

** Symbol ** is used to point out that additional provisions are provided in the Appendices as indicated by the letters.

### A1.2 Terms

Many of the definitions in *Specification* Section A1.2 for ASD, LRFD and LSD are self-explanatory. Only those which are not self-explanatory are briefly discussed below.

#### General Terms

**Effective Design Width**

The effective design width is a concept which facilitates taking account of local buckling and postbuckling strength for compression elements. The effect of shear lag on short, wide flanges is also handled by using an effective design width. These matters are treated in *Specification* Chapter B, and the corresponding effective widths are discussed in the Commentary on that chapter.
Multiple-Stiffened Elements

Multiple-stiffened elements of two sections are shown in Figure C-A1.2-1. Each of the two outer sub-elements of section (1) are stiffened by a web and an intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of section (2) are stiffened by a web and the attached intermediate middle stiffener.

---

Stiffened or Partially Stiffened Compression Elements

Stiffened compression elements of various sections are shown in Figure C-A1.2-2, in which sections (1) through (5) are for flexural members, and sections (6) through (9) are for compression members. Sections (1) and (2) each have a web and a lip to stiffen the compression element (i.e., the compression flange), the ineffective portion of which is shown shaded. For the explanation of these ineffective portions, see the discussion of Effective Design Width and Chapter B. Sections (3), (4), and (5) show compression elements stiffened by two webs. Sections (6) and (8) show edge stiffened flange elements that have a vertical element (web) and an edge stiffener (lip) to stiffen the elements while the web itself is stiffened by the flanges. Section (7) has four compression elements stiffening each...
Lipped Channel

I-Beam Made of Two Lipped Channels Back-to-Back

Hat-Section

Box-Type Section

Inverted "U"-Type Section

Flexural Members, Such as Beams (Top Flange in Compression)

Lipped Channel

Box-Type Section

I-Section Made of Two Lipped Channels Back-to-Back

Lipped Angle

Compression Members, Such as Columns

Figure C-A1.2-2 Stiffened Compression Elements

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other, and section (9) has each stiffened element stiffened by a lip and the other stiffened element.

**Thickness**

In calculating section properties, the reduction in thickness that occurs at corner bends is ignored, and the base metal thickness of the flat steel stock, exclusive of coatings, is used in all calculations for load-carrying purposes.

**Torsional-Flexural Buckling**

The 1968 edition of the *Specification* pioneered methods for computing column loads of cold-formed steel sections prone to buckle by...
simultaneous twisting and bending. This complex behavior may result in lower column loads than would result from primary buckling by flexure alone.

**Unstiffened Compression Elements**

Unstiffened elements of various sections are shown in Figure C-A1.2-3, in which sections (1) through (4) are for flexural members and sections (5) through (8) are for compression members. Sections (1), (2), and (3) have only a web to stiffen the compression flange element. The legs of section (4) provide mutual stiffening action to each other along their common edges. Sections (5), (6), and (7), acting as columns have vertical stiffened elements (webs) which provide support for one edge of the unstiffened flange elements. The legs of section (8) provide mutual stiffening action to each other.

**ASD and LRFD Terms (USA and Mexico)**

**ASD (Allowable Stress Design, herein referred to as Allowable Strength Design)**

Allowable Strength Design (ASD) is a method of designing structural components such that the allowable design value (stress, force, or moment) permitted by various sections of the Specification is not exceeded when the structure is subjected to all appropriate combinations of nominal loads as given in Section A4.1.2 of Appendix A or C of the Specification.

**LRFD (Load and Resistance Factor Design)**

Load and Resistance Factor Design (LRFD) is a method of designing structural components such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A5.1.2 of Appendix A or C of the Specification. See also Specification Section A5.1.1 for LRFD strength requirements.

**LSD Terms (Canada)**

**LSD (Limit States Design)**

Limit States Design (LSD) is a method of designing structural components such that the applicable limit state is not exceeded when the structure is subjected to all appropriate load combinations as given in Section A6.1.2 of Appendix B of the Specification. See also Specification Section A6.1.1 for LSD requirements.

In the 2001 North American Specification, the terminologies for limit states design (LSD) are given in brackets parallel to those for load and resistance factor design (LRFD). The inclusion of LSD terminology is intended to help engineers who are familiar with LSD better understand the Specification.

It should be noted that the design concept used for the LRFD and the LSD methods is the same, except that the load factors, load combinations, assumed dead-to-live ratios, and target reliability indexes are slightly
different. In most cases, same nominal strength [nominal resistance] equations are used for ASD, LRFD, and LSD approaches.

A1.3 Units of Symbols and Terms

The non-dimensional character of the majority of the Specification provisions is intended to facilitate design in any compatible systems of units (U.S. customary, SI or metric, and MKS systems).

The conversion of U.S. customary into SI metric units and MKS systems are given in parentheses throughout the entire text of the Specification and Commentary. Table C-A1.3-1 is a conversion table for these three different units.

<table>
<thead>
<tr>
<th>Table C-A1.3-1 Conversion Table</th>
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<td>To Convert</td>
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A2 Material

A2.1 Applicable Steels

The American Society for Testing and Materials (ASTM) is the basic source of steel designations for use with the Specification. Section A2.1 contains the complete list of ASTM Standards for steels that are accepted by the
Dates of issue are included in Section A9. Other standards that are applicable to a specific country are listed in the corresponding Appendix.

In the AISI 1996 Specification, the ASTM A446 Standard was replaced by the ASTM A653/A653M Standard. At the same time, the ASTM A283/A283M Standard, High-Strength, Low-Alloy Steel (HSLAS) Grades 70 (480) and 80 (550) of ASTM A653/A653M and ASTM A715 were added.

In 2001, the ASTM A1008/A1008M and ASTM A1011/A1011M Standards replaced the ASTM A570/A570M, ASTM A607, ASTM A611, and ASTM A715 Standards. ASTM A1003/A1003M was added to the list of Specification Section A2.1.

The important material properties for the design of cold-formed steel members are: yield point, tensile strength, and ductility. Ductility is the ability of a steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold forming. It is usually measured by the elongation in a 2-inch (51 mm) gage length. The ratio of the tensile strength to the yield point is also an important material property; this is an indication of strain hardening and the ability of the material to redistribute stress.

For the listed ASTM Standards, the yield points of steels range from 24 to 80 ksi (165 to 552 MPa or 1690 to 5620 kg/cm²) and the tensile strengths vary from 42 to 100 ksi (290 to 690 MPa or 2950 to 7030 kg/cm²). The tensile-to-yield ratios are no less than 1.13, and the elongations are no less than 10 percent. Exceptions are ASTM A653/A653M SS Grade 80 (550), ASTM A1008/A1008M SS Grade 80 (550), and ASTM A792/A792M SS Grade 80 (550) steels with a specified minimum yield point of 80 ksi (550 MPa or 5620 kg/cm²), a specified minimum tensile strength of 82 ksi (565 MPa or 5770 kg/cm²), and with no stipulated minimum elongation in 2 inches (51 mm). These low ductility steels permit only limited amounts of cold forming, require fairly large corner radii, and have other limits on their applicability for structural framing members. Nevertheless, they have been used successfully for specific applications, such as decks and panels with large corner radii and little, if any, stress concentrations. The conditions for use of these SS Grade 80 (550) steels are outlined in Specification Section A2.3.2.

For ASTM A1003/A1003M steel, even though the minimum tensile strength is not specified in the ASTM Standard for each of Types H and L Steels, the footnote of Table 2 of the Standard states that for Type H steels the ratio of tensile strength to yield strength shall not be less than 1.08. Thus, a conservative value of $F_u = 1.08 F_y$ can be used for the design of cold-formed steel members using Type H steels. Based on the same Standard, a conservative value of $F_u = F_y$ can be used for the design of purlins and girts using Type L steels.

### A2.2 Other Steels

Comments on other steels are provided in the corresponding Appendices of this Commentary.
A2.3 Ductility

The nature and importance of ductility and the ways in which this property is measured were briefly discussed in Commentary Section A2.1.

Low-carbon sheet and strip steels with specified minimum yield points from 24 to 50 ksi (165 to 345 MPa or 1690 to 3520 kg/cm²) need to meet ASTM specified minimum elongations in a 2-inch (51 mm) gage length of 11 to 30 percent. In order to meet the ductility requirements, steels with yield points higher than 50 ksi (345 MPa or 3520 kg/cm²) are often low-alloy steels. However, SS Grade 80 (550) of ASTM A653/A653M, SS Grade 80 (550) of A1008/A1008M, SS Grade 80 (550) of A792/A792M, and SS Grade 80 (550) of A875/A875M steels are carbon steels, for which specified minimum yield strength is 80 ksi (552 MPa or 5620 kg/cm²) and no elongation requirement is specified. These differ from the array of steels listed under Specification Section A2.1.

In 1968, because new steels of higher strengths were being developed, sometimes with lower elongations, the question of how much elongation is really needed in a structure was the focus of a study initiated at Cornell University. Steels were studied that had yield strengths ranging from 45 to 100 ksi (310 to 690 MPa or 3160 to 7030 kg/cm²), elongations in 2 inches (51 mm) ranging from 50 to 1.3 percent, and tensile-to-yield strength ratios ranging from 1.51 to 1.00 (Dhalla, Errera and Winter, 1971; Dhalla and Winter, 1974a; Dhalla and Winter, 1974b). The investigators developed elongation requirements for ductile steels. These measurements are more accurate but cumbersome to make; therefore, the investigators recommended the following determination for adequately ductile steels: (1) The tensile-to-yield strength ratio shall not be less than 1.08 and (2) the total elongation in a 2-inch (51-mm) gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch (203-mm) gage length. Also, the Specification limits the use of Chapters B through E to adequately ductile steels. 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 by Dhalla and Winter (1974a) (Yu, 2000).

Further information on the test procedure should be obtained from “Standard Methods for Determination of Uniform and Local Ductility”, Cold-Formed Steel Design Manual, Part VIII (AISI, 2002). Because of limited experimental verification of the structural performance of members using materials having a tensile-to-yield strength ratio less than 1.08 (Macadam et al., 1988), 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, C3.1.3, C3.1.4, and C3.1.5. Thus, the use of such steels in other applications (compression members, tension members, other flexural members including those whose strength [resistance] 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 Specification Section C5.2 are met and \( \Omega_{cp}/P_n \) does not exceed 0.15 for allowable strength design,
Pu/\phi P_n does not exceed 0.15 for the Load and Resistance Factor Design, and Pf/\phi P_n does not exceed 0.15 for the Limit States Design.

SS Grade 80 (550) of ASTM A653/A653M, SS Grade 80 (550) of ASTM A1008/A1008M, SS Grade 80 (550) of A792/A792M, and SS Grade 80 of A875/A875M steels do not have adequate ductility as defined by Specification Section A2.3.1. Their use has been limited in Specification Section A2.3.2 to particular multiple-web configurations such as roofing, siding, and floor decking.

In the past, the limit of the yield point used in design to 75 percent of the specified minimum yield point, or 60 ksi (414 MPa or 4220 kg/cm²), and the tensile strength used in design to 75 percent of the specified minimum tensile strength, or 62 ksi (427 MPa or 4360 kg/cm²) whichever was lower, introduced a higher factor of safety, but still made low ductility steels, such as SS Grade 80 and Grade E, useful for the named applications.

Based on the recent UMR research findings (Wu, Yu, and LaBoube, 1996), Equation A2.3.2-1 was added in Specification Section A2.3.2 under an Exception Clause to determine the reduced yield point, RbFy, for the calculation of the nominal flexural strength [moment resistance] of multiple-web sections such as roofing, siding and floor decking (AISI, 1999). For the unstiffened compression flange, Equation A2.3.2-2 is added on the basis of a 1988 UMR study (Pan and Yu, 1988). This revision allows the use of a higher nominal bending strength [resistance] than previous editions of the AISI Specification. When the multiple-web section is composed of both stiffened and unstiffened compression flange elements, the smallest Rb should be used to determine the reduced yield point for use on the entire section. Different values of the reduced yield point could be used for positive and negative moments.

The equations provided in the Exception Clause can also be used for calculating the nominal flexural strength [resistance] when the design strengths [factored resistances] are determined on the basis of tests as permitted by the alternative method.

It should be noted that the Exception Clause does not apply to the steel deck used for composite slabs when the deck is used as the tensile reinforcement. This limitation is to prevent the possible sudden failure of the composite slab due to lack of ductility of the steel deck.

For the calculation of web crippling strength [resistance] of deck panels, although the UMR study (Wu, Yu, and LaBoube, 1997) shows that the specified minimum yield point can be used to calculate the web crippling strength [resistance] of deck panels, the Specification is adopting a conservative approach in Section C3.4. The lesser of 0.75 Fy and 60 ksi (414 MPa or 4220 kg/cm²) is used to determine both the web crippling strength [resistance] and the shear strength [resistance] for the low ductility steels. This is consistent with the previous edition of the AISI Specification.

Another UMR study (Koka, Yu, and LaBoube, 1997) confirmed that for the connection design using SS Grade 80 (550) of A653/A653M steel, the
tensile strength used in design should be taken as 75 percent of the specified minimum tensile strength or 62 ksi (427 MPa or 4360 kg/cm²), whichever is less. It should be noted that the current design provisions are limited only to the design of members and connections subjected to static loading without the considerations of fatigue strength.

Load tests are permitted, but not for the purpose of using higher loads than can be calculated under Specification Chapters B through G.

**A2.4 Delivered Minimum Thickness**

Sheet and strip steels, both coated and uncoated, may be ordered to nominal or minimum thickness. If the steel is ordered to minimum thickness, all thickness tolerances are over (+) and nothing under (-). If the steel is ordered to nominal thickness, the thickness tolerances are divided equally between over and under. Therefore, in order to provide the similar material thickness between the two methods of ordering sheet and strip steel, it was decided to require that the delivered thickness of a cold-formed product be at least 95 percent of the design thickness. Thus, it is apparent that a portion of the factor of safety or resistance factor may be considered to cover minor negative thickness tolerances.

Generally, thickness measurements should be made in the center of flanges. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. Thickness measurements should not be made closer to edges than the minimum distances specified in ASTM A568 Standard.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

**A3 Loads**

Comments on loads and load combinations for different countries are provided in the corresponding Appendices of this Commentary.

**A4 Allowable Strength Design**

**A4.1 Design Basis**

The Allowable Strength Design method has been featured in AISI specifications beginning with the 1946 edition. It is included in the current Specification along with the LRFD and the LSD methods for use in the United States, Mexico, and Canada.

**A4.1.1 ASD Requirements**

In the allowable strength design approach, the required allowable strengths (bending moments, axial forces, and shear forces) in structural members are computed by accepted methods of structural analysis for the specified nominal or working loads for all applicable load combinations.
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determined according to Specification Section A4.1.2. These required allowable strengths are not to exceed the allowable design strengths permitted by the Specification. According to Specification Section A4.1.1, the allowable design strength is determined by dividing the nominal strength by a factor of safety as follows:

\[
R \leq \frac{R_n}{\Omega} \quad \text{(C-A4.1.1-1)}
\]

where

- \( R \) = required allowable strength
- \( R_n \) = nominal strength
- \( \Omega \) = factor of safety

The fundamental nature of the factor of safety is to compensate for uncertainties inherent in the design, fabrication, or erection of building components, as well as uncertainties in the estimation of applied loads. Appropriate factors of safety are explicitly specified in various sections of the Specification. Through experience it has been established that the present factors of safety provide satisfactory design. It should be noted that the ASD method employs only one factor of safety for a given condition regardless of the type of load.

A4.1.2 Load Combinations for ASD

Comments for load combinations are provided in Appendix A or C of this Commentary.

A5 Load and Resistance Factor Design

A5.1 Design Basis

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., postbuckling 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 (Winter, 1970; Pekoz, 1986b; and Yu, 2000), and a continuing research effort provides further improvement in understanding them.

Two types of limit states are considered in the load and resistance factor design method. They are: (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 two limit states are usually referred to as the limit state of strength and limit state of serviceability. Like the ASD method, the LRFD method focuses on the limit state of strength in Specification Section A5.1.1 and the limit state of serviceability in Specification Section A8.
A5.1.1 LRFD Requirements

For the limit state of strength, the general format of the LRFD method is expressed by the following equation:

$$\Sigma \gamma_i Q_i \leq \phi R_n$$  \hfill (C-A5.1.1-1)

or

$$R_u \leq \phi R_n$$

where

- $R_u = \Sigma \gamma_i Q_i$ = required strength
- $R_n$ = nominal resistance
- $\phi$ = resistance factor
- $\gamma_i$ = load factors
- $Q_i$ = load effects
- $\phi R_n$ = design strength

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 (i.e., bending moment, axial force, or shear force) determined from the specified nominal 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 for LRFD are discussed in the Commentary on Appendices A for the United States and C for Mexico.

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 ASD method.

(a) 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 Figure C-A5.1.1-1). 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 Figure C-A5.1.1-2. 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 $\beta$ (Figure C-A5.1.1-2), since $\beta \sigma_{\ln(R/Q)} = \ln(R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{\frac{V^2_R}{R} + \frac{V^2_Q}{Q}}} \quad \text{(C-A5.1.1-2)}$$

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 for 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.

The ASD design requirement of the Specification for such a beam is

\[ S_e F_y / \Omega = \left( \frac{L_s^2 s}{8} \right) (D + L) \]  

where

- \( S_e \) = elastic section modulus based on the effective section
- \( \Omega \) = 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 (Ravindra and Galambos, 1978)

\[ R_m = R_n (P_m M_m F_m) \]  

In the above equation, \( R_n \) is the nominal resistance, which in this case is

\[ R_n = S_e F_y \]  

that is, the nominal moment predicted on the basis of the postbuckling strength of the compression flange and the web. 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:

- \( P_m \) = 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 actual 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

\[ V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \]  

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 from research (Hsiao, Yu, and Galambos, 1988a and 1990; Hsiao, 1989) 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

\[ Q_m = \left( \frac{L_s^2 s}{8} \right) (D_m + L_m) \]  

and
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 a study of the National Bureau of Standards (NBS) (Ellingwood et al., 1980), where it was shown that $D_m = 1.05D$, $V_D = 0.1$; $L_m = L$, $V_L = 0.25$.

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 Equations C-A5.1.1-7 and C-A5.1.1-8 gives

$$Q_m = \frac{L_s^2 s}{8} \left( \frac{1.05D}{L} + 1 \right) L$$

(C-A5.1.1-9)

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_L^2}}{(1.05D/L + 1)}$$

(C-A5.1.1-10)

$Q_m$ and $V_Q$ thus depend on the dead-to-live load ratio. Cold-formed steel beams typically have small $D/L$ ratio, which may vary for different applications. Different $D/L$ ratio may be assumed by different countries for developing design criteria. For the purposes of checking the reliability of these LRFD criteria it has been assumed that $D/L = 1/5$, and so $Q_m = 1.21L(L_s^2 s/8)$ and $V_Q = 0.21$.

From Equations C-A5.1.1-3 and C-A5.1.1-5, the nominal resistance, $R_n$, can be obtained for $D/L = 1/5$ and $\Omega = 5/3$ as follows:

$$R_n = 2L(L_s^2 s/8)$$

In order to determine the reliability index, $\beta$, from Equation C-A5.1.1-2, the $R_m/Q_m$ ratio is required by considering $R_m = 1.22R_n$:

$$\frac{R_m}{Q_m} = \frac{1.22 \times 2.0L(L_s^2 s/8)}{1.21L(L_s^2 s/8)} = 2.02$$

Therefore, from Equation C-A5.1.1-2,

$$\beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79$$

Of itself $\beta = 2.79$ for beams having different compression flanges with partially and fully effective flanges and webs designed by the Specification means nothing. However, when this is compared to $\beta$ for other types of cold-formed steel 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 (Galambos et al., 1982).
(b) 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 by Ravindra and Galambos (1978), 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 several research reports of the University of Missouri-Rolla (Hsiao, Yu, and Galambos, 1988a; Rang, Galambos, and Yu, 1979a, 1979b, 1979c, and 1979d; Supornsilaphachai, Galambos, and Yu, 1979), 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 by Ellingwood, Galambos, MacGregor, and Cornell (Ellingwood et al., 1980; Galambos et al., 1982; Ellingwood et al., 1982) using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions and to describe the true distributions more realistically. The details of this extensive reanalysis are presented by the investigators. Only the final conclusions from the analysis are summarized below.

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 by Ellingwood et al. (1982) 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 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 ASCE 7-98 Load Standard (ASCE, 1998).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the allowable strength design method in the current Specification or to any previous version of the AISI specification, it was shown that for the representative dead-to-live load ratio of 1/5 the reliability index \( \beta = 2.79 \). 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 2.79, a somewhat lower target reliability index of $\beta_0 = 2.5$ is recommended as a lower limit in the United States. The resistance factors $\phi$ were selected such that $\beta_0 = 2.5$ 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 $\beta_0 = 3.5$ is recommended for joints and fasteners in the United States. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by the ASCE 7-98 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the AISC LRFD Specification (AISC, 1999). For wind loading, the same ASCE target value of $\beta_0 = 2.5$ is used for connections in the US LRFD method. For flexural members such as individual purlins, girts, panels, and roof decks subjected to the combination of dead and wind loads, the target $\beta_0$ value used in the United States is reduced to 1.5. With this reduced target reliability index, the design based on the US LRFD method is comparable to the US allowable strength design method.

(c) Resistance Factors

The following portions of this Commentary present the background for the resistance factors $\phi$ which are recommended for various members and connections in Chapters B through E (AISI, 1996). These $\phi$ factors are determined in conformance with the ASCE 7 load factors to provide approximately a target $\beta_0$ of 2.5 for members and 3.5 for connections, respectively, for a typical 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} (C-A5.1.1-11)

where $c$ is the deterministic influence coefficient translating load intensities to load effects.

By assuming $D/L = 1/5$, Equations C-A5.1.1-11 and C-A5.1.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi)$$  \hspace{1cm} (C-A5.1.1-12)

$$Q_m = (1.05D/L+1)cL = 1.21cL$$  \hspace{1cm} (C-A5.1.1-13)

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n)$$  \hspace{1cm} (C-A5.1.1-14)

The $\phi$ factor can be computed from Equation C-A5.1.1-15 on the basis of Equations C-A5.1.1-2, C-A5.1.1-4 and C-A5.1.1-14 (Hsiao, Yu and Galambos, 1988b, AISI 1996):

$$\phi = 1.521 (P_m M_m F_m) \exp(-\beta_0 \sqrt{V^2 + V_Q^2})$$  \hspace{1cm} (C-A5.1.1-15)

in which, $\beta_0$ is the target reliability index. Other symbols were defined previously.
By knowing the $\phi$ factor, the corresponding factor of safety, $\Omega$, for allowable strength design can be computed for the load combination $1.2D+1.6L$ as follows:

$$\Omega = \frac{(1.2D/L + 1.6)}{\phi(D/L + 1)}$$  \hfill (C-A5.1.1-16)

where $D/L$ is the dead-to-live load ratio for the given condition.

### A5.1.2 Load Factors and Load Combinations for LRFD

Comments for load factors and load combinations are provided in Appendix A or C of this Commentary.

### A6 Limit States Design

#### A6.1 Design Basis

Same as the LRFD method, 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., postbuckling 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.

Two types of limit states are considered in the Limit States Design method. They are: (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 two limit states are usually referred to as the limit state of strength and limit state of serviceability. The LSD method focuses on the limit state of strength in Specification Section A6.1.1 and the limit state of serviceability in Specification Section A8.

#### A6.1.1 LSD Requirements

For the limit state of strength, the general format of the LSD method is expressed by the following equation:

$$\phi R_n \geq \Sigma \gamma_i Q_i$$ \hfill (C-A6.1.1-1)

or

$$\phi R_n \geq R_f$$

where

- $R_f = \Sigma \gamma_i Q_i =$ effect of factored loads
- $R_n =$ nominal resistance
- $\phi =$ resistance factor
- $\gamma_i =$ load factors
- $Q_i =$ load effects
- $\phi R_n =$ factored resistance
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 resistance. 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 (i.e., bending moment, axial force, or shear force) determined from the specified nominal 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 for LSD are discussed in the Commentary on Appendix B.

Since the design basis for the LSD and the LRFD is the same, further discussions on how to obtain resistance factor using probability analysis can be obtained from Section A5.1.1 (c) of the Commentary. However, attention should be paid that target values for members and connections as well as the dead-to-live load ratio may vary from country to country. These variations lead to the differences in resistance factors. The dead-to-live ratio used in Canada is assumed to be 1/3, and the target of the reliability index for cold-formed steel structural members is 3.0 for members and 4.0 for connections. These target values are consistent with those used in other CSA design standards.

**A6.1.2 Load Factors and Load Combinations for LSD**

Comments for load factors and load combinations are provided in Appendix B of this Commentary.

**A7 Yield Point and Strength Increase from Cold Work of Forming**

**A7.1 Yield Point**

The strength [resistance] of cold-formed steel structural members depends on the yield point or yield strength, except in those cases where elastic local buckling or overall buckling is critical. Because the stress-strain curve of steel sheet or strip can be either sharp-yielding type (Figure C-A7.1-1(a)) or gradual-yielding type (Figure C-A7.1-1(b)), the method for determining the yield point for sharp-yielding steel and the yield strength for gradual-yielding steel are based on the ASTM Standard A370 (ASTM, 1997). As shown in Figure C-A7.1-2(a), the yield point for sharp-yielding steel is defined by the stress level of the plateau. For gradual-yielding steel, the stress-strain curve is rounded out at the “knee” and the yield strength is determined by either the offset method (Figure C-A7.1-2(b)) or the extension under the load method (Figure C-A7.1-2(c)). The term **yield point** used in the Specification applies to either yield point or yield strength. Section 1.2 of the AISI Design Manual (AISI, 2002) lists the minimum mechanical properties specified by the ASTM specifications for various steels.
The strength [resistance] of members that are governed by buckling depends not only on the yield point but also on the modulus of elasticity, $E$, and the tangent modulus, $E_t$. The modulus of elasticity is defined by the slope of the initial straight portion of the stress-strain curve (Figure C-A7.1-1). The measured values of $E$ on the basis of the standard methods usually range from 29,000 to 30,000 ksi (200 to 207 GPa or $2.0 \times 10^6$ to $2.1 \times 10^6$ kg/cm²). A value of 29,500 ksi (203 GPa or $2.07 \times 10^6$ kg/cm²) is used in the Specification for design purposes. The tangent modulus is defined by the slope of the stress-strain curve at any stress level, as shown in Figure C-A7.1-1(b).

For sharp-yielding steels, $E_t = E$ up to the yield point, but with gradual-yielding steels, $E_t = E$ only up to the proportional limit, $f_{pr}$. Once the stress exceeds the proportional limit, the tangent modulus $E_t$ becomes progressively smaller than the initial modulus of elasticity.

Various buckling provisions of the Specification have been written for gradual-yielding steels whose proportional limit is not lower than about 70 percent of the specified minimum yield point.

Determination of proportional limits for information purposes can be done simply by using the offset method shown in Figure C-A7.1-2(b) with the distance “om” equal to 0.0001 length/length (0.01 percent offset) and calling the stress $R$ where “mn” intersects the stress-strain curve at “r”, the proportional limit.
A7.2 Strength Increase from Cold Work of Forming

The mechanical properties of the flat steel sheet, strip, plate or bar, such as yield point, tensile strength, and elongation may be substantially different from the properties exhibited by the cold-formed steel sections. Figure C-A7.2-1 illustrates the increase of yield strength and tensile strength from those of the virgin material at the section locations in a cold-formed steel channel section and a joist chord (Karren and Winter, 1967). This difference can be attributed to cold working of the material during the cold-forming process.

The influence of cold work on mechanical properties was investigated by Chajes, Britvec, Winter, Karren, and Uribe at Cornell University in the 1960s (Chajes, Britvec, and Winter, 1963; Karren, 1967; Karren and Winter, 1967; Winter and Uribe, 1968). It was found that the changes of mechanical properties due to cold-stretching are caused mainly by strain-hardening and strain-aging, as illustrated in Figure C-A7.2-2 (Chajes, Britvec, and Winter, 1963). In this figure, curve A represents the stress-strain curve of the virgin material. Curve B is due to unloading in the strain-hardening range, curve C represents immediate reloading, and curve D is the stress-strain curve of reloading after strain-aging. It is interesting to note that the yield points of both curves C and D are higher than the yield point of the virgin material and that the ductilities decrease after strain hardening and strain aging.

Cornell research also revealed that the effects of cold work on the mechanical properties of corners usually depend on (1) the type of steel, (2) the type of stress (compression or tension), (3) the direction of stress with respect to the direction of cold work (transverse or longitudinal), (4) the
F_u/F_y ratio, (5) the inside radius-to-thickness ratio (R/t), and (6) the amount of cold work. Among the above items, the F_u/F_y and R/t ratios are the most important factors to affect the change in mechanical properties of formed sections. Virgin material with a large F_u/F_y ratio possesses a large potential for strain hardening. Consequently as the F_u/F_y ratio increases, the effect of cold work on the increase in the yield point of steel increases. Small inside radius-to-thickness ratios, R/t, correspond to a large degree of cold work in a corner, and therefore, for a given material, the smaller the R/t ratio, the larger the increase in yield point.

Investigating the influence of cold work, Karren derived the following equations for the ratio of corner yield strength-to-virgin yield strength (Karren, 1967):

\[
\frac{F_{yc}}{F_{yyv}} = \frac{B_c}{(R/t)^m}
\]

where

\[
B_c = 3.69 \frac{F_{uv}}{F_{yyv}} - 0.819 \left( \frac{F_{uv}}{F_{yyv}} \right)^2 - 1.79
\]

and

\[
m = 0.192 \frac{F_{uv}}{F_{yyv}} - 0.068
\]

F_{yc} = corner yield strength
F_{yyv} = virgin yield strength
F_{uv} = virgin ultimate tensile strength
R = inside bend radius
t = sheet thickness

With regard to the full-section properties, the tensile yield strength of the full section may be approximated by using a weighted average as follows:

\[
F_{ya} = CF_{yc} + (1 - C)F_{yf}
\]

where

F_{ya} = full-section tensile yield strength
F_{yc} = average tensile yield strength of corners = B_cF_{yyv}/(R/t)^m
F_{yf} = average tensile yield strength of flats
C = ratio of corner area to total cross-sectional area. For flexural members having unequal flanges, the one giving a smaller C value is considered to be the controlling flange

Good agreements between the computed and the tested stress-strain characteristics for a channel section and a joist chord section were demonstrated by Karren and Winter (Karren and Winter, 1967).
In the last three decades, additional studies have been made by numerous investigators. These investigations dealt with the cold-formed sections having large R/t ratios and with thick materials. They also considered residual stress distribution, simplification of design methods, and other related subjects. For details, see Yu (2000).

In 1962, the AISI Specification permitted the utilization of cold work of forming on the basis of full section tests. Since 1968, the AISI Specification has allowed the use of the increased average yield point of the section, \( F_{ya} \), to be determined by (1) full section tensile tests, (2) stub column tests, or (3) computed in accordance with Equation C-A7.2-2. However, such a strength increase is limited only to relatively compact sections designed according to Specification Section C3.1 (bending strength excluding the use of inelastic...
reserve capacity), Section C4 (concentrically loaded compression members), Section C5 (combined axial load and bending), Section C6 (closed cylindrical tubular members) and Section D4 (wall studs). In the 1996 edition of the AISI

*Specification*, the strength increase from cold work of forming is also allowed for the design of axially loaded tension members as specified in *Specification* Section C2. Design Example of the 2002 *Cold-Formed Steel Design Manual* (AISI, 2002) demonstrates the use of strength increase from cold work of forming for a channel section to be used as a beam.

In some cases, when evaluating the effective width of the web, the reduction factor $\rho$ according to Section B2 of the *Specification* may be less than unity but the sum of $b_1$ and $b_2$ of Figure B2.3-1 of the *Specification* may be such that the web is fully effective, and cold work of forming may be used. This situation only arises when the web width to flange width ratio, $h_o/b_o$, is less than or equal to 4.

In the development of the AISI LRFD *Specification*, the following statistical data on material and cross-sectional properties were developed by Rang, Galambos and Yu (1979a and 1979b) for use in the derivation of resistance factors $\phi$:

- $$(F_y)_m = 1.10 F_y; \quad M_m = 1.10; \quad V_{Fy} = V_M = 0.10$$
- $$(F_{ya})_m = 1.10 F_{ya}; \quad M_m = 1.10; \quad V_{Fya} = V_M = 0.11$$
- $$(F_u)_m = 1.10 F_u; \quad M_m = 1.10; \quad V_{Fu} = V_M = 0.08$$
- $$F_m = 1.00; \quad V_F = 0.05$$

In the above expressions, $m$ refers to mean value, $V$ represents coefficient of variation, $M$ and $F$ are, respectively, the ratios of the actual-to-the nominal material property and 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 statistical data are based on the analysis of many samples (Rang et al., 1978) and they are representative properties of materials and cross sections used in the industrial application of cold-formed steel structures.

A8 Serviceability

Serviceability limit states are conditions under which a structure can no longer perform its intended functions. Safety and strength [resistance] considerations are generally not affected by serviceability limit states. However, serviceability criteria are essential to ensure functional performance and economy of design.

Common conditions which may require serviceability limits are:
1. Excessive deflections or rotations which may affect the appearance or functional use of the structure. Deflections which may cause damage to non-structural elements should be considered.
2. Excessive vibrations which may cause occupant discomfort of equipment malfunctions.
3. Deterioration over time which may include corrosion or appearance considerations.

When checking serviceability, the designer should consider appropriate service loads, the response of the structure, and the reaction of building occupants.

Service loads that may require consideration include static loads, snow or rain loads, temperature fluctuations, and dynamic loads from human activities, wind-induced effects, or the operation of equipment. The service loads are actual loads that act on the structure at an arbitrary point in time. Appropriate service loads for checking serviceability limit states may only be a fraction of the nominal loads.

The response of the structure to service loads can normally be analyzed assuming linear elastic behavior. However, members that accumulate residual deformations under service loads may require consideration of this long-term behavior.

Serviceability limits depend on the function of the structure and on the perceptions of the observer. In contrast to the strength [resistance] limit states, it is not possible to specify general serviceability limits that are applicable to all structures. The Specification does not contain explicit requirements, however, guidance is generally provided by the applicable building code. In the absence of specific criteria, guidelines may be found in Fisher and West (1990), Ellingwood (1989), Murray (1991), AISC (1997) and ATC (1999).

A9 Referenced Documents

Other specifications and standards to which the Specification makes references to have been listed and updated in Specification Section A9 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 listed at the end of the *Commentary.*
B. ELEMENTS

In cold-formed steel construction, individual elements of steel structural members are thin and the width-to-thickness ratios are large as compared with hot-rolled steel shapes. These thin elements may buckle locally at a stress level lower than the yield point of steel when they are subjected to compression in flexural bending, axial compression, shear, or bearing. Figure C-B-1 illustrates some local buckling patterns of certain beams and columns (Yu, 2000).

Because local buckling of individual elements of cold-formed steel sections is a major design criterion, the design of such members should provide sufficient safety against the failure by local instability with due consideration given to the postbuckling strength of structural components. Chapter B of the Specification contains the design requirements for width-to-thickness ratios and the design equations for determining the effective widths of stiffened compression elements, unstiffened compression elements, elements with edge stiffeners or intermediate stiffeners, and beam webs. The design provisions are provided for the use of stiffeners in Specification Section C3.6 for flexural members.

![Figure C-B-1 Local Buckling of Compression Elements](image)

(a) beams, (b) columns

B1 Dimensional Limits and Considerations

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

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

Section B1.1 (a) of the Specification contains limitations on permissible flat-width-to-thickness ratios of compression elements. To some extent,
these limitations are arbitrary. They do, however, reflect a long time experience and are intended to delimit practical ranges (Winter, 1970).

The limitation to a maximum w/t of 60 for the compression flanges having one longitudinal edge connected to a web and the other edge is stiffened by a simple lip is based on the fact that if the w/t ratio of such a flange exceeds 60, a simple lip with a relatively large depth would be required to stiffen the flange (Winter, 1970). The local instability of the lip would necessitate a reduction of the bending capacity to prevent premature buckling of the stiffening lip. This is the reason why the w/t ratio is limited to 60 for stiffened compression elements having one longitudinal edge connected to a web or flange element and the other is stiffened by a simple lip.

The limitation to w/t = 90 for compression flanges with any other kind of stiffeners indicates that thinner flanges with large w/t ratios are quite flexible and liable to be damaged in transport, handling and erection. The same is true for the limitation to w/t = 500 for stiffened compression elements with both longitudinal edges connected to other stiffened elements and for the limitation to w/t = 60 for unstiffened compression elements. The note specifically states that wider flanges are not unsafe, but that when the w/t ratio of unstiffened flanges exceeds 30 and the w/t ratio of stiffened flanges exceeds 250, it is likely to develop noticeable deformation at the full design strength [resistance], without affecting the ability of the member to develop required strength [resistance]. In both cases the maximum w/t is set at twice that ratio at which first noticeable deformations are likely to appear, based on observations of such members under tests. These upper limits will generally keep such deformations to reasonable limits. In such cases where the limits are exceeded, tests in accordance with Specification Chapter F are required.

(b) Flange Curling

In beams which have unusually wide and thin, but stable flanges, (i.e., primarily tension flanges with large w/t ratios), there is a tendency for these flanges to curl under bending. That is, the portions of these flanges most remote from the web (edges of I-beams, center portions of flanges of box or hat beams) tend to deflect toward the neutral axis. An approximate, analytical treatment of this problem was given by Winter (1948b). Equation B1.1-1 of the Specification permits one to compute the maximum permissible flange width, w_f, for a given amount of flange curling, c_f.

It should be noted that Section B1.1(b) does not stipulate the amount of curling which can be regarded as tolerable, but an amount of curling in the order of 5 percent of the depth of the section is not excessive under usual conditions. In general, flange curling is not a critical factor to govern the flange width. However, when the appearance of the section is important, the out-of-plane distortion should be closely controlled in practice. Example of the AISI Cold-Formed Steel Design Manual (AISI, 2002) illustrates the design consideration for flange curling.
(c) *Shear Lag Effects - Short Spans Supporting Concentrated Loads*

For the beams of usual shapes, the normal stresses are induced in the flanges through shear stresses transferred from the web to the flange. These shear stresses produce shear strains in the flange which, for ordinary dimensions, have negligible effects. However, if flanges are unusually wide (relative to their length) these shear strains have the effect that the normal bending stresses in the flanges decrease with increasing distance from the web. This phenomenon is known as shear lag. It results in a non-uniform stress distribution across the width of the flange, similar to that in stiffened compression elements (see Section B2 of the Commentary), though for entirely different reasons. The simplest way of accounting for this stress variation in design is to replace the non-uniformly stressed flange of actual width $w_f$ by one of reduced, effective width subject to uniform stress (Winter, 1970).

Theoretical analyses by various investigators have arrived at results which differ numerically (Roark, 1965). The provisions of Section B1.1(c) are based on the analysis and supporting experimental evidence obtained by detailed stress measurements on eleven beams (Winter, 1940). In fact, the values of effective widths in Specification Table B1.1(c) are taken directly from Curve A of Figure 4 of Winter (1940).

It will be noted that according to Specification Section B1.1(c), the use of a reduced width for stable, wide flanges is required only for concentrated load as shown in Figure C-B1.1-1. For uniform load it is seen from Curve B of the figure that the width reduction due to shear lag for any unrealistically large width-span ratios is so small as to be practically negligible.

![Figure C-B1.1-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams](image)

The phenomenon of shear lag is of considerable consequence in naval architecture and aircraft design. However, in cold-formed steel construction it is infrequent that beams are so wide as to require
significant reductions according to Specification Section B1.1(c). For design purpose, see Example of the AISI Design Manual (AISI, 2002).

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

Prior to 1980, the maximum web depth-to-thickness ratio, $h/t$, was limited to (a) 150 for cold-formed steel members with unreinforced webs and (b) 200 for members which are provided with adequate means of transmitting concentrated loads and/or reactions into the web. Based on the studies conducted at the University of Missouri-Rolla in the 1970s (LaBoube and Yu, 1978a, 1978b, and 1982b; Hettrakul and Yu, 1978 and 1980; Nguyen and Yu, 1978a and 1978b), the maximum h/t ratios were increased to (a) 200 for unreinforced webs, (b) 260 for using bearing stiffeners and (c) 300 for using bearing and intermediate stiffeners in the 1980 edition of the AISI Specification. These h/t limitations are the same as that used in the AISC Specification (AISC, 1989) for plate girders and are retained in the current edition of the Specification. Because the definition for “$h$” was changed in the 1986 edition of the AISI Specification from the “clear distance between flanges” to the “depth of flat portion,” measured along the plane of web, the prescribed maximum h/t ratio may appear to be more liberal. An unpublished study by LaBoube concluded that the present definition for $h$ had negligible influence on the web strength [resistance].

### B2 Effective Widths of Stiffened Elements

It is well known that the structural behavior and the load-carrying capacity of the stiffened compression element such as the compression flange of the hat section depend on the $w/t$ ratio and the supporting condition along both longitudinal edges. If the $w/t$ ratio is small, the stress in the compression flange can reach the yield point of steel and the strength [resistance] of the compression element is governed by yielding. For the compression flange with large $w/t$ ratios, local buckling (Figure C-B2-1) will occur at the following elastic critical buckling stress:

\[
f_{cr} = \frac{k \pi^2 E}{12(1-\mu^2)(w/t)^2}
\]

(C-B2-1)

where

- $E$ = modulus of elasticity of steel
- $k$ = plate buckling coefficient (Table C-B2-1)
- $k = 4$ for stiffened compression elements supported by a web on each longitudinal edge
- $t$ = thickness of the compression element
- $w$ = flat width of the compression element
- $\mu$ = Poisson’s ratio = 0.3 for steel in the elastic range

When the elastic critical buckling stress computed according to Equation C-B2-1 exceeds the proportional limit of the steel, the compression element will buckle in the inelastic range (Yu, 2000).
Unlike one-dimensional structural members such as columns, stiffened compression elements will not collapse when the buckling stress is reached. An additional load can be carried by the element after buckling by means of a redistribution of stress. This phenomenon is known as postbuckling strength [resistance] of the compression elements and is most pronounced for stiffened compression elements with large w/t ratios. The mechanism of the postbuckling action of compression elements was discussed by Winter in previous editions of the AISI Commentary (Winter, 1970).

Imagine for simplicity a square plate uniformly compressed in one direction, with the unloaded edges simply supported. Since it is difficult to visualize the performance of such two-dimensional elements, the plate will be replaced by a model which is shown on Figure C-B2-2. It consists of a grid of longitudinal and transverse bars in which the material of the actual plate is thought to be concentrated. Since the plate is uniformly compressed, each of the longitudinal struts represents a column loaded by $P/5$, if $P$ is the total load on the plate. As the load is gradually increased the compression stress in each of these struts will reach the critical column buckling value and all five struts will tend to buckle simultaneously. If these struts were simple columns, unsupported except at the ends, they would simultaneously collapse through unrestrained increasing lateral deflection. It is evident that this cannot occur in the grid model of the plate. Indeed, as soon as the longitudinal struts start deflecting at their buckling stress, the transverse bars which are connected to them must stretch like ties in order to accommodate the imposed deflection. Like any structural material, they resist stretch and, thereby, have a restraining effect on the deflections of the longitudinal struts.

The tension forces in the horizontal bars of the grid model correspond to the so-called membrane stresses in a real plate. These stresses, just as in the grid model, come into play as soon as the compression stresses begin to cause buckling waves. They consist mostly of transverse tension, but also of some shear stresses, and they counteract increasing wave deflections, i.e. they tend to stabilize the plate against further buckling under the applied increasing longitudinal compression. Hence, the resulting behavior of the model is as follows: (a) there is no collapse by unrestrained deflections, as in unsupported
columns, and (b) the various struts will deflect unequal amounts, those nearest the supported edges being held almost straight by the ties, those nearest the center being able to deflect most.

**Table C-B2-1 Values of Plate Buckling Coefficients**

<table>
<thead>
<tr>
<th>Case</th>
<th>Boundary condition</th>
<th>Type of stress</th>
<th>Value of k for long plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>s.s. s.s. s.s.</td>
<td>Compression</td>
<td>4.0</td>
</tr>
<tr>
<td>(b)</td>
<td>Fixed s.s. s.s.</td>
<td>Compression</td>
<td>6.97</td>
</tr>
<tr>
<td>(c)</td>
<td>s.s. s.s.</td>
<td>Compression</td>
<td>0.425</td>
</tr>
<tr>
<td>(d)</td>
<td>Fixed s.s.</td>
<td>Compression</td>
<td>1.277</td>
</tr>
<tr>
<td>(e)</td>
<td>Fixed s.s.</td>
<td>Compression</td>
<td>5.42</td>
</tr>
<tr>
<td>(f)</td>
<td>S.S. S.S.</td>
<td>Shear</td>
<td>5.34</td>
</tr>
<tr>
<td>(g)</td>
<td>Fixed Fixed Fixed</td>
<td>Shear</td>
<td>8.98</td>
</tr>
<tr>
<td>(h)</td>
<td>s.s. s.s.</td>
<td>Bending</td>
<td>23.9</td>
</tr>
<tr>
<td>(i)</td>
<td>Fixed Fixed Fixed</td>
<td>Bending</td>
<td>41.8</td>
</tr>
</tbody>
</table>

In consequence of (a), the model will not collapse and fail when its buckling stress (Equation C-B2-1) is reached; in contrast to columns it will merely develop slight deflections but will continue to carry increasing load. In consequence of (b), the struts (strips of the plate) closest to the center, which deflect most, “get away from the load,” and hardly participate in carrying any further load increases. These center strips may in fact, even transfer part of their pre-buckling load to their neighbors. The struts (or strips) closest to the edges, held straight by the ties, continue to resist increasing load with hardly any increasing deflection. For the plate, this means that the hitherto uniformly distributed compression stress re-distributes itself in a manner shown on Figure C-B2-3, the stresses being largest at the edges and smallest in the center. With further increase in load this non-uniformity increases further, as also shown on Figure C-B2-3. The plate fails,
i.e., refuses to carry any further load increases, only when the most highly stressed strips, near the supported edges, begin to yield, i.e., when the compression stress $f_{\text{max}}$ reaches the yield point $F_y$.

This postbuckling strength [resistance] of plates was discovered experimentally in 1928, and an approximate theory of it was first given by Th. v. Karman in 1932 (Bleich, 1952). It has been used in aircraft design ever since. A graphic illustration of the phenomenon of postbuckling strength [resistance] can be found in the series of photographs on Figure 7 of Winter (1959b).

The model of Figure C-B2-2 is representative of the behavior of a compression element supported along both longitudinal edges, as the flange in Figure C-B2-1. In fact, such elements buckle into approximately square waves.
In order to utilize the postbuckling strength [resistance] of the stiffened compression element for design purposes, the AISI Specification has used the effective design width approach to determine the sectional properties since 1946. In Section B2 of the present Specification, design equations for computing the effective widths are provided for the following four cases: (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular holes, (3) webs and stiffened elements with stress gradient, and (4) C-section webs with holes under stress gradient. The background information on various design requirements is discussed in subsequent sections.

**B2.1 Uniformly Compressed Stiffened Elements**

(a) **Effective Width for Strength [Resistance] Determination**

In the “effective design width” approach, instead of considering the nonuniform distribution of stress over the entire width of the plate \( w \), it is assumed that the total load is carried by a fictitious effective width \( b \), subject to a uniformly distributed stress equal to the edge stress \( f_{\text{max}} \), as shown in Figure C-B2-3. The width \( b \) is selected so that the area under the curve of the actual nonuniform stress distribution is equal to the sum of the two parts of the equivalent rectangular shaded area with a total width \( b \) and an intensity of stress equal to the edge stress \( f_{\text{max}} \).

Based on the concept of “effective width” introduced by von Karman et al. (von Karman, Sechler and Donnell, 1932) and the extensive investigation on light-gage, cold-formed steel sections at Cornell University, the following equation was developed by Winter in 1946 for determining the effective width \( b \) for stiffened compression elements simply supported along both longitudinal edges:

\[
b = 1.9t \sqrt{\frac{E}{f_{\text{max}}}} \left[ 1 - 0.475 \left( \frac{t}{w} \right) \frac{E}{f_{\text{max}}} \right] \quad \text{(C-B2.1-1)}
\]

The above equation can be written in terms of the ratio of \( F_{\text{cr}}/f_{\text{max}} \) as follows:

\[
\frac{b}{w} = \sqrt{\frac{F_{\text{cr}}}{f_{\text{max}}}} \left[ 1 - 0.25 \frac{F_{\text{cr}}}{f_{\text{max}}} \right] \quad \text{(C-B2.1-2)}
\]

where \( F_{\text{cr}} \) is the critical elastic buckling stress of a plate, and is expressed in Eq. C-B2-1.

Thus, the effective width expression (e.g., C-B2.1-1) provides a prediction of the nominal strength [resistance] based only on the critical elastic buckling stress and the applied stress of the plate. During the period from 1946 to 1968, the AISI design provision for the determination of the effective design width was based on Equation C-B2.1-1. A long-time accumulated experience has indicated that a more realistic equation, as shown below may be used for the determination of the effective width \( b \) (Winter, 1970):
The correlation between the test data on stiffened compression elements and Equation C-B2.1-3 is illustrated by Yu (2000). It should be noted that Equation C-B2.1-3 may also be rewritten in terms of the $F_{cr}/f_{max}$ ratio as follows:

$$\frac{b}{w} = \frac{F_{cr}}{f_{max}} \left(1 - 0.22 \frac{F_{cr}}{f_{max}} \right)$$

(C-B2.1-4)

Therefore, the effective width, $b$, can be determined as

$$b = \rho w$$

(C-B2.1-5)

where $\rho = \text{reduction factor}$

$$(1 - 0.22 / \sqrt{f_{max}/F_{cr}}) / \sqrt{f_{max}/F_{cr}} = (1 - 0.22 / \lambda) / \lambda \leq 1$$

(C-B2.1-6)

In Equation C-B2.1-6, $\lambda$ is a slenderness factor determined below.

$$\lambda = \sqrt{f_{max}/F_{cr}}$$

(C-B2.1-7)

Figure C-B2.1-1 shows the relationship between $\rho$ and $\lambda$. It can be seen that when $\lambda \leq 0.673$, $\rho = 1.0$.

Based on Equations C-B2.1-5 through C-B2.1-7 and the unified approach proposed by Pekoz (1986b and 1986c), the 1986 edition of the AISI Specification adopted the nondimensional format in Section B2.1 for determining the effective design width, $b$, for uniformly compressed stiffened elements. The same design equations were used in the 1996 edition of the AISI Specification and is retained in this first edition of the North American Specification. For design examples, see Part I of the AISI Design Manual (AISI, 2002).

(b) **Effective Width for Serviceability Determination**

The effective design width equations discussed above for strength [resistance] determination can also be used to obtain a conservative effective width, $b_{dl}$, for serviceability determination. It is included in

![Figure C-B2.1-1 Reduction Factor, $\rho$, vs. Slenderness Factor, $\lambda$](image-url)
Section B2.1(b) of the *Specification* as Procedure I.

For stiffened compression elements supported by a web on each longitudinal edge, a study conducted by Weng and Pekoz (1986) indicated that Equations B2.1-8 through B2.1-11 of the *Specification* can yield a *more accurate estimate* of the effective width, \( b_d \), for serviceability. These equations are given in Procedure II for additional design information. The design engineer has the option of using one of the two procedures for determining the effective width to be used for serviceability determination.

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

In cold-formed steel structural members, holes are sometimes provided in webs and/or flanges of beams and columns for duct work, piping, and other construction purposes. The presence of such holes may result in a reduction of the strength [resistance] of individual component elements and the overall strength [resistance] and stiffness of the members depending on the size, shape, and arrangement of holes, the geometric configuration of the cross section, and the mechanical properties of the material.

The exact analysis and the design of steel sections having perforations are complex, particularly when the shapes and the arrangement of holes are unusual. The limited design provisions included in Section B2.2 of the *Specification* for uniformly compressed stiffened elements with circular holes are based on a study conducted by Ortiz-Colberg and Pekoz at Cornell University (Ortiz-Colberg and Pekoz, 1981). For additional information on the structural behavior of perforated elements, see Yu and Davis (1973a) and Yu (2000).

**B2.3 Webs and other Stiffened Elements under Stress Gradient**

When a beam is subjected to bending moment, the compression portion of the web may buckle due to the compressive stress caused by bending. The theoretical critical buckling stress for a flat rectangular plate under pure bending can be determined by Equation C-B2-1, except that the depth-to-thickness ratio, \( h/t \), is substituted for the width-to-thickness ratio, \( w/t \), and the plate buckling coefficient, \( k \), is equal to 23.9 for simple supports as listed in Table C-B2-1.

Prior to 1986, the design of cold-formed steel beam webs was based on the full web depth with the allowable bending stress specified in the AISI *Specification*. In order to unify the design methods for web elements and compression flanges, the “effective design depth” approach was adopted in the 1986 edition of the AISI *Specification* on the basis of the studies made by Pekoz (1986b), Cohen and Pekoz (1987). This is a different approach as compared with the past practice of using a full area of the web element in conjunction with a reduced stress to account for local buckling and postbuckling strength (LaBoube and Yu, 1982b; Yu, 1985).

Prior to 2001, the \( b_1 \) and \( b_2 \) expressions used in the AISI *Specification* for the effective width of webs (Equations B2.3-3 through B2.3-5) implicitly
assumed that the flange provided beneficial restraint to the web. Collected data (Cohen and Peköz (1987), Elhour and Murray (1985), Ellifritt et al (1997), Hancock et al (1996), LaBoube and Yu (1978), Moreyra and Peköz (1993), Rogers and Schuster (1995), Schardt and Schrade (1982), Schuster (1992), Shan et al (1994), and Willis and Wallace (1990) as summarized in Schafer and Peköz (1999)) on flexural tests of C’s and Z’s indicate that Specification equations B2.3-3 through B2.3-5 can be unconservative if the overall web width ($h_o$) to overall flange width ($b_o$) ratio exceeds 4. Consequently, in 2001, in the absence of a comprehensive method for handling local web and flange interaction, the North American Specification adopted a two-part approach for the effective width of webs: an additional set of alternative expressions (Eqs B2.3-6 and B2.3-7), originally developed by Cohen and Pekoz (1987) were adopted for $h_o/b_o > 4$; while the expressions adopted in the 1986 edition of the AISI Specification (Eqs B2.3-3 through B2.3-5) remain for $h_o/b_o \leq 4$. For flexural members with local buckling in the web, the effect of these changes is that the strengths [resistances] will be somewhat lower when $h_o/b_o > 4$ compared with the 1996 AISI Specification (AISI, 1996). When compared with the CSA S136 (CSA, 1994) Standard, there are only minor changes for members with $h_o/b_o > 4$, but an increase in strength [resistance] will be experienced when $h_o/b_o \leq 4$.

It should be noted that in the North American Specification, the stress ratio $\psi$ is defined as an absolute value. As a result, some signs for $\psi$ have been changed in Specification Equations B2.3-2, B2.3-3, B2.3-6 and B2.3-7 as compared with the 1996 edition of the AISI Specification (AISI, 1996).

**B2.4 C-Section Webs with Holes under Stress Gradient**

Studies of the behavior of web elements with holes conducted at the University of Missouri-Rolla (UMR) serve as the basis for the design recommendations for bending alone, shear, web crippling, combinations of bending and shear, and bending and web crippling (Shan et al., 1994; Langan et al., 1994; Uphoff, 1996; Deshmukh, 1996). The Specification considers a hole to be any flat punched opening in the web without any edge stiffened openings.

The UMR design recommendations for a perforated web with stress gradient are based on the tests of full-scale C-section beams having $h/t$ ratios as large as 200 and $d_0/h$ ratios as large as 0.74. The test program considered only stud and joist industry standard web holes. These holes were rectangular with fillet corners, punched during the rolling process. For non-circular holes, the corner radii recommendation was adopted to avoid the potential of high stress concentration at the corners of a hole. Webs with circular holes and a stress gradient were not tested, however, the provisions are conservatively extended to cover this case. Other shaped holes must be evaluated by the virtual hole method described below, by test, or by other provisions of the Specification. The Specification is not intended to cover cross sections having repetitive $\frac{1}{2}$ in. diameter holes.
Based on the study by Shan et al. (1994), it was determined that the nominal bending strength [resistance] of a C-section with a web hole is unaffected when $d_0/h < 0.38$. For situations where the $d_0/h \geq 0.38$, the effective depth of the web can be determined by treating the flat portion of the remaining web that is in compression as an unstiffened compression element.

Although these provisions are based on tests of singly-symmetric C-sections having the web hole centered at mid-depth of the section, the provisions may be conservatively applied to sections for which the full unreduced compression region of the web is less than the tension region. However, for cross sections having a compression region greater than the tension region, the web strength [resistance] must be determined by test in accordance with Section F1.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. For example, Figure C-B2.4-1 illustrates the $b$ and $d_0$ that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the $d_0$ that may be used for a rectangular hole that exceeds the 2.5 in. (64 mm) by 4.5 in. (114 mm) limit but still fits within an allowed circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole, not the actual hole or holes.

The effects of holes on shear strength [resistance] and web crippling strength [resistance] of C-section webs are discussed in Sections C3.2.2 and C3.4.2 of the Commentary, respectively.

![Figure C-B2.4-1 Virtual Hole Method for Multiple Openings](image1)

![Figure C-B2.4-2 Virtual Hole Method for Opening Exceeding Limit](image2)
B3 Effective Widths of Unstiffened Elements

Similar to stiffened compression elements, the stress in the unstiffened compression elements can reach to the yield point of steel if the w/t ratio is small. Because the unstiffened element has one longitudinal edge supported by the web and the other edge is free, the limiting width-to-thickness ratio of unstiffened elements is much less than that for stiffened elements.

Figure C-B3-1 Local Buckling of Unstiffened Compression Flange

When the w/t ratio of the unstiffened element is large, local buckling (Figure C-B3-1) will occur at the elastic critical stress determined by Equation C-B2-1 with a value of k=0.43. This buckling coefficient is listed in Table C-B2-1 for case (c). For the intermediate range of w/t ratios, the unstiffened element will buckle in the inelastic range. Figure C-B3-2 shows the relationship between the maximum stress for unstiffened compression elements and the w/t ratio, in which Line A is the yield point of steel, Line B represents the inelastic buckling stress, Curves C and D illustrate the elastic buckling stress. The equations for
Curves A, B, C, and D have been developed from previous experimental and analytical investigations and used for determining the allowable design stresses in the AISI Specification up to 1986 (Winter, 1970; Yu, 2000). Also shown in Figure C-B3-2 is Curve E, which represents the maximum stress on the basis of the postbuckling strength of the unstiffened element. The correlation between the test data on unstiffened elements and the predicted maximum stresses is shown in Figure C-B3-3 (Yu, 2000).

Prior to 1986, it had been a general practice to design cold-formed steel members with unstiffened flanges by using the allowable stress design approach. The effective width equation was not used in earlier editions of the AISI Specification due to lack of extensive experimental verification and the concern for excessive out-of-plane distortions under service loads.

In the 1970s, the applicability of the effective width concept to unstiffened elements under uniform compression was studied in detail by Kalyanaraman, Pekoz, and Winter at Cornell University (Kalyanaraman, Pekoz, and Winter, 1977; Kalyanaraman and Pekoz, 1978). The evaluation of the test data using $k=0.43$ was presented and summarized by Pekoz in the AISI report (Pekoz, 1986b), which indicates that Equation C-B2.1-6 developed for stiffened compression elements gives a conservative lower bound to the test results of unstiffened compression elements. In addition to the strength determination, the same study also investigated the out-of-plane deformations in unstiffened elements. The results of theoretical calculations and the test results on the sections having unstiffened elements with $w/t=60$ were presented by Pekoz in the same report. It was found that the maximum amplitude of the out-of-plane deformation at failure can be twice the thickness as the $w/t$ ratio approaches 60. However, the deformations are significantly less under the service loads. Based
on the above reasons and justifications, the effective design width approach was adopted for the first time in Section B3 of the 1986 AISI Specification for the design of cold-formed steel members having unstiffened compression elements.

**B3.1 Uniformly Compressed Unstiffened Elements**

In the present Specification, it is specified that the effective widths, \( b \), of uniformly compressed unstiffened elements can be determined in accordance with Section B2.1(a) of the Specification with the exception that the buckling coefficient \( k \) is taken as 0.43. This is a theoretical value for long plates. See case (c) in Table C-B2-1. For serviceability determination, the effective widths of uniformly compressed unstiffened elements can only be determined according to Procedure I of Section B2.1(b) of the Specification, because Procedure II was developed only for stiffened compression elements. See Part I of the AISI Design Manual for design examples (AISI, 2002).

**B3.2 Unstiffened Elements and Edge Stiffeners under Stress Gradient**

In concentrically loaded compression members and in flexural members where the unstiffened compression element is parallel to the neutral axis, the stress distribution is uniform prior to local buckling. However, when edge stiffeners of the beam section are turned in or out, the compressive stress in the edge stiffener is not uniform but varies in proportion to the distance from the neutral axis.

There is very limited amount of information on the behavior of unstiffened compression elements with a stress gradient. Cornell research on the behavior of edge stiffeners for flexural members has demonstrated that by using Winter’s effective width equation (Equation C-B2.1-4) with a \( k=0.43 \), good correlation was achieved between the tested and calculated capacity (Pekoz, 1986b). The same trend was also true for serviceability determination. Therefore, in Section B3.2 of the Specification, the effective widths of unstiffened elements and edge stiffeners with stress gradient are treated as uniformly compressed elements with stress \( f \) to be the maximum compression stress in the element.

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

For cold-formed steel beams such as hat, box or inverted U-type sections (Sections (3), (4), and (5) in Figure C-A1.2-2), the compression flange is supported along both longitudinal edges by webs. In this case, if the webs are properly designed, they provide adequate stiffening for the compression elements by preventing their longitudinal edges from out-of-plane displacements. On the other hand, in many cases only one longitudinal edge is stiffened by the web, while the other edge is supported by an edge stiffener. In most cases, the edge stiffener takes the form of a simple lip, such as in the C-section and I-section as shown in Figure C-A1.2-2 for Sections (1) and (2).
The structural efficiency of a stiffened element always exceeds that of an unstiffened element with the same \( w/t \) ratio by a sizeable margin, except for low \( w/t \) ratios, for which the compression element is fully effective. When stiffened elements with large \( w/t \) ratios are used, the material is not employed economically inasmuch as an increasing proportion of the width of the compression element becomes ineffective. On the other hand, in many applications of cold-formed steel construction, such as panels and decks, maximum coverage is desired and, therefore, large \( w/t \) ratios are called for. In such cases, structural economy can be improved by providing intermediate stiffeners between webs. Such intermediate stiffeners provide optimum stiffening if they do not participate in the wave-like distortion of the compression element. In that case they break up the wave pattern so that the two strips to each side of intermediate stiffener distort independently of each other, each in a pattern similar to that shown for a simple, stiffened element in Figure C-B2-1. Compression elements furnished with such intermediate stiffeners are designated as “multiple-stiffened elements.” Illustrative examples are given in Part I of the *Design Manual* (AISI, 2002).

As far as the design provisions are concerned, the 1980 and earlier editions of the AISI *Specification* included the requirements for the minimum moment of inertia of stiffeners to provide sufficient rigidity. When the size of the actual stiffener does not satisfy the required moment of inertia, the load-carrying capacity of the beam had to be determined either on the basis of a flat element disregarding the stiffener or through tests.

In 1986, the AISI *Specification* included the revised provisions in Section B4 for determining the effective widths of elements with an edge stiffener or one intermediate stiffener on the basis of Pekoz’s research findings in regard to stiffeners (Pekoz, 1986b). These design provisions were based on both critical local buckling and postbuckling strength [resistance] criteria recognizing the interaction of plate elements. Also, for the first time, the design provisions could be used for analyzing partially stiffened and adequately stiffened compression elements using different sizes of stiffeners.

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

The buckling behavior of rectangular plates with central stiffeners is discussed by Bulson (1969). For the design of cold-formed steel beams using intermediate stiffeners, the 1980 AISI *Specification* contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. Subsequent research conducted by Desmond, Pekoz, and Winter (1981b) has developed expressions for evaluating the required stiffener rigidity based upon the geometry of the contiguous flat elements.

In view of the fact that for some cases the design requirements for intermediate stiffeners included in the 1980 *Specification* could be unduly conservative (Pekoz, 1986b), the AISI design provisions were revised in 1986 according to Pekoz’s research findings (Pekoz, 1986b and 1986c). In this method, the buckling coefficient for determining the effective width of sub-
elements and the reduced area of the stiffener are to be calculated by using the ratio $I_s/I_a$. In the foregoing expression, $I_s$ is the actual stiffener moment of inertia and $I_a$ is the adequate moment of inertia of the stiffener determined from the applicable equations in the Specification sections. However, a discontinuity could occur in the previous design expressions. To eliminate the discontinuity, Dinovitzer’s expressions (Dinovitzer, et al, 1992) for $n$ (Eq. B4.1-4) was adopted in 2001. This revised equation gives $n = 1/2$ for $b_o/t = S$ and $n = 1/3$ for $b_o/t = 3S$, in which $S$ is the maximum $b_o/t$ ratio for a stiffened element to be fully effective.

**B4.2 Uniformly Compressed Elements with an Edge Stiffener**

An edge stiffener is used to provide a continuous support along a longitudinal edge of the compression flange to improve the buckling stress. Even though in most cases, the edge stiffener takes the form of a simple lip, other types of edge stiffeners can also be used for cold-formed steel members.

In order to provide necessary support for the compression element, the edge stiffener must possess sufficient rigidity. Otherwise it may buckle perpendicular to the plane of the element to be stiffened.

Both theoretical and experimental studies on the local stability of compression flanges stiffened by edge stiffeners have been carried out in the past. The design requirements included in Section B4.2 of the 1986 AISI Specification were based on the investigations on adequately stiffened and partially stiffened elements conducted by Desmond, Pekoz and Winter (1981a), with additional research work of Pekoz and Cohen (Pekoz, 1986b). These design provisions were developed on the basis of the critical buckling criterion and the postbuckling strength [resistance] criterion.

Specification Section B4.2 recognizes that the necessary stiffener rigidity depends upon the slenderness ($w/t$) of the plate element being stiffened. The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for $k$, $d_s$, and $A_s$ (Pekoz, 1986b).

In the 1996 edition of the AISI Specification (AISI, 1996), the design equations for buckling coefficient were changed for further clarity. For $w/t > S/3$, the equation for $k_a = 5.25-5 (D/w) \leq 4.0$ is applicable only for simple lip stiffeners because the term $D/w$ is meaningless for other types of edge stiffeners. It should be noted that the provisions in this section were based on research dealing only with simple lip stiffeners and extension to other types of stiffeners was purely intuitive. The requirement of $140^\circ \geq \theta \geq 40^\circ$ for the applicability of these provisions was also decided on an intuitive basis. For design examples, see Part I of the Cold-Formed Steel Manual (AISI, 2002).

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.

The 1996 Commentary provided a warning to the user that lip lengths with a $d/t$ ratio greater than 14 may give unconservative results. Examination of available experimental data on both flexural members (Rogers and
Schuster, 1996, Schafer and Pekoz, 1999) and compression members (Schafer, 2000) with edge stiffeners indicates that the Specification does not have an inherent problem for members with large d/t ratios. Existing experimental data covers d/t ratios as high as 35 for both flexural and compression members.

In 2001, Dinovitzer’s expressions (Dinovitzer, et al., 1992) for n (Eq. B4.2-11) were adopted, which eliminated a discontinuity that existed in the previous design expressions. The revised equation gives n =1/2 for w/t = 0.328S and n = 1/3 for w/t = S, in which S is also the maximum w/t ratio for a stiffened element to be fully effective.

**B5 Effective Widths of Stiffened Elements with Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffeners**

**B5.1 Effective Width of Uniformly Compressed Stiffened Elements with Multiple Intermediate Stiffeners**

Prior to 2001, the AISI Specification and the Canadian Standard provided design provisions for determination of the effective widths of uniformly compressed stiffened elements with multiple intermediate stiffeners or edge stiffened elements with intermediate stiffeners. In the AISI Specification, the design requirements of Section B5 dealt with (1) the minimum moment of inertia of the intermediate stiffener, (2) the number of intermediate stiffeners considered to be effective, (3) the “equivalent element” of multiple-stiffened element having closely spaced intermediate stiffeners, (4) the effective width of sub-element with w/t > 60, and (5) the reduced area of stiffeners. In the Canadian Standard, a different design equation was used to determine the equivalent thickness.

In 2001, Specification Section B5.1 was revised to reflect recent research findings for flexural members with multiple intermediate stiffeners in the compression flange (Papazian et al. 1994, Schafer and Peköz 1998, Acharya and Schuster 1998). The method is based on determining the plate buckling coefficient for the two competing modes of buckling: local buckling, in which the stiffener does not move; and distortional buckling in which the stiffener buckles with the entire plate. See Figure C-B5.1-1. Experimental research shows that the distortional mode is prevalent for members with multiple intermediate stiffeners.

The reduction factor, $\rho$, is applied to the entire element (gross area of the element/thickness) instead of only the flat portions. Reducing the entire element to an effective width, which ignores the geometry of the stiffeners, for effective section property calculation allows distortional buckling to be treated consistently with the rest of the Specification, rather than as an “effective area” or other method. The resulting effective width must act at the centroid of the original element including the stiffeners. This insures that the neutral axis location for the member is unaffected by the use of the simple effective width, which replaces the more complicated geometry of the element with multiple intermediate stiffeners. One possible result of this approach is that the calculated effective width ($b_{e}$) may be greater than $b_{o}$. This may occur...
when \( \rho \) is near 1, and is due to the fact that \( b_e \) includes contributions from the stiffener area and \( b_o \) does not. As long as the calculated \( b_e \) is placed at the centroid of the entire element the use of \( b_e > b_o \) is correct.

\[
\text{Plate Sub-element}
\]

(a) Local Buckling

\[
\text{(a) Distortional Buckling}
\]

Figure C-B5.1-1 Local and Distortional Buckling of a Uniformly Compressed Element with Multiple Intermediate Stiffeners

**B5.2 Edge Stiffened Elements with Intermediate Stiffeners**

The buckling modes for edge stiffened elements with intermediate stiffeners include: local sub-element buckling, distortional buckling of the intermediate stiffener, and distortional buckling of edge stiffener, as shown in Figure C-B5.2-1. If the edge stiffened element is stocky (\( b_o/t < 0.328S \)) or the stiffener is large enough (\( I_s > I_a \) and thus \( k = 4 \), per the rules of Specification Section B4.2) then the edge stiffened element performs as a stiffened element. In this case, effective width for local sub-element buckling and distortional buckling of the intermediate stiffener may be predicted by the rules of Specification Section B5.1. However, an edge stiffened element does not have the same web rotational restraint as a stiffened element, therefore the constant \( R \) of Specification Section B5.1 is conservatively limited to be less than or equal to 1.0.

Figure C-B5.2-1 Buckling Modes in an Edge Stiffened Element with Intermediate Stiffeners
If the edge stiffened element is partially effective ($b_o/t > 0.3285$ and $I_S < I_a$ and thus $k < 4$, per the rules of Specification Section B4.2) then the intermediate stiffener(s) should be ignored and the provisions of Specification Section B4.2 followed. Elastic buckling analysis of the distortional mode for an edge stiffened element with intermediate stiffener(s) indicates that the effect of intermediate stiffener(s) on the distortional buckling stress is ±10% for practical intermediate and edge stiffener sizes.

When applying section B5.2 for effective width determination of edge stiffened elements with intermediate stiffeners, the effective width of the intermediately stiffened flange, $b_{ew}$, is replaced by an equivalent flat section (as shown in Fig. B5.1-2). The edge stiffener should not be used in determining the centroid location of the equivalent flat effective width, $b_{ew}$, for the intermediately stiffened flange.
C. MEMBERS

This Chapter provides the design requirements for (a) tension members, (b) flexural members, (c) concentrically loaded compression members, (d) combined axial load and bending, and (e) closed cylindrical tubular members. To simplify the use of the Specification, all design provisions for a given specific member type have been assembled in a particular section within the Specification. In general, a common nominal strength [resistance] equation is provided in the Specification for a given limit state with a required factor of safety ($\Omega$) for allowable strength design (ASD) and a resistance factor ($\phi$) for load and resistance factor design (LRFD) or limit state design (LSD). Design provisions that are applicable to a specific country are provided in the corresponding Appendix.

C1 Properties of Sections

The geometric properties of a member (i.e., area, moment of inertia, section modulus, radius of gyration, etc.) are evaluated using conventional methods of structural design. These properties are based upon either full cross-section dimensions, effective widths or net section, as applicable.

For the design of tension members, both gross and net sections are employed when computing the nominal tensile strength [resistance] of the axially loaded tension members.

For flexural members and axially loaded compression members, both full and effective dimensions are used to compute sectional properties. The full dimensions are used when calculating the critical load or moment, while the effective dimensions, evaluated at the stress corresponding to the critical load or moment, are used to calculate the nominal strength [resistance]. For serviceability consideration, the effective dimension should be determined for the compressive stress in the element corresponding to the service load. Pekoz (1986a and 1986b) discussed this concept in more detail.

Section 3 of Part I of the AISI Design Manual (AISI, 2002) deals with the calculation of sectional properties for C-sections, Z-sections, angles, hat sections, and decks.

C2 Tension Members

The design provisions of this section are given in Section C2 of the Appendices. The discussion for this section is provided in the Commentary on the corresponding Appendix.

C3 Flexural Members

For the design of cold-formed steel flexural members, consideration should be given to several design features: (a) bending strength [resistance] and serviceability, (b) shear strength [resistance] of webs and combined bending and shear, (c) web crippling strength [resistance] and combined bending and web crippling, and (d) bracing requirements. For some cases, special consideration should also be given to shear lag and flange curling due to the use of thin...
material. The design provisions for Items (a), (b) and (c) are provided in Specification Section C3, while the requirements for lateral bracing are given in Specification Section D3. The treatments for flange curling and shear lag were discussed in Section B1.1(b) and (c) of the Commentary, respectively.

Example problems are given in Part II of the AISI Cold-Formed Steel Design Manual (AISI, 2002) for the design of flexural members.

C3.1 Bending

Bending strengths [resistances] 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 [resistance] (Specification Section C3.1.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Specification Section C3.1.2). For C- or Z-sections 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 (Specification Section C3.1.3). For C- or Z-sections supporting a standing seam roof system under gravity or uplift loads, the bending capacity is greater than that of an unbraced member and may be equal to that of a fully braced member (Specification Section C3.1.4). Similarly, for standing seam roof systems, design provisions are provided in Specification Section C3.1.5 for evaluating the bending strength of the system based on tests. The governing nominal bending strength [resistance] is the smallest of the values determined from the applicable conditions.

C3.1.1 Nominal Section Strength [Resistance]

Specification Section C3.1.1 includes two design procedures for calculating the nominal section strength [resistance] of flexural members. Procedure I is based on Initiation of Yielding and Procedure II is based on Inelastic Reserve Capacity.

(a) Procedure I - Based on Initiation of Yielding

In Procedure I, the nominal moment, \( M_{ny} \), of the cross section is the effective yield moment, \( M_y \), determined on the basis of the effective areas of flanges and the beam web. The effective width of the compression flange and the effective depth of the web can be computed from the design equations given in Chapter B of the Specification.

Similar to the design of hot-rolled steel shapes, the yield moment \( M_y \) of a cold-formed steel beam is defined as the moment at which an outer fiber (tension, compression, or both) first attains the yield point of the steel. This is the maximum bending capacity to be used in elastic design. Figure C-C3.1.1-1 shows several types of stress distributions for yield moment based on different locations of the neutral axis. For balanced sections (Figure C-C3.1.1-1(a)) the outer fibers in the compression and tension flanges reach the yield point at the same time. However, if the neutral axis is eccentrically located, as shown in
Figures C-C3.1.1-1(b) and (c), the initial yielding takes place in the tension flange for case (b) and in the compression flange for case (c).

Accordingly, the nominal section strength [resistance] for initiation of yielding is calculated by using Equation C-C3.1.1-1:

\[ M_n = S_e F_y \]  

where

- \( F_y \) = design yield stress
- \( S_e \) = elastic section modulus of the effective section calculated with the extreme compression or tension fiber at \( F_y \).

For cold-formed steel design, \( S_e \) is usually computed by using one of the following two cases:

1. If the neutral axis is closer to the tension than to the compression flange, the maximum stress occurs in the compression flange, and therefore the plate slenderness ratio \( \lambda \) and the effective width of the compression flange are determined by the w/t ratio and \( f = F_y \). Of course, this procedure is also applicable to those beams for which the neutral axis is located at the mid-depth of the section.
2. If the neutral axis is closer to the compression than to the tension flange, the maximum stress of $F_y$ occurs in the tension flange. The stress in the compression flange depends on the location of the neutral axis, which is determined by the effective area of the section. The latter cannot be determined unless the compressive stress is known. The closed-form solution of this type of design is possible but would be a very tedious and complex procedure. It is therefore customary to determine the sectional properties of the section by successive approximation.

For determining the design flexural strength [factored resistance], $\phi_bM_{Py}$, by using the LRFD approach, slightly different resistance factors are used for the sections with stiffened or partially stiffened compression flanges and the sections with unstiffened compression flanges. These $\phi_b$ values were derived from the test results and a dead-to-live load ratio of 1/5. They provide the $\beta$ values from 2.53 to 4.05 (AISI, 1991; Hsiao, Yu and Galambos, 1988a).

(b) Procedure II - Based on Inelastic Reserve Capacity

Prior to 1980, the inelastic reserve capacity of beams was not included in the AISI Specification because most cold-formed steel shapes have large width-to-thickness ratios that are considerably in excess of the limits required by plastic design.

In the 1970s and early 1980s, research work on the inelastic strength of cold-formed steel beams was carried out by Reck, Pekoz, Winter, and Yener at Cornell University (Reck, Pekoz and Winter, 1975; Yener and Pekoz, 1985a, 1985b). These studies showed that the inelastic reserve strength [resistance] of cold-formed steel beams due to partial plastification of the cross section and the moment redistribution of...
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statically indeterminate beams can be significant for certain practical shapes. With proper care, this reserve strength [resistance] can be utilized to achieve more economical design of such members.

In order to utilize the available inelastic reserve strength [resistance] of certain cold-formed steel beams, design provisions based on the partial plastification of the cross section were added in the 1980 edition of the AISI Specification. The same provisions are retained in the 2001 edition of the Specification. According to Procedure II of Section C3.1.1(b) of the Specification, the nominal section strength [resistance], \( M_n \), of those beams satisfying certain specific limitations can be determined on the basis of the inelastic reserve capacity with a limit of 1.25\( M_y \), where \( M_y \) is the effective yield moment. The ratio of \( M_n/M_y \) represents the inelastic reserve strength [resistance] of a beam cross section.

The nominal moment \( M_n \) is the maximum bending capacity of the beam by considering the inelastic reserve strength [resistance] through partial plastification of the cross section. The inelastic stress distribution in the cross section depends on the maximum strain in the compression flange, \( \varepsilon_{cu} \). Based on the Cornell research work on hat sections having stiffened compression flanges (Reck, Pekoz and Winter, 1975), the AISI design provision limits the maximum compression strain to be \( C_y \varepsilon_y \), where \( C_y \) is a compression strain factor determined by using the equations provided in Specification Section C3.1.1(b) as shown in Figure C-C3.1.1-2.

On the basis of the maximum compression strain \( \varepsilon_{cu} \) allowed in the Specification, the neutral axis can be located by using Equation C-C3.1.1-2 and the nominal moment \( M_n \) can be determined by using Equation C-C3.1.1-3:

\[
\int \sigma \, dA = 0 \quad (C-C3.1.1-2)
\]

\[
\int \sigma y \, dA = M_n \quad (C-C3.1.1-3)
\]

where \( \sigma \) is the stress in the cross section.

The calculation of \( M_n \) based on inelastic reserve capacity is illustrated in Part I of the AISI Cold-Formed Steel Design Manual (AISI, 2002) and the textbook by Yu (2000).

In 2001, the shear force upper limit was clarified. The stress upper limit is 0.35\( F_y \) for ASD and 0.6\( F_y \) for LRFD and LSD in the North American Specification.

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

C3.1.2.1 Lateral-Torsional Buckling Strength [Resistance] for Open Cross Section Members

The bending capacity of flexural members is not only governed by the strength [resistance] of the cross section, but can also be limited by the lateral-torsional buckling strength [resistance] of the member if braces
are not adequately provided. The design provisions for determining the nominal lateral-torsional buckling strength [resistance] are given in Specification Section C3.1.2.1.

If a doubly-symmetric or singly-symmetric member in bending is laterally unbraced, it can fail in lateral-torsional buckling. For a beam having simply supported end conditions both laterally and torsionally, the elastic critical lateral-torsional buckling stress can be determined by Equation C-C3.1.2.1-1.

\[
\sigma_{cr} = \frac{\pi}{L_s f} \sqrt{\frac{E I_y G}{E I_y G} \left(1 + \frac{\pi^2 E C_w}{G J L^2}\right)} \tag{C-C3.1.2.1-1}
\]

For other than simply supported end conditions, Equation C-C3.1.2.1-1 can be generalized as given in Equation C-C3.1.2.1-1a (Galambos, 1998):

\[
\sigma_{cr} = \frac{\pi}{(K_y L_y) S_f} \sqrt{\frac{E I_y G}{E I_y G} \left[1 + \frac{\pi^2 E C_w}{G J (K_t L_t)^2}\right]} \tag{C-C3.1.2.1-1a}
\]

In the above equation, \(K_y\) and \(K_t\) are effective length factors and \(L_y\) and \(L_t\) are unbraced lengths for bending about the y-axis and for twisting, respectively, \(E\) is the modulus of elasticity, \(G\) is the shear modulus, \(S_f\) is the elastic section modulus of the full unreduced section relative to the extreme compression fiber, \(I_y\) is the moment of inertia about the y-axis, \(C_w\) is the torsional warping constant, \(J\) is the Saint-Venant torsion constant, and \(L\) is the unbraced length.

For equal-flanged I-members with simply supported end conditions both laterally and torsionally, Equation C-C3.1.2.1-2 can be used to calculate the elastic critical buckling stress (Winter, 1947a; Yu, 2000):

\[
\sigma_{cr} = \frac{\pi^2 E}{2(L/d)^2} \sqrt{\left(\frac{I_y}{2I_x}\right)^2 + \left(\frac{I_{y y}}{2(1+\mu)I_x^2}\right) \left(\frac{L}{\pi d}\right)^2} \tag{C-C3.1.2.1-2}
\]

In Equation C-C3.1.2.1-2, the first term under the square root represents the lateral bending rigidity of the member, and the second term represents the Saint-Venant torsional rigidity. For thin-walled cold-formed steel sections, the first term usually exceeds the second term by a considerable margin.

For simply supported I-members with unequal flanges, the following equation has been derived by Winter for the lateral-torsional buckling stress (Winter, 1943):

\[
\sigma_{cr} = \frac{\pi^2 E d}{2L^2 S_f} \left(I_{y c} - I_{y t} + I_y \sqrt{1 + \frac{4GJL^2}{\pi^2 I_y E d^2}}\right) \tag{C-C3.1.2.1-3}
\]

where \(I_{y c}\) and \(I_{y t}\) are the moments of inertia of the compression and tension portions of the full section, respectively, about the centroidal
axis parallel to the web. Other symbols were defined previously. For equal-flange sections, $I_{yc} = I_{yt} = I_y/2$, Equations C-C3.1.2.1-2 and C-C3.1.2.1-3 are identical.

For other than simply supported end conditions, Equation C-C3.1.2.1-3 can be generalized as given in Equation C-C3.1.2.1-3a:

$$\sigma_{cr} = \frac{\pi^2 Ed}{2(K_y L_y)^2 S_f} \left( I_{yc} - I_{yt} + I_y \left( 1 + \frac{4GJ(K_t L_t)^2}{\pi^2 I_y Ed^2} \right) \right)$$  \hspace{1cm} (C-C3.1.2.1-3a)

In Equation C-C3.1.2.1-3a, the second term under the square root represents the Saint-Venant torsional rigidity, which can be neglected without any loss in economy. Therefore, Equation C-C3.1.2.1-3a can be simplified as shown in Equation C-C3.1.2.1-4 by considering $I_y = I_{yc} + I_{yt}$ and neglecting the term $4GJ(K_t L_t)^2/\pi^2 I_y Ed^2$:

$$\sigma_{cr} = \frac{\pi^2 Ed I_{yc}}{(K_y L_y)^2 S_f}$$  \hspace{1cm} (C-C3.1.2.1-4)

Equation C-C3.1.2.1-4 was derived on the basis of a uniform bending moment and is conservative for other cases. For this reason $\sigma_{cr}$ is modified by multiplying the right hand side by a bending coefficient $C_b$, to account for non-uniform bending and the symbol $F_e$ is used for $\sigma_{cr}$, i.e.,

$$F_e = \frac{C_b \pi^2 Ed I_{yc}}{(K_y L_y)^2 S_f}$$  \hspace{1cm} (C-C3.1.2.1-5)

where $C_b$ is the bending coefficient, which can conservatively be taken as unity, or calculated from

$$C_b = 1.75 + 1.05 (M_1/M_2) + 0.3 (M_1/M_2)^2 \leq 2.3$$  \hspace{1cm} (C-C3.1.2.1-6)

in which $M_1$ is the smaller and $M_2$ the larger bending moment at the ends of the unbraced length.

The above Equation was used in the 1968, 1980, 1986, and 1991 editions of the AISI Specification. Because it is valid only for straight-line moment diagrams, Equation C-C3.1.2.1-6 was replaced by the following equation for $C_b$ in the 1996 edition of the AISI Specification and is retained in this edition of the Specification:

$$C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_C}$$  \hspace{1cm} (C-C3.1.2.1-7)

where

- $M_{\text{max}}$ = absolute value of maximum moment in the unbraced segment
- $M_A$ = absolute value of moment at quarter point of unbraced segment
- $M_B$ = absolute value of moment at centerline of unbraced
segment

\[ M_C = \text{absolute value of moment at three-quarter point of unbraced segment} \]

Equation C-C3.1.2.1-7, derived from Kirby and Nethercot (1979), can be used for various shapes of moment diagrams within the unbraced segment. It gives more accurate solutions for fixed-end members in bending and moment diagrams which are not straight lines. This equation is the same as that being used in the AISC LRFD Specification (AISC, 1999).

Figure C-C3.1.2.1-1 shows the differences between Equations C-C3.1.2.1-6 and C-C3.1.2.1-7 for a straight line moment diagram.

In 2001, the unbraced length, \( L \), in Specification Equations C3.1.2.1-14 and C3.1.2.1-15 was replaced with \( K_y L_y \) on the basis of Equation C-C3.1.2.1-5, where \( K_y \) is the effective length factor for bending about the \( y \)-axis. The \( K_y \) factor provides for other than simply supported end conditions. In addition, Specification Equation C3.1.2.1-14 is permitted to be used for the design of singly-symmetric C-sections. The use of this equation was also permitted in the 1968 and 1980 editions of the AISI Specification.

Also in 2001, the requirement of taking \( C_b \) equal to unity when considering axial load and bending moment in Specification Section C5 was removed. This requirement was in place since both \( C_b \) and \( C_m \) in Specification Section C5 are adjustments for the moment gradient in the member and it was conservative to take \( C_b \) equal to unity. \( C_b \) is an adjustment to the critical moment for lateral-torsional buckling when the bending moment is not constant and \( C_m \) adjusts the magnitude of
the second order p-delta moment in the member. Since these are two separate quantities, it is appropriate to use both $C_b$ and $C_m$ in evaluating the member under combined loads. However, it is still conservative to take $C_b$ equal to unity.

It should be noted that Equations C-C3.1.2.1-1a and C-C3.1.2.1-5 apply only to elastic buckling of cold-formed steel members in bending when the computed theoretical buckling stress is less than or equal to the proportional limit. When the computed stress exceeds the proportional limit, the beam behavior will be governed by inelastic buckling. The inelastic buckling stress, $F_c$, can be computed from Equation C-C3.1.2.1-8 (Yu, 2000):

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{56F_e}\right)$$  \hspace{1cm} \text{(C-C3.1.2.1-8)}

where $F_e$ is the elastic critical lateral-torsional buckling stress.

Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 with $K_y = 1.0$ and $L_y = L$ were used in the 1968, 1980 and 1986 editions of the AISI Specification to develop the allowable stress design equations for lateral-torsional buckling of I-members. In the 1986 edition of the AISI Specification, in addition to the use of Equations C-C3.1.2.1-5 and C-C3.1.2.1-8 for determining the critical stresses, more design equations (Specification Equations C3.1.2.1-5 and C3.1.2.1-6) for elastic critical stress were added as alternative methods. These additional equations were developed from the previous studies conducted by Pekoz, Winter and Celebi on torsional-flexural buckling of thin-walled sections under eccentric loads (Pekoz and Winter, 1969a; Pekoz and Celebi, 1969b) and are retained in this edition of the Specification. These general design equations can be used for singly-, doubly- and point-symmetric sections. Consequently, the elastic critical lateral-torsional buckling stress can be determined by the following equation:

$$F_e = \frac{C_b Ar_0}{S_f} \sqrt{\sigma_{ey} \sigma_t}$$  \hspace{1cm} \text{(C-C3.1.2.1-9)}

where $\sigma_{ey}$ and $\sigma_t$ are the elastic buckling stresses as defined in Specification Equations C3.1.2.1-8 and C3.1.2.1-9, respectively.

It should be noted that point-symmetric sections such as Z-sections with equal flanges will buckle laterally at lower strengths than doubly- and singly-symmetric sections. A conservative design approach has been and is being used in the Specification, in which the elastic critical buckling stress is taken to be one-half of that for I-members.

Regarding the inelastic critical buckling stress, the following equation was used for calculating the critical moment in Section C3.1.2(a) of the 1986 edition of the AISI Specification instead of using Equation C-C3.1.2.1-8 for inelastic critical buckling stress:

$$(M_{cr})_I = M_y \left[1 - \frac{M_y}{4(M_{cr})_e}\right]$$  \hspace{1cm} \text{(C-C3.1.2.1-10)}
in which \((M_{cr})_e\) is the elastic critical buckling moment. In 1996, the basic inelastic lateral-torsional buckling curve for singly-, doubly-, and point-symmetric sections in AISI Specification Section C3.1.2.1(a) was redefined to be consistent with the inelastic lateral-torsional buckling curve for I- or Z-sections in Specification Section C3.1.2.1(b). The general shape of the curve as represented by Equation C-C3.1.2.1-8 is also consistent with the preceding edition of the Specification (AISI, 1986).

As specified in Specification Section C3.1.2.1, lateral-torsional buckling is considered to be elastic up to a stress equal to 0.56\(F_y\). The inelastic region is defined by a Johnson parabola from 0.56\(F_y\) to \((10/9)F_y\) at an unsupported length of zero. The \((10/9)\) factor is based on the partial plastification of the section in bending (Galambos, 1963). A flat plateau is created by limiting the maximum stress to \(F_y\) which enables the calculation of the maximum unsupported length for which there is no stress reduction due to lateral-torsional instability. This maximum unsupported length can be calculated by setting \(F_y\) equal to \(F_c\) in Equation C-C3.1.2.1-8.

This liberalization of the inelastic lateral-torsional buckling curve for singly-, doubly-, and point-symmetric sections has been confirmed by research in beam-columns (Pekoz and Sumer, 1992) and wall studs (Niu and Pekoz, 1994).

The elastic and inelastic critical stresses for the lateral-torsional buckling strength are shown in Figure C-C3.1.2.1-2. For any unbraced length, \(L\), less than \(L_u\), lateral-torsional buckling does not need to be considered, where \(L_u\) is determined by setting \(F_c = 2.78F_y\) and \(L_u = L_y = L_t\). \(L_u\) may be calculated using the expression given below (AISI, 1996):

(a) for Singly-, doubly- and point-symmetric sections:
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\[ L_u = \left\{ \frac{GJ}{2C_1} + \left[ \frac{C_2}{C_1} + \left( \frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \]  

(C-C3.1.2.1-11)

where

\[ C_1 = \frac{7.72}{AE} \left[ \frac{K_y F_y S_f}{C_b \pi r_y} \right]^2 \] for singly- and doubly-symmetric sections  

(C-C3.1.2.1-12)

\[ C_1 = \frac{30.9}{AE} \left[ \frac{K_y F_y S_f}{C_b \pi r_y} \right]^2 \] for point-symmetric sections  

(C-C3.1.2.1-13)

\[ C_2 = \frac{\pi^2 EC_w}{(K_t)^2} \]  

(C-C3.1.2.1-14)

(b) for I-, C- or Z-sections bent about the centroidal axis perpendicular to the web, the following equations may be used in lieu of (a) (AISI, 1996):

For doubly-symmetric I-sections and singly-symmetric C-sections:

\[ L_u = \left[ \frac{0.36 C_b \pi^2 E d l_{yc}}{F_y S_f} \right]^{0.5} \]  

(C-C3.1.2.1-15)

For point-symmetric Z-sections:

\[ L_u = \left[ \frac{0.18 C_b \pi^2 E d l_{yc}}{F_y S_f} \right]^{0.5} \]  

(C-C3.1.2.1-16)

The above discussion dealt only with the lateral-torsional buckling strength [resistance] of locally stable beams. For locally unstable beams, the interaction of the local buckling of the compression elements and overall lateral-torsional buckling of members may result in a reduction of the lateral-torsional buckling strength [resistance] of the member. The effect of local buckling on the critical moment is considered in Section C3.1.2.1 of the Specification by using the elastic section modulus \( S_c \) based on an effective section. i.e.,

\[ M_n = F_c S_c \]  

(C-C3.1.2.1-17)

where

\[ F_c = \text{Elastic or inelastic critical lateral-torsional buckling stress} \]

\[ S_c = \text{Elastic section modulus of effective section calculated at a stress } F_c \text{ relative to the extreme compression fiber} \]

Using the above nominal lateral buckling strength [resistance] with a resistance factor of \( \phi_b = 0.90 \), the values of \( \beta \) vary from 2.4 to 3.8 for the LRFD method.

The research conducted by Ellifritt, Sputo and Haynes (1992) has indicated that when the unbraced length is defined as the spacing
between intermediate braces, the equations used in Specification Section C3.1.2.1 may be conservative for cases where one mid-span brace is used, but may be unconservative where more than one intermediate brace is used.

The above mentioned research (Ellifritt, Sputo, and Haynes, 1992) and the study of Kavanagh and Ellifritt (1993 and 1994) have shown that a discretely braced beam, not attached to deck and sheathing, may fail either by lateral-torsional buckling between braces, or by distortional buckling at or near the braced point. The distortional buckling strength of C- and Z-sections has been studied extensively at the University of Sydney by Lau and Hancock (1987); Hancock, Kwon and Bernard (1994); and Hancock (1995).

![Combined Sheet-Stiffener Sections](image1)

![Lateral Buckling of U-Shaped Beam](image2)

The problems discussed above dealt with the type of lateral-torsional buckling of I-members, C-sections, and Z-shaped sections for which the entire cross section rotates and deflects in the lateral direction as a unit. But this is not the case for U-shaped beams and the combined sheet-stiffener sections as shown in Figure C-C3.1.2.1-3. For this case, when the section is loaded in such a manner that the brims and the flanges of stiffeners are in compression, the tension flange of the beam remains straight and does not displace laterally; only the compression flange tends to buckle separately in the lateral direction, accompanied by out-of-plane bending of the web, as shown in Figure C-C3.1.2.1-4, unless adequate bracing is provided.

The precise analysis of the lateral buckling of U-shaped beams is
rather complex. The compression flange and the compression portion of the web act not only like a column on an elastic foundation, but the problem is also complicated by the weakening influence of the torsional action of the flange. For this reason, the design procedure outlined in Section 2 of Part VII (Supplementary Information) of the AISI Cold-Formed Steel Design Manual (AISI, 2002) for determining the allowable design strength [resistance] for laterally unbraced compression flanges is based on the considerable simplification of an analysis presented by Douty (1962).

In 1964, Haussler presented rigorous methods for determining the strength [resistance] of elastically stabilized beams (Haussler, 1964). In his methods, Haussler also treated the unbraced compression flange as a column on an elastic foundation and maintained more rigor in his development.

A comparison of Haussler’s method with Douty’s simplified method indicates that the latter may provide a lower value of critical stress.

An additional study of laterally unbraced compression flanges has been made at Cornell University (Serrette and Pekoz, 1992, 1994 and 1995). An analytical procedure has been developed for determining the distortional buckling strength [resistance] of the standing seam roof panel. The predicted maximum capacities have been compared with experimental results.

### C3.1.2.2 Lateral-Torsional Buckling Strength [Resistance] for Closed Box Members

Due to the high torsional stiffness of closed box sections, lateraltorsional buckling is not critical in typical design considerations, even for bending about the major axis. Deflection limits will control most designs due to the large values of $L_u$. However, lateral-torsional buckling can control the design when the unbraced length is larger than $L_u$, which is determined by setting the inelastic buckling stress of Specification Equation C3.1.2.1-3 equal to $F_y$, with $F_e$ set equal to Specification Equation C3.1.2.2-2.

In computing the lateral-torsional buckling stress of closed box sections, the warping constant, $C_w$, may be neglected since the effect of non-uniform warping of box sections is small. The development of Specification Equation C3.1.2.2-2 can be found in the SSRC Guide (Galambos, 1998). As a result of adding Section C3.1.2.2 to the Specification, Specification Section D3.3 has been deleted.

The Saint-Venant torsional constant, $J$, of a box section, neglecting the corner radii, may be conservatively determined as follows:

$$J = \frac{2(ab)^2}{(a/t_1) + (b/t_2)}$$

(C-C3.1.2.2-1)

where

$a$ = distance between web centerlines
In 2001, the unbraced length, $L$, in Specification Equation C3.1.2.2-2 was replaced with $K_yL_y$, where $K_y$ is the effective length factor for bending about the y-axis. The $K_y$ factor provides for other than simply supported end conditions. Detailed discussions are provided in Section C3.1.2.1 of the Commentary.

**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 (Pekoz and Soroushian, 1981 and 1982; LaBoube, 1986; Haussler and Pahers, 1973; LaBoube, et al., 1988; Haussler, 1988; Fisher, 1996).

The R factors for simple span C-sections and Z-sections up to 8.5 inches (216 mm) in depth have been increased from the 1986 Specification, and a member design yield strength limit is added based on the work by Fisher (1996).

As indicated by LaBoube (1986), the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. To ensure adequate rotational stiffness of the roof and wall systems designed using the AISI provisions, Specification 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 a maximum positive moment, $M = 0.08\, wL^2$.

The provisions of Specification Section C3.1.3 apply to beams for 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 (Pekoz and Soroushian, 1981 and 1982; LaBoube and Thompson, 1982a; LaBoube, et al., 1988; LaBoube and Golovin, 1990) 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.
For the LRFD method, the use of the reduced nominal flexural strength [resistance] (Specification Equation C3.1.3-1) with a resistance factor of $\phi_b = 0.90$ provides the $\beta$ values varying from 1.5 to 1.60 which are satisfactory for the target value of 1.5. This analysis was based on the load combination of 1.17 W - 0.9D using a reduction factor of 0.9 applied to the load factor for the nominal wind load, where W and D are nominal wind and dead loads, respectively (Hsiao, Yu and Galambos, 1988a; AISI, 1991).

**C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System**

The design provision of this section is only applicable to the United States and Mexico. The discussion for this section is provided in the Commentary on Appendix A.

**C3.1.5 Strength [Resistance] of Standing Seam Roof Panel Systems**

The nominal strength [nominal resistance] of a standing seam roof panel system is determined using the ASTM E1592-95 (1995) test procedure. A methodology of interpreting test results is specified in the Specification Section C3.1.5.

Clarification and extension of the ASTM E1592-95 (1995) test procedure is presented in the Standard Procedures for Panel and Anchor Structural Tests as published by AISI. The Specification Section C3.1.5 provides the method for determining a factor of safety or resistance factor for one or more tests.

The relationship of strength [resistance] to serviceability limits may be taken as strength limit/serviceability limit = 1.25, or

$$\Omega_{\text{serviceability}} = \frac{\Omega_{\text{strength}}}{1.25} \quad \text{(C-C3.1.5-1)}$$

It should be noted that the purpose of the test procedure specified in Specification Section C3.1.5 is not to set up guidelines to establish the serviceability limit. The purpose is to define the method of determining the controlling allowable load whether based on the serviceability limit or on the ultimate load. The Corps of Engineers Procedure CEGS 07416 (1991) requires a factor of safety of 1.65 on strength [resistance] and 1.3 on serviceability. A buckling or crease does not have the same consequences as a failure of a clip. In the latter case, the roof panel itself may become detached and expose the contents of a building to the elements of the environment. Further, Galambos (1988a) recommended a value of 2.0 for the target reliability index, $\beta_0$, when slight damage is expected and a value of 2.5 when moderate damage is expected. The resulting ratio is 1.25.

In Specification Section C3.1.5, a target reliability index of 2.5 is used for connection limits. It is used because the consequences of a panel fastener failure ($\beta_0 = 2.5$) are not nearly so severe as the consequences of a primary frame connection failure ($\beta_0 = 3.5$). The intermittent nature of
wind load as compared to the relatively long duration of snow load further justifies the use of $\beta_0 = 2.5$ for panel anchors. In Specification Section C3.1.5, the coefficient of variation of the material factor, $V_{Mf}$, is recommended to be 0.08 for failure limited by anchor or connection failure, and 0.10 for limits caused by flexural or other modes of failure. Specification Section C3.1.5 also eliminates the limit on coefficient of variation of the test results, $V_{p}$, because consistent test results often lead to $V_{p}$ values lower than the 6.5% value set in Specification Section F1. The elimination of the limit will be beneficial when test results are consistent.

The value for the number of tests for fasteners is set as the number of anchors tested with the same tributary area as the anchor that failed. This is consistent with design practice where anchors are checked using a load calculated based on tributary area. Actual anchor loads are not calculated from a stiffness analysis of the panel in ordinary design practice.

**C3.2 Shear**

**C3.2.1 Shear Strength [Resistance] of Webs Without Holes**

The shear strength [resistance] 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 nominal shear strength [resistance] is governed by shear yielding, i.e.,

$$V_n = A_w \tau_y = A_w F_y \sqrt{\frac{3}{2}} = 0.60F_y h t$$

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 nominal shear strength [resistance] is governed by elastic shear buckling (Yu, 2000), i.e.,

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2}$$

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 [resistance], $V_n$, can be determined as follows:

$$V_n = 0.904E k_v t^3 / h$$

For beam webs having moderate h/t ratios, the nominal shear strength [resistance] is based on inelastic shear buckling (Yu, 2000), i.e.,

$$V_n = 0.64t^2 \sqrt{k_v F_y E}$$

The Specification provisions are applicable for the design of webs of beams and decks either with or without transverse web stiffeners.

The nominal strength [resistance] equations of Section C3.2.1 of the Specification are similar to the nominal shear strength [resistance] equations given in the AISI LRFD Specification (AISI, 1991). The acceptance of these
nominal strength [resistance] equations for cold-formed steel sections has been considered in the study summarized by LaBoube and Yu (1978a).

Previous editions of the AISI ASD Specification (AISI, 1986) used three different factors of safety when evaluating the allowable shear strength [resistance] of an unreinforced web because it was intended to use the same nominal strength [resistance] equations for the AISI and AISC Specifications. To simplify the design of shear using only one factor of safety for ASD and one resistance factor for LRFD, Craig (Craig, 1999) carried out a calibration using the data by LaBoube and Yu (LaBoube, 1978a). Based on this work, the constant used in Specification Equation C3.2.1-3 was reduced from 0.64 to 0.60. In addition, the ASD factor of safety for yielding, elastic and inelastic buckling is now taken as 1.60, with a corresponding resistance factor of 0.95 for LRFD and 0.80 for LSD.

C3.2.2 Shear Strength [Resistance] of C-Section Webs With Holes

For C-section webs with holes, Schuster et al. (1995) and Shan et al. (1994) investigated the degradation in web shear strength [resistance] due to the presence of a web perforation. The test program considered a constant shear distribution across the perforation, and included \(d_0/h\) ratios ranging from 0.20 to 0.78, and \(h/t\) ratios of 91 to 168. Schuster’s \(q_s\) equation was developed with due consideration for the potential range of both punched and field cut holes. Three hole geometries, rectangular with corner fillets, circular, and diamond, were considered in the test program. Eiler (1997) extended the work of Schuster and Shan for the case of constant shear along the longitudinal axis of the perforation. He also studied linearly varying shear but this case is not included in the Specification. The development of Eiler’s reduction factor, \(q_s\), utilized the test data of both Schuster et al. (1995) and Shan et al. (1994). The focus of the test programs was on the behavior of slender webs with holes. Thus for stocky web elements with \(h/t \leq 0.96 \sqrt{E_k v/F_y}\), an anomaly exists; the calculated design shear strength [resistance] is independent of \(t\) when \(h\) is constant. In this region, the calculated design shear strength [resistance] is valid but may be somewhat conservative.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the \(b\) and \(d_0\) that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the \(d_0\) that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.

C3.3 Combined Bending and Shear

For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at the supports. Such beam webs must be
safeguarded against buckling due to the combination of bending and shear stresses.

For disjointed flat rectangular plates, the critical combination of bending and shear stresses can be approximated by the following interaction equation (Bleich, 1952):

\[
\frac{f_b}{f_{cr}}^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1.0
\]

(C-C3.3-1)

where \(f_b\) is the actual compressive bending stress, \(f_{cr}\) is the theoretical buckling stress in pure bending, \(\tau\) is the actual shear stress and \(\tau_{cr}\) is the theoretical buckling stress in pure shear. The above equation was found to be conservative for beam webs with adequate transverse stiffeners, for which a diagonal tension field action can be developed. Based on the studies made by LaBoube and Yu (1978b), Equation C-C3.3-2 was developed for beam webs with transverse stiffeners satisfying the requirements of Specification Section C3.6.

\[
0.6 \frac{f_b}{f_{b\text{max}}} + \frac{\tau}{\tau_{\text{max}}} = 1.3
\]

(C-C3.3-2)

The above equation was added to the AISI Specification in 1980. The correlations between Equation C-C3.3-2 and the test results of beam webs having a diagonal tension field action are shown in Figure C-C3.3-1.
C3.3.1 ASD Method

Since 1986, the AISI ASD Specification uses strength ratios (i.e., moment ratio for bending and force ratio for shear) instead of stress ratios for the interaction equations. Specification Equations C3.3.1-1 and C3.3.1-2 are based on Equations C-C3.3-1 and C-C3.3-2, respectively, by using the allowable design moment, $M_{n\phi}/\Omega_b$, and the allowable design shear force, $V_n/\Omega_v$.

C3.3.2 LRFD and LSD Methods

For the load and resistance factor design and the limit states design, the interaction equations for combined bending and shear are also based on Equations C-C3.3-1 and C-C3.3-2 as given in Specification Equations C3.3.2-1 and C3.3.2-2 by using the required and design strengths. In both equations, different symbols are used for the required flexural strength [factored moment] and the required shear strength [factored shear] according to the LRFD and the LSD methods.

C3.4 Web Crippling

C3.4.1 Web Crippling Strength [Resistance] of Webs Without Holes

Since cold-formed steel flexural members generally have large web slenderness ratios, the webs of such members may cripple due to the high local intensity of the load or reaction. Figure C-C3.4.1-1 shows typical web crippling failure modes of unreinforced single hat sections (Figure C-C3.4.1-1(a)) and of I-sections (Figure C-C3.4.1-1(b)) unfastened to the support.

![Figure C-C3.4.1-1 Web Crippling of Cold-Formed Steel Sections](image)

In the past, the buckling problem of plates and the web crippling behavior of cold-formed steel members under locally distributed edge loading have been studied by numerous investigators (Yu, 2000). It has been found that the theoretical analysis of web crippling for cold-formed steel flexural members is rather complicated because it involves the following factors: (1) nonuniform stress distribution under the applied load and adjacent portions of the web, (2) elastic and inelastic stability of the web element, (3) local yielding in the immediate region of load application,
(4) bending produced by eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web, (5) initial out-of-plane imperfection of plate elements, (6) various edge restraints provided by beam flanges and interaction between flange and web elements, and (7) inclined webs for decks and panels.

For these reasons, the present AISI design provision for web crippling is based on the extensive experimental investigations conducted at Cornell University by Winter and Pian (1946) and Zetlin (1955a); at the University of Missouri-Rolla by Hetrakul and Yu (1978 and 1979), Yu (1981), Santaputra (1986), Santaputra, Parks and Yu (1989), Bhakta, LaBoube and Yu (1992), Langan, Yu and LaBoube (1994), Cain, LaBoube and Yu (1995) and Wu, Yu and LaBoube (1997); at the University of Waterloo by Wing (1981), Wing and Schuster (1982), Prabakaran (1993), Gerges (1997), Gerges and Schuster (1998), Prabakaran and Schuster (1998), Beshara (1999), and Beshara and Schuster (2000 and 2000a); and at the University of Sydney by Young and Hancock (1998). In these experimental investigations, the web crippling tests were carried out under the following four loading conditions for beams having single unreinforced webs and I-beams, single hat sections and multi-web deck sections:
1. End one-flange (EOF) loading
2. Interior one-flange (IOF) loading
3. End two-flange (ETF) loading
4. Interior two-flange (ITF) loading

All loading conditions are illustrated in Figure C-C3.4.1-2. In Figures (a) and (b), the distances between bearing plates were kept to no less than 1.5 times the web depth in order to avoid the two-flange loading action. Application of the various load cases is shown in Figure C-C3.4.1-3.
and the assumed reaction or load distributions are illustrated in Figure C-C3.4.1-4.

Figure C-C3.4.1-3 Application of Loading Cases
Figure C-C3.4.1-4 Assumed Distribution of Reaction or Load
In the 1996 edition of the AISI Specification, and in previous editions, different web crippling equations were used for the various loading conditions stated above. These equations were based on experimental evidence (Winter, 1970; Hettrakul and Yu, 1978) and the assumed distributions of loads or reactions acting on the web as shown in Figure C-C3.4.1-4. The equations were also based on the type of section geometry, i.e., shapes having single webs and I-sections (made of two channels connected back to back, by welding two angles to a channel, or by connecting three channels). C-and Z-sections, single hat sections and multi-web deck sections were considered in the single web member category. I-sections made of two channels connected back to back by a line of connectors near each flange or similar sections that provide a high degree of restraint against rotation of the web were treated separately. In addition, different equations were used for sections with stiffened or partially stiffened flanges and sections with unstiffened flanges.

Prabakaran (1993) and Prabakaran and Schuster (1998) developed one consistent unified web crippling equation with variable coefficients (Specification Equation C3.4.1-1). These coefficients accommodate one or two flange loading for both end and interior loading conditions of various section geometries. Beshara (1999) extended the work of Prabakaran and Schuster (1998) by developing new web crippling coefficients using the available data as summarized by Beshara and Schuster (2000). The web crippling coefficients are summarized in Tables C3.4.1-1 to C3.4.1-5 of the Specification and the parametric limitations given are based on the experimental data that was used in the development of the web crippling coefficients. From Specification Equation C3.4.1-1, it can be seen that the nominal web crippling strength of cold-formed steel members depends on an overall web crippling coefficient, \( C \), the web thickness, \( t \), the yield stress, \( F_y \), the web inclination angle, \( \theta \), the inside bend radius coefficient, \( C_R \), the inside bend radius ratio, \( R/t \), the bearing length coefficient, \( C_N \), the bearing length ratio, \( N/t \), the web slenderness coefficient, \( C_{h'} \), and the web slenderness ratio, \( h/t \).

This new equation is presented in a normalized format and is non-dimensional, allowing for any consistent system of measurement to be used. Consideration was given to whether or not the test specimens were fastened to the bearing plate/support during testing. It was discovered that some of the test specimens in the literature were not fastened to the bearing plate/support during testing, which can make a considerable difference in the web crippling capacity of certain sections and loading conditions. Therefore, it was decided to separate the data on the basis of members being fastened to the bearing plate/support and those not being fastened to the bearing plate/support. The fastened to the bearing plate/support data in the literature were primarily based on specimens being bolted to the bearing plate/support, hence, a few control tests were carried out by Schuster, the results of which are contained in (Beshara
1999), using self-drilling screws to establish the web crippling integrity in comparison to the bolted data. Based on these tests, the specimens with self-drilling screws performed equally well in comparison to the specimens with bolts. Fastened to the bearing plate/support in practice can be achieved by either using bolts, self-drilling/self-tapping screws or by welding. What is important is that the flange elements are restrained from rotating at the location of load application. In fact, in most cases, the flanges are frequently completely restrained against rotation by some type of sheathing material that is attached to the flanges.

The data was further separated based on section type, as follows.
1) Built-up sections (Table C3.4.1-1);
2) Single web channel and C-sections (Table C3.4.1-2);
3) Single web Z-sections (Table C3.4.1-3);
4) Single hat sections (Table C3.4.1-4); and
5) Multi-web deck sections (Table C3.4.1-5).

In the case of unfastened built-up members such as I-sections (not fastened to the bearing plate/support), the available data was for specimens that were fastened together with a row of fasteners near each flange line of the member (Winter and Pian 1946) and Hetrakul and Yu (1978) as shown in Figure C-C3.4.1-5(a). For the fastened built-up member data of I-sections (fastened to the bearing plate/support), the specimens were fastened together with two rows of fasteners located symmetrically near the centerline length of the member, as shown in Figure C-C3.4.1-5(b) (Bhakta, LaBoube and Yu, 1992).

Calibrations were carried out by Beshara and Schuster (2000) in accordance with Supornsilaphachai, Galambos and Yu (1979) to establish the factors of safety, \( \Omega \), and the resistance factors, \( \phi \), for each web crippling case. Based on these calibrations, different factors of safety and corresponding resistance factors are presented in the web crippling coefficient tables for the particular load case and section type. Also, a minimum bearing length of 3/4 in. (19 mm) was introduced based on the data used in the development of the web crippling coefficients. For fastened to support single web C- and Z-section members under interior two-flange loading or reaction, the distance from the edge of bearing to the end of the member (Fig; C-C3.4.1-2(d)) must be extended at least 2.5h. This requirement is necessary because a total of 5h specimen length was used for the test setup shown in Figure C-C3.4.1-2(d) (Beshara, 1999). The 2.5h length is conservatively taken from the edge of bearing rather than the centerline of bearing.

The assumed distributions of loads or reactions acting on the web of a member, as shown in Figure C-C3.4.1-4, are independent of the flexural response of the member. Due to the flexural action, the point of bearing will vary relative to the plane of bearing, resulting in a non-uniform bearing load distribution on the web. The value of \( P_n \) will vary because of a transition from the interior one-flange loading (Figure C3.4.1-4(b)) to the end one-flange loading (Figure C3.4.1-4(a)) condition. These discrete conditions represent the experimental basis on which the design
provisions were founded (Winter, 1970; Hettrakul and Yu, 1978).

Recent research indicates that a Z-section having its end support flange bolted to the section’s supporting member through two 1/2-in. (12.7 mm) diameter bolts will experience an increase in end-one-flange web crippling capacity (Bhakta, LaBoube and Yu, 1992; Cain, LaBoube and Yu, 1995). The increase in load-carrying capacity was shown to range from 27 to 55 percent for the sections under the limitations prescribed in the Specification. A lower bound value of 30 percent increase was permitted in Specification Section C3.4 of the 1996 Specification. This is now incorporated under “Fastened to Support” condition.

![Diagram of Z-section with fasteners](image)

Figure C3.4.1-5 Typical Bolt Pattern for I-Section Test Specimens

For two nested Z-sections, the 1996 AISI Specification permitted the use of a slightly different factor of safety and resistance factor for the interior one flange loading condition. This is no longer required since the new web crippling approach now takes this into account in Table C3.4.1-3 of the Specification under the category of “Fastened to Support” for the interior one flange loading case.

**C3.4.2 Web Crippling Strength [Resistance] of C-Section Webs with Holes**

Studies by Langan et al. (1994), Uphoff (1996) and Deshmukh (1996) quantified the reduction in web crippling capacity when a hole is present in a web element. These studies investigated both the end-one-flange and interior-one-flange loading conditions for h/t and d₀/h ratios as large as 200 and 0.81, respectively. The studies revealed that the reduction in web crippling strength is influenced primarily by the size of the hole as reflected in the d₀/h ratio and the location of the hole, x/h ratio.

The provisions for circular and non-circular holes also apply to any hole pattern that fits within an equivalent virtual hole. Figure C-B2.4-1 illustrates the b and d₀ that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-B2.4-2 illustrates the d₀ that may be used for a rectangular hole that fits within a circular virtual hole. For each case, the design provisions apply to the geometry of the virtual hole geometry, not the actual hole or holes.
C3.5 Combined Bending and Web Crippling

C3.5.1 ASD Method

This Specification contains interaction equations for the combination of bending and web crippling. Specification Equations C3.5.1-1 and C3.5.1-2 are based on the studies conducted at the University of Missouri-Rolla for the effect of bending on the reduction of web crippling loads with the applicable factors of safety used for bending and web crippling (Hetrakul and Yu, 1978 and 1980; Yu, 1981 and 2000). For embossed webs, crippling strength [resistance] should be determined by tests according to Specification Chapter F.

The exception clause included in Specification Section C3.5.1 for single unreinforced webs applies to the interior supports of continuous spans using decks and beams, as shown in Figure C-C3.5-1. Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the postbuckling behavior of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This postbuckling strength [resistance] enables the member to redistribute the moments in continuous spans. For this reason, Specification Equation C3.5.1-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans. This exception clause applies only to the members shown in Figure C-C3.5-1 and similar situations explicitly described in Specification Section C3.5.1.

The exception clause should be interpreted to mean that the effects of combined bending and web crippling need not be checked for determining load-carrying capacity. Furthermore the positive bending resistance of the beam should be at least 90 percent of the negative bending resistance in order to insure the safety implied by the Specification.
Using this procedure the service loads may (1) produce slight deformations in the member over the support, (2) increase the actual compressive bending stresses over the support to as high as 0.8 \( F_y \), and (3) result in additional bending deflection of up to 22 percent due to elastic moment redistribution.

If load-carrying capacity is not the primary design concern because of the above behavior, the designer is urged to use Specification Equation C3.5.1-1.

With regard to Equation C3.5.1-2, previous tests indicate that when the \( h/t \) ratio of an I-beam web does not exceed \( 2.33 / \sqrt{F_y / E} \) and when \( \lambda \leq 0.673 \) for all elements, the bending moment has little or no effect on the web crippling load (Yu, 2000). For this reason, the allowable reaction or concentrated load can be determined by the equation given in Specification Section C3.4 without reduction for the presence of bending.

In 1996, additional design information was added to Specification Section C3.5.1(c) for two nested Z-shapes. These design provisions are based on the research conducted at the University of Wisconsin-Milwaukee, University of Missouri-Rolla, and a metal building manufacturer (LaBoube, Nunnery and Hodges, 1994). The web crippling and bending behavior of unreinforced nested web elements is enhanced because of the interaction of the nested webs. The design equation is based on the experimental results obtained from 14 nested web configurations. These configurations are typically used by the metal building industry.

In 2001, the interaction equation for the combined effects of bending and web crippling was re-evaluated because new web crippling equation was adopted for Section C3.4.1 of the Specification. Based on the same test data of LaBoube, Nunnery, and Hodges (1994), the following interaction equation was derived.

\[
\frac{M}{M_{no}} + 0.8631 \frac{P}{P_n} \leq 1.6521 \tag{C-C3.5.1-1}
\]

Using the statistical data from this analysis, new values for \( \phi \) and \( \Omega \) were calculated to be 0.9042 and 1.7696.

The constants in Equation C-C3.5.1-1 have been rounded and presented in Specification Equation C3.5.1-3 as:

\[
\frac{M}{M_{no}} + 0.85 \frac{P}{P_n} \leq \frac{1.65}{\Omega} \tag{C-C3.5.1-2}
\]

with \( \Omega = 1.75 \).

**C3.5.2 LRFD and LSD Methods**

For the load and resistance factor design and the limit states design methods, Specification Equations C3.5.2-1 and C3.5.2-2 are based on the original equations using the required and design strengths. In both equations, different symbols are used for the required strength [resistance] for the concentrated load or reaction due to factored loads, and the
required flexural strength [factored moment] according to the LRFD and
the LSD methods.

In the development of the LRFD equations, a total of 551 tests were
calibrated for combined bending and web crippling strength [resistance].
Based on $\phi_w = 0.75$ for single unreinforced webs and $\phi_w = 0.80$ for I-
sections, the values of reliability index vary from 2.5 to 3.3 as summarized
in the AISI Commentary (AISI, 1991).

For two nested Z-shapes, Specification Equation C3.5.2-3 was
derived from the same research work discussed in Section C3.5.1 for
Specification Equation C3.5.1-3. The new interaction equation for LRFD and
LSD is

$$\frac{M}{M_{no}} + 0.85 \frac{P}{P_n} \leq 1.65\phi \quad \text{(C-C3.5.2-1)}$$

where $\phi = 0.90$.

C3.6 Stiffeners

C3.6.1 Transverse Stiffeners

Design requirements for attached transverse stiffeners and for shear
stiffeners were added in the 1980 AISI Specification and were unchanged in
the 1986 Specification. The same design equations are retained in Section
C3.6 of the current Specification. The nominal strength [resistance] equation
given in Item (a) of Specification Section C3.6.1 serves to prevent end
crushing of the transverse stiffeners, while the nominal strength [resistance] equation given in Item (b) is to prevent column-type buckling
of the web-stiffeners. The equations for computing the effective areas ($A_b$
and $A_c$) and the effective widths ($b_1$ and $b_2$) were adopted from Nguyen
and Yu (1978a) with minor modifications.

The available experimental data on cold-formed steel transverse
stiffeners were evaluated by Hsiao, Yu and Galambos (1988a). A total of 61
tests were examined. The resistance factor of 0.85 used for the LRFD
method was selected on the basis of the statistical data. The corresponding
reliability indices vary from 3.32 to 3.41.

In 1999, the upper limit of $w/t_s$ ratio for the unstiffened elements of
cold-formed steel transverse stiffeners was revised from $0.37\sqrt{E/F_{ys}}$ to
$0.42\sqrt{E/F_{ys}}$ for the reason that the former was calculated based on the
allowable strength design approach, while the latter is based on the
effective area approach. The revision provided the same basis for the
stiffened and unstiffened elements of cold-formed steel transverse
stiffeners.

C3.6.2 Shear Stiffeners

The requirements for shear stiffeners included in Specification
Section C3.6.2 were primarily adopted from the AISC Specification (1978).
The equations for determining the minimum required moment of inertia (Equation C3.6.2-1) and the minimum required gross area (Equation C3.6.2-2) of attached intermediate stiffeners are based on the studies summarized by Nguyen and Yu (1978a). In Equation C3.6.2-1, the minimum value of \((h/50)^4\) was selected from the AISC Specification (AISC, 1978).

For the LRFD method, the available experimental data on the shear strength [resistance] of beam webs with shear stiffeners were calibrated by Hsiao, Yu and Galambos (1988a). The statistical data used for determining the resistance factor were summarized in the AISI Design Manual (AISI, 1991). Based on these data, the reliability index was found to be 4.10 for \(\phi = 0.90\).

### C3.6.3 Non-Conforming Stiffeners

Tests on rolled-in transverse stiffeners covered in Specification Section C3.6.3 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the design strength [resistance] of members and the allowable design loads should be determined by special tests.

### C4 Concentrically Loaded Compression Members

Axially loaded compression members should be designed for the following limit states depending on the configuration of the cross-section, thickness of material, unbraced length, and end restraint: (1) yielding, (2) overall column buckling (flexural buckling, torsional buckling, or torsional-flexural buckling), and (3) local buckling of individual elements. For the design tables and example problems on columns, see Parts I and III of the AISI Cold-Formed Steel Design Manual (AISI, 2002).

#### A. Yielding

It is well known that a very short, compact column under an axial load may fail by yielding. The yield load is determined by Equation C-C4-1:

\[
P_y = A_g F_y
\]

where \(A_g\) is the gross area of the column and \(F_y\) is the yield point of steel.

#### B. Flexural Buckling of Columns

(a) Elastic Buckling Stress

A slender, axially loaded column may fail by overall flexural buckling if the cross-section of the column is a doubly-symmetric shape, closed shape (square or rectangular tube), cylindrical shape, or point-symmetric shape. For singly-symmetric shapes, flexural buckling is one of the possible failure modes. Wall studs connected with sheathing material can also fail by flexural buckling.

The elastic critical buckling load for a long column can be determined by the following Euler equation:
(P_{cr})_e = \frac{\pi^2 EI}{(KL)^2} \quad (C-C4-2)

where (P_{cr})_e is the column buckling load in the elastic range, E is the modulus of elasticity, I is the moment of inertia, K is the effective length factor, and L is the unbraced length. Accordingly, the elastic column buckling stress is

\[
(F_{cr})_e = \frac{(P_{cr})_e}{A_g} = \frac{\pi^2 E}{(KL / r)^2} \quad (C-C4-3)
\]

in which \( r \) is the radius of gyration of the full cross section, and \( KL / r \) is the effective slenderness ratio.

(b) Inelastic Buckling Stress

When the elastic column buckling stress computed by Equation C-C4-3 exceeds the proportional limit, \( F_{pr} \), the column will buckle in the inelastic range. Prior to 1996, the following equation was used in the AISI Specification for computing the inelastic column buckling stress:

\[
(F_{cr})_I = F_y \left(1 - \frac{F_y}{4(F_{cr})_e}\right) \quad (C-C4-4)
\]

It should be noted that because the above equation is based on the assumption that \( F_{pr} = F_y / 2 \), it is applicable only for \( (F_{cr})_e \geq F_y / 2 \).

By using \( \lambda_c \) as the column slenderness parameter instead of slenderness ratio, \( KL / r \), Equation C-C4-4 can be rewritten as follows:

\[
(F_{cr})_I = \left(1 - \frac{\lambda_c^2}{4}\right) F_y \quad (C-C4-5)
\]

where

\[
\lambda_c = \sqrt{\frac{F_y}{(F_{cr})_e}} = KL \sqrt{\frac{F_y}{F_{cr}} / E} \quad (C-C4-6)
\]

Accordingly, Equation C-C4-5 is applicable only for \( \lambda_c \leq \sqrt{2} \).

(c) Nominal Axial Strength [Compressive Resistance] for Locally Stable Columns

If the individual components of compression members have small \( w / t \) ratios, local buckling will not occur before the compressive stress reaches the column buckling stress or the yield point of steel. Therefore, the nominal axial strength [compressive resistance] can be determined by the following equation:

\[
P_n = A_g F_{cr} \quad (C-C4-7)
\]

where

- \( P_n \) = nominal axial strength
- \( A_g \) = gross area of the column
- \( F_{cr} \) = column buckling stress

(d) Nominal Axial Strength [Compressive Resistance] for Locally Unstable Columns
For cold-formed steel compression members with large w/t ratios, local buckling of individual component plates may occur before the applied load reaches the nominal axial strength [compressive resistance] determined by Equation C-C4-7. The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength [resistance]. From 1946 through 1986, the effect of local buckling on column strength was considered in the AISI Specification by using a form factor Q in the determination of allowable stress for the design of axially loaded compression members (Winter, 1970; Yu, 2000). Even though the Q-factor method was used successfully for the design of cold-formed steel compression members, research work conducted at Cornell University and other institutions have shown that this method is capable of improvement. On the basis of the test results and analytical studies of DeWolf, Pekoz, Winter, and Mulligan (DeWolf, Pekoz and Winter, 1974; Mulligan and Pekoz, 1984) and Pekoz’s development of a unified approach for the design of cold-formed steel members (Pekoz, 1986b), the Q-factor method was eliminated in the 1986 edition of the AISI Specification. In order to reflect the effect of local buckling on the reduction of column strength, the nominal axial strength [compressive resistance] is determined by the critical column buckling stress and the effective area, $A_e$, instead of the full sectional area. When $A_e$ cannot be calculated, such as when the compression member has dimensions or geometry beyond the range of applicability of the AISI Specification, the effective area $A_e$ can be determined experimentally by stub column tests using the procedure given in Part VIII of the AISI Design Manual (AISI, 2002). For a more in-depth discussion of the background for these provisions, see Pekoz (1986b). Therefore, the nominal axial strength [compressive resistance] of cold-formed steel compression members can be determined by the following equation:

$$P_n = A_eF_{cr}$$

(C-C4-8)

where $F_{cr}$ is either elastic buckling stress or inelastic buckling stress whichever is applicable, and $A_e$ is the effective area at $F_{cr}$.

An exception for Equation C-C4-8 is for C- and Z-shapes, and single angle sections with unstiffened flanges. For these cases, the nominal axial strength [compressive resistance] is also limited by the following capacity, which is determined by the local buckling stress of the unstiffened element and the area of the full cross-section:

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

(C-C4-9)

The above equation was included in Section C4(b) of the 1986 edition of the AISI Specification when the unified design approach was adopted. A study conducted by Rasmussen at the University of Sydney (Rasmussen, 1994) indicated that the design provisions of Section C4(b) of the 1986 AISI Specification leads to unnecessarily and excessively conservative results. This conclusion was based on analytical studies carefully
validated against test results as reported by Rasmussen and Hancock (1992). Consequently, Section C4(b) of Specification (Equation C-C4-9) was deleted in the 1996 AISI Specification.

In the 1996 AISI Specification, the design equations for calculating the inelastic and elastic flexural buckling stresses have been changed to those used in the AISC LRFD Specification (AISC, 1993). As given in the Specification Section C4(a), these design equations are as follows:

\[
\text{For } \lambda_c \leq 1.5: \quad F_n = (0.658 \lambda_c^2) F_y
\]
\[
\text{For } \lambda_c > 1.5: \quad F_n = \left[ \frac{0.877}{\lambda_c^2} \right] F_y
\]

where \( F_n \) is the nominal flexural buckling stress which can be either in the elastic range or in the inelastic range depending on the value of \( \lambda_c = \sqrt{F_y / F_e} \), and \( F_e \) is the elastic flexural buckling stress calculated by using Equation C-C4-3. Consequently, the equation for determining the nominal axial strength [compressive resistance] can be written as

\[
P_n = A_e F_n
\]

which is Equation C4-1 of the Specification.

The reasons for changing the design equations from Equation C-C4-4 to Equation C-C4-10 for inelastic buckling stress and from Equation C-C4-3 to Equation C-C4-11 for elastic buckling stress are:

1. The revised column design equations (Equations C-C4-10 and C-C4-11) are based on a different basic strength [resistance] model and were shown to be more accurate by Pekoz and Sumer (1992). In this study, 299 test results on columns and beam-columns were evaluated. The test specimens included members with component elements in the post-local buckling range as well as those that were locally stable. The test specimens included members subject to flexural buckling as well as torsional-flexural buckling.

2. Because the revised column design equations represent the maximum strength [resistance] with due consideration given to initial crookedness and can provide the better fit to test results, the required factor of safety can be reduced. In addition, the revised equations enable the use of a single factor of safety for all \( \lambda_c \) values even though the nominal axial strength [compressive resistance] of columns decreases as the slenderness increases because of initial out-of-straightness. By using the selected factor of safety and resistance factor, the results obtained from the ASD and LRFD approaches would be approximately the same for a live-to-dead load ratio of 5.0.

The design provisions included in the AISI ASD Specification (AISI, 1986), the LRFD Specification (AISI, 1991), the 1996 Specification and the current Specification (AISI, 2001) are compared in Figures C-C4-1, C-C4-2, and C-C4-3.
Figure C-C4-1 shows a comparison of the critical flexural buckling stresses used in the 1986, 1991, 1996 and 2001 Specifications. The equations used to plot these two curves are indicated in the figure. Because of the use of a relatively smaller factor of safety in the 2001 Specification, it can be
seen from Figure C-C4-2 that the design capacity is increased for thin columns with low slenderness parameters and decreased for high slenderness parameters. However, the differences would be less than 10%. For the LRFD method, the differences between the nominal axial strengths [compressive resistances] used for the 1991 and the 2001 LRFD design provisions are shown in Figure C-C4-3. The curve for the LSD
provisions would be the same as the curve for LRFD.

\( e \) **Effective Length Factor, \( K \)**

The effective length factor \( K \) accounts for the influence of restraint against rotation and translation at the ends of a column on its load-carrying capacity. For the simplest case, a column with both ends hinged and braced against lateral translation, buckling occurs in a single half-wave and the effective length \( KL \), being the length of this half-wave, is equal to the actual physical length of the column (Figure C-C4-4);
correspondingly, for this case, \( K = 1 \). This situation is approached if a
given compression member is part of a structure which is braced in such
a manner that no lateral translation (sidesway) of one end of the column
relative to the other can occur. This is so for columns or studs in a
structure with diagonal bracing, diaphragm bracing, shear-wall
construction or any other provision which prevents horizontal
displacement of the upper relative to the lower column ends. In these
situations it is safe and only slightly, if at all, conservative to take \( K = 1 \).

If translation is prevented and abutting members (including
foundations) at one or both ends of the member are rigidly connected to
the column in a manner which provides substantial restraint against
rotation, \( K \)-values smaller than 1 (one) are sometimes justified. Table C-
C4-1 provides the theoretical \( K \) values for six idealized conditions in
which joint rotation and translation are either fully realized or
nonexistent. The same table also includes the \( K \) values recommended by
the Structural Stability Research Council for design use (Galambos, 1998).

In trusses, the intersection of members provides rotational restraint to
the compression members at service loads. As the collapse load is
approached, the member stresses approach the yield point which greatly
reduces the restraint they can provide. For this reason \( K \) value is usually
taken as unity regardless of whether they are welded, bolted, or
connected by screws. However, when sheathing is attached directly to the
top flange of a continuous compression chord, recent research (Harper,
LaBoube and Yu, 1995) has shown that the \( K \) values may be taken as 0.75
(AISI, 1995).

On the other hand, when no lateral bracing against sidesway is present,
such as in the portal frame of Figure C-C4-5, the structure depends on its
own bending stiffness for lateral stability. In this case, when failure occurs
by buckling of the columns, it invariably takes place by the sidesway
motion shown. This occurs at a lower load than the columns would be
able to carry if they where braced against sidesway and the figure shows
that the half-wave length into which the columns buckle is longer than
the actual column length. Hence, in this case \( K \) is larger than 1 (one) and
its value can be read from the graph of Figure C-C4-6 (Winter et al., 1948a
and Winter, 1970). Since column bases are rarely either actually hinged or
completely fixed, \( K \)-values between the two curves should be estimated
depending on actual base fixity.

Figure C-C4-6 can also serve as a guide for estimating \( K \) for other simple
situations. For multi-bay and/or multi-story frames, simple alignment
charts for determining \( K \) are given in the AISC Commentaries (AISC,
1989; 1999). For additional information on frame stability and second
order effects, see SSRC Guide to Stability Design Criteria for Metal
Structures (Galambos, 1998) and the AISC Specifications and
Commentaries.

If roof or floor slabs, anchored to shear walls or vertical plane bracing
systems, are counted upon to provide lateral support for individual
columns in a building system, their stiffness must be considered when
functioning as horizontal diaphragms (Winter, 1958a).

C. Torsional Buckling of Columns

It was pointed out at the beginning of this section that purely torsional buckling, i.e., failure by sudden twist without concurrent bending, is also possible for certain cold-formed open shapes. These are all point-symmetric shapes (in which shear center and centroid coincide), such as doubly-symmetric I-shapes, anti-symmetric Z-shapes, and such unusual sections as cruciforms, swastikas, and the like. Under concentric load, torsional buckling of such shapes very rarely governs design. This is so because such members of realistic slenderness will buckle flexurally or by a combination of flexural and local buckling at loads smaller than those which would produce torsional buckling. However, for relatively short members of this type, carefully dimensioned to minimize local buckling, such torsional buckling cannot be completely ruled out. If such buckling is elastic, it occurs at the critical stress $\sigma_t$ calculated as follows (Winter, 1970):

$$\sigma_t = \frac{1}{A r_0^2} \left[ GJ + \frac{\pi^2 E C_w}{(Kt L_t)^2} \right]$$  \hspace{1cm} (C-C4-13)

The above equation is the same as Specification Equation C3.1.2.1-9, in which $A$ is the full cross-sectional area, $r_0$ is the polar radius of gyration of the cross section about the shear center, $G$ is the shear modulus, $J$ is Saint-Venant torsion constant of the cross section, $E$ is the modulus of elasticity, $C_w$ is the torsional warping constant of the cross section, and $Kt L_t$ is the effective length for twisting.

For inelastic buckling, the critical torsional buckling stress can also be calculated according to Equation C-C4-10 by using $\sigma_t$ as $F_e$ in the calculation of $\lambda_c$. 

Figure C-C4-6 Effective Length Factor $K$ in Laterally Unbraced Portal Frames
D. Torsional-Flexural Buckling of Columns

As discussed previously, concentrically loaded columns can buckle in the flexural buckling mode by bending about one of the principal axes; or in the torsional buckling mode by twisting about the shear center; or in the torsional-flexural buckling mode by simultaneous bending and twisting. For singly-symmetric shapes such as channels, hat sections, angles, T-sections, and I-sections with unequal flanges, for which the shear center and centroid do not coincide, torsional-flexural buckling is one of the possible buckling modes as shown in Figure C-C4-7. Unsymmetric sections will always buckle in the torsional-flexural mode.

It should be emphasized that one needs to design for torsional-flexural buckling only when it is physically possible for such buckling to occur. This means that if a member is so connected to other parts of the structure such as wall sheathing that it can only bend but cannot twist, it needs to be designed for flexural buckling only. This may hold for the entire member or for individual parts. For instance, a channel member in a wall or the chord of a roof truss is easily connected to girts or purlins in a manner which prevents twisting at these connection points. In this case torsional-flexural buckling needs to be checked only for the unbraced lengths between such connections. Likewise, a doubly-symmetric compression member can be made up by connecting two spaced channels at intervals by batten plates. In this case each channel constitutes an “intermittently fastened component of a built-up shape.” Here the entire member, being doubly-symmetric, is not subject to torsional-flexural buckling so that this mode needs to be checked only for the individual component channels between batten connections (Winter, 1970).
The governing elastic torsional-flexural buckling load of a column can be found from the following equation, (Chajes and Winter, 1965; Chajes, Fang and Winter, 1966; Yu, 2000):

\[ P_n = \frac{1}{2\beta} \left( (P_x + P_z) - \sqrt{(P_x + P_z)^2 - 4\beta P_x P_z} \right) \]  

(C-C4-14)

If both sides of this equation are divided by the cross-sectional area \( A \), one obtains the equation for the elastic, torsional-flexural buckling stress \( F_e \) as follows:

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

(C-C4-15)

For this equation, as in all provisions which deal with torsional-flexural buckling, the \( x \)-axis is the axis of symmetry; \( \sigma_{ex} = \pi^2 E/(K_x L_x/r_x)^2 \) is the flexural Euler buckling stress about the \( x \)-axis, \( \sigma_t \) is the torsional buckling stress (Equation C-C4-13) and \( \beta = 1-(x_o/r_o)^2 \). It is worth noting that the torsional-flexural buckling stress is always lower than the Euler stress \( \sigma_{ex} \) for flexural buckling about the symmetry axis. Hence, for these singly-symmetric sections, flexural buckling can only occur, if at all, about the \( y \)-axis which is the principal axis perpendicular to the axis of symmetry.

For inelastic buckling, the critical torsional-flexural buckling stress can also be calculated by using Equation C-C4-10.

An inspection of Equation C-C4-15 will show that in order to calculate \( \beta \) and \( \sigma_t \), it is necessary to determine \( x_o = \) distance between shear center and centroid, \( J = \) Saint-Venant torsion constant, and \( C_w = \) warping constant, in addition to several other, more familiar cross-sectional properties. Because of these complexities, the calculation of the torsional-flexural buckling stress cannot be made as simple as that for flexural buckling. However, a variety of design aids as given in Part VII of the Design Manual (AISI, 2002), simplify these calculations at least for the most common cold-formed steel shapes.

For one thing, any singly-symmetric shape can buckle either flexurally about the \( y \)-axis or torsional-flexurally, depending on its detailed dimensions. For instance, a channel stud with narrow flanges and wide web will generally buckle flexurally about the \( y \)-axis (axis parallel to web); in contrast a channel stud with wide flanges and a narrow web will generally fail in torsional-flexural buckling. One can determine the mode which governs by using the charts in Part VII of the AISI Design Manual. These design charts were developed for common shapes. They permit one to determine which of the two buckling modes governs, depending on simple combinations of the cross-sectional dimensions and the length of the member. If torsional-flexural buckling is indicated, the information and design aids in Parts I and VII of the AISI Design Manual (AISI, 2002) facilitate and expedite the necessary calculations.

The above discussion refers to members subject to torsional-flexural buckling, but made up of elements whose \( w/t \) ratios are small enough so that no local buckling will occur. For shapes which are sufficiently thin, i.e., with
w/t ratios sufficiently large, local buckling can combine with torsional-flexural buckling similar to the combination of local with flexural buckling. For this case, the effect of local buckling on the torsional-flexural buckling strength can also be handled by using the effective area, \( A_e \), determined at the stress \( F_n \) for torsional-flexural buckling.

E. Additional Design Consideration for Angles

During the development of a unified approach to the design of cold-formed steel members, Pekoz realized the possibility of a reduction in column strength due to initial sweep (out-of-straightness) of angle sections. Based on an evaluation of the available test results, an initial out-of-straightness of \( L/1000 \) was recommended by Pekoz for the design of concentrically loaded compression angle members and beam-columns in the 1986 edition of the AISI Specification. Those requirements were retained in Sections C4, C5.2.1, and C5.2.2 of the 1996 edition of the Specification. A recent study conducted at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) indicated that for the design of singly-symmetric unstiffened angles sections under the axial compression load, the required additional moment about the minor principal axis due to initial sweep should only be applied to those angle sections, for which the effective area at stress \( F_y \) is less than the full, unreduced cross-sectional area. Consequently, clarifications have been made in Sections C5.2.1 and C5.2.2 of the current edition of the AISI Specification to reflect the recent research findings.

F. Slenderness Ratios

The slenderness ratio, \( KL/r \), of all compression members preferably should not exceed 200, except that during construction only, \( KL/r \) should not exceed 300. In 1999, the above recommendations were moved from the Specification to the Commentary.

The maximum slenderness ratios on compression and tension members have been stipulated in steel design standards for many years but are not mandatory in the AISI Specification. The \( KL/r \) limit of 300 is still recommended for most tension members in order to control serviceability issues such as handling, sag and vibration. The limit is not mandatory, however, because there are a number of applications where it can be shown that such factors are not detrimental to the performance of the structure or assembly of which the member is a part. Flat strap tension bracing is a common example of an acceptable type of tension member where the \( KL/r \) limit of 300 is routinely exceeded.

The compression member \( KL/r \) limits are recommended not only to control handling, sag and vibration serviceability issues but also to flag possible strength [resistance] concerns. The AISI Specification provisions adequately predict the capacities of slender columns and beam-columns but the resulting strengths [resistances] are quite small and the members relatively inefficient. Slender members are also very sensitive to eccentrically applied axial load because the moment magnification factors given by \( 1/\alpha \) will be large.
C4.1 Sections Not Subject to Torsional or Torsional-Flexural Buckling

If concentrically loaded compression members can buckle in the flexural buckling mode by bending about one of the principal axes, the nominal flexural buckling strength [resistance] of the column should be determined by using Equation C4-1 of the Specification. The elastic flexural buckling stress is given in Equation C4.1-1 of the Specification, which is the same as Equation C-C4-3 of the Commentary. This provision is applicable to doubly-symmetric sections, closed cross sections and any other sections not subject to torsional or torsional-flexural buckling.

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

As discussed previously in Section C4, torsional buckling is one of the possible buckling modes for doubly- and point-symmetric sections. For singly-symmetric sections, torsional-flexural buckling is one of the possible buckling modes. The other possible buckling mode is flexural buckling by bending about the y-axis (i.e., assuming x-axis is the axis of symmetry).

For torsional buckling, the elastic buckling stress can be calculated by using Equation C-C4-13. For torsional-flexural buckling, Equation C-C4-15 can be used to compute the elastic buckling stress. The following simplified equation for elastic torsional-flexural buckling stress is an alternative permitted by the AISI Specification:

\[ F_c = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \]  

(C-C4-16)

The above equation is based on the following interaction relationship given by Pekoz and Winter (1969a):

\[ \frac{1}{P_n} = \frac{1}{P_x} + \frac{1}{P_z} \]  

or

\[ \frac{1}{F_c} = \frac{1}{\sigma_{ex}} + \frac{1}{\sigma_t} \]  

(C-C4-17)

(C-C4-18)

Research at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) has shown that singly-symmetric unstiffened cold-formed steel angles, which have a fully effective cross-section under yield point, do not fail in a torsional-flexural mode and can be designed based on flexural buckling alone as specified in Specification Section C4.1. There is also no need to include a load eccentricity for these sections when using Specification Section C5.2.1 or Section C5.2.2 as explained in Item E of Section C4.

C4.3 Point-Symmetric Sections

This section of the Specification is for the design of discretely braced point-symmetric section subjected to axial compression. An example of a point-symmetric section is a lipped or unlipped Z-section with equal flanges.
The critical elastic buckling stress of point-symmetric sections is the lesser of the two possible buckling modes, the elastic torsional buckling stress, $\sigma_t$, as defined in *Specification* Equation C3.1.2.1-9 or the elastic flexural buckling stress about its minor principal axis, as defined in *Specification* Equation C4.1-1. Figure C-D3.2.2-5 shows the relationship of the principal axes to the x and y axes of a lipped Z-section. The elastic flexural buckling stress should be calculated for axis 2.

**C4.4 Nonsymmetric Sections**

For nonsymmetric open shapes the analysis for torsional-flexural buckling becomes extremely tedious unless its need is sufficiently frequent to warrant computerization. For one thing, instead of the quadratic equations, cubic equations have to be solved. For another, the calculation of the required section properties, particularly $C_w$, becomes quite complex. The method of calculation is given in Parts I and VII of the AISI *Design Manual* (AISI, 2002) and the book by Yu (2000). Section C4.4 of the *Specification* states that calculation according to this section shall be used or tests according to Chapter F shall be made when dealing with nonsymmetric open shapes.

**C4.5 Built-Up Members**

Compression members composed of two shapes joined together at discrete points have a reduced shear rigidity. The influence of this reduced shear rigidity on the buckling stress is taken into account by modifying the slenderness ratio used to calculate the elastic critical buckling stress (Bleich, 1952). The overall slenderness and the local slenderness between connected points both influence the compressive resistance. The combined action is expressed by the modified slenderness ratio given by the following:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_{oi}^2 + \left(\frac{a}{r_i}\right)^2}$$  \hspace{1cm} (C-C4.5-1)

Note that in this expression, the overall slenderness ratio, $(KL/r)_{oi}$, is computed about the same axis as the modified slenderness ratio, $(KL/r)_{mi}$. Further, the modified slenderness ratio, $(KL/r)_{mi}$, replaces $KL/r$ in the *Specification* Section C4 for both flexural and torsional-flexural buckling.

This modified slenderness approach is used in other steel standards, including the AISC (AISC, 1999), CSA S136 (CSA S136, 1994), and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994).

To prevent the flexural buckling of the individual shapes between intermediate connectors, the intermediate fastener spacing, $a$, is limited such that $a/r_i$ does not exceed one half the governing slenderness ratio of the built-up member (i.e. $a/r_i \leq 0.5(KL/r)_{oi}$). This intermediate fastener spacing requirement is consistent with the previous edition of the AISI *Specification* with the one half factor included to account for any one of the connectors becoming loose or ineffective. Note that the previous edition of S136 (S136,
1994) had no limit on fastener spacing.

The importance of preventing shear slip in the end connection is addressed by the prescriptive requirements in Specification Section C4.5(2) adopted from the AISC (AISC, 1999) and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994). These provisions are new to both the AISI Specification and CSA S136 Standard.

Intermediate connectors are required to transmit a shear force equal to 2.5% of the nominal force for ASD and factored force for LRFD and LSD in the built-up member. This requirement has been adopted from CSA S136-94 and is new to the AISI Specification.

Note that the provision in Specification Section C4.5 has been substantially taken from research in hot-rolled built-up members connected with bolts or welds. These hot-rolled provisions have been extended to include other fastener types common in cold-formed steel construction (such as screws) provided they meet the 2.5% requirement for shear strength [resistance] and the conservative spacing requirement \( a/r_1 \leq 0.5(KL/r)_{o} \).

**C4.6 Compression Members Having One Flange Through-Fastened to Deck or Sheathing**

For axially loaded C- or Z- sections having one flange attached to deck or sheathing and the other flange unbraced, e.g., a roof purlin or wall girt subjected to wind or seismic generated compression forces, the axial load capacity is less than a fully braced member, but greater than an unbraced member. The partial restraint relative to weak axis buckling is a function of the rotational stiffness provided by the panel-to-purlin connection. Specification Equation C4.6-1 is used to calculate the weak axis capacity. This equation is not valid for sections attached to standing seam roofs. The equation was developed by Glaser, Kaehler and Fisher (1994) and is also based on the work contained in the reports of Hatch, Easterling and Murray (1990) and Simaan (1973).

A limitation on the maximum yield point of the C- or Z- section is not given in the Specification since Specification Equation C4.6-1 is based on elastic buckling criteria. A limitation on minimum length is not contained in the Specification because Equation C4.6-1 is conservative for spans less than 15 feet.

As indicated in the Specification, the strong axis axial load capacity is determined assuming that the weak axis of the strut is braced.

The controlling axial capacity (weak or strong axis) is suitable for usage in the combined axial load and bending equations in Section C5 of the Specification (Hatch, Easterling, and Murray, 1990).

**C5 Combined Axial Load and Bending**

In the 1996 edition of the AISI Specification, the design provisions for combined axial load and bending were expanded to include expressions for the design of members subject to combined tensile axial load and bending. In this edition, combined axial and bending for the limit states design (LSD) method has
been added. The design approach of the LSD method is the same as the LRFD method.

**C5.1 Combined Tensile Axial Load and Bending**

These provisions apply to concurrent bending and tensile axial load. If bending can occur without the presence of tensile axial load, the member must also conform to the provisions of Specification Section C3. Care must be taken not to overestimate the tensile load as this could be unconservative.

**C5.1.1 ASD Method**

*Specification* Equation C5.1.1-1 provides a design criterion to prevent yielding of the tension flange of a member under combined tensile axial load and bending. *Specification* Equation C5.1.1-2 provides a design criterion to prevent failure of the compression flange.

**C5.1.2 LRFD and LSD Methods**

Similar to the ASD method, two interaction equations are included in *Specification* Section C5.1.2 for the LRFD and the LSD methods. *Specification* Equations C5.1.2-1 and C5.1.2-2 are used to prevent the failure of the tension flange and compression flange, respectively. In both equations, different symbols are used for the required tensile axial strength [factored tension] and the required flexural strength [factored moment] according to the LRFD and the LSD methods.

**C5.2 Combined Compressive Axial Load and Bending**

Cold-formed steel members under a combination of compressive axial load and bending are usually referred to as beam-columns. The bending may result from eccentric loading, transverse loads, or applied moments. Such members are often found in framed structures, trusses, and exterior wall studs. For the design of such members, interaction equations have been developed for locally stable and unstable beam-columns on the basis of thorough comparison with rigorous theory and verified by the available test results (Pekoz, 1986a; Pekoz and Sumer, 1992).

The structural behavior of beam-columns depends on the shape and dimensions of the cross section, the location of the applied eccentric load, the column length, the end restraint, and the condition of bracing. In this edition of the *Specification*, the ASD method is included in Section C5.2.1. *Specification* Section C5.2.2 is for the LRFD and the LSD methods.

**C5.2.1 ASD Method**

When a beam-column is subject to an axial load P and end moments M as shown in Figure C-C5.2-1(a), the combined axial and bending stress in compression is given in Equation C-C5.2.1-1 as long as the member remains straight:
\[ f = \frac{P}{A} + \frac{M}{S} \]  
\[ = f_a + f_b \]  
\[
\text{(C-C5.2.1-1)}
\]

where

\[
\begin{align*}
  f &= \text{combined stress in compression} \\
  f_a &= \text{axial compressive stress} \\
  f_b &= \text{bending stress in compression} \\
  P &= \text{applied axial load} \\
  A &= \text{cross-sectional area} \\
  M &= \text{bending moment} \\
  S &= \text{section modulus}
\end{align*}
\]

It should be noted that in the design of such a beam-column by using the ASD method, the combined stress should be limited by certain allowable stress \( F \), that is,

\[
\begin{align*}
  f_a + f_b &\leq F \\
  \text{or} \\
  \frac{f_a}{F} + \frac{f_b}{F} &\leq 1.0
\end{align*}
\]  
\[
\text{(C-C5.2.1-2)}
\]

As specified in Sections C3.1 and C4 of the *Specification*, the factor of safety \( \Omega_c \) for the design of compression members is different from the factor of safety \( \Omega_b \) for beam design. Therefore Equation C-C5.2.1-2 may be modified as follows:

\[
\begin{align*}
  \frac{f_a}{F_a} + \frac{f_b}{F_b} &\leq 1.0
\end{align*}
\]  
\[
\text{(C-C5.2.1-3)}
\]

where

\[
\begin{align*}
  F_a &= \text{allowable stress for the design of compression members} \\
  F_b &= \text{allowable stress for the design of beams}
\end{align*}
\]

If the strength ratio is used instead of the stress ratio, Equation C-C5.2.1-3 can be rewritten as follows:
\[
\frac{P}{P_a} + \frac{M}{M_a} \leq 1.0 \tag{C-C5.2.1-4}
\]
where
\[P = \text{applied axial load} = Af_a\]
\[P_a = \text{allowable axial load} = AF_a\]
\[M = \text{applied moment} = Sf_b\]
\[M_a = \text{allowable moment} = SF_b\]

According to Equation C-A4.1.1-1,
\[P_a = \frac{P_n}{\Omega_c} \]
\[M_a = \frac{M_n}{\Omega_b} \]

In the above equations, \(P_n\) and \(\Omega_c\) are given in Specification Section C4, while \(M_n\) and \(\Omega_b\) are specified in Specification Section C3.1. Substituting the above expressions into Equation C-C5.2.1-4, the following interaction equation (Specification Equation C5.2.1-3), can be obtained:
\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{M_n} \leq 1.0 \tag{C-C5.2.1-5}
\]

Equation C-C5.2.1-4 is a well-known interaction equation, which has been adopted in several specifications for the design of beam-columns. It can be used with reasonable accuracy for short members and members subjected to a relatively small axial load. It should be realized that in practical applications, when end moments are applied to the member, it will be bent as shown in Figure C-C5.2-1(b) due to the applied moment \(M\) and the secondary moment resulting from the applied axial load \(P\) and the deflection of the member. The maximum bending moment at midlength (point C) can be represented by
\[M_{\text{max}} = \Phi M \tag{C-C5.2.1-6}\]

where
\[M_{\text{max}} = \text{maximum bending moment at mid-length}\]
\[M = \text{applied end moments}\]
\[\Phi = \text{amplification factor}\]

It can be shown that the amplification factor \(\Phi\) may be computed by
\[
\Phi = \frac{1}{1 - \frac{P}{P_E}} \tag{C-C5.2.1-7}
\]
where \(P_E = \text{elastic column buckling load (Euler load)} = \pi^2 EI/(KL_b)^2\).

Applying a safety factor \(\Omega_c\) to \(P_E\), Equation C-C5.2.1-7 may be rewritten as
\[
\Phi = \frac{1}{1 - \frac{\Omega_c P}{P_E}} \tag{C-C5.2.1-8}
\]

If the maximum bending moment \(M_{\text{max}}\) is used to replace \(M\), the following interaction equation can be obtained from Equations C-C5.2.1-5 and C-C5.2.1-8:
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\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b M}{(1 - \frac{\Omega_c P}{P_E}) M_n} \leq 1.0 \quad (\text{C-C5.2.1-9})
\]

It has been found that Equation C-C5.2.1-9, developed for a member subjected to an axial compressive load and equal end moments, can be used with reasonable accuracy for braced members with unrestrained ends subjected to an axial load and a uniformly distributed transverse load. However, it could be conservative for compression members in unbraced frames (with sidesway), and for members bent in reverse curvature. For this reason, the interaction equation given in Equation C-C5.2.1-9 should be further modified by a coefficient \( C_m \) as shown in Equation C-C5.2.1-10, to account for the effect of end moments:

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_m M}{\alpha M_n} \leq 1.0 \quad (\text{C-C5.2.1-10})
\]

The above equation is Specification Equation C5.2.1-1, in which \( \alpha = 1 - \Omega_c P/P_E \).

In Equation C-C5.2.1-10, \( C_m \) can be determined for one of the three cases defined in Specification Section C5.2.1. For Case 1, \( C_m \) is given as 0.85. In Case 2, it can be computed by Equation C-C5.2.1-11 for restrained compression members braced against joint translation and not subject to transverse loading:

\[
C_m = 0.6 - 0.4 \frac{M_1}{M_2} \quad (\text{C-C5.2.1-11})
\]

where \( M_1/M_2 \) is the ratio of smaller to the larger end moments. For Case 3, \( C_m \) may be approximated by using the value given in the AISC Commentaries for the applicable condition of transverse loading and end restraint (AISC, 1989 and 1999).

Figure C-C5.2-2 illustrates the interaction relation. In order to simplify the illustration, bending about only one axis is considered in Figure C-C5.2-2 and the factors of safety, \( \Omega_c \) and \( \Omega_b \), are taken as unity. The ordinate is the compressive axial load on the member and the abscissa is the bending moment. When the moment is zero, the limiting axial load is \( P_n \) determined in accordance with Specification Section C4, which is based on column buckling and local buckling. When the axial load is zero, the limiting moment, \( M_n \), is determined in accordance with Specification Section C3 and is the lowest of the effective yield moment, the moment based on inelastic reserve capacity (if applicable) or the moment based on lateral-torsional buckling. The interaction relation cannot exceed either of these limits.

When Specification Equation C5.2.1-1 is plotted in Figure C-C5.2-2, the axial load limit is \( P_n \) and the moment limit is \( M_n/C_m \) which will exceed \( M_n \) when \( C_m < 1 \). Therefore, Specification Equation C5.2.1-2 is used as a mathematical stratagem to limit the moment to \( M_n \) and match the rigorous solution at low axial loads. The interaction limit is the lower of...
the two equations as shown by hash marks. Specification Equation C5.2.1-2 is a linear relation between the nominal axial yield strength $P_{no} = F_y A_e$ and $M_n$, and does not represent a failure state over its whole range. If Specification Equation C5.2.1-2 uses the moment capacity based only on yield or local buckling, $M_{no} = F_y S_{eff}$, it would be represented by the dashed line, which could exceed an $M_n$ limit based on lateral-torsional buckling. Clearly, load combinations in the shaded region would be unconservative. If $M_n$ is determined by $M_{no}$, the relation in Figure C-C5.2-2 still apply. If $C_m/\alpha \geq 1$, Specification Equation C5.2.1-1 controls.

For low axial loads, Specification Equation C5.2.1-3 may be used. This is a conservative simplification of the interaction relation defined by Specification Equations C5.2.1-1 and C5.2.1-2.

In 2001, a new requirement of each individual ratio in Eqs. C5.2.1-1 to C5.2.1-3 not exceeding unity was added to avoid situations of the load $\Omega_c P$ exceeding the Euler buckling load $P_E$, which leads to amplification factor $\Phi$ (given in Eq. C-C5.2.1-8) negative.

For the design of angle sections using the ASD method, the required additional bending moment of $PL/1000$ about the minor principal axis is discussed in Item E of Section C4 of the Commentary.

**C5.2.2 LRFD and LSD Methods**

The LRFD and the LSD methods use the same interaction equations as the ASD method, except that $\phi_c P_n$ and $\phi_b M_n$ are used for design strengths [factored resistances]. In addition, the required axial strength [factored compressive force], $P_{u}$, and the required flexural strength [factored moment], $M_{u}$, are to be determined from factored loads.
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according to the requirements of Section A5.1.2 of the Specification Appendices A and C for USA and Mexico, and the requirements of Section A6.1.2 of Specification Appendix B for Canada. In Specification Equations C5.2.2-1 through C5.2.2-3, different symbols are used for the required compressive axial strength [factored compressive force] and the required flexural strength [factored moment] according to the LRFD and the LSD methods.

It should be noted that, as compared with the 1991 edition of the AISI LRFD Specification, the definition of factor $\alpha$ was changed in the AISI 1996 and this edition of the Specification by eliminating the $\phi_c$ term because the term $P_E$ is a deterministic value and hence does not require a resistance factor.

The interaction equations used in Specification Section C5.2.2 are the same as that used in the AISI LRFD Specification (AISI, 1991) but they are different as compared with the AISC LRFD Specification (AISC, 1999) due to the lack of sufficient evidence for cold-formed steel columns to adopt the AISC LRFD criteria.

Similar to Specification Section C5.2.1, ASD Method, the new requirement of each individual ratio in Specification Equations C5.2.2-1 to C5.2.2-3 not exceeding unity was added in 2001.

For the design of angle sections using the LRFD and the LSD methods, the required additional bending moment of PL/1000 about the minor principal axis was discussed in Item E of Section C4 of the Commentary.

C6 Closed Cylindrical Tubular Members

Closed thin-walled cylindrical tubular members are economic sections for compression and torsional members because of their large ratio of radius of gyration to area, the same radius of gyration in all directions, and the large torsional rigidity. Like other cold-formed steel compression members, cylindrical tubes must be designed to provide adequate safety not only against overall column buckling but also against local buckling. It is well known that the classical theory of local buckling of longitudinally compressed cylinders overestimates the actual buckling strength [resistance] and that inevitable imperfections and residual stresses reduce the actual strength [resistance] of compressed tubes radically below the theoretical value. For this reason, the design provisions for local buckling have been based largely on test results.

Local Buckling Stress

Considering the postbuckling behavior of the axially compressed cylinder and the important effect of the initial imperfection, the design provisions included in the AISI Specification were originally based on Plantema’s graphic representation and the additional results of cylindrical shell tests made by Wilson and Newmark at the University of Illinois (Winter, 1970).

From the tests of compressed tubes, Plantema found that the ratio $F_{ult}/F_Y$ depends on the parameter $(E/F_Y)(t/D)$, in which $t$ is the wall thickness, $D$ is the
mean diameter of the tube, and $F_{\text{ult}}$ is the ultimate stress or collapse stress. As shown in Figure C-C6-1, line 1 corresponds to the collapse stress below the proportional limit, line 2 corresponds to the collapse stress between the proportional limit and the yield point, and line 3 represents the collapse stress occurring at yield point. In the range of line 3, local buckling will not occur before yielding. In ranges 1 and 2, local buckling occurs before the yield point is reached. The cylindrical tubes should be designed to safeguard against local buckling.

Based on a conservative approach, the Specification specifies that when the $D/t$ ratio is smaller than or equal to $0.112E/F_y$, the tubular member shall be designed for yielding. This provision is based on point A1, for which $(E/F_y)(t/D) = 8.93$.

When $0.112E/F_y < D/t < 0.441E/F_y$, the design of tubular members is based on the inelastic local buckling criteria. For the purpose of developing a design equation for inelastic buckling, point $B_1$ was selected to represent the proportional limit. For point $B_1$,

$$
\left( \frac{E}{F_y} \right) \left( \frac{t}{D} \right) = 2.27, \quad \frac{F_{\text{ult}}}{F_y} = 0.75
$$

Equation (C-C6-1)

Using line $A_1B_1$, the maximum stress of cylindrical tubes can be represented by

$$
\frac{F_{\text{ult}}}{F_y} = 0.037 \left( \frac{E}{F_y} \right) \left( \frac{t}{D} \right) + 0.667
$$

Equation (C-C6-2)
When $D/t \geq 0.441E/F_y$, the following equation represents Line 1 for elastic local buckling stress:

$$\frac{F_{ult}}{F_y} = 0.328 \left( \frac{E}{F_y} \right) \left( \frac{t}{D} \right)$$

(C-C6-3)

The correlations between the available test data and Equations C-C6-2 and C-C6-3 are shown in Figure C-C6-2. The definition of symbol “D” was changed from “mean diameter” to “outside diameter” in the 1986 AISI Specification in order to be consistent with the general practice.

It should be noted that the design provisions of Specification Section C6 are applicable only for members having a ratio of outside diameter-to-wall thickness, $D/t$, not greater than 0.441E/F_y because the design of extremely thin tubes will be governed by elastic local buckling resulting in an uneconomical design. In addition, cylindrical tubular members with unusually large $D/t$ ratios are very sensitive to geometric imperfections.

### C6.1 Bending

For thick cylinders in bending, the initiation of yielding does not represent a failure condition as is generally assumed for axial loading. Failure is at the plastic moment capacity which is at least 1.29 times the moment at first yielding. In addition, the conditions for inelastic local buckling are not as severe as in axial compression due to the stress gradient.

Specification Equations C6.1-2, C6.1-3 and C6.1-4 are based upon the work reported by Sherman (1985) and an assumed minimum shape factor of 1.25. This slight reduction in the inelastic range has been made to limit the
maximum bending stress to 0.75$F_y$, a value typically used for solid sections in bending for the ASD method. The reduction also brings the criteria closer to a lower bound for inelastic local buckling. A small range of elastic local buckling has been included so that the upper D/t limit of 0.441$E/F_y$ is the same as for axial compression.

All three equations for determining the nominal flexural strength [moment resistance] of closed cylindrical tubular members are shown in Figure C-C6.1-1. These equations have been used in the AISI Specification since 1986 and are retained in this Specification. In 1999, the limiting D/t ratios for Specification Equations C6.1-2 and C6.1-3 have been revised to provide an appropriate continuity. The safety factor $\Omega_B$ and the resistance factor $\phi_B$ are the same as that used in Specification Section C3.1.1 for sectional bending strength.

### C6.2 Compression

When closed cylindrical tubes are used as concentrically loaded compression members the nominal axial strength [compressive resistance] is determined by the same equation as given in Specification Section C4, except that (1) the nominal buckling stress, $F_{ce}$, is determined only for flexural buckling and (2) the effective area, $A_{ce}$, is calculated by Equation C-C6.2-1:
\[ A_e = [1 - (1 - R^2)(1 - A_o / A)]A \]  \hspace{2cm} (C-C6.2-1)

where

\[ R = \sqrt{\frac{F_y}{2F_e}} \]  \hspace{2cm} (C-C6.2-2)

\[ A_o = \left[ \frac{0.037}{DF_y / tE} + 0.667 \right] A \leq A \]  \hspace{2cm} (C-C6.2-3)

and \( A = \) area of the unreduced cross section. The factor of safety \( \Omega_c \) and the resistance factor \( \phi_c \) are the same as that used in Specification Section C4 for compression members.

Equation C-C6.2-3 is used for computing the reduced area due to local buckling. It is derived from Equation C-C6-2 for inelastic local buckling stress (Yu, 2000).

In 1999, the coefficient, \( R \), was limited to one (1.0) so that the effective area, \( A_e \), will always be less than or equal to the unreduced cross sectional area, \( A \). To simplify the equations, \( R = F_y / 2F_e \) rather than \( R = \sqrt{F_y / 2F_e} \) as in the previous edition of the AISI Specification.

**C6.3 Combined Bending and Compression**

The interaction equations presented in Specification Section C5 can also be used for the design of closed cylindrical tubular members when these members are subject to combined bending and compression.
D. STRUCTURAL ASSEMBLIES

D1  Built-Up Sections

D1.1  I-Sections Composed of Two C-Sections

I-Sections made by connecting two C-sections back to back are often used as either compression or flexural members. Cases (2) and (8) of Figure C-A1.2-2 and Cases (3) and (7) of Figure C-A1.2-3 show several built-up I-sections.

(a) Compression Members

For the special case of built-up compression members composed of two C-sections, reference is made to the general provisions for built-up compression members in Specification Section C4.5.

(b) Flexural Members

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equation D1.1-1 of the Specification. The first requirement is an arbitrarily selected limit to prevent any possible excessive distortion of the top flange between connectors. The second is based on the strength [resistance] and arrangement of connectors and the intensity of the load acting on the beam (Yu, 2000).

The second requirement for maximum spacing of connectors required by Specification Equation D1.1-1 is based on the fact that the shear center of the C-section is neither coincident with nor located in the plane of the web; and that when a load Q is applied in the plane of the web, it produces a twisting moment Qm about its shear center, as shown in Figure C-D1.1-1. The tensile force of the top connector Ts can then be computed from the equality of the twisting moment Qm and the resisting moment Ts,g, that is

\[ Qm = Ts,g \]  

\[ Ts = \frac{Qm}{g} \]  

Considering that q is the intensity of the load and that s is the spacing of connectors as shown in Figure C-D1.1-2, the applied load is Q=qs/2. The maximum spacing smax used in the Specification can easily be obtained by substituting the above value of Q into Equation C-D1.1-2 of this
Commentary. The determination of the load intensity $q$ is based upon the type of loading applied to the beam. For simple C-sections without stiffening lips at the outer edges,

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

For C-sections with stiffening lips at the outer edges,

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

$w_f =$Projection of flanges from the inside face of the web (For C-sections with flanges of unequal width, $w_f$ shall be taken as the width of the wider flange)

d = Depth of C-section or beam

D = Overall depth of lip

$I_x =$Moment of inertia of one C-section about its centroidal axis normal to the web

In addition to the above considerations on the required strength [effect of factored loads] of connections, the spacing of connectors should not be so great as to cause excessive distortion between connectors by separation along the top flange. In view of the fact that C-sections are connected back to back and are continuously in contact along the bottom flange, a maximum spacing of $L/3$ may be used. Considering the possibility that one connection may be defective, a maximum spacing of $s_{max} = L/6$ is the first requirement in Specification Equation D1.1-1.

**D1.2 Spacing of Connections in Compression Elements**

When compression elements are joined to other parts of built-up members by intermittent connections, these connectors must be closely spaced to develop the required strength [resistance] of the connected element. Figure C-D1.2-1 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, $w$, equal to the
distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section D1.2 of the *Specification*.

Section D1.2(a) of the *Specification* requires that the necessary shear strength [resistance] be provided by the same standard structural design procedure that is used in calculating flange connections in bolted or welded plate girders or similar structures.

Section D1.2(b) of the *Specification* ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure C-D1.2-1) at a stress less than $1.67f_c$, where $f_c$ is the stress at service load in the connected compression element (Winter, 1970; Yu, 2000). The AISI requirement is based on the following Euler equation for column buckling:

$$
\sigma_{cr} = \frac{\pi^2 E}{(KLr)^2}
$$

by substituting $\sigma_{cr} = 1.67f_c$, $K = 0.6$, $L = s$, and $r = t/\sqrt{12}$. This provision is conservative because the length is taken as the center distance instead of the clear distance between connectors, and the coefficient $K$ is taken as 0.6 instead of 0.5, which is theoretical value for a column with fixed end supports.

Section D1.2(c) ensures satisfactory spacing to make a row of connectors act as a continuous line of stiffening for the flat sheet under most conditions (Winter, 1970; Yu, 2000).

### D2 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications also must be satisfied.

### D3 Lateral Bracing

Bracing design requirements were expanded in the 1986 AISI *Specification* to include a general statement regarding bracing for symmetrical beams and columns and specific requirements for the design of roof systems subjected to gravity load. These requirements are retained in this *Specification*.  

Figure C-D1.2-1 Spacing of Connectors in Composite Section
D3.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the required strength [resistance] and stiffness for discrete braces in steel construction. Winter (1960) offered a partial solution and others have extended this knowledge (Haussler, 1964; Haussler and Pahers, 1973; Lutz and Fisher, 1985; Salmon and Johnson, 1990; Yura, 1993; SSRC, 1993). The design engineer is encouraged to seek out the stated references to obtain guidance for design of a brace or brace system.

D3.2 C-Section and Z-Section Beams

C-sections and Z-sections used as beams to support transverse loads applied in the plane of the web may twist and deflect laterally unless adequate lateral supports are provided. Section D3.2 of the Specification includes two subsections. The first subsection (Section D3.2.1) deals with the bracing requirements when one flange of the beam is connected to deck or sheathing material. The second subsection (Section D3.2.2) covers the requirements for spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material.

D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load with Top Flange Connected to Sheathing

In metal roof systems attached to C- or Z-purlins, 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 (Murray and Elhouar, 1985). For the design of lateral bracing, Specification Equations D3.2.1-2 through D3.2.1-7 can be used to determine the restraint forces for single-span and multiple-span systems with braces at various locations. These design equations are written in terms of the cross sectional dimensions of the purlin, number of purlin lines, number of spans, span length for multiple-span systems, and the total load applied to the system. The accuracy of these design equations has been verified by Murray and Elhouar using their experimental results of six prototype and 33 quarter-scale tests.

In the 1986 edition of the AISI ASD Specification and the 1991 edition of the AISI LRFD Specification, the brace force equations included in Section D3.2.1(b) were restricted only to through-fastened roof systems. Results of seven single-span tests and six multiple-span tests of standing seam roof systems have shown that Specification Equations D3.2.1-2 through D3.2.1-7 are also applicable to standing seam roof systems (Rivard and Murray, 1986). In addition, Section D3.2.1(b) of previous editions of the AISI Specification required a diaphragm stiffness of the roof system of at least 2000 lb/in. (350 N/mm) for Z-sections. Because the maximum lateral
displacement of the top flange with respect to the purlin reaction points is limited to not exceed L/360, the minimum requirement for a diaphragm stiffness is not needed. Therefore, this requirement was eliminated in the 1996 edition of the AISI Specification.

In 1999, an explicit requirement was indicated for purlins facing opposite directions to resist the down-slope component of the total gravity load. To have a consistent approach in calculating the restraint force for C- and Z-sections, Specification Equation D3.2.1-1 is added for calculating the anchorage force for C-sections. In addition, the “cosθ” term is added to the first term of Specification Equation D3.2.1-1 for C-sections and Equations D3.2.1-2 through D3.2.1-7 for Z-sections. The original research was done assuming the roof was flat and the applied loading was parallel to the purlin webs. In the equations, Wcosθ is the component of the vertical loading parallel to the purlin webs.

D3.2.2 Neither Flange Connected to Sheathing

(a) Bracing of C-Section Beams

If C-sections are used singly as beams, rather than being paired to form I-sections, they should be braced at intervals so as to prevent them from rotating in the manner indicated in Figure C-D3.2.2-1. Figure C-D3.2.2-2, for simplicity, shows two C-sections braced at intervals against each other. The situation is evidently much the same as in the composite I-section of Figure C-D1.1-2, except that the role of the connectors is now played by the braces. The difference is that the two C-sections are not in contact, and that the spacing of braces is generally considerably larger than the connector spacing. In consequence, each C-section may actually rotate very slightly between braces, and this will cause some additional stresses which superimpose on the usual, simple bending stresses. Bracing should be so arranged that: (1) these additional stresses are small enough not to reduce the load-carrying capacity of the C-section (as compared to what it would be in the continuously braced condition); and (2) rotations should be kept small enough to be unobjectionable of the order of 1 to 2 degrees.

In order to develop information on which to base appropriate bracing provisions, different C-section shapes have been tested at Cornell University (Winter, 1970). Each of these was tested with full, continuous bracing; without any bracing; and with intermediate bracing at two different spacings. In addition to this experimental work, an approximate method of analysis was developed and checked against the test results. A condensed account of this was given by Winter, Lansing and McCalley (1949b). It is indicated in that reference that the above requirements are satisfied for most distributions of beam load if between supports not less than three equidistant braces are placed (i.e., at quarter-points of the span, or closer). The exception is the case where a large part of the total load of the beam is concentrated over a short portion of the span; in this case an additional brace should
be placed at such a load. Correspondingly, previous editions of the AISI Specification (AISI, 1986; AISI, 1991) provided that the distance between braces shall not be greater than one-quarter of the span; it also defined the conditions under which an additional brace should be placed at a load concentration.

![Figure C-D3.2.2-1 Rotation of C-Section Beams](image1)

**Figure C-D3.2.2-1 Rotation of C-Section Beams**

For such braces to be effective it is not only necessary that their spacing be appropriately limited; in addition, their strength [resistance] should suffice to provide the force required to prevent the C-section from rotating. It is, therefore, necessary also to determine the forces which will act in braces, such as those forces shown in Figure C-D3.2.2-3. These forces are found if one considers that the action of a load applied in the plane of the web (which causes a torque \( Q_m \)) is equivalent to that same load when applied at the shear center (where it causes no torque) plus two forces \( P = Q_m / d \) which, together, produce the same torque \( Q_m \). As is sketched in Figure C-D3.2.2-4, and shown in some detail by Winter, Lansing and McCalley (1949b), each half of the channel can then be regarded as a continuous beam loaded by the horizontal forces and supported at the brace points. The horizontal brace force is then, simply, the appropriate reaction of this continuous beam. The provisions of Specification Section D3.2.2 represent a simple
and conservative approximation for determining these reactions, which
are equal to the force $P_L$ which the brace is required to resist at each
flange.

(b) Bracing of Z-Section Beams

Most Z-sections are anti-symmetrical about the vertical and
horizontal centroidal axes, i.e. they are point-symmetrical. In view of
this, the centroid and the shear center coincide and are located at the
midpoint of the web. A load applied in the plane of the web has, then,
no lever arm about the shear center ($m = 0$) and does not tend to
produce the kind of rotation a similar load would produce on a C-
section. However, in Z-sections the principal axes are oblique to the
web (Figure C-D3.2.2-5). A load applied in the plane of the web,
resolved in the direction of the two axes, produces deflections in each
of them. By projecting these deflections onto the horizontal and vertical
planes it is found that a Z-beam loaded vertically in the plane of the
web deflects not only vertically but also horizontally. If such deflection
is permitted to occur then the loads, moving sideways with the beam,
are no longer in the same plane with the reactions at the ends. In
consequence, the loads produce a twisting moment about the line
connecting the reactions. In this manner it is seen that a Z-beam,
unbraced between ends and loaded in the plane of the web, deflects laterally and also twists. Not only are these deformations likely to interfere with a proper functioning of the beam, but the additional stresses caused by them produce failure at a load considerably lower than when the same beam is used fully braced.

In order to obtain information for developing appropriate bracing provisions, tests have been carried out on three different Z-sections at Cornell University, unbraced as well as with variously spaced intermediate braces. In addition, an approximate method of analysis has been developed and checked against the test results. An account of this was given by Zetlin and Winter (1955b). Briefly, it is shown that intermittently braced Z-beams can be analyzed in much the same way as intermittently C-beams. It is merely necessary, at the point of each actual vertical load \( Q \), to apply a fictitious horizontal load \( P = Q(l_{xy}/l_{x}) \) or \( P = Q(l_{xy}/(2l_{x})) \) to each flange. One can then compute the vertical and horizontal deflections, and the corresponding stresses, in conventional ways by utilizing the convenient axes \( x \) and \( y \) (rather than \( 1 \) and \( 2 \), Figure C-D3.2.2-5), except that certain modified section properties have to be used.

In this manner it has been shown that as to location of braces the same provisions which apply to C-sections are also adequate for Z-sections. Likewise, the forces in the braces are again obtained as the reactions of continuous beams horizontally loaded by fictitious loads \( P \). It should, however, be noted that the direction of the bracing forces in Z-beams is different from the direction in C-beams. In the Z-beam, the bracing forces are acting in the same direction, as shown in Fig. C-D3.2.2-5 in order to constrain bending of the section about the axis \( x-x \) in Figure C-D3.2.2-5. The directions of the bracing forces in the C-beam flanges are in the opposite direction as shown in Figure C-D3.2.2-3 in order to

![Figure C-D3.2.2-5 Principal Axis of Z-Section](image-url)
resist the torsion caused by the applied load. In the previous edition of the *Specification*, the magnitude of the Z-beam bracing force was shown as $P = Q(I_{xy}/I_x)$ on each flange. In 2001, this force was corrected to $P = Q[I_{xy}/(2I_x)]$.

(c) Spacing of Braces

During the period from 1956 through 1996, the AISI *Specification* required that braces be attached both to the top and bottom flanges of the beam, 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. The lateral-torsional buckling equations provided in *Specification* Section C3.1.2.1 can be used to predict the moment capacity of the member. Beam tests conducted by Ellifritt, Sputo and Haynes (1992) have shown that for typical sections, a mid-span brace may reduce service load horizontal deflections and rotations by as much as 80 percent when compared to a completely unbraced beam. However, the restraining effect of braces may change the failure mode from lateral-torsional buckling to distortional buckling of the flange and lip at a brace point. The natural tendency of the member under vertical load is to twist and translate in such a manner as to relieve the compression on the lip. When such movement is restrained by intermediate braces, the compression on the stiffening lip is not relieved, and may increase. In this case, local distortional buckling may occur at loads lower than that predicted by the lateral-torsional buckling equations of *Specification* Section C3.1.2.1.

Research (Ellifritt, Sputo and Haynes, 1992) has also shown that the lateral-torsional buckling equations of *Specification* Section C3.1.2.1 predict loads which are conservative for cases where one mid-span brace is used but may be unconservative where more than one intermediate brace is used. Based on such research findings, Section D3.2.2 of the *Specification* was revised in 1996 to eliminate the requirement of quarter-point bracing. It is suggested that, minimally, a mid-span brace be used for C-section and Z-section beams to control lateral deflection and rotation at service loads. The lateral-torsional buckling strength [resistance] of an open cross section member should be determined by *Specification* Section C3.1.2.1 using the distance between center lines of braces “$a$” as the unbraced length of the member “$L$” in all design equations. In any case, the user is permitted to perform tests, in accordance with *Specification* Section F1, as an alternative, or use a rigorous analysis which accounts for biaxial bending and torsion.

Section D3.2.2 of the *Specification* provides the lateral forces for which these discrete braces must be designed.

The *Specification* permits omission of discrete braces when all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member
against torsional rotation and lateral displacement. Frequently, this occurs in the end walls of metal buildings.

**D4 Wall Studs and Wall Stud Assemblies**

It is well known that column strength [resistance] can be increased considerably by using adequate bracing, even though the bracing is relatively flexible. This is particularly true for those sections generally used as load-bearing wall studs which have large \( I_x/I_y \) ratios.

Cold-formed I-, C-, Z-, or box-type studs are generally used in walls with their webs placed perpendicular to the wall surface. The walls may be made of different materials, such as fiberboard, pulp board, plywood, or gypsum board. If the wall material is strong enough and there is adequate attachment provided between wall material and studs for lateral support of the studs, then the wall material can contribute to the structural economy by increasing the usable strength [resistance] of the studs substantially.

In order to determine the necessary requirements for adequate lateral support of the wall studs, theoretical and experimental investigations were conducted in the 1940s by Green, Winter, and Cuykendall (1947). The study included 102 tests on studs and 24 tests on a variety of wall material. Based on the findings of this earlier investigation, specific AISI provisions were developed for the design of wall studs.

In the 1970s, the structural behavior of columns braced by steel diaphragms was a special subject investigated at Cornell University and other institutions. The renewed investigation of wall-braced studs has indicated that the bracing provided for studs by steel panels is of the shear diaphragm type rather than the linear type, which was considered in the 1947 study. Simaan (1973) and Simaan and Pekoz (1976), which are summarized by Yu (2000), contain procedures for computing the strength [resistance] of C- and Z-section wall studs that are braced by sheathing materials. The bracing action is due to both the shear rigidity and the rotational restraint supplied by the sheathing material. The treatment by Simaan (1973) and Simaan and Pekoz (1976) is quite general and includes the case of studs braced on one as well as on both flanges. However, the provisions of Section D4 of the 1980 AISI *Specification* dealt only with the simplest case of identical sheathing material on both sides of the stud. For simplicity, only the restraint due to the shear rigidity of the sheathing material was considered.

The 1989 Addendum to the AISI 1986 *Specification* included the design limitations from the *Commentary* and introduced stub column tests and/or rational analysis for the design of studs with perforations (Davis and Yu, 1972; Rack Manufacturers Institute, 1990).

In 1996, the design provisions were revised to permit (a) all steel design and (b) sheathing braced design of wall studs with either solid or perforated webs. For sheathing braced design, in order to be effective, sheathing must retain its design strength [resistance] and integrity for the expected service life of the wall. Of particular concern is the use of gypsum sheathing in a moist environment. The values given in Table D4 of the *Specification* for gypsum are based on dry service conditions.
D4.1 Compression

The provisions in this Specification Section D4.1 are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners (Figure C-D4.1-1) even if one fastener is missing or otherwise ineffective. Provision (b) contains equations for critical stresses for overall column buckling (Figure C-D4.1-2). Essential to these provisions is the magnitude of the shear rigidity of the sheathing material. A table of values and an equation for determining the shear rigidity is provided in the Specification. These values are based on the small scale tests described by Simaan (1973) and Simaan and Pekoz (1976). For other types of materials, the sheathing parameters can be determined by using the procedures described in these references.

Provision (c) is intended to ensure that the sheathing has sufficient ability to distort without rupture. The procedure involves assuming a value of the ultimate stress and checking whether the shear strain at the load corresponding to the ultimate stress exceeds the permissible 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 iteration is not needed.

In the 1986 Specification, the Q-factor method for treating the local buckling effects was eliminated. The approach recommended was to find the overall buckling stress on the basis of the full unreduced section. The ultimate load was determined by multiplying the buckling stress by the effective area determined at the buckling stress.

In the 1989 Addendum, the effective length factors $K_x$, $K_y$ and $K_t$ were eliminated from Equations D4.1-8, D4.1-10, and D4.1-11, respectively. This is consistent with the 1980 Edition of the AISI Specification. Inclusion of the effective length factors could lead to unconservative designs where both sheathing and strap or C-section bracing are present. The equations are based on tests with only sheathing as bracing.

The approach of determining effective areas in accordance with Specification Section D4(a) is currently being used in the RMI Specification (Rack Manufacturers Institute, 1997) for the design of perforated rack columns and was verified extensively for such structures as reported by Pekoz (1988a). The validity of this approach for wall studs was verified in a Cornell University project on wall studs reported by Miller and Pekoz (1989 and 1994).

The limitations included in Specification Section D4(a) for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program. For sections with perforations which do not meet these limits, the effective area, $A_e$, can be determined by stub column tests.

In the Specification, the web is defined as the component element of the section perpendicular to the wall and the flange is parallel to the plane of the wall.
Figure C-D4.1-1 Buckling of Studs between Fully Effective Fasteners

Figure C-D4.1-2 Overall Column Buckling of Studs
Studs with sheathing on one flange only, or with sheathing on both flanges that is not identical, or having rotational restraint that is not neglected, or having any combination of the above, can be designed in accordance with the same basic analysis principles used in deriving the provisions of this section (Simaan and Pekoz, 1976).

For the ASD method, in the 1996 Specification as well as this edition, a constant factor of safety of 1.80 is used for Specification Sections D4.1(a), D4.1(b) and D4.1(c) in order to be consistent with Specification Section C4 for the design of concentrically loaded compression members.

D4.2 Bending

The design provisions for wall studs in bending were provided in the 1986 AISI Specification. The footnote for unusual cases was moved to Section D4.1 of the Commentary in 1996. It should be noted that the nominal flexural strength [moment resistance] of wall studs is determined by the “all steel design” approach neglecting the structural contribution of the attached sheathing material.

D4.3 Combined Axial Load and Bending

The general interaction equations of Specification Section C5 are also applicable to wall studs with the exception that the nominal flexural strength [moment resistance] be evaluated by excluding lateral-torsional buckling considerations.

D5 Floor, Roof or Wall Steel Diaphragm Construction

In building construction, it has been a common practice to provide a separate bracing system to resist horizontal loads due to wind load, blast force, or earthquake. However, steel floor and roof panels, with or without concrete fill, are capable of resisting horizontal loads in addition to the beam strength [resistance] for gravity loads if they are adequately interconnected to each other and to the supporting frame. The effective use of steel floor and roof decks can therefore eliminate separate bracing systems and result in a reduction of building costs. For the same reason, wall panels can provide not only enclosure surface and support normal loads, but they can also provide diaphragm action in their own planes.

The structural performance of a diaphragm construction can be evaluated by either calculations or tests. Several analytical procedures exist, and are summarized in the literature (Steel Deck Institute, 1988; Department of Army, 1985; and ECCS, 1977). Tested performance is measured by the procedures of the Standard Method for Static Load Testing of Framed Floor, Roof and Wall Diaphragm Construction for Buildings, ASTM E455. Part VIII of the AISI Design Manual (AISI, 2002) contains the Test Procedure with Commentary on Cantilever Test Method for Cold-Formed Diaphragms. A general discussion of structural diaphragm behavior is given by Yu (2000).
The factors of safety and resistance factors required in the *Specification* are based on statistical studies of the nominal and mean resistances from full scale tests (Steel Deck Institute, 1981). The study concluded that the quality of mechanical connectors is easier to control than welded connections. The variation in the strength [resistance] of mechanical connectors is smaller than that for welded connections, and their performance is more predictable. Therefore, a smaller factor of safety, or larger resistance factor, is justified for mechanical connections.

The factors of safety for earthquake loading are slightly larger than those for wind due to the ductility demands required by seismic loading. Factors of safety for load combinations not involving wind or seismic load should be greater than those involving wind and seismic loads, thus the *Specification* provides for appropriate factors of safety. Resistance factors have been determined accordingly.
Chapter E, Connections and Joints

E. CONNECTIONS AND JOINTS

E1 General Provisions

Welds, bolts, screws, rivets, and other special devices such as metal stitching and adhesives are generally used for cold-formed steel connections (Brockenbrough, 1995). The 2001 edition of the Specification contains provisions in Chapter E for welded connections, bolted connections, and screw connections. Among the above three commonly used types of connections, the design provisions for using screws were developed in 1993 and were included in the 1996 AISI Specification for the first time. The following brief discussions deal with the applications of rivets and other special devices:

(a) Rivets

While hot rivets have little application in cold-formed steel construction, cold rivets find considerable use, particularly in special forms, such as blind rivets (for application from one side only), tubular rivets (to increase bearing area), high shear rivets, and explosive rivets. For the design of connections using cold rivets, the provisions for bolted connections may be used as a general guide, except that the shear strength [resistance] of rivets may be quite different from that of bolts. Additional design information on the strength [resistance] of rivets should be obtained from manufacturers or from tests.

(b) Special devices

Special devices include: (1) metal stitching, achieved by tools that are special developments of the common office stapler, and (2) connecting by means of special clinching tools that draw the sheets into interlocking projections.

Most of these connections are proprietary devices for which information on strength [resistance] of connections must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in Specification Chapter F are to be used in these tests.

The plans and/or specifications are to contain adequate information and design requirement data for the adequate detailing of each connection if the connection is not detailed on the engineering design drawings.

In this edition of the Specification, the ASD, LRFD and LSD design provisions for welded and bolted connections were based on the 1996 edition of the AISI Specification with some revisions and additions which will be discussed in subsequent sections. The development of the factors of safety and the resistance factors for each country is discussed in Section E3a of the Commentary on Appendix A, B or C.

As far as the LRFD and the LSD methods are concerned, the resistance factors were derived for a target reliability index, $\beta_0 = 3.5$ for USA and Mexico and 4 for Canada, for the connections subjected to gravity loading. For the tensile strength [resistance] of connectors used to join roof decks and wall panels to purlins and girts, two cases were considered in the determination of $\phi$ factors,
i.e., (1) $1.2D + 1.6L$ with $\beta_o = 3.5$ for USA and Mexico and $4$ for Canada, and (2) $1.17W - 0.9D$ with $\beta_o = 2.5$ for USA and Mexico and $3.0$ for Canada. Case (2) represents the counteracting loads acting according to a load combination of dead load and wind uplift with a reduction factor $0.9$ applied to the load factor for the nominal wind load (AISI, 1996). Screws loaded by wind uplift can also be designed for a target reliability index $\beta_o = 2.5$ for USA and Mexico and $? \text{ for Canada.}$ Other statistical data for developing the LRFD criteria for connections were documented by Hsiao, Yu and Galambos (1988a) and summarized in the AISI LRFD Design Manual (AISI, 1991)

**E2 Welded Connections**

Welds used for cold-formed steel construction may be classified as fusion welds (or arc welds) and resistance welds. Fusion welding is used for connecting cold-formed steel members to each other as well as connecting such members to heavy, hot-rolled steel framing (such as floor panels to beams of the steel frame). It is used in groove welds, arc spot welds, arc seam welds, fillet welds, and flare groove welds.

The design provisions contained in this Specification section for fusion welds have been based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. The results of this program are reported by Pekoz and McGuire (1979) and summarized by Yu (2000). All possible failure modes are covered in the provisions of the AISI 1996 Specification and also this Specification, whereas the earlier provisions mainly dealt with shear failure.

For most of the connection tests reported by Pekoz and McGuire (1979), the onset of yielding was either poorly defined or followed closely by failure. Therefore, in the provisions of this section, rupture rather than yielding is used as a more reliable criterion of failure.

The welded connection tests, which served as the basis of the provisions given in Specification Sections E2.1 through E2.5, were conducted on sections with single and double sheets. See Specification Figures E2.2-1 and E2.2-2. The largest total sheet thickness of the cover plates was approximately 0.15 inch (3.81 mm). However, within this Specification, the validity of the equations was extended to welded connections in which the thickness of the thinnest connected part is 0.18 inch (4.57 mm) or less. For arc spot welds, the maximum thickness of a single sheet (Specification Figure E2.2.1-1) and the combined thickness of double sheets (Specification Figure E2.2.1-2) are set at 0.15 inch (3.81 mm).

In 2001, the factors of safety and resistance factors in this section were modified for consistency based on the research work by Tangorra, Schuster, and LaBoube (2001).

For design tables and example problems on welded connections, see Part IV of the Design Manual (AISI, 2002).


**E2.1 Groove Welds in Butt Joints**

The design equations for determining nominal strength [resistance] for groove welds in butt joints have been taken from the AISC LRFD Specification (AISC, 1993). Therefore, the AISC definition for the effective throat thickness, \( t_e \), is equally applicable to this section of the Specification. Prequalified joint details are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

**E2.2 Arc Spot Welds**

Arc spot welds (puddle welds) used for connecting thin sheets are similar to plug welds used for relatively thicker plates. The difference between plug welds and arc spot welds is that the former are made with prepunched holes, but for the latter no prepunched holes are required. Instead, a hole is burned in the top sheet by the arc and then filled with weld metal to fuse it to the bottom sheet or a framing member. The provisions of Section E2.2 apply to plug welds as well as spot welds.

**E2.2.1 Shear**

The Cornell tests (Pekoz and McGuire, 1979) identified four modes of failure for arc spot welds, which are addressed in this Specification section. They are: (1) shear failure of welds in the fused area, (2) tearing of the sheet along the contour of the weld with the tearing spreading the sheet at the leading edge of the weld, (3) sheet tearing combined with buckling near the trailing edge of the weld, and (4) shearing of the sheet behind weld. It should be noted that many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Figure C-E2.2-1. This form of behavior is similar to that observed in wide, pin-connected plates. Such behavior should be avoided by closer spacing of welds. When arc spot welds are used to connect two sheets to a framing member as shown in Specification Figure E2.2.1-2, consideration should also be given to the possible shear failure between thin sheets.

![Figure C-E2.2-1 Out of Plane Distortion of Welded Connection](image-url)
The thickness limitation of 0.15 inch (3.81 mm) is due to the range of the test program that served as the basis of these provisions. On sheets below 0.028 inch (0.711 mm) thick, weld washers are required to avoid excessive burning of the sheets and, therefore, inferior quality welds.

As compared with previous editions of the AISI Specification, several minor revisions were made in the 1996 Specification concerning the determination of the nominal shear strength [resistance] of welds (Specification Equation E2.2.1-1) and the limiting $F_u/F_{sy}$ ratios for using Specification Equations E2.2.1-6a and E2.2.1-6b. Specification Equation E2.2.1-1 was revised to be consistent with the research report (Pekoz and McGuire, 1979), and the limiting $F_u/F_{sy}$ ratios were changed to be consistent with Specification Section A2.3.1.

In 2001, the equation used for determining $d_a$ for multiple sheets was revised to be $(d-t)$.

### E2.2.2 Tension

For tensile capacity of arc spot welds, the design provisions in the 1989 Addendum were based on the tests reported by Fung (1978) and the study made by Albrecht (1988). Those provisions were limited to sheet failure with restrictive limitations on material properties and sheet thickness. These design criteria were revised in 1996 because the recent tests conducted at the University of Missouri-Rolla (LaBoube and Yu, 1991 and 1993) have shown that two potential limit states may occur. The most common failure mode is that of sheet tearing around the perimeter of the weld. This failure condition was found to be influenced by the sheet thickness, the average weld diameter, and the material tensile strength. In some cases, it was found that tensile failure of the weld can occur. The strength [resistance] of the weld was determined to be a function of the cross-section of the fused area and tensile strength of the weld material. Based on analysis by LaBoube (LaBoube, 2001), the nominal strength [resistance] equation was changed in 2001 to reflect the ductility of the sheet, $F_u/F_{y}$, and the sheet thickness, the average weld diameter, and the material tensile strength.

The multiple factors of safety and resistance factors recognize the behavior of a panel system with many connections versus the behavior of a

![Figure C-E2.2-2 Interior Weld, Exterior Weld and Lap Connection](image-url)
member connection and the potential for a catastrophic failure in each application. In *Specification* Section E2.2.2 a target reliability index of 3.0 for the United States and Mexico and 3.5 for Canada is used for the panel connection limit, whereas a target reliability index of 3.5 for the United States and Mexico and 4 for Canada is used for the other connection limit. Precedence for the use of a smaller target reliability index for systems was established in Section C3.1.5 of the *Specification*.

Tests (LaBoube and Yu, 1991 and 1993) have also shown that when reinforced by a weld washer, thin sheet weld connections can achieve the design strength [resistance] given by *Specification* Equation E2.2.2-2 using the thickness of the thinner sheet.

The equations given in the *Specification* were derived from the tests for which the applied tension load imposed a concentric load on the weld, as would be the case, for example, for the interior welds on a roof system subjected to wind uplift. Welds on the perimeter of a roof or floor system would experience an eccentric tensile loading due to wind uplift. Tests have shown that as much as a 50 percent reduction in nominal connection strength [resistance] could occur because of the eccentric load application (LaBoube and Yu, 1991 and 1993). Eccentric conditions may also occur at connection laps depicted by Figure C-E2.2-2.

At a lap connection between two deck sections as shown in Figure C-E2.2-2, the length of the unstiffened flange and the extent of the encroachment of the weld into the unstiffened flange have a measurable influence on the strength [resistance] of the welded connection (LaBoube and Yu, 1991). The *Specification* recognizes the reduced capacity of this connection detail by imposing a 30 percent reduction on the calculated nominal strength [resistance].

### E2.3 Arc Seam Welds

The general behavior of arc seam welds is similar to that of arc spot welds. No simple shear failures of arc seam welds were observed in the Cornell tests (Pekoz and McGuire, 1979). Therefore, *Specification* Equation E2.3-1, which accounts for shear failure of welds, is adopted from the AWS welding provisions for sheet steel (AWS, 1998).

*Specification* Equation E2.3-2 is intended to prevent failure by a combination of tensile tearing plus shearing of the cover plates.

### E2.4 Fillet Welds

For fillet welds on the lap joint specimens tested in the Cornell research (Pekoz and McGuire, 1979), the dimension, $w_1$, of the leg on the sheet edge generally was equal to the sheet thickness; the other leg, $w_2$, often was two or three times longer than $w_1$ (See *Specification* Figure E2.4-1). In connections of this type, the fillet weld throat commonly is larger than the throat of a conventional fillet welds of the same size. Usually ultimate failure of fillet
welded joints has been found to occur by the tearing of the plate adjacent to
the weld, See Figure C-E2.4-1.

![Diagram of fillet weld failure modes](image)

*Figure C-E2.4-1 Fillet Weld Failure Modes*

In most cases, the higher strength of the weld material prevents weld shear failure, therefore, the provisions of this *Specification* section are based on sheet tearing. Because specimens up to 0.15 inch (3.81 mm) thickness were tested in the Cornell research (Pekoz and McGuire, 1979), the last provision in this section is to cover the possibility that for sections thicker than 0.15 inch (3.81 mm), the throat dimension may be less than the thickness of the cover plate and the tear may occur in the weld rather than in the plate material. Recent research at the University of Sydney (Zhao and Hancock, 1995) has further indicated that weld throat failure may even occur between the thickness of 0.10 in. (2.54 mm) to 0.15 in. (3.81 mm). Accordingly, the *Specification* was revised, in 2001, to require weld strength [resistance] check when the plate thickness is greater than 0.10 in. (2.54 mm). For high strength materials with yield stress of 65 ksi (448 MPa) or higher, recent research at the University of Sydney (Teh and Hancock, 2000) has shown that weld throat failure does not occur in materials less than 0.10 in. (2.54 mm) thick and that the AISI *Specification* provisions based on sheet strength are satisfactory for high strength material less than 0.10 in. (2.54 mm) thick. Prequalified fillet welds are given in AWS D1.3-98 (AWS, 1998) or other equivalent weld standards.

**E2.5 Flare Groove Welds**

The primary mode of failure in cold-formed steel sections welded by flare groove welds, loaded transversely or longitudinally, also was found to be sheet tearing along the contour of the weld. See Figure C-E2.5-1.

Except for *Specification* Equation E2.5-4, the provisions of this *Specification* section are intended to prevent shear tear failure. *Specification* Equation E2.5-4 covers the possibility that thicker sections may have effective throats less than the thickness of the channel and weld failure may become critical.
In the 1996 edition of the AISI Specification, the former Specification Figure E2.5-4 was replaced by four new drawings to describe in more detail the different possible flare bevel groove weld uses. Specification Figures E2.5-4 and E2.5-5 show the condition where the weld is filled flush to the surface. This weld is a prequalified weld in AWS D1.3-98 (AWS, 1998) which provides the definition of the effective throat for this type of weld. The distinction of double and single shear requirements in the Specification for flare groove welds is indicated on these figures. Specification Figures E2.5-6 and E2.5-7 show flare bevel groove welds which are frequently used in cold-formed steel construction in which the weld is not filled flush to the surface. The vertical leg of the weld can either be greater, Figure E2.5-6, or less, Figure E2.5-7, than the radius of outside bend surface. The definition of the horizontal leg of the weld in each case is slightly different as indicated. No change was needed in the Specification requirements from previous editions except in the definitions of the effective throat for use in Specification Equation E2.5-4.

In 2001, the Specification was revised to require that weld strength be checked when the plate thickness is greater than 0.10 in. (2.54 mm) based on the research by Zhao and Hancock (1995).

### E2.6 Resistance Welds

The shear values for outside sheets of 0.125 inch (3.18 mm) or less in thickness are based on “Recommended Practice for Resistance Welding Coated Low-Carbon Steels,” AWS C1.3-70, (Table 2.1 - Spot Welding Galvanized Low-Carbon Steel). Shear values for outside sheets thicker than 0.125 inch (3.18 mm) are based upon “Recommended Practices for Resistance Welding,” AWS C1.1-66, (Table 1.3 - Pulsation Welding Low-Carbon Steel) and apply to pulsation welding as well as spot welding. They are applicable for all structural grades of low-carbon steel, uncoated or galvanized with 0.90 oz/ft² (275 g/m²) of sheet, or less, and are based on values selected from AWS C1.3-70, Table 2.1; and AWS C1.1-66, Table 1.3. The above values may also be applied to medium carbon and low-alloy steels. Spot welds in such steels give somewhat higher shear strengths than those upon which the above values are based; however, they may require special welding conditions. In all
cases, welding shall be performed in accordance with AWS C1.3-70 and AWS C1.1-66 (AWS, 1966 and 1970).

In this edition of the *Specification*, a design equation is used to determine the nominal shear strength [resistance] which replaces the tabulated values given in the previous specifications. The upper limit of *Specification* Equations E2.6-1, E2.6-3 and E2.6-5 is selected to best fit the data provided in AWS C1.3-70, Table 2.1 and AWS C1.1-66, Table 1.3. Shear strength [resistance] values for welds with the thickness of the thinnest outside sheet greater than 0.180 in. (4.57 mm) have been excluded in *Specification* Equations E2.6-2, E2.6-4 and E2.6-6 due to the thickness limit set forth in *Specification* Section E2.

**E2.7 Fracture in Net Section of Members other than Flat Sheets (Shear Lag)**

Shear lag has a debilitating effect on the nominal tensile strength of a cross section. The AISI *Specification* addresses the shear lag effect on tension members other than flat sheets in welded connections. The AISC Specification’s design approach has been adopted.

When computing \( U \) for combinations of longitudinal and transverse welds, \( L \) is taken as the length of the longitudinal weld because the transverse weld does little to minimize shear lag. For angle or channel sections, the distance, \( \bar{x} \), from shear plane to centroid of the cross section is defined in Figure C-E2.7.

![Figure C-E2.7](image)

**Figure C-E2.7 \( \bar{x} \) Definition for Sections with Fillet Welding**

**E3 Bolted Connections**

The structural behavior of bolted connections in cold-formed steel construction is somewhat different from that in hot-rolled heavy construction, mainly because of the thinness of the connected parts. Prior to 1980, the provisions included in the AISI *Specification* for the design of bolted connections were developed on the basis of the Cornell tests (Winter, 1956a, 1956b). These provisions were updated in 1980 to reflect the results of additional research performed in the United States (Yu, 1982) and to provide a better coordination with the specifications of the Research Council on Structural Connections (RCSC, 1980) and the AISC (1978). In 1986, design provisions for maximum size of bolt holes and the allowable tension stress for bolts were added in the AISI
Specification (AISI, 1986). In the 1996 edition of the AISI Specification, minor changes of the factors of safety were made for computing the nominal tensile and shear strengths [resistances] of bolts. The allowable tension stress for the bolts subject to the combination of shear and tension is determined by the equations provided in Specification Table E3.4-2 with the applicable factor of safety.

(a) Scope

Previous studies and practical experiences have indicated that the structural behavior of bolted connections used for joining relatively thick cold-formed steel members is similar to that for connecting hot-rolled shapes and built-up members. The AISI Specification criteria are applicable only to cold-formed steel members or elements less than 3/16 inch (4.76 mm) in thickness. For materials not less than 3/16 inch (4.76 mm), reference is made to the specifications or standards stipulated in Section E3a of Appendix A, B or C.

Because of lack of appropriate test data and the use of numerous surface conditions, this Specification does not provide design criteria for slip-critical (also called friction-type) connections. When such connections are used with cold-formed members where the thickness of the thinnest connected part is less than 3/16 inch (4.76 mm), it is recommended that tests be conducted to confirm their design capacity. The test data should verify that the specified design capacity for the connection provides a sufficient safety against initial slip at least equal to that implied by the provisions of the specifications or standards listed in Section E3a of the Appendix A, B or C. In addition, the safety against ultimate capacity should be at least equal to that implied by this Specification for bearing-type connections.

The Specification provisions apply only when there are no gaps between plies. The designer should recognize that the connection of a rectangular tubular member by means of bolt(s) through such members may have less strength [resistance] than if no gap existed. Structural performance of connections containing unavoidable gaps between plies would require tests in accordance with Specification Section F1.

(b) Materials

This section lists five different types of fasteners which are normally used for cold-formed steel construction. In view of the fact that A325 and A490 bolts are available only for diameters of 1/2 inch (12.7 mm) and larger, A449 and A354 Grade BD bolts should be used as an equivalent of A325 and A490 bolts, respectively, whenever smaller bolts (less than 1/2 inch (12.7 mm) in diameter) are required.

During recent years, other types of fasteners, with or without special washers, have been widely used in steel structures using cold-formed steel members. The design of these fasteners should be determined by tests in accordance with Chapter F of this Specification.

(c) Bolt Installation

Bolted connections in cold-formed steel structures use either mild or high-strength steel bolts and are designed as a bearing-type connection. Bolt pretensioning is not required because the ultimate strength of a bolted connection is independent of the level of bolt preload. Installation must
ensure that the bolted assembly will not come apart during service. Experience has shown that bolts installed to a snug tight condition do not loosen or “back-off” under normal building conditions and not subject to vibration or fatigue.

Bolts in slip-critical connections, however, must be tightened in a manner which assures the development of the fastener tension forces required by the Research Council on Structural Connections (1985 and 2000) for the particular size and type of bolts. Turn-of-nut rotations specified by the Research Council on Structural Connections may not be applicable because such rotations are based on larger grip lengths than are encountered in usual cold-formed construction. Reduced turn-of-the-nut values would have to be established for the actual combination of grip and bolt. A similar test program (RCSC, 1985 and 1988) could establish a cut-off value for calibrated wrenches. Direct tension indicators (ASTM F959), whose published clamping forces are independent of grip, can be used for tightening slip-critical connections.

(d) Hole Sizes

Design information for oversized and slotted holes is included in the Appendices because such holes are often used in practice to meet dimensional tolerances during erection.

E3.1 Shear, Spacing and Edge Distance

The design provisions of this section are given in Section E3.1 of the Appendices. The discussion for this section is provided in the Commentary on the corresponding Appendix.

E3.2 Fracture in Net Section (Shear Lag)

The design provisions of this section are given in Section E3.2 of the Appendices. The discussion for this section is provided in the Commentary on the corresponding Appendix.

E3.3 Bearing

Previous bolted connection tests have shown that the bearing strength [resistance] of bolted connections depends on (1) the tensile strength $F_u$ of the connected parts, (2) the thickness of connected parts, (3) the diameter of bolt, (4) joints with single shear and double shear conditions, (5) the $F_u/F_y$ ratio, and (6) the use of washers (Winter, 1956a and 1956b; Chong and Matlock, 1974; Yu, 1982 and 2000). These design parameters were used in the 1996 and earlier editions of the AISI Specification for determining the bearing strength [resistance] between bolt and connected parts (AISI, 1996).

In the Canadian Standard (CSA, 1994), the $d/t$ ratio was also used in the design equation for determining the bearing strength [resistance] of bolted connections.

In this edition of the Specification, the design format and tables for determining the bearing strength [resistance] without consideration of bolt
hole deformation were revised in 2001 on the basis of the recent research work conducted at the University of Sydney (Rogers and Hancock, 1998) and at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a and 2001b).

E3.3.1 Strength [Resistance] Without Consideration of Bolt Hole Deformation

Rogers and Hancock (Rogers and Hancock, 1998) developed the design equation for bearing of bolted connections with washers (Specification Table E3.3.1-1). Based on research at the University of Waterloo (Wallace, Schuster, and LaBoube, 2001a), the Rogers and Hancock equation was extended to bolted connections without washers and to the inside sheet of double shear connections with or without washers (Specification Table E3.3.1-2). In Specification Table E3.3.1-1, the bearing factor C depends on the ratio of bolt diameter to member thickness, d/t. The design equations in the Specification Section E3.3.1 are based on available test data. Thus, for sheets thinner than 0.024 in. (0.61 mm), tests must be performed to determine the structural performance.

The factor of safety and resistance factor are based on calibration of available test data (Wallace, Schuster, and LaBoube, 2001b).

E3.3.2 Strength [Resistance] With Consideration of Bolt Hole Deformation

Based on research at the University of Missouri-Rolla (LaBoube and Yu, 1995), design equations have been developed that recognize the presence of hole elongation prior to reaching the limited bearing strength [resistance] of a bolted connection. The researchers adopted an elongation of 0.25 in. (6.4 mm) as the acceptable deformation limit. This limit is consistent with the permitted elongation prescribed for hot-rolled steel.

E3.4 Shear and Tension in Bolts

The design provisions of this section are given in Section E3.4 of the Appendices. The discussion for this section is provided in the Commentary on the corresponding Appendices.

E4 Screw Connections

Results of over 3500 tests worldwide were analyzed to formulate screw connection provisions (Pekoz, 1990). European Recommendations (1987) and British Standards (1992) were considered and modified as appropriate. Since the provisions apply to many different screw connections and fastener details, a greater degree of conservatism is implied than is otherwise typical within this Specification. These provisions are intended for use when a sufficient number of test results is not available for the particular application. A higher degree of accuracy can be obtained by testing any particular application (AISI, 1992).
Screw connection tests used to formulate the provisions included single fastener specimens as well as multiple fastener specimens. However, it is recommended that at least two screws should be used to connect individual elements. This provides redundancy against under-torquing, over-torquing, etc., and limits lap shear connection distortion of flat unformed members such as straps.

Proper installation of screws is important to achieve satisfactory performance. Power tools with adjustable torque controls and driving depth limitations are usually used.

For the convenience of designers, Table C-E4-1 gives the correlation between the common number designation and the nominal diameter for screws. See Figure C-E4-1 for the measurement of nominal diameters.

### E4.1 Minimum Spacing

Minimum Spacing is the same as specified for bolts.

### E4.2 Minimum Edge and End Distances

In 2001, the minimum edge distance was decreased from 3d to 1.5d with a provision added for nominal shear strength based on end distance.

---

**Table C-E4-1 Nominal Diameter for Screws**

<table>
<thead>
<tr>
<th>Number Designation</th>
<th>Nominal Diameter, d in.</th>
<th>Nominal Diameter, d mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.060</td>
<td>1.52</td>
</tr>
<tr>
<td>1</td>
<td>0.073</td>
<td>1.85</td>
</tr>
<tr>
<td>2</td>
<td>0.086</td>
<td>2.18</td>
</tr>
<tr>
<td>3</td>
<td>0.099</td>
<td>2.51</td>
</tr>
<tr>
<td>4</td>
<td>0.112</td>
<td>2.84</td>
</tr>
<tr>
<td>5</td>
<td>0.125</td>
<td>3.18</td>
</tr>
<tr>
<td>6</td>
<td>0.138</td>
<td>3.51</td>
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<tr>
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<td>0.151</td>
<td>3.84</td>
</tr>
<tr>
<td>8</td>
<td>0.164</td>
<td>4.17</td>
</tr>
<tr>
<td>10</td>
<td>0.190</td>
<td>4.83</td>
</tr>
<tr>
<td>12</td>
<td>0.216</td>
<td>5.49</td>
</tr>
<tr>
<td>1/4</td>
<td>0.250</td>
<td>6.35</td>
</tr>
</tbody>
</table>

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**Figure C-E4-1 Nominal Diameter for Screws**
E4.3 Shear

E4.3.1 Connection Shear Limited by Tilting and Bearing

Screw connections loaded in shear can fail in one mode or in combination of several modes. These modes are screw shear, edge tearing, tilting and subsequent pull-out of the screw, and bearing of the joined materials.

Tilting of the screw followed by threads tearing out of the lower sheet reduces the connection shear capacity from that of the typical connection bearing strength (Figure C-E4.3-1).

These provisions are focused on the tilting and bearing failure modes. Two cases are given depending on the ratio of thicknesses of the connected members. Normally, the head of the screw will be in contact with the thinner material as shown in Figure C-E4.3-2. However, when both members are the same thickness, or when the thicker member is in contact with the screw head, tilting must also be considered as shown in Figure C-E4.3-3.

It is necessary to determine the lower bearing capacity of the two members based on the product of their respective thicknesses and tensile strengths.
E4.3.2 Connection Shear Limited by End Distance

The provisions of this section are given in Section E4.3.2. of the Appendices. The discussion of this section is provided in the Commentary on the corresponding Appendix.

E4.3.3 Shear in Screws

Shear strength [resistance] of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order to avoid the brittle and sudden shear fracture of the screw, the Specification limits the shear strength [resistance] to 0.80 times the shear strength of the screw as reported by the manufacturer or determined by independent laboratory testing.

E4.4 Tension

Screw connections loaded in tension can fail either by pulling out of the screw from the plate (pull-out) or pulling of material over the screw head and the washer, if a washer is present, (pull-over) or by tensile fracture of the screw. The serviceability concerns of gross distortion are not covered by the equations given in Specification Section E4.4.

Diameter and rigidity of the fastener head assembly as well as sheet thickness and tensile strength have a significant effect on the pull-over failure load of a connection.

There are a variety of washers and head styles in use. Washers must be at least 0.050 inch (1.27 mm) thick to withstand bending forces with little or no deformation.

E4.4.1 Pull-Out

For the limit state of pull-out, Specification Equation E4.4.1-1 was derived on the basis of the modified European Recommendations and the results of a large number of tests. The statistic data on pull-out design considerations were presented by Pekoz (1990).

E4.4.2 Pull-Over

For the limit state of pull-over, Specification Equation E4.4.2-1 was derived on the basis of the modified British Standard and the results of a series of tests as reported by Pekoz (1990).

E4.4.3 Tension in Screws

Tensile strength [resistance] of the screw fastener itself should be known and documented from testing. Screw strength [resistance] should be established and published by the manufacturer. In order to prevent the brittle and sudden tensile fracture of the screw, the Specification limits the
tensile strength of the screw to 0.80 times the tensile strength of the screw as reported by the manufacturer or determined by independent laboratory testing.

**E5 Rupture**

The design provisions of this section are given in Section E5 of the Appendices. The discussion of this section is provided in the *Commentary* on the corresponding Appendix.

**E6 Connections to other Materials**

**E6.1 Bearing**

The design provisions for the nominal bearing strength [resistance] on the other materials should be derived from appropriate material specifications.

**E6.2 Tension**

This Section is included in the *Specification* to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

**E6.3 Shear**

This Section is included in the *Specification* to raise the awareness of the design engineer regarding the transfer of shear forces from steel components to adjacent components of other materials.
F. TESTS FOR SPECIAL CASES

All tests for (1) the determination and confirmation of structural performance and (2) the determination of mechanical properties must be made by an independent testing laboratory or by a manufacturer’s testing laboratory. Information on tests for cold-formed steel diaphragms can be found in *Design of Light Gage Steel Diaphragms* (AISI, 1967). A general discussion of structural diaphragms is given in *Cold-Formed Steel Design* (Yu, 2000).

F1 Tests for Determining Structural Performance

This *Specification* section contains provisions for proof of structural adequacy by load tests. This section is restricted to those cases permitted under Section A1.1 of the *Specification* or specifically permitted elsewhere in the *Specification*.

F1.1 Load and Resistance Factor Design and Limit States Design

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same procedures used to calibrate the LRFD design criteria, for which the $\phi$ factor can be computed from Equation C-A5.1.1-15. The correction factor $C_P$ is used in *Specification* Equation F1.1-2 for determining the $\phi$ factor to account for the influence due to a small number of tests (Pekoz and Hall, 1988b and Tsai, 1992). It should be noted that when the number of tests is large enough, the effect of the correction factor is negligible. In the 1996 edition of the AISI *Specification*, Equation F1.1-3 was revised because the old formula for $C_P$ could be unconservative for combinations of a high $V_P$ and a small sample size (Tsai, 1992). This revision enables the reduction of the minimum number of tests from four to three identical specimens. Consequently, the $\pm 10\%$ deviation limit was relaxed to $\pm 15\%$. The use of $C_P$ with a minimum $V_P$ reduces the need for this restriction. In *Specification* Equation F1.1-3, a numerical value of $C_P = 5.7$ was found for $n = 3$ by comparison with a two-parameter method developed by Tsai (1992). It is based on the given value of $V_Q$ and other statistics listed in *Specification* Table F1, assuming that $V_P$ will be no larger than about 0.20. The requirements of *Specification* Section F1.1(a) for $n = 3$ help to ensure this.

The $6.5\%$ minimum value of $V_P$, when used in *Specification* Equation F1.1-2 for the case of three tests, produces factors of safety similar to those of the 1986 edition of the AISI ASD *Specification*, i.e. approximately 2.0 for members and 2.5 for connections. The LRFD calibration reported by Hsiao, Yu and Galambos (1988a) indicates that $V_P$ is almost always greater than 0.065 for common cold-formed steel components, and can sometimes reach values of 0.20 or more. The minimum value for $V_P$ helps to prevent potential unconservatism compared to values of $V_P$ implied in LRFD design criteria.
In evaluating the coefficient of variation $V_P$ from test data, care must be taken to use the coefficient of variation for a sample. This can be calculated as follows:

$$V_P = \sqrt{\frac{s^2}{R_m}}$$

where

$$s^2 = \text{sample variance of all test results}$$

$$= \frac{1}{n-1} \sum_{i=1}^{n} (R_i - R_m)^2$$

$$R_m = \text{mean of all test results}$$

$$R_i = \text{test result i of n total results}$$

Alternatively, $V_P$ can be calculated as the sample standard deviation of $n$ ratios $R_i/R_m$.

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 the resistance factor are listed in Specification Table F1. The data listed for screw connections were added in 1996 on the basis of the study of bolted connections reported by Rang, Galambos, and Yu (1979b). The same statistical data of $M_{m}, V_{M}, F_{m},$ and $V_F$ have been used by Pekoz in the development of the design criteria for screw connections (Pekoz, 1990).

In 1999, two entries were added to Table F1, one for "Structural Members Not Listed Above" and the other for "Connections Not Listed Above". It was considered necessary to include these values for members and connections not covered by one of the existing classifications. The statistical values were taken as the most conservative values in the existing table.

**F1.2 Allowable Strength Design**

The equation for the factor of safety $\Omega$ (Specification Equation F1.2-2) converts the resistance factor $\phi$ from LRFD test procedures in Specification Section F1.1 to an equivalent factor of safety for the allowable strength design. The average of the test results, $R_{n}$, is then divided by the factor of safety to determine an allowable design strength [resistance]. It should be noted that Specification Equation F1.2-2 is identical with Equation C-A5.1.1-16 for $D/L=0$.

**F2 Tests for Confirming Structural Performance**

Members, connections and assemblies which can be designed according to the provisions of Chapters A through E of the Specification need no confirmation.
of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design is in accordance with the Specification, all that is needed is that the tested specimen or assembly demonstrates a strength [resistance] not less than the applicable nominal resistance, \( R_n \).

**F3 Tests for Determining Mechanical Properties**

**F3.1 Full Section**

Explicit methods for utilizing the effects of cold work are incorporated in Section A7.2 of the Specification. In that section, it is specified that as-formed mechanical properties, in particular the yield strength, can be determined either by full-section tests or by calculating the strength of the corners and computing the weighted average for the strength of corners and flats. The strength of flats can be taken as the virgin strength of the steel before forming, or can be determined by special tension tests on specimens cut from flat portions of the formed section. This Specification section spells out in considerable detail the types and methods of these tests, and their number as required for use in connection with Specification Section A7.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see AISI Specification (1968), Chajes, Britvec and Winter (1963), and Karren (1967). A Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns is included in Part VIII of the AISI Design Manual (AISI, 2002).

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

Specification Section F3.2 provides the basic requirements for determining the mechanical properties of flat elements of formed sections. These tested properties are to be used in Specification Section A7.2 for calculating the average yield point of the formed section by considering the strength increase from cold work of forming.

**F3.3 Virgin Steel**

For steels other than the ASTM Specifications listed in Specification Section A2.1, the tensile properties of the virgin steel used for calculating the increased yield point of the formed section should also be determined in accordance with the Standard Methods of ASTM A370 (1997).
G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

Fatigue in a cold-formed steel member or connection is the process of initiation and subsequent growth of a crack under the action of a cyclic or repetitive load. The fatigue process commonly occurs at a stress level less than the static failure condition.

When fatigue is a design consideration, its severity is determined primarily by three factors: (1) the number of cycles of loading, (2) the type of member and connection detail, and (3) the stress range at the detail under consideration (Fisher et al. 1998).

Fluctuation in stress, which does not involve tensile stress, does not cause crack propagation and is not considered to be a fatigue situation.

When fabrication details involving more than one category occur at the same location in a member, the design stress range at the location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

For axially stressed angle members the Specification allows the effects of eccentricity on the weld group to be ignored provided the weld lengths $L_1$ and $L_2$ are proportional such that the centroid of the weld group falls between “$\bar{x}$” and “$b/2$” in Figure C-G1(a). When the weld lengths $L_1$ and $L_2$ are so proportioned, the effects of eccentric loads causing moment about $x-x$ in Figure C-G1(b) also need not be considered.

developed fatigue information on the behavior of sheet and plate steel weldments and mechanical connections. Although research indicates that the values of $F_y$ and $F_u$ do not influence fatigue behavior, the *Specification* provisions are based on tests using ASTM A715 (Grade 80), ASTM A607 Grade 60, and SAE 1008 ($F_y = 30$ ksi). Using regression analysis, mean fatigue life curves (S-N curves) with the corresponding standard deviation were developed. The fatigue resistance S-N curve has been expressed as an exponential relationship between stress range and life cycle (Fisher et al, 1970). The general relationship is often plotted as a linear log-log function, Eq. C-G1.

$$\log N = C_f - m \log F_{SR} \quad \text{(C-G1)}$$

$$C_f = b - (n s) \quad \text{(C-G2)}$$

where

- $N$ = Number of full stress cycles
- $m$ = Slope of the mean fatigue analysis curve
- $F_{SR}$ = Effective stress range
- $B$ = Intercept of the mean fatigue analysis curve from Table C-G1
- $n$ = Number of standard deviations to obtain a desired confidence level
  - $= 2$ for $C_f$ given in the Table G1 of the *Specification*
- $s$ = Approximate standard deviation of the fatigue data
  - $= 0.25$ (Klippstein, 1988)

The database for these design provisions are based upon cyclic testing of real joints; therefore, stress concentrations have been accounted for by the categories in Table G1 of the *Specification*. It is not intended that the allowable stress ranges should be compared to “hot-spot” stresses determined by finite element analysis. Also, calculated stresses computed by ordinary analysis need not be amplified by stress concentration factors at geometrical discontinuities and changes of cross section. All categories were found to have a common slope with $m = -3$. Equation G3-1 of the *Specification* is to be used to calculate the design stress range for the chosen design life, $N$. Table G1 of the *Specification* provides a classification system for the various stress categories. This also provides the constant $C_f$ that is applicable to the stress category that is required for calculating the design stress range $F_{SR}$.

<table>
<thead>
<tr>
<th>Stress Category</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>11.0</td>
</tr>
<tr>
<td>II</td>
<td>10.5</td>
</tr>
<tr>
<td>III</td>
<td>10.0</td>
</tr>
<tr>
<td>IV</td>
<td>9.5</td>
</tr>
</tbody>
</table>

The provisions for bolts and threaded parts were taken from the AISC Specification (AISC, 1999).
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Appendix A:

Commentary on Provisions
Applicable to the United States

2001 Edition
APPENDIX A: COMMENTARY ON PROVISIONS APPLICABLE TO THE UNITED STATES

The Commentary on Appendix A provides a record of reasoning behind, and justification for, provisions that are applicable only to the United States. The format used herein is consistent with that used in Appendix A.

A1.1a Scope and Limits of Applicability

The ASD/LRFD Specification (AISI, 2001) is limited to the design of steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The design can be made by using either the Allowable strength Design method or the Load and Resistance Factor Design method. Even though both methods are equally acceptable, these two methods must not be mixed in designing various components and connections of structures.

A2.2 Other Steels

Although the use of ASTM-designated steels listed in Specification Section A2.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in this provision.

A3 Loads

A3.1 Nominal Loads

The Specification does not establish the dead, live, snow, wind, earthquake or other loading requirements for which a structure should be designed. These loads are typically covered by the applicable building code. Otherwise, the American Society of Civil Engineers Standard (ASCE, 1998) should be used as the basis for design.

Recognized engineering procedures should be employed to reflect the effect of impact loads on a structure. For building design, reference may be made to AISC publications (AISC, 1989; AISC 1999).

When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads acting in combination with wind or earthquake loads.

A4.1.2 Load Combinations for ASD

In 2001, the Specification was revised to specify that all loads and load combinations are required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE, 1998).

The combined effects of two or more loads, excluding dead load, are permitted to be multiplied by 0.75. However, the effect of reduced loads plus the dead load should not be less than the effect of dead load.
plus any single load that produces the largest effect. This reduction is based on the low probability of occurrence of two or more loads all attaining their maximum value at the same time, and is also consistent with ASCE 7. The requirement that the 0.75 factor only applies to load combinations containing two or more load effects, excluding dead load, indicates that the 0.75 factor cannot be applied to load combinations such as dead load plus wind load, or dead load plus earthquake load.

When steel decks are used for roof and floor composite construction, steel decks should be designed to carry the concrete dead load, the steel dead load, and the construction live load. The construction load is based on the sequential loading of concrete as specified in the ANSI/ASCE Standard 3-91 (ASCE, 1991) and in the Design Manual of Steel Deck Institute (SDI, 1995).

**A5.1.2 Load Factors and Load Combinations for LRFD**

In 2001, the Specification was revised to specify that all loads and load combinations were required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE, 1998).

In view of the fact that building codes and ASCE Standard 7 do not provide load factors and load combinations for roof and floor composite construction using cold-formed steel deck, the following load combination may be used for this type of composite construction:

\[ 1.2D_s + 1.6C_w + 1.4C \]

where

- \( D_s \) = weight of steel deck
- \( C_w \) = weight of wet concrete during construction
- \( C \) = construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.

The above load combination provides safety 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 is comparable to the allowable strength design method.

**C2 Tension Members**

As described in Specification Section C2, the nominal tensile strength [resistance] of axially loaded cold-formed steel tension members is determined either by yielding of the gross area of the cross-section or by fracture of the net area of the cross section. At locations of connections, the nominal tensile strength [resistance] is also limited by the capacities specified in Specification Sections E2.7, E3, and E5 for tension in connected parts.
Yielding in the gross section indirectly provides a limit on the deformation that a tension member can achieve. The definition of yielding in the gross section to determine the tensile strength [resistance] is well established in hot-rolled steel construction.

For the LRFD Method, the resistance factor of \( \phi_t = 0.75 \) used for fracture of the net section is consistent with the \( \phi \) factor used in the AISC LRFD Specification (AISC, 1999). The resistance factor \( \phi_t = 0.90 \) used for yielding in the gross section was also selected to be consistent with the AISC LRFD Specification (AISC, 1999).

**C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System**

For beams supporting a standing seam roof system, e.g. a roof purlin subjected to dead plus live load, or uplift from wind load, the bending capacity is greater than the bending strength of an unbraced member and may be equal to the bending strength of a fully braced member. The bending capacity is governed by the nature of the loading, gravity or uplift, and the nature of the particular standing seam roof system. Due to the availability of many different types of standing seam roof systems, an analytical method for determining positive and negative bending capacities has not been developed at the present time. However, in order to resolve this issue relative to the gravity loading condition, Section C3.1.4 was added in the 1996 edition of the AISI Specification for determining the nominal flexural strength of beams having one flange fastened to a standing seam roof system. In Specification Equation C3.1.4-1, the reduction factor, \( R \), can be determined by the test procedures, which were established in 1996 and are included in Part VIII of the AISI Cold-Formed Steel Design Manual (AISI, 2002). Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

**E2a Welded Connections**

The design provisions for welded connections were developed based primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. In addition, the Cornell research provided the experimental basis for the AWS Structural Welding Code for Sheet Steel (AWS, 1998). In most cases, the provisions of the AWS code are in agreement with this Specification section.

The terms used in this Specification section agree with the standard nomenclature given in the AWS Welding Structural Code for Sheet Steel (AWS, 1998).

For welded material thicknesses greater than 0.18 in. (4.57 mm), AISC specifications for ASD or LRFD should be followed.

**E3a Bolted Connections**

In Table E3a of Appendix A, the maximum size of holes for bolts having
diameters not less than 1/2 inch (12.7 mm) is based on the specifications of the Research Council on Structural Connections and the American Institute of Steel Construction (RCSC, 2000; AISC, 1989 and 1999), except that for the oversized hole diameter, a slightly larger hole diameter is permitted.

For bolts having diameters less than 1/2 inch (12.7 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (0.794 mm). This maximum size of bolt holes is based on previous editions of the AISI Specification.

When using oversized holes care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working loads. Excessive deformations which can occur in the direction of the slots may be prevented by requiring bolt pretensioning.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or backup plates should be used over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by tests. For connections using long-slotted holes, Specification Section E3.4 requires the use of washers or back-up plates and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

**E3.1 Shear, Spacing and Edge Distance**

The provisions for minimum spacing and edge distance were revised in 1980 to include additional design requirements for bolted connections with standard, oversized, and slotted holes. The minimum edge distance of each individual connected part, e_{min}, is determined by using the tensile strength of steel (F_u) and the thickness of connected part. According to the different ranges of the F_u/F_{sy} ratio, two different factors of safety are used for determining the required minimum edge distance. These design provisions are based on the following basic equation established from the test results:

\[ e = \frac{P}{F_u t} \]  

in which e is the required minimum edge distance to prevent shear failure of the connected part for a force, P, transmitted by one bolt, and t is the thickness of the thinnest connected part. For design purpose, a factor of safety of 2.0 was used for F_u/F_{sy} ≥ 1.08, and 2.22 for F_u/F_{sy} < 1.08, according to the degree of correlation between the above equation and the test data. As a result, whenever F_u/F_{sy} ≥ 1.08, the AISI requirement is the same as the AISC specification except for the measurement of distance “e”. In addition, several requirements were added to the AISI Specification in 1980 concerning (1) the minimum distance between centers of holes, as required for installation of bolts, (2) the required clear distance between edges of two adjacent holes, and (3) the minimum distance between the edge of the hole and the end of the member. The same design provisions were retained in the 1986 AISI Specification and were also used in the 1996 AISI Specification, except that the limiting F_u/F_{sy} ratio has been reduced from 1.15 to 1.08 for the consistency with Specification Section A2.3.1. The test data used for the development of
Equation C-E3.1-1 are documented by Winter (1956a and 1956b) and Yu (1982, 1985, and 2000).

**E3.2 Fracture in Net Section (Shear Lag)**

In the AISI Specification, the nominal tensile strength [resistance], $P_n$, of the net section of bolt connected parts is based on the loads determined by Specification Sections C2 and E3.2, whichever is smaller. In the use of the equations provided in Specification Section E3.2, the following design features should be noted:

1. The provisions are applicable only to the thinnest connected part less than 3/16 inch (4.76 mm) in thickness. For materials thicker than 3/16 inch (4.76 mm), the design should follow the specifications or standards stipulated in Section E3a of Appendix A, B, or C.
2. The nominal tensile strength [resistance], $P_n$, on the net section of a bolt connected member is determined by the tensile strength of the connected part ($F_u$), and the ratio “$d/s$” for connections with a single bolt or a single row of bolts perpendicular to the force.
3. Different equations are given for bolted connections with and without washers (Chong and Matlock, 1975).
4. The nominal tensile strength [resistance] on the net section of a connected member is based on the type of joint, either a single shear lap joint or a double shear butt joint.

The presence of staggered or diagonal hole patterns in a bolted connection has long been recognized as increasing the net section area for the limit state of fracture in the net section. LaBoube and Yu (1995) summarized the findings of a limited study of the behavior of bolted connections having staggered hole patterns. The research showed that when a staggered hole pattern is present, the width of a fracture plane can be adjusted by use of $s'^2/4g$.

Because of the lack of test data necessary for a more accurate design formulation, a discontinuity between this Specification and the specifications or standards, stipulated in Appendix A, B or C, may occur. The presence of a discontinuity should not be a significant design issue because the use of the staggered hole patterns is not common in cold-formed steel applications.

Shear lag has a debilitating effect on the tensile capacity of a cross

![Figure C-E3.2-1](image-url)
section. Based on UMR research (LaBoube and Yu, 1995) design equations have been developed that can be used to estimate the influence of the shear lag. The research demonstrated that the shear lag effect differs for an angle and a channel. For both cross sections, however, the key parameters that influence shear lag are the distance from the shear plane to the center of gravity of the cross section and the length of the bolted connection (Fig. C-E3.2-1). The research showed that for cold-formed sections using single bolt connections, bearing usually controlled the nominal strength, not fracture in the net section.

Previous tests showed that for flat sheet connections using a single bolt or a single row having multiple bolts perpendicular to the force (Chong and Matlock, 1975; Carill, LaBoube and Yu, 1994), the joint rotation and out-of-plane deformation of flat sheets are excessive. The strength reduction due to tearing of steel sheets in the net section is considered by Specification Equations. E3.2-2 and E3.2-4 according to the d/s ratio and the use of washers (AISI, 1996). For flat sheet connections using multiple bolts in the line of force and having less out-of-plane deformations, the strength reduction is not required in this edition of the Specification (Rogers and Hancock, 1998).

For flat sheet connections having staggered hole patterns as shown in Figure C-E3.2-2, the nominal tensile strength of path ABDE can be determined by Specification Section E3.2(1). In addition, the nominal tensile strength of the staggered path ABCDE can be determined by Specification Section E3.2(2). For this case, Specification Equation E3.2-2 can be used to compute $F_t$ as long as each line of bolts parallel to the force has only one bolt.

![Figure C-E3.2-2 Flat Sheet Connections Having Staggered Holes](image)

The value for $\phi$ used with Specification Equation E3.2-8 is based on statistical analysis of the test data with a corresponding value of $\beta = 3.5$ for LRFD. The $\Omega$ values are unchanged from previous editions of the AISI ASD Specification.

**E3.4 Shear and Tension in Bolts**

For the design of bolted connections, the allowable shear stresses for bolts have been provided in the AISI Specification for cold-formed steel design since 1956. However, the allowable tension stresses were not provided in Specification Section E3.4 for bolts subject to tension until 1986. In Specification
Table E3.4-1, the allowable stresses specified for A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC Specification (1978). It should be noted that the same values are also used in Table J3.2 of the AISC ASD Specification (1989). For A307, A449, and A354 bolts with diameters less than 1/2-inch (12.7 mm), the allowable tension stresses were reduced by 10 percent, as compared with these bolts having diameters not less than 1/2 inch (12.7 mm), because the average ratio of (tensile-stress area)/(gross-area) for 1/4-inch (6.35 mm) and 3/8-inch (9.53 mm) diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for 1/2-inch (12.7 mm) and 1-inch (25.4 mm) diameter bolts. In the AISI ASD/LRFD Specification (1996), Table E3.4-1 provides nominal tensile strengths for various types of bolts with applicable factors of safety. The allowable tension stresses computed from \( F_{nt}/\Omega \) are approximately the same as that permitted by the AISI 1986 ASD Specification. The same table also gives the resistance factor to be used for the LRFD method.

The design provisions for bolts subjected to a combination of shear and tension were added in Specification Section E3.4 in 1986. The design equations given in the Specification were based on Section 1.6.3 of the AISC Specification (AISC, 1978) for the design of bolts used for bearing-type connections. The design equations used for A354, A449, and A307 bolts with d < 1/2 inch (12.7 mm) were derived from the following equation for the ASD method:

\[
F'_t = 1.25F_t - Af_v \leq F_t \tag{C-E3.4-1}
\]

in which

- \( F'_t \) = reduced allowable tension stress for bolts subject to a combination of shear and tension
- \( F_t \) = allowable tension stress for bolts subject only to tension
- \( A \) = 1.8 for threads not excluded from shear planes
- \( A \) = 1.4 for threads excluded from shear planes
- \( f_v \) = shear stress in bolt

In 1996, the equations for determining the reduced nominal tension stress, \( F'_{nt} \) for bolts subjected to the combination of shear and tension were included in the Specification and those equations are retained in this edition of the Specification. For bolted connection design, the possibility of pullover of the connected sheet at the bolt head, nut, or washer should also be considered when bolt tension is involved, especially for thin sheathing material. For unsymmetrical sections, such as C- and Z-sections used as purlins or girts, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications, other literature, or tests.

For design tables and example problems on bolted connections, see Part IV of the Design Manual (AISI, 2002).

**E4.3.2 Connection Shear Limited by End Distance**

The nominal shear per fastener as limited by edge distance is the
same as that specified for bolts.

**E5 Rupture**

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coped beams a tearing failure mode as shown in Figure C-E5-1(a) can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-E5-1(b) and Figure C-E5-2. The provisions provided in *Specification* Section E5 for shear rupture have been adopted from the AISC Specification (AISC, 1978). For additional design information on tension rupture strength [resistance] and block shear rupture strength [resistance] of connections (Figures C-E5-1 and C-E5-2), refer to the AISC Specifications (AISC, 1989 and 1999).

![Failure Modes for Block Shear Rupture](image1.png)

*Figure C-E5-1 Failure Modes for Block Shear Rupture*

![Block Shear Rupture in Tension](image2.png)

*Figure C-E5-2 Block Shear Rupture in Tension*

Block shear is a limit state in which the resistance is determined by the sum of the shear strength [resistance] on a failure path(s) parallel to the force and the tensile strength [resistance] on the segment(s) perpendicular to the force, as shown in Figure C-E5-2. A comprehensive test program does not exist regarding block shear for cold-formed steel members. However, a limited study conducted at the University of Missouri-Rolla indicates that the AISC LRFD equations may be applied to cold-formed steel members. The $\phi$ (LRFD) and $\Omega$ (ASD) values for
block shear were taken from the AISI 1996 edition of the *Specification*, and are based on the performance of fillet welds. In calculating the net web area $A_{WN}$ for coped beams, the web depth is taken as the flat portion of the web as illustrated in Fig. C-E5-3.

![Figure C-E5-3 Definition of $h_{WC}$](image)
Appendix B:
Commentary on Provisions
Applicable to Canada

2001 EDITION
APPENDIX B: COMMENTARY ON PROVISIONS APPLICABLE TO CANADA

This commentary on Appendix B of the Specification provides a record of reasoning behind, and justification for, provisions that are applicable only to Canada. Only those sections of Appendix B of the Specification are addressed herein or where additional commentary is required beyond what is already contained in the Commentary on the 2001 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members (hereinafter referred to as the Commentary). The format used herein is consistent with that used in Appendix B of the Specification.

In comparison to the 1994 edition of CSA Standard S136, a number of changes have been incorporated into the Specification. The most significant ones are as follows:

a) Changes have been made to the steel thickness tolerances.
b) A different approach in calculating the increase in yield strength due to cold work of forming is presented.
c) A new method for the design of multiple intermediate stiffeners has been introduced.
d) Information on the design of standing seam roof panel systems has been added.
e) A new expression for the bending coefficient, $C_b$, has been added and $C_b > 1$ is now permitted for beam-columns.
f) A new method for beams having one flange through-fastened to deck or sheathing has been added.
g) A section on the shear resistance of C-sections with holes has been added.
h) A new method has been introduced for concentrically loaded compression members.
i) A new method for compression members having one flange through-fastened to deck or sheathing has been added.
j) Interaction equations for combined axial tensile load and bending have been added.
k) A new section has been added, entitled, “Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing”
l) The section on welded connections has been revised and expanded.
m) Some changes have been made regarding bearing of bolted connections.
n) A new section on screw connections has been added.
o) A new section has been added on tests for special cases.
p) A new chapter has been added that deals with the design of cold-formed steel structural members and connections for cyclic loading (fatigue).
A2.1a Applicable Steels

CSA Standard G40.20/G40.21 is widely used in Canada for structural quality bars and plate. A number of ASTM Standards are included in Section A2.1 because they are also widely used in Canada.

A2.2 Other Steels

This section in Appendix B is essentially the same as in CSA Standard S136-94, with only some changes that were made in the section on ductility.

A2.3 Ductility

An exception is included for multi-web configurations such as decks, where reduced yield strength can be used for determining the nominal flexural resistance of such sections. See Commentary for detailed information.

A2.4a Delivered Minimum Thickness

This section of the Specification includes a significant change in the requirements for minimum delivered thickness. CSA Standard S136-94 provided a series of tables that specified the maximum under-tolerances applicable to different sheet classifications. This has been replaced with the requirement that 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% of the thickness used in its design. This approach is simpler and unifies the practice across North America. Table B-A2.4-1 of Appendix B provides values for Hot-Dipped Metallic Coating Thickness Allowances, which is similar to what was contained in CSA Standard S136-94.

A6 Limit States Design

In limit states design, the resistance of a structural component is checked against the various limit states. For the ultimate limit states resistance, the structural member must retain its load-carrying capacity up to the factored load levels. For serviceability limit states, the performance of the structure must be satisfactory at specified load levels. Specified loads are those prescribed by the National Building Code of Canada. Examples of serviceability requirements include deflections and the possibility of vibrations.

Section A6 of the Specification sets forth the fundamental safety criterion that must be met, namely:

Factored resistance $\geq$ effect of factored loads

The factored resistance is given by the product $\phi R_n$, where $\phi$ is the resistance factor which is applied to the nominal member resistance, $R_n$. The resistance factor is intended to take into account the fact that the resistance of the member may be less than anticipated, due to variability of the material properties, dimensions, and workmanship, and also to take into account the type of failure and uncertainty in the prediction of the resistance.

The resistance factor does not, however, cover gross human errors. Human errors cause most structural failures and typically these human errors are “gross”
errors. Gross errors are completely unpredictable and are not covered by the overall safety factor inherent in buildings.

In limit states design, structural reliability is specified in terms of a safety index, $\beta$, determined through a statistical analysis of the loads and resistances. The safety index is directly related to the structural reliability of the design; hence, increasing $\beta$ increases the reliability, and decreasing $\beta$ decreases the reliability. The safety index, $\beta$, is also directly related to the load and resistance factors used in the design.

The *National Building Code of Canada* defines a set of load factors, load combination factors, and specified minimum loads to be used in the design, hence fixing the position of the nominal load distribution and the factored load distribution. The design Standard is then obligated to specify the appropriate resistance function.

Those responsible for writing a design Standard are given the load distribution and load factors, and must calibrate the resistance factors, $\phi$, such that the safety index, $\beta$, reaches a certain target value. The technical committee responsible for CSA Standard S136 elected to use a target safety index of 3.0 for members and 4.0 for connections.

In order to determine the loading for calibration, it was assumed that 80% of cold-formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20% for structural sections (purlins, girts, studs, etc.). An effective load factor was arrived at by assuming live-to-dead load ratios and their relative frequencies of occurrence.

Probabilistic studies show that consistent probabilities of failure are determined for all live-to-dead load ratios when a live load factor of 1.50 and a dead load factor of 1.25 are used.

### C2 Tension Members

The general provisions for the design of tension members have not changed with respect to the CSA Standard S136-94. The only change that was made involves staggered connections.

#### C2.2 Fracture of Net Section

Some reformatting has been done and the critical path involving stagger has been reduced by 10%. This reduction is justified on the basis of the recent research by LaBoube and Yu (1995). See *Commentary* for detailed explanation. Two new sections have been added for shear lag effect, i.e., Section E2.7 of the *Specification* for welded connections and Section E3.2 of Appendix B for bolted connections. As a result, Clause 6.3.3 of CSA Standard S136-94 has been deleted.

Examples of tension members are shown in Figures B-C2.2-1 and B-C2.2-2. Block tear-out can also occur at the end of a coped beam, where the applied force is a shear at the end of a beam. This force causes tension on horizontal planes and shear on vertical planes. An example is shown in Figure B-C2.2-3. Other possible failure paths should also be checked.
Appendix B, Provisions Applicable to Canada

Failure Path 1, 2, 3, 4
\[ L_c = L_t \]
\[ L_t = (w_g - h) \]
\[ L_c = (w_g - h) \]

Failure Path 5, 2, 3, 6
\[ L_c = 0.6L_v \]
\[ L_v = 2(e - h/2) \]
\[ L_c = 0.6[2(e - h/2)] = 1.2e - 0.6h \]

Figure B-C2.2-1 Potential Failure Paths of Single Lap Joint

Failure Path 3, 2, 4, 6, 7, 8
\[ L_c = 0.9[L_t + L_s + 0.6L_v] \]
\[ L_t = (2g - h) \]
\[ L_s = 2(g + s^2/4g - h) \]
\[ L_v = (2e - h) \]
\[ L_c = 0.9[(2g - h) + 2(g + s^2/4g - h) + 0.6(2e - h)] \]

Figure B-C2.2-2 Potential Failure Paths of Stiffened Channel

Failure Path 1, 2, 3, 4, 5, 6
\[ L_c = L_t + 0.6L_v \]
\[ L_t = (s + e_2 - 1.5h) \]
\[ L_v = (e_1 + 2g - 2.5h) \]
\[ L_c = (s + e_2 - 1.5h) + 0.6(e_1 + 2g - 2.5h) \]

Figure B-C2.2-3 Potential Failure Path of Coped Stiffened Channel
C3.4 Web Crippling

The basic web crippling equation is the same as in Clause 6.4.7 of CSA Standard S136-94; however, more detailed web crippling coefficients are presented based on recent research and calibrations. See Commentary for detailed information. A new section has been added for web crippling of C-section webs with holes.

D3a Lateral Bracing

The provisions of this section cover members loaded in the plane of the web. Conditions may occur that cause a lateral component of the load to be transferred through the bracing member to supporting structural members. In such a case, these lateral forces shall be additive to the requirements of this section. The provisions in the Specification recognize the distinctly different behavior of the members to be braced, as defined in Section D3.1 and D3.2 of this Appendix. The term “discrete braces” is used to identify those braces that are only connected to the member to be braced for this express purpose.

D3.1a Symmetrical Beams and Columns

D3.1.1 Discrete Bracing

This section maintains the 2% requirement of either the compressive force in the compressive member at the braced location or the compressive force in the compressive flange of the flexural member at the braced location.

D3.2a C-Section and Z-Section Beams

This section covers bracing requirements of channel and Z-sections and any other section in which the applied load in the plane of the web induces twist.

D3.2.3 Discrete Bracing

This section provides for brace intervals to prevent the member from rotating about the shear center for channels or from rotating about the point of symmetry for Z-sections. The spacing must be such that any stresses due to the rotation tendency are small enough so that they will not significantly reduce the load-carrying capacity of the member. The rotation must also be small enough (in the order of 2°) to be not objectionable as a service requirement.

Based on tests and the study by Winter et al. (1949b), it was found that these requirements are satisfied for any type of load if braces are provided at intervals of one-quarter of the span, with the exception of concentrated loads requiring braces near the point of application.

Fewer brace points may be used if it can be shown to be acceptable by rational analysis or testing in accordance with Chapter F of the Specification, recognizing the variety of conditions, including the case where loads are applied out of the plane of the web.
For sections used as purlins with a standing seam roof, the number of braces per bay is often determined by rational analysis and/or testing. The requirement for a minimum number of braces per bay is to recognize that predictability of the lateral support and rotational restraint is limited on account of the many variables such as fasteners, insulation, friction coefficients, and distortion of roof panels under load.

**D3.2.4 One Flange Braced by Deck, Slab, or Sheathing**

Forces generated by the tendency for lateral movement and/or twist of the beams, whether cumulative or not, must be transferred to a sufficiently stiff part of the framing system. There are several ways in which this transfer may be accomplished:

(a) by the deck, slab, or sheathing providing a rigid diaphragm capable of transferring the forces to the supporting structure;
(b) by arranging equally loaded pairs of members facing each other;
(c) by direct axial force in the covering material that can be transferred to the supporting structure or balanced by opposing forces;
(d) by a system of sag members such as rods, angles, or channels that transfer the forces to the supporting structure; or
(e) by any other method that designers may select to transfer forces to the supporting structure.

For all types of single web beams, the flange that is not attached to the deck or sheathing material may be subject to compressive stresses under certain loading arrangements, such as beams continuous over supports or under wind load. The elastic lateral support to this flange provided through the web may allow an increase in limit stress over that calculated by assuming that the compressive flange is a column, with pinned ends at points of lateral bracing. Research indicates that the compressive limit stress is also sensitive to the rotational flexibility of the joint between the beam and the deck or sheathing material.

This section is intended to apply even when the flange that is not attached to the sheathing material is in tension.

**E2a Welded Connections**

The section has been revised and expanded and replaces Clause 7.2 of CSA Standard S136-94. See *Commentary* for detailed information. Both fabricators and erectors must be certified under CSA Standard W47.1 for arc welding and CSA Standard W55.3 for resistance welding. This provision extends the certification requirements to the welding of cold-formed members or components to other construction, e.g., welding steel deck to structural steel framing.

**E3 Bolted Connections**

**E3.3 Bearing**

Improvements have been made to this section in comparison to Clause 7.3.5.1 of CSA Standard S136-94. Section E3.3.2 has been added, giving consideration to bolt hole deformation. See *Commentary* for detailed information.
Appendix C:
Provisions Applicable to Mexico

2001 EDITION
APPENDIX C: COMMENTARY ON PROVISIONS APPLICABLE TO MEXICO

The Commentary on Appendix C provides a record of reasoning behind, and justification for, provisions that are applicable only to Mexico. The format used herein is consistent with that used in Appendix C.

A1.1a Scope and Limits of Applicability

The ASD/LRFD Specification (AISI, 2001) is limited to the design of steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The design can be made by using either the Allowable strength Design method or the Load and Resistance Factor Design method. Even though both methods are equally acceptable, these two methods must not be mixed in designing various components and connections of structures.

A2.2 Other Steels

Although the use of ASTM-designated steels listed in Specification Section A2.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in this provision.

A3 Loads

A3.1 Nominal Loads

The Specification does not establish the dead, live, snow, wind, earthquake or other loading requirements for which a structure should be designed. These loads are typically covered by the applicable building code. Otherwise, the American Society of Civil Engineers Standard (ASCE, 1998) should be used as the basis for design.

Recognized engineering procedures should be employed to reflect the effect of impact loads on a structure. For building design, reference may be made to AISC publications (AISC, 1989; AISC 1999).

When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads acting in combination with wind or earthquake loads.

A4.1.2 Load Combinations for ASD

In 2001, the Specification was revised to specify that all loads and load combinations are required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE, 1998).

The combined effects of two or more loads, excluding dead load, are permitted to be multiplied by 0.75. However, the effect of reduced loads plus the dead load should not be less than the effect of dead load plus any single load that produces the largest effect. This reduction is
based on the low probability of occurrence of two or more loads all attaining their maximum value at the same time, and is also consistent with ASCE 7. The requirement that the 0.75 factor only applies to load combinations containing two or more load effects, excluding dead load, indicates that the 0.75 factor cannot be applied to load combinations such as dead load plus wind load, or dead load plus earthquake load.

When steel decks are used for roof and floor composite construction, steel decks should be designed to carry the concrete dead load, the steel dead load, and the construction live load. The construction load is based on the sequential loading of concrete as specified in the ANSI/ASCE Standard 3-91 (ASCE, 1991) and in the Design Manual of Steel Deck Institute (SDI, 1995).

A5.1.2 Load Factors and Load Combinations for LRFD

In 2001, the Specification was revised to specify that all loads and load combinations were required to follow the applicable building code. In the absence of an applicable building code, loads and load combinations should be determined according to the American Society of Civil Engineers Standard, Minimum Design Loads for Buildings and Other Structures, ASCE 7 (ASCE, 1998).

In view of the fact that building codes and ASCE Standard 7 do not provide load factors and load combinations for roof and floor composite construction using cold-formed steel deck, the following load combination may be used for this type of composite construction:

$$1.2D_s + 1.6C_w + 1.4C$$

where

- $D_s = \text{weight of steel deck}$
- $C_w = \text{weight of wet concrete during construction}$
- $C = \text{construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete}$

The above load combination provides safety 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 is comparable to the allowable strength design method.

C3.1.4 Beams Having One Flange Fastened to a Standing Seam Roof System

For beams supporting a standing seam roof system, e.g. a roof purlin subjected to dead plus live load, or uplift from wind load, the bending capacity is greater than the bending strength of an unbraced member and may be equal to the bending strength of a fully braced member. The bending capacity is governed by the nature of the loading, gravity or uplift, and the nature of the particular standing seam roof
system. Due to the availability of many different types of standing seam roof systems, an analytical method for determining positive and negative bending capacities has not been developed at the present time. However, in order to resolve this issue relative to the gravity loading condition, Section C3.1.4 was added in the 1996 edition of the AISI Specification for determining the nominal flexural strength of beams having one flange fastened to a standing seam roof system. In Specification Equation C3.1.4-1, the reduction factor, R, can be determined by the test procedures, which were established in 1996 and are included in Part VIII of the AISI Cold-Formed Steel Design Manual (AISI, 2002). Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

C2 Tension Members

As described in Specification Section C2, the nominal tensile strength [resistance] of axially loaded cold-formed steel tension members is determined either by yielding of the gross area of the cross-section or by fracture of the net area of the cross section. At locations of connections, the nominal tensile strength [resistance] is also limited by the capacities specified in Specification Sections E2.7, E3, and E5 for tension in connected parts.

Yielding in the gross section indirectly provides a limit on the deformation that a tension member can achieve. The definition of yielding in the gross section to determine the tensile strength [resistance] is well established in hot-rolled steel construction.

For the LRFD Method, the resistance factor of $\phi_t = 0.75$ used for fracture of the net section is consistent with the $\phi$ factor used in the AISC LRFD Specification (AISC, 1999). The resistance factor $\phi_t = 0.90$ used for yielding in the gross section was also selected to be consistent with the AISC LRFD Specification (AISC, 1999).

E2a Welded Connections

The design provisions for welded connections were developed primarily on experimental evidence obtained from an extensive test program conducted at Cornell University. In addition, the Cornell research provided the experimental basis for the AWS Structural Welding Code for Sheet Steel (AWS, 1998). In most cases, the provisions of the AWS code are in agreement with this Specification section.

The terms used in this Specification section agree with the standard nomenclature given in the AWS Welding Structural Code for Sheet Steel (AWS, 1998).

For welded material thicknesses greater than 0.18 in. (4.57 mm), AISC specifications for ASD or LRFD should be followed.

E3a Bolted Connections

In Table E3a of Appendix A, the maximum size of holes for bolts having diameters not less than 1/2 inch (12.7 mm) is based on the specifications of the
Appendix C, Provisions Applicable to Mexico

Research Council on Structural Connections and the American Institute of Steel Construction (RCSC, 2000; AISC, 1989 and 1999), except that for the oversized hole diameter, a slightly larger hole diameter is permitted.

For bolts having diameters less than 1/2 inch (12.7 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (0.794 mm). This maximum size of bolt holes is based on previous editions of the AISI Specification.

When using oversized holes care must be exercised by the designer to ensure that excessive deformation due to slip will not occur at working loads. Excessive deformations which can occur in the direction of the slots may be prevented by requiring bolt pretensioning.

Short-slotted holes are usually treated in the same manner as oversized holes. Washers or backup plates should be used over oversized or short-slotted holes in an outer ply unless suitable performance is demonstrated by tests. For connections using long-slotted holes, Specification Section E3.4 requires the use of washers or back-up plates and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

E3.1 Shear, Spacing and Edge Distance

The provisions for minimum spacing and edge distance were revised in 1980 to include additional design requirements for bolted connections with standard, oversized, and slotted holes. The minimum edge distance of each individual connected part, \( e_{\text{min}} \), is determined by using the tensile strength of steel \( F_u \) and the thickness of connected part. According to the different ranges of the \( F_u/F_{sy} \) ratio, two different factors of safety are used for determining the required minimum edge distance. These design provisions are based on the following basic equation established from the test results:

\[
e = \frac{P}{F_u t}
\]

in which \( e \) is the required minimum edge distance to prevent shear failure of the connected part for a force, \( P \), transmitted by one bolt, and \( t \) is the thickness of the thinnest connected part. For design purpose, a factor of safety of 2.0 was used for \( F_u/F_{sy} \geq 1.08 \), and 2.22 for \( F_u/F_{sy} < 1.08 \), according to the degree of correlation between the above equation and the test data. As a result, whenever \( F_u/F_{sy} \geq 1.08 \), the AISI requirement is the same as the AISC specification except for the measurement of distance “\( e \)”. In addition, several requirements were added to the AISI Specification in 1980 concerning (1) the minimum distance between centers of holes, as required for installation of bolts, (2) the required clear distance between edges of two adjacent holes, and (3) the minimum distance between the edge of the hole and the end of the member. The same design provisions were retained in the 1986 AISI Specification and were also used in the 1996 AISI Specification, except that the limiting \( F_u/F_{sy} \) ratio has been reduced from 1.15 to 1.08 for the consistency with Specification Section A2.3.1. The test data used for the development of Equation C-E3.1-1 are documented by Winter (1956a and 1956b) and Yu (1982, 1985, and 2000).
E3.2 Fracture in Net Section (Shear Lag)

In the AISI Specification, the nominal tensile strength [resistance], $P_n$, of the net section of bolt connected parts is based on the loads determined by Specification Sections C2 and E3.2, whichever is smaller. In the use of the equations provided in Specification Section E3.2, the following design features should be noted:

1. The provisions are applicable only to the thinnest connected part less than 3/16 inch (4.76 mm) in thickness. For materials thicker than 3/16 inch (4.76 mm), the design should follow the specifications or standards stipulated in Section E3a of Appendix A, B, or C.

2. The nominal tensile strength [resistance], $P_n$, on the net section of a bolt connected member is determined by the tensile strength of the connected part ($F_{u}$), and the ratio “$d/s$” for connections with a single bolt or a single row of bolts perpendicular to the force.

3. Different equations are given for bolted connections with and without washers (Chong and Matlock, 1975).

4. The nominal tensile strength [resistance] on the net section of a connected member is based on the type of joint, either a single shear lap joint or a double shear butt joint.

The presence of staggered or diagonal hole patterns in a bolted connection has long been recognized as increasing the net section area for the limit state of fracture in the net section. LaBoube and Yu (1995) summarized the findings of a limited study of the behavior of bolted connections having staggered hole patterns. The research showed that when a staggered hole pattern is present, the width of a fracture plane can be adjusted by use of $s^2/4g$.

Because of the lack of test data necessary for a more accurate design formulation, a discontinuity between this Specification and the specifications or standards, stipulated in Appendix A, B or C, may occur. The presence of a discontinuity should not be a significant design issue because the use of the staggered hole patterns is not common in cold-formed steel applications.

Shear lag has a debilitating effect on the tensile capacity of a cross

![Figure C-E3.2-1](#)
section. Based on UMR research (LaBoube and Yu, 1995) design equations have been developed that can be used to estimate the influence of the shear lag. The research demonstrated that the shear lag effect differs for an angle and a channel. For both cross sections, however, the key parameters that influence shear lag are the distance from the shear plane to the center of gravity of the cross section and the length of the bolted connection (Fig. C-E3.2-1). The research showed that for cold-formed sections using single bolt connections, bearing usually controlled the nominal strength, not fracture in the net section.

Previous tests showed that for flat sheet connections using a single bolt or a single row having multiple bolts perpendicular to the force (Chong and Matlock, 1975; Carill, LaBoube and Yu, 1994), the joint rotation and out-of-plane deformation of flat sheets are excessive. The strength reduction due to tearing of steel sheets in the net section is considered by Specification Equations. E3.2-2 and E3.2-4 according to the d/s ratio and the use of washers (AISI, 1996). For flat sheet connections using multiple bolts in the line of force and having less out-of-plane deformations, the strength reduction is not required in this edition of the Specification (Rogers and Hancock, 1998).

For flat sheet connections having staggered hole patterns as shown in Figure C-E3.2-2, the nominal tensile strength of path ABDE can be determined by Specification Section E3.2(1). In addition, the nominal tensile strength of the staggered path ABCDE can be determined by Specification Section E3.2(2). For this case, Specification Equation E3.2-2 can be used to compute $F_t$ as long as each line of bolts parallel to the force has only one bolt.

![Figure C-E3.2-2 Flat Sheet Connections Having Staggered Holes](image)

The value for $\phi$ used with Specification Equation E3.2-8 is based on statistical analysis of the test data with a corresponding value of $\beta = 3.5$ for LRFD. The $\Omega$ values are unchanged from previous editions of the AISI ASD Specification.

**E3.4 Shear and Tension in Bolts**

For the design of bolted connections, the allowable shear stresses for bolts have been provided in the AISI Specification for cold-formed steel design since 1956. However, the allowable tension stresses were not provided in Specification Section E3.4 for bolts subject to tension until 1986. In Specification
Table E3.4-1, the allowable stresses specified for A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC Specification (1978). It should be noted that the same values are also used in Table J3.2 of the AISC ASD Specification (1989). For A307, A449, and A354 bolts with diameters less than 1/2-inch (12.7 mm), the allowable tension stresses were reduced by 10 percent, as compared with these bolts having diameters not less than 1/2 inch (12.7 mm), because the average ratio of (tensile-stress area)/(gross-area) for 1/4-inch (6.35 mm) and 3/8-inch (9.53 mm) diameter bolts is 0.68, which is about 10 percent less than the average area ratio of 0.75 for 1/2-inch (12.7 mm) and 1-inch (25.4 mm) diameter bolts. In the AISI ASD/LRFD Specification (1996), Table E3.4-1 provides nominal tensile strengths for various types of bolts with applicable factors of safety. The allowable tension stresses computed from $F_{nt}/\Omega$ are approximately the same as that permitted by the AISI 1986 ASD Specification. The same table also gives the resistance factor to be used for the LRFD method.

The design provisions for bolts subjected to a combination of shear and tension were added in Specification Section E3.4 in 1986. The design equations given in the Specification were based on Section 1.6.3 of the AISC Specification (AISC, 1978) for the design of bolts used for bearing-type connections. The design equations used for A354, A449, and A307 bolts with d < 1/2 inch (12.7 mm) were derived from the following equation for the ASD method:

$$F'_t = 1.25F_t - Af_v \leq F_t$$  \hspace{1cm} (C-E3.4-1)

in which

- $F'_t$ = reduced allowable tension stress for bolts subject to a combination of shear and tension
- $F_t$ = allowable tension stress for bolts subject only to tension
- $A$ = 1.8 for threads not excluded from shear planes
- $A$ = 1.4 for threads excluded from shear planes
- $f_v$ = shear stress in bolt

In 1996, the equations for determining the reduced nominal tension stress, $F'_{nt}$ for bolts subjected to the combination of shear and tension were included in the Specification and those equations are retained in this edition of the Specification. For bolted connection design, the possibility of pullover of the connected sheet at the bolt head, nut, or washer should also be considered when bolt tension is involved, especially for thin sheathing material. For unsymmetrical sections, such as C- and Z-sections used as purlins or girts, the problem is more severe because of the prying action resulting from rotation of the member which occurs as a consequence of loading normal to the sheathing. The designer should refer to applicable product code approvals, product specifications, other literature, or tests.

For design tables and example problems on bolted connections, see Part IV of the Design Manual (AISI, 2002).

**E4.3.2 Connection Shear Limited by End Distance**

The nominal shear per fastener as limited by edge distance is the
same as that specified for bolts.

**E5 Rupture**

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coping beams a tearing failure mode as shown in Figure C-E5-1(a) can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-E5-1(b).

![Figure C-E5-1 Failure Modes for Block Shear Rupture](image)

Figure C-E5-1 Failure Modes for Block Shear Rupture

![Figure C-E5-2 Block Shear Rupture in Tension](image)

Figure C-E5-2 Block Shear Rupture in Tension

and Figure C-E5-2. The provisions provided in Specification Section E5 for shear rupture have been adopted from the AISC Specification (AISC, 1978). For additional design information on tension rupture strength [resistance] and block shear rupture strength [resistance] of connections (Figures C-E5-1 and C-E5-2), refer to the AISC Specifications (AISC, 1989 and 1999).

Block shear is a limit state in which the resistance is determined by the sum of the shear strength [resistance] on a failure path(s) parallel to the force and the tensile strength [resistance] on the segment(s) perpendicular to the force, as shown in Figure C-E5-2. A comprehensive test program does not exist regarding block shear for cold-formed steel members. However, a limited study conducted at the University of Missouri-Rolla indicates that the AISC LRFD equations may be applied to cold-formed steel members. The $\phi$ (LRFD) and $\Omega$ (ASD) values for
block shear were taken from the AISI 1996 edition of the *Specification*, and are based on the performance of fillet welds. In calculating the net web area $A_{WNV}$ for coped beams, the web depth is taken as the flat portion of the web as illustrated in Fig. C-E5-3.

![Figure C-E5-3 Definition of $h_{WC}$](image-url)