AISI STANDARD

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

2016 EDITION WITH SUPPLEMENT 1

Approved in Canada by CSA Group
Endorsed in Mexico by CANACERO
DISCLAIMER

The material contained herein has been developed by a joint effort of the American Iron and Steel Institute (AISI) Committee on Specifications, CSA Group 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.

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DEDICATION

This edition of AISI S100 is dedicated to Roger L. Brockenbrough, P.E., who served as chairman of the AISI Committee on Specifications from 1991 to 2016. Roger led the development of the first unified ASD and LRFD steel design specification, as well as the first harmonized North American Cold-Formed Steel Specification. The Direct Strength Method was introduced under his leadership, and is incorporated into the main body of this edition of AISI S100. The Committee recognizes his significant contributions to the development of AISI S100, AISI S310, AISI test standards, and AISI design guides and manuals. The members of the AISI Committee on Specifications have valued Roger’s open-minded leadership approach and his willingness to promote new ideas and suggestions. Roger has been instrumental in the successes of the Committee on Specifications. The staff and members of AISI, along with the members of the Committee, greatly appreciate his dedication and contributions toward advancing the cold-formed steel industry.
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PREFACE

The North American Specification for the Design of Cold-Formed Steel Structural Members, as its name implies, is intended for use throughout Canada, Mexico, and the United States. This Specification supersedes the 2012 and previous editions of the North American Cold-Formed Steel Specification, the previous editions of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute (AISI), and the previous editions of CSA Group S136, Cold Formed Steel Structural Members, published by CSA Group.

The Specification was developed by a joint effort of the American Iron and Steel Institute Committee on Specifications, CSA Group 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 members from the AISI Committee on Specifications and the CSA Group S136 Committee.

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 M and Appendices 1 and 2, that is intended for use in all three countries, and two country-specific appendices (A and B). Appendix A is for use in both the United States and Mexico, and Appendix B is for use in Canada. A symbol (☞A,B) 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 (ϕ) for use with LRFD and LSD and the appropriate safety factors (Ω) for use with ASD. It should be noted that the use of LSD is limited to Canada and the use of ASD and LRFD is limited to the United States and Mexico.

The Specification also contains some terminology that is defined differently in Canada, the United States, and Mexico. These differences are set out in Section A1.3, “Definitions.” In the Specification, the terms that are specifically applicable to LSD are included in square brackets.

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 previous editions of the Specification.

The AISI and CSA Group consensus committees responsible for developing these provisions provide a balanced forum, with representatives of 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 and the United States. AISI, CSA Group, and CANACERO acknowledge the continuing dedication of the members of the specifications committees and their subcommittees. The membership of these committees follows this Preface.

The 2016 Edition of the Specification has been reorganized by incorporating the Direct Strength Method design provisions into Chapters A through M. Also, the chapters are laid out to be more in line with ANSI/AISC 360-2010. A section reference table of the 2012 Edition of the
**Specification** and this edition is provided.

In addition to content reorganization, the following changes and additions are made in this edition:

- **Section A2, Referenced Specifications, Codes and Standards.** All the references, including those specific to U.S. and Mexico or Canada, are listed in the main body of the **Specification**. All the referenced standards are updated.

- **Section A3.2, Other Steels.** The country-specific provisions are consolidated by bringing the provisions into the main body of the **Specification**.

- **Section B2, Loads and Load Combinations.** The applicable building codes for determining the loads and load combinations are introduced for the U.S., Mexico, and Canada.

- **Section B3, Design Basis.** This section introduces three design methods: ASD and LRFD are applicable to the U.S. and Mexico, and LSD is applicable to Canada. It references **Specification** chapters or sections that provide design provisions for *required strength* [effect due to factored loads] and *available strengths* [factored resistances], structural members, connections, stability, structural assemblies and systems, serviceability, ponding, fatigue, and corrosion effects.

- **Section B4, Dimensional Limits and Considerations.** The limitations for applying the **Effective Width Method** and the **Direct Strength Method** are streamlined.

- **Section C1, Design for System Stability.** The provisions consider Appendix 2, Second-Order Analysis, included in the 2012 Edition of the **Specification**, and incorporate system stability analysis approaches provided in ANSI/AISC 360.

- **Chapters E, F and G.** The provisions of the **Direct Strength Method** included in Appendix 1 of the 2012 Edition of the **Specification** are incorporated into these chapters.

- **Section F2.1.1, Singly- or Doubly-Symmetric Sections Bending About Symmetric Axis.** Simplified Equation F2.1.1-6 to determine elastic buckling stress, $F_{cre}$, is no longer applicable to *singly-symmetric C-Sections*.

- **Section H1, Combined Axial Load and Bending.** The interaction check equations for ASD, LRFD, and LSD are combined into one format, as applicable.

- **Section H1.2, Combined Compressive Axial Load and Bending.** The interaction check equations are revised with the moment magnification effect taken into consideration through the system stability effect in accordance with Section C1.

- **Section I2, Floor, Roof, or Wall Steel Diaphragm Construction.** AISI S310, AISI S240, and AISI S400 are introduced for *diaphragm* design, and the table of Safety and Resistance Factors for Diaphragms is moved to AISI S310.

- **Section I4, Cold-Formed Steel Light-Frame Construction.** The cold-formed steel framing standards are updated.

- **Section I5, Special Bolted Moment Frame Systems.** Special bolted moment frame systems should be designed in accordance with AISI S400.

- **Section I6.1, Members Strength: General Cross-Sections and System Connectivity.** This section permits the bending and compression strengths of purlins and girts to be determined analytically provided the lateral, rotational, and composite stiffness provided by the deck or sheathing, bridging and bracing, and span continuity are included.

- **Section I7, Rack Systems.** Rack system design should be in accordance with ANSI MH16.1.

- **Section J2, Welded Connections.** The country-specific standards are brought into the main
body of the *Specification*.  
Section J3, Bolted Connections. The table of Nominal Tensile and Shear Strengths for Bolts in Appendix A has been updated to be consistent with those in ANSI/AISC 360, and values for bolt diameters less than 0.5 in. (12 mm) have been revised.  
Section J7.2, Power-Actuated Fasteners (PAFs) in Concrete. The PAF pull-out strength in shear in cold-formed steel framing track-to-concrete connections is added.  
Section K1, Test Standards. The AISI S900 series of test standards are introduced, and the standards are also referenced in Section A2.  
Section K2, Test for Special Cases. The sentence that the provisions shall not apply to cold-formed steel *diaphragms* was deleted.  
Section K2.1.1, Load and Resistance Factor Design and Limit States Design. The table of Statistical Data for the Determination of Resistance Factor is simplified. The sentence that Section K2.1.1(b) is not applicable to floor, roof or wall steel *diaphragm* was deleted.  
Appendix 1, Effective Width of Elements. This appendix provides provisions for determining the effective width of elements as needed for the *Effective Width Method*.  
Appendix 2, Elastic Buckling Analysis of Members. This new appendix provides analytical and numerical approaches to determine the local, distortional, and global buckling strengths.

In the 2nd printing, Errata 1, published on March 20, 2018, has been incorporated.  
In the 3rd printing, Supplement 1 to the 2016 Edition of the North American Specification has been incorporated. The following changes are included in Supplement 1:
Section A3.3.2, Strength Increase From Cold Work of Forming. Revisions are made to the first paragraph to remove the requirement of no distortional buckling for considering strength increase from cold work of forming.  
Section E2.2, Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling. The last paragraph is revised so that the provisions can be applicable for members with holes.  
Section H1.2, Combined Compression Axial Load and Bending. The second paragraph is revised so that the provisions can be applicable for members with holes.  
Section J7.2, Power-Actuated Fasteners (PAFs) in Concrete. This section is removed to avoid unconservative designs of track and other cold-formed steel structural member attachments to concrete and to avoid unintended interpretation of the validity of these provisions in different applications.

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CSA Group  
Camara Nacional de la Industria del Hierro y del Acero
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## North American Specification Committee

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- R. L. Brockenbrough, *Chairman*
- H. H. Chen

**CSA Group**
- R. M. Schuster, *Chairman*
- S. R. Fox, *Secretary*

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- T. B. Pekoz

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Personnel

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Bailey Metal Products Ltd.

D. Allen
Super Stud Building Products

D. Bak
Steelway Building Systems

P. Bodwell
Verco Decking Inc.

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R. L. Brockenbrough and Associates

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University of Sydney

A. J. Harrold
BlueScope Buildings North America

J. M. Klaiman
ADTEK Engineers

R. A. LaBoube
Wei-Wen Yu Center for Cold-Formed Steel Structures

Z. Li
SUNY Polytechnic Institute

R. L. Madsen
Supreme Steel Framing System Association

B. Mandelzys
Steelrite

J. R. Martin
Verco Decking, Inc.

J. A. Mattingly
Consultant

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IRC Building Sciences Group

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ICC Evaluation Service, Inc.

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Keymark

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Vicwest

C. Moen
Virginia Polytechnic Institute and State University

J. R. U. Mujagic
Consultant

T. M. Murray
Consultant

J. D. Musselwhite
Southern Code Consulting International, LLC

R. V. Nunna
S. B. Barnes Associates

J. N. Nunnery
Consultant

T. B. Pekoz
Consultant
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## Section Numbering Comparison — AISI S100-12 Versus AISI S100-16

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NORTH AMERICAN SPECIFICATION FOR THE DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

This chapter addresses the scope and applicability of the Specification, lists the definitions of the terminology used, summarizes referenced specifications, codes, and standards, and provides requirements for materials.

This chapter is organized as follows:
A1 Scope, Applicability, and Definitions
A2 Referenced Specifications, Codes, and Standards
A3 Material

A1 Scope, Applicability, and Definitions

A1.1 Scope

This Specification applies to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than 1 in. (25.4 mm) in thickness and used for load-carrying purposes in (a) Buildings, and (b) Structures other than buildings provided allowances are made for dynamic effects.

A1.2 Applicability

This Specification includes Symbols, Chapters A through M, Appendices A and B, and Appendices 1 and 2 that shall apply as follows:

- Chapters A through M, Appendices 1 and 2—the United States, Mexico, and Canada,
- Appendix A—the United States and Mexico, and
- Appendix B—Canada.

The symbol \( \equiv \times \) is used to point out that additional provisions that are specific to a certain country are provided in the corresponding appendices indicated by the letter(s) “x.”

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:

- ASD and LRFD—the United States and Mexico, and
- LSD—Canada.

In this Specification, bracketed terms are equivalent terms that apply particularly to LSD.

The nominal strength [resistance] and stiffness of cold-formed steel components such as elements, members, assemblies, connections, and details shall be determined in accordance with the provisions in Chapters B through M, Appendices A and B, and Appendices 1 and 2 of the Specification.

Where the composition or configuration of the components is such that calculation of available strength [factored resistance] or stiffness cannot be made in accordance with these provisions (excluding those in Chapter K), structural performance shall be established from
one of the following:
(a) *Available strength* [factored resistance] or stiffness by tests only. Specifically, the *available strength* [factored resistance] is determined from tested *nominal strength* [resistance] by applying the *safety factors* or the *resistance factors* evaluated in accordance with Section K2.1.1(a);
(b) *Available strength* [factored resistance] by rational engineering analysis with confirmatory tests. Specifically, the *available strength* [factored resistance] is determined from the calculated *nominal strength* [resistance] by applying the *safety factors* or *resistance factors* evaluated in accordance with Section K2.1.1(b);
(c) *Available strength* [factored resistance] or stiffness by rational engineering analysis based on appropriate theory and engineering judgment. Specifically, the *available strength* [factored resistance] is determined from the calculated *nominal strength* [resistance] by applying the following *safety factors* or *resistance factors*:

For members
\[ \Omega = 2.00 \ (ASD) \]
\[ \phi = 0.80 \ (LRFD) \]
\[ = 0.75 \ (LSD) \]

For connections
\[ \Omega = 3.00 \ (ASD) \]
\[ \phi = 0.55 \ (LRFD) \]
\[ = 0.50 \ (LSD) \]

When rational engineering analysis is used in accordance with Section A1.2(b) or A1.2(c) to determine the *nominal strength* [resistance] for a limit state already provided in this *Specification*, the *safety factor* shall not be less than the applicable *safety factor* (\( \Omega \)), nor shall the *resistance factor* exceed the applicable *resistance factor* (\( \phi \)) for the prescribed limit state.

### A1.3 Definitions

In this *Specification*, “shall” is used to express a mandatory requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the *Specification*; and “is permitted” is used to express an option or that which is permissible within the limits of the *Specification*. In standards developed by the CSA Group, “is permitted” is expressed by “may.”

The following terms are italicized when they appear in the *Specification*. Definitions listed under the *ASD* and *LRFD* Terms sections shall apply to the USA and Mexico, while definitions listed under the *LSD* Terms section shall apply in Canada.

Terms designated with * are usually qualified by the type of *load effect*; for example, *nominal tensile strength*, *available compressive strength*.

Terms designated with + are common AISC-AISI terms that are coordinated between the two standards developers.

### General Terms

*Applicable Building Code*+. Building code under which the structure is designed.
*Bearing*+. In a *connection*, limit state of shear forces transmitted by the mechanical fastener to the *connection elements*.
*Bearing (Local Compressive Yielding)*+. Limit state of local compressive *yielding* due to the action
of a member bearing against another member or surface.

**Block Shear Rupture**+. In a connection, limit state of tension rupture along one path and shear yielding or shear rupture along another path.

**Braced Frame**+. Essentially vertical truss system that provides resistance to lateral loads and provides stability for the structural system.

**Buckling**+. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

**Buckling Strength**+. Nominal strength [resistance] for instability limit states.

**Cold-Formed Steel Structural Member**+. Shape 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 in accordance with this Specification or its specific references, or rational engineering analysis, in order to compare actual to calculated performance.

**Connection**+. Combination of structural elements and joints used to transmit forces between two or more members.

**Cross-Sectional Area:**

- **Effective Area.** Effective area, $A_{ef}$, calculated using the effective widths of component elements in accordance with Appendix 1. If the effective widths of all component elements, determined in accordance with Appendix 1, are equal to the actual flat widths, it equals the gross or net area, as applicable.

- **Full, Unreduced Area.** Full, unreduced area, $A$, calculated without considering local buckling in the component elements, which equals either the gross area or 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.

**Curtain Wall Stud.** A member in a steel-framed exterior wall system that transfers transverse (out-of-plane) loads and is limited to a superimposed axial load, exclusive of sheathing materials, of not more than 100 lb/ft (1460 N/m or 1.49 kg/cm), or a superimposed axial load of not more than 200 lbs (890 N or 90.7 kg) per stud.

**Diaphragm**+. Roof, floor, or other membrane or bracing system that transfers in-plane forces to the lateral force-resisting system.

**Direct Analysis Method.** Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of members by reducing stiffness and applying notional loads in a second-order analysis.

**Direct Strength Method.** A design method that provides predictions of member strengths without the use of effective widths.

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

**Effective Length.** Length of an otherwise identical column of the same strength when analyzed
with pinned end conditions.

**Effective Length Factor, K.** Ratio between the *effective length* and the *unbraced length* of the member.

**Effective Length Method.** A method of design that addresses stability through calculation of *available strength* [factored resistance] using the *effective length factor*.

**Effective Width Method.** A method that considers the local buckling of cold-formed steel members by reducing the gross cross-section under a non-linear stress distribution to an effective cross-section under a simplified linear stress distribution.

**Factored Load**. Product of a load factor and the nominal load [specified load].

**Fatigue**. *Limit state* of crack initiation and growth resulting from repeated application of live loads.

**First-Order Analysis.** Structural analysis in which equilibrium conditions are formulated on the undeformed structure; second-order effects are neglected.

**Flange of a Section in Bending (Flange).** 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.

**Flexural Buckling**. Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

**Flexural-Torsional Buckling**. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

**Girt**. Horizontal *structural member* that supports wall panels and is primarily subjected to bending under horizontal loads, such as wind load.

**In-Plane Instability**. *Limit state* involving buckling in the plane of the frame or the member.

**Instability**. *Limit state* reached in the loading of a structural component, frame, or structure in which a slight disturbance in the loads or geometry produces large displacements.

**Joint**. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and the method of force transfer.

**Lateral-Torsional Buckling**. Buckling mode of a flexural member involving deflection out of the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

**Limit State**. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength [resistance] limit state).

**Load**. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

**Load Effect**. Forces, stresses, and deformations produced in a structural component by the applied loads.

**Load Factor.** A factor defined by the applicable building code to take into account the variability in loads and the analysis of their effects.

**Local Bending**. *Limit state* of large deformation of a flange under a concentrated transverse force.
Local Buckling. Limit state of buckling of a compression element where the line junctions between elements remain straight and angles between elements do not change.

Local Yielding*. Yielding that occurs in a local area of an element.

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

Moment Frame*. Framing system that provides resistance to lateral loads and provides stability to the structural system primarily by shear and flexure of the framing members and their connections.

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.

Non-symmetric Section. Section not symmetric about either an axis or a point.

Notional Load. Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Out-of-Plane Buckling*. Limit state of a beam, column, or beam-column involving lateral or lateral-torsional buckling.

Patterned hole. Repeated pattern of holes along the longitudinal axis of a member, excluding those holes in the corners of a cross-section.

Performance Test. Test made on structural members, connections, and assemblies whose performance cannot be determined in accordance with Chapters A through J and L through M of this Specification or its specific references.

Permanent Load*. Load in which variations over time are rare or of small magnitude. All other loads are variable loads.

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

Power-Actuated Fastener (PAF). Hardened steel fastener driven through steel members into embedment material using either powder cartridges or compressed gas as the energy-driving source.

Power-Actuated Fastener Point. Portion of pointed end of PAF shank with varying diameter.

Published Specification. Requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (a) are generally available in the public domain or are available to the public upon request, (b) are established before the steel is ordered, and (c) as a minimum, specify minimum mechanical properties, chemical composition limits, and, if coated sheet, coating properties.

Purlin*. Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind, or dead loads.

P-δ Effect. Effect of loads acting on the deflected shape of a member between joints or nodes.

P-∆ Effect. Effect of loads acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.

Rational Engineering Analysis*. Analysis based on theory that is appropriate for the situation, any relevant test data, if available, and sound engineering judgment.
Resistance Factor, $\phi^+$. Factor that accounts for unavoidable deviations of the nominal strength [resistance] from the actual strength and for the manner and consequences of failure.

Rupture Strength$. Strength limited by breaking or tearing of members or connecting elements.

Second-Order Analysis. Structural analysis in which equilibrium conditions are formulated on the deformed structure; second-order effects (both $P-\delta$ and $P-\Delta$ effects, unless specified otherwise) are included.

Second-Order Effect. Effect of loads acting on the deformed configuration of a structure; includes $P-\delta$ effect and $P-\Delta$ effect.

Service Load$. Load under which serviceability limit states are evaluated.

Serviceability Limit State$. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery, under normal usage.

Shear Buckling$. Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear Wall$. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

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

Specified Minimum Yield Stress$. Lower limit of yield stress specified for a material as defined by ASTM.

Stability. Condition in the loading of a structural component, frame, or structure in which a slight disturbance in the loads or geometry does not produce large displacements.

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.

Stiffness. Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

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

Structural Analysis$. Determination of load effects on members and connections based on principles of structural mechanics.

Structural Component$. Member, connector, connecting element, or assemblage.

Structural Members. See the definition of Cold-Formed Steel Structural Member.

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 (of Material)$®. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tension and Shear Rupture$. In a bolt or other type of mechanical fastener, limit state of rupture due to simultaneous tension and shear force.

Thickness. The thickness, $t$, of any element or section is the base steel thickness, exclusive of coatings.

Top Arc Seam Sidelap Weld. Arc seam weld applied to the top sidelap connection.

Top Sidelap Connection. A connection formed by a vertical sheet leg (edge stiffener of deck)
inside an overlapping sheet hem, or by vertical sheet legs back-to-back.

Torsional Buckling. Buckling mode in which a compression member twists about its shear center axis.

Unbraced length. Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Unstiffened Compression Elements. Flat compression element stiffened at only one edge parallel to the direction of stress.

Variable Load. Load not classified as permanent load.

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 stress, tensile strength, and elongation.

Wall Diaphragm. A wall, load-bearing or non-load-bearing, designed to resist forces acting in the plane of the wall (commonly referred to as a “vertical diaphragm” or “shear wall”).

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.

Web Crippling. Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

Yield Moment. In a member subjected to bending, the moment at which the extreme outer fiber first attains the yield stress.

Yield Point. First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield Strength. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield Stress. Generic term to denote either yield point or yield strength, as appropriate for the material.

Yielding. Limit state of inelastic deformation that occurs when the yield stress is reached.

Yielding (Plastic Moment). Yielding throughout the cross-section of a member as the bending moment reaches the plastic moment.

Yielding (Yield Moment). Yielding at the extreme fiber on the cross-section of a member when the bending moment reaches the yield moment.

ASD and LRFD Terms (United States and Mexico):

ASD (Allowable Strength Design). Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD Load Combination. Load combination in the applicable building code intended for allowable strength design (allowable stress design).

Allowable Strength. Nominal strength divided by the safety factor, \( R_n/\Omega \).

Available Strength. Design strength or allowable strength as appropriate.

Design Earthquake. The ground motion represented by the design response spectrum as specified in the applicable building code.

Design Load. Applied load determined in accordance with either LRFD load combinations or
ASD load combinations, whichever is applicable.

Design Strength**: Resistance factor multiplied by the nominal strength, $\phi R_n$.

LRFD (Load and Resistance Factor Design)**: Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD Load Combination**: Load combination in the applicable building code intended for strength design (Load and Resistance Factor Design).

Nominal Load**: The magnitudes of the load specified by the applicable building code.

Nominal Strength**: Strength of a structure or component (without the resistance factor or safety factor applied) to resist the load effects, as determined in accordance with this Specification.

Required Strength**: Forces, stresses, and deformations acting on a structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as appropriate, or as specified by this Specification.

Resistance. See the definition of Nominal Strength.

Risk Category. A categorization of buildings and other structures for determination of flood, wind, snow, ice, and earthquake loads based on the risk associated with unacceptable performance.

Safety Factor, $\Omega$**: Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic Design Category (SDC). A classification assigned by the applicable building code to a structure based on its risk category and the severity of the design earthquake ground motion at the site.

Span Continuity. Ability of a member to develop moment over a support.

Strength Limit State**: Limiting condition, in which the maximum strength of a structure or its components is reached.

**LSD Terms (Canada):**

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

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.

Nominal Resistance (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.

Specified Loads. The magnitudes of the loads specified by the applicable building code, not including load factors.

**A1.4 Units of Symbols and Terms**

Any compatible system of measurement units is permitted to be used in the Specification, except where explicitly stated otherwise. The unit systems considered in those sections shall include U.S. customary units (force in kilopounds and length in inches), SI units (force in
Newton and length in millimeters), and MKS units (force in kilograms and length in centimeters).

**A2 Referenced Specifications, Codes, and Standards**

The following documents or portions thereof are referenced in this Specification and shall be considered part of the requirements of this Specification. Country-specific codes and standards are listed in Section A2.1 for the United States and Mexico, and Section A2.2 for Canada.

1. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800, Washington, DC 20001:
   - AISI S240-15, North American Standard for Cold-Formed Steel Structural Framing
   - AISI S310-16, North American Standard for the Design of Profiled Steel Diaphragm Panels
   - AISI S400-15, North American Standard for Seismic Design of Cold-Formed Steel Structural Systems
   - AISI S901-13, Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies
   - AISI S902-13, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns
   - AISI S903-13, Standard Method for Determination of Uniform and Local Ductility
   - AISI S904-13, Standard Test Methods for Determining the Tensile and Shear Strength of Screws
   - AISI S905-13, Test Standard for Cold-Formed Steel Connections
   - AISI S906-13, Standard Procedures for Panel and Anchor Structural Tests
   - AISI S907-13, Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms
   - AISI S909-13, Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Beams
   - AISI S910-13, Test Method for Distortional Buckling of Cold-Formed Steel Hat-Shaped Compression Members
   - AISI S911-13, Method for Flexural Testing Cold-Formed Steel Hat-Shaped Beams
   - AISI S912-13, Test Procedure for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection
   - AISI S913-13, Test Standard for Hold-Downs Attached to Cold-Formed Steel Structural Framing
   - AISI S914-15, Test Standard for Joist Connectors Attached to Cold-Formed Steel Structural Framing
   - AISI S915-15, Test Standard for Through-the-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors
   - AISI S916-15, Test Standard for Cold-Formed Steel Framing—Nonstructural Interior Partition Walls With Gypsum Board

2. American Society of Mechanical Engineers (ASME), Two Park Avenue, New York, NY 10016-5990:
   - ASME B46.1-2009, Surface Texture, Surface Roughness, Waviness, and Lay

3. ASTM International (ASTM), 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959:
   - ASTM A36/A36M-14, Standard Specification for Carbon Structural Steel
ASTM A194/A194M-15a, Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both
ASTM A242/A242M-13, Standard Specification for High-Strength Low-Alloy Structural Steel
ASTM A283/A283M-13, Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
ASTM A307-14, Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength
ASTM A354-11, Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
ASTM A370-15, Standard Test Methods and Definitions for Mechanical Testing of Steel Products
ASTM A449-14, Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use
ASTM A500/A500M-13, Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A529/A529M-14, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality
ASTM A572/A572M-15, Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
ASTM A653/A653M-15, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process
ASTM A792/A792M-10(2015), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process
ASTM A847/A847M-14, Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing With Improved Atmospheric Corrosion Resistance
ASTM A875/A875M-13, Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process
ASTM A1003/A1003M-15, Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members
ASTM A1008/A1008M-15, Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, Solution Hardened, and Bake Hardenable
ASTM A1011/A1011M-15, Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy With Improved Formability, and Ultra-High Strength
ASTM A1039/A1039M-13, Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial, Structural, and High-Strength Low-Alloy, Produced by the Twin-Roll Casting Process
ASTM A1058-14, Standard Test Methods for Mechanical Testing of Steel Products—Metric
ASTM A1063/A1063M-11a, Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process
ASTM F436-11, Standard Specification for Hardened Steel Washers
ASTM F436M-11, Standard Specification for Hardened Steel Washers (Metric)
ASTM F844-07a(2013), Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use
ASTM F959-15, Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use With Structural Fasteners
ASTM F959M-13, Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use With Structural Fasteners (Metric)
ASTM F3125-15, Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions

User Note:
ASTM F3125 is an umbrella standard including Grades A325, A325M, A490, and A490M, which were previously separate standards.

4. CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada, M9W 1R3:
   G40.20-13/G40.21-13, General requirements for rolled or welded structural quality steel/Structural quality steel

5. Factory Mutual, Corporate Offices, 270 Central Avenue, Johnston, RI 02919-4949:
   FM 4471, Approval Standard for Class 1 Metal Roofs, 2010

6. Rack Manufacturers Institute, 8720 Red Oak Boulevard, Suite 201, Charlotte, NC 28217-3996:

7. Steel Deck Institute, P.O. Box 25, Fox River Grove, IL 60021-0025
   ANSI/SDI C-2011, Standard for Composite Steel Floor Deck – Slabs

8. U. S. Army Corps of Engineers, 441 G Street NW, Washington, DC 20314-1000:

A2.1 Referenced Specifications, Codes, and Standards for United States and Mexico

1. American Concrete Institute (ACI), 38800 Country Club Dr., Farmington Hills, MI 48331:
   ACI 318-14, Building Code Requirements for Structural Concrete

2. American Institute of Steel Construction (AISC), 130 East Randolph Street, Suite 2000, Chicago, IL 60601-6219:
   ANSI/AISC 360-10, Specification for Structural Steel Buildings

3. American Iron and Steel Institute (AISI), 25 Massachusetts Avenue, NW, Suite 800,
Chapter A, General Provisions

Washington, DC 20001:
AISI S908-13, *Base Test Method for Purlins Supporting a Standing Seam Roof System*

4. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston, VA 20191:
ASCE/SEI 7-10 Including Supplement No. 1, *Minimum Design Loads in Buildings and Other Structures*

5. American Welding Society (AWS), 8669 NW 36 Street, # 130, Miami, FL 33166-6672:
AWS D1.1/D1.1M-2010, *Structural Welding Code–Steel*
AWS D1.3-2008, *Structural Welding Code–Sheet Steel*
AWS C1.1/C1.1M-2012, *Recommended Practices for Resistance Welding*

6. ASTM International (ASTM), 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959:
ASTM A924/A924M-14, *Standard Specification for General Requirements for Steel Sheet, Metallic-Coated by the Hot Dip Process*

7. Steel Deck Institute, P.O. Box 25, Fox River Grove, IL 60021:
ANSI/SDI C-2011, *Standard for Composite Steel Floor Deck - Slabs*

A2.2 Referenced Specifications, Codes, and Standards for Canada

1. CSA Group, 178 Rexdale Boulevard, Toronto, Ontario, Canada, M9W 1R3:
CAN/CSA A23.3-14, *Design of Concrete Structures*
S16-14, *Design of steel structures*
W47.1-09 (R2014), *Certification of companies for fusion welding of steel*
W55.3-08 (R2013), *Certification of companies for resistance welding of steel and aluminum*
W59-13, *Welded steel construction (metal arc welding)*

2. National Research Council of Canada (NRC), 1200 Montreal Road, Bldg. M-58, Ottawa, Ontario, Canada, K1A 0R6:
*National Building Code of Canada (NBCC), 2015*

A3 Material

This *Specification* requires the use of steels intended for structural applications as defined in general by the specifications of ASTM International listed in this section. The term SS designates structural steels and the terms HSLAS and HSLAS-F designate high-strength low-alloy steels. Steels that do not meet the requirements specified in Sections A3.1 are permitted to be used for structural applications provided Section A3.2 is met.

A3.1 Applicable Steels

This section shall apply to steels that are based on specifications providing mandatory mechanical properties and requiring test reports to confirm those properties.

Steels used in *structural members*, decks, and *connections* shall follow uses and restrictions outlined in this section and sub-sections, as applicable.
**Exception:** For steels used in composite slabs, the requirements of ANSI/SDI C shall be followed exclusively.

Applicable steels have been grouped by their minimum elongation requirements over a two-inch (50-mm) gage length.

### A3.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater

(Elongation $\geq 10\%$)

Steel grades listed below, as well as any other steel for structural applications, are permitted to be used without restriction under the provisions of this Specification provided:

(a) Ratio of tensile strength to yield stress is not less than 1.08; and

(b) The minimum elongation is greater than or equal to either 10 percent in a two-inch (50-mm) gage length or 7 percent in an eight-inch (200-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058.

The following steel grades and standards fall within this range of permitted elongations:

- ASTM A242/A242M, *Standard Specification for High-Strength Low-Alloy Structural Steel*
- ASTM A500, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*
- ASTM A529/A529M, *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*
- ASTM A572/A572M, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
- ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Class 1, Class 3 and Class 4, 55 (380) and 60 (410); HSLAS and HSLAS-F, Grades 40 (275), 50 (340), 55 (380) Class 1 and 2, 60 (410), 70 (480), and 80 (550)), *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*
  **Exception:** SS Grade 60 (410) with thicknesses less than or equal to 0.028 in. (0.71 mm) is excluded from this elongation group.
- ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), 50 (340) Class 1 and Class 4, and 60 (410)), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*
  **Exception:** Grade 60 (410) with thicknesses less than or equal to 0.028 in. (0.71 mm) is excluded from this elongation group.
- ASTM A847/A847M, *Standard Specification for 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 and HSLAS-F, Grades 50 (340), 60 (410), 70 (480), and 80 (550)), Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process


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)), Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1011/A1011M (SS Grades 30 (205), 33 (230), 36 (250) Types 1 and 2, and 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)), Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy With Improved Formability

ASTM A1039/A1039M (SS Grades 40 (275), 50 (340), 55 (380), 60 (410), 70 (480), and 80 (550); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), and 65 (450)), Standard Specification for Steel, Sheet, Hot-Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process

Exception: SS Grades 55 (380), 60 (410), 70 (480), and 80 (550) with thicknesses outside the range of 0.064 in. (1.6 mm) to 0.078 in. (2.0 mm) are excluded from this elongation group.

ASTM A1063/A1063M (SS Grades 40 (275), 50 (340); HSLAS Classes 1 and 2, Grades 45 (310), 50 (340), 55 (380), 60 (410), 65 (450)), Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process

CSA G40.20-13/G40.21-13, General requirements for rolled or welded structural quality steel/Structural quality steel

A3.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steel grades listed below, as well as any other steel for structural applications that has a minimum elongation of 3 percent in a two-inch (50-mm) gage length standard specimen tested in accordance with ASTM A370 or ASTM A1058, are permitted to be used provided that the available strengths [factored resistances] of structural members and connections are calculated in accordance with Chapters B through M (excluding welded connections in Chapter J), Appendices A and B, and Appendices 1 and 2. For the purposes of these calculations, a reduced yield stress 0.9 F_{sy} shall be used in place of F_{sy}, and a reduced tensile strength of 0.9 F_{u} shall be used in place of F_{u}.

The following steel grades and standards fall within this range of permitted elongations:

ASTM A653/A653M (SS Grades 60 (410), 70 (480), and 80 (550) Class 3), Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

Exception: SS Grade 60 (410) with thicknesses greater than 0.028 in. (0.71 mm) is
excluded from this elongation group.

ASTM A792/A792M (Grades 60 (410), 70 (480), and 80 (550) Class 3), *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*

**Exception:** Grade 60 (410) with **thicknesses** greater than 0.028 in. (0.71 mm) is excluded from this elongation group.

ASTM A1039/A1039M (SS Grades 55 (380), 60 (410), 70 (480), and 80 (550); HSLAS Classes 1 and 2, Grades 70 (480) and 80 (550)), *Standard Specification for Steel Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process*

**Exception:** SS grades with **thicknesses** greater than or equal to 0.064 in. (1.6 mm) are excluded from this elongation group.

ASTM A1063/A1063M (SS Grades 55 (380), 60 (410), 70 (480), Grade 80 (550) Class 1); (HSLAS Grade 70 (480) Classes 1 and 2, Grade 80 (550) Classes 1 and 2), *Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process*

### A3.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

Steel grades listed below, as well as other steel grades that do not meet the requirements of A3.1.1 or A3.1.2, are permitted to be used only for multiple web configurations such as roofing, siding, and floor decking provided the following adjustments are made to the design parameters:

(a) A reduced specified minimum yield stress, $R_b F_{sy}$, is used for determining the **nominal flexural strength** [resistance] in Chapter F, for which the reduction factor, $R_b$, is determined in accordance with (1) or (2):

1. For stiffened and partially stiffened compression flanges
   
   For $w/t \leq 0.067E/F_{sy}$
   
   $R_b = 1.0$
   
   For $0.067E/F_{sy} < w/t < 0.974E/F_{sy}$
   
   $R_b = 1.0 - 0.26 \left[ \frac{wF_{sy}}{tE} - 0.067 \right]^{0.4} \quad (Eq. A3.1.3-1)$
   
   For $0.974E/F_{sy} \leq w/t \leq 500$
   
   $R_b = 0.75$

2. For unstiffened compression flanges
   
   For $w/t \leq 0.0173E/F_{sy}$
   
   $R_b = 1.0$
   
   For $0.0173E/F_{sy} < w/t \leq 60$
   
   $R_b = 1.079 - 0.6 \sqrt{\frac{wF_{sy}}{tE}} \quad (Eq. A3.1.3-2)$

where

- $w$ = Flat width of compression flange
- $t$ = Thickness of section
- $E$ = Modulus of elasticity of steel
- $F_{sy}$ = Specified minimum yield stress determined in accordance with Section A3.3.1 $\leq 80$ ksi (550 MPa, or 5620 kg/cm²)

(b) The yield stress, $F_y$, used for determining **nominal strength** [resistance] in Appendix 1 and
Chapters C to J exclusive of Section F2.4 is taken as 75 percent of the specified minimum yield stress or 60 ksi (414 MPa or 4220 kg/cm²), whichever is less, and

(c) The tensile strength, $F_u$, used for determining nominal strength [resistance] in Chapter J 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 in accordance with the provisions of Section K2.1. Available strengths [factored resistances] based on these tests shall not exceed the available strengths [factored resistances] calculated in accordance with Chapters C through J, Appendices A and B, and Appendices 1 and 2, using the specified minimum yield stress, $F_{sy}$, and the specified minimum tensile strength, $F_u$.

The following steel grades and standards fall within this range of permitted elongations:

ASTM A653/A653M (SS Grade 80 (550) Classes 1 and 2), Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M (Grade 80 (550) Classes 1 and 2), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A875/A875M (SS Grade 80 (550)), Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1008/A1008M (SS Grade 80 (550)), Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy With Improved Formability, Solution Hardened, and Bake Hardenable

ASTM A1063/A1063M (SS Grade 80 (550) Class 2), Standard Specification for Steel Sheet, Twin-Roll Cast, Zinc-Coated (Galvanized) by the Hot-Dip Process

### A3.2 Other Steels

The listing in Section A3.1 shall not exclude the use of steel up to and including 1 in. (25.4 mm) in thickness, ordered or produced to other than the listed specifications, provided the following requirements are met:

(a) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other published specification. $F_y$ and $F_u$ shall be the specified minimum values as given in the specified reference specification.

(b) The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the specified reference specification including all general requirements standards cited therein.

(c) The coating properties of coated sheet shall be determined by the producer, the supplier, or the purchaser, in accordance with ASTM A924/A924M.

(d) If the steel is to be welded, its suitability for the intended welding process shall be established by the producer, the supplier, or the purchaser, in accordance with AWS D1.1, AWS D1.3 or CSA W59, as applicable.

These steels shall also meet the permitted uses and restrictions of Section A3.1, as appropriate.

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (a) through (d) in Specification Section A3.2, the
manufacturer of the cold-formed steel product shall establish that the *yield stress* and *tensile strength* of the *master coil* are at least 10 percent greater than specified in the referenced published specification.

### A3.2.1 Ductility Requirements of Other Steels

Steels not listed in Section A3.1 and used for *structural members* and *connections* in accordance with Section A3.2 shall comply with the following ductility requirements:

(a) Minimum local elongation in a 1/2-inch (12.7 mm) gage length across the fracture is 20 percent, and

(b) Minimum uniform elongation outside the fracture is three percent.

When material ductility is determined on the basis of these criteria, the use of such material shall be restricted to the design of *purlins*, *girts*, and *curtain wall studs* in accordance with Chapter F, and Sections I6.2.1, I6.2.2, and I6.3.1. *Curtain wall studs* shall also be subject to the restrictions specified in Section A3.2.1.1. For *purlins*, *girts*, and *curtain wall studs* subject to combined axial *load* and bending moment (Section H1), \( \frac{\Omega P}{P_n} \) shall not exceed 0.15 for *ASD*, \( \frac{P_u}{\phi_c P_n} \) shall not exceed 0.15 for *LRFD*, and \( \frac{P_f}{\phi_c P_n} \) shall not exceed 0.15 for *LSD*.

#### A3.2.1.1 Restrictions for Curtain Wall Studs

The use of *curtain wall studs* shall be limited to a wall assembly whose dead *load* divided by its surface area is no greater than 15 psf (0.72 kN/m² or 7.32 g/cm²) in accordance with the following:

(a) In the United States and Mexico, where the building is assigned to *Seismic Design Category* D, E, or F; and

(b) In Canada, where the building has a specified short period spectral acceleration ratio \( I_{F}F_{S}S_{A}(0.2) \) greater than 0.35, determined in accordance with the NBCC.

### A3.3 Yield Stress and Strength Increase From Cold Work of Forming

#### A3.3.1 Yield Stress

The *yield stress*, \( F_y \), used in design shall not exceed the *specified minimum yield stress* of steels as listed in Section A3.1, as established in accordance with Section K2, or as increased for cold work of forming in Section A3.3.2.

#### A3.3.2 Strength Increase From Cold Work of Forming

Strength increase from cold work of forming is permitted by substituting \( F_{ya} \) for \( F_y \), where \( F_{ya} \) is the average *yield stress* of the full section. Such increase shall be limited to Chapters D, E, F (excluding Section F2.4), Sections H1, I4, and I6.2 and to sections not subject to strength reduction from *local buckling*. The limits and methods for determining \( F_{ya} \) shall be in accordance with (a), (b) and (c).
(a) The design yield stress, \( F_{ya} \), of the steel shall be determined on the basis of one of the following methods:

1. Full section tensile tests [see paragraph (a) of Section K2.3.1],
2. Stub column tests [see paragraph (b) of Section K2.3.1],
3. Computed in accordance with Eq. A3.3.2-1:

\[
F_{ya} = CF_{yc} + (1 - C) F_{yf} \leq F_{uv}\]

where

\( F_{ya} \) = Average yield stress of full unreduced section of compression members or full flange sections of flexural members

\( C \) = 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

\( F_{yc} \) = \( B_c F_{yv} / (R/t)^m \), tensile yield stress of corners

Eq. A3.3.2-2 applies only when \( F_{uv}/F_{yv} \geq 1.2 \), \( R/t \leq 7 \), and the included angle \( \leq 120^\circ \)

where

\( B_c = 3.69 \left( F_{uv}/F_{yv} \right) - 0.819 \left( F_{uv}/F_{yv} \right)^2 - 1.79 \) (Eq. A3.3.2-3)

\( F_{yv} \) = Tensile yield stress of virgin steel specified by Section A3 or established in accordance with Section K2.3.3

\( R \) = Inside bend radius

\( t \) = Thickness of section

\( m = 0.192 \left( F_{uv}/F_{yv} \right) - 0.068 \) (Eq. A3.3.2-4)

\( F_{uv} \) = Tensile strength of virgin steel specified by Section A3 or established in accordance with Section K2.3.3

\( F_{yf} \) = Weighted average tensile yield stress of flat portions established in accordance with Section K2.3.2 or virgin steel yield stress if tests are not made

(b) For axially loaded tension members, the yield stress 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.
B. DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of cold-formed steel structural members, assemblies, and systems applicable to the whole Specification.

The chapter is organized as follows:

B1 General Provisions
B2 Loads and Load Combinations
B3 Design Basis
B4 Dimensional Limits and Considerations
B5 Member Properties
B6 Fabrication and Erection (reserved)
B7 Quality Control and Quality Assurance
B8 Evaluation of Existing Structures (reserved)

B1 General Provisions

The design of structural members and connections shall be consistent with the intended behavior of cold-formed steel structures and the assumptions made in the structural analysis.

B2 Loads and Load Combinations

Loads and load combinations shall be as stipulated by the applicable building code.

Where no building code is stipulated, the loads, load combinations, and nominal loads [specified loads] shall be those stipulated as follows:

(a) In the United States and Mexico, ASCE/SEI 7, Minimum Design Loads for Buildings and Other Structures; and

(b) In Canada, National Building Code of Canada.

B3 Design Basis

No applicable strength or serviceability limit state shall be exceeded when the structure is subjected to the applicable load combinations.

Design shall be in accordance with the following methods:

(a) ASD, LRFD, or a combination of ASD and LRFD — the United States and Mexico; and

(b) LSD — Canada.

B3.1 Required Strength [Effect Due to Factored Loads]

The required strength [effect due to factored loads] of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Section B2.

The required strength [effect due to factored loads] shall be noted as follows:

\[ R = \begin{cases} \text{Required strength [effect due to factored loads]} \\
R \text{ in accordance with ASD load combinations} \\
R_u \text{ in accordance with LRFD load combinations} \\
R_f \text{ in accordance with LSD load combinations} \end{cases} \]
### B3.2 Design for Strength

*Structural members* and their *connections* shall be designed to have strength such that the *available strength* [factored resistance], $R_a$, equals or exceeds the *required strength* [effect due to factored loads], $\bar{R}$.

Design for strength shall be in accordance with:
- (a) Section B3.2.1 for the *Allowable Strength Design (ASD)*,
- (b) Section B3.2.2 for the *Load and Resistance Factor Design (LRFD)*, or
- (c) Section B3.2.3 for the *Limit States Design (LSD)*.

#### B3.2.1 Allowable Strength Design (ASD) Requirements

The design shall be performed in accordance with Eqs. B3.2.1-1 and B3.2.1-2:

$$R \leq R_a \quad (Eq. \text{ B3.2.1-1})$$

$$R_a = \frac{R_n}{\Omega} \quad (Eq. \text{ B3.2.1-2})$$

where

- $R$ = *Required strength*
- $R_a$ = *Allowable strength*
- $R_n$ = *Nominal strength* specified in Chapters C through K, and M
- $\Omega$ = *Safety factor* specified in Chapters C through K, and M

All provisions of this *Specification* shall apply, except for those provisions that are designated specifically for LRFD or LSD.

#### B3.2.2 Load and Resistance Factor Design (LRFD) Requirements

The design shall be performed in accordance with Eqs. B3.2.2-1 and B3.2.2-2:

$$R_u \leq R_a \quad (Eq. \text{ B3.2.2-1})$$

$$R_a = \phi R_n \quad (Eq. \text{ B3.2.2-2})$$

where

- $R_u$ = *Required strength*
- $R_a$ = *Design strength*
- $\phi$ = *Resistance factor* specified in Chapters C through K, and M
- $R_n$ = *Nominal strength* specified in Chapters C through K, and M

All provisions of this *Specification* shall apply, except for those provisions that are designated specifically for ASD or LSD.

#### B3.2.3 Limit States Design (LSD) Requirements

The design shall be performed in accordance with Eqs. B3.2.3-1 and B3.2.3-2:

$$R_a \geq R_f \quad (Eq. \text{ B3.2.3-1})$$

$$R_a = \phi R_n \quad (Eq. \text{ B3.2.3-2})$$

where

- $R_a$ = *Factored resistance*
- $R_f$ = *Effect of factored loads*
\[ \phi = \text{Resistance factor specified in Chapters C through K, and M} \]
\[ R_n = \text{Nominal resistance specified in Chapters C through K, and M} \]

All provisions of this Specification shall apply, except for those provisions that are designated specifically for ASD or LRFD.

B3.3 Design for Structural Members

The available strength [factored resistance] of cold-formed steel structural members that meet the geometric and material limitations provided in Section B4 shall be determined in accordance with Chapters D, E, F, G, and H, as applicable, with the safety and resistance factors provided in the corresponding sections. Cold-formed steel structural members outside the limitations provided in Section B4 are permitted to be designed in accordance with Section A1.2.

B3.4 Design for Connections

Connection elements shall be designed in accordance with the provisions of Chapter J. The forces and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in structural analysis. Self-limiting inelastic deformations of the connections are permitted. At the points of support, beams and trusses shall be restrained against rotation about their longitudinal axis unless other means of restraints against rotation are provided.

B3.4.1 Design for Anchorage to Concrete

Cold-formed steel to concrete anchorage shall be designed according to the applicable building code. For cast-in-place or post-installed anchors, connection strength controlled by cold-formed steel members or connector components shall be designed in accordance with the provisions of Section J3.

B3.5 Design for Stability

Stability of a structural system and its members shall be determined in accordance with Chapter C.

B3.6 Design of Structural Assemblies and Systems

Cold-formed steel assemblies and systems including diaphragms and collectors shall be designed for load effects that result from loads as stipulated in Section B2. Structural assemblies and systems shall be designed in accordance with the provisions of Chapter I, and in accordance with the provisions of Chapters C through H, and J through M, as applicable.

B3.7 Design for Serviceability

A structure shall be designed to perform its required functions during its expected life. Serviceability limit states shall be chosen based on the intended function of the structure and shall be evaluated using realistic loads and load combinations. The serviceability determination shall be in accordance with Chapter L.
B3.8 Design for Ponding

The roof system shall be investigated through rational engineering analysis to ensure strength and stability under ponding conditions, unless the roof surface is configured to prevent the accumulation of water.

B3.9 Design for Fatigue

Fatigue shall be considered in accordance with Chapter M for cold-formed steel structural members and their connections subject to repeated loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure. Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components.

B3.10 Design for Corrosion Effects

Where corrosion may impair the strength or serviceability of a structure, structural components shall be protected against corrosion or shall be designed to tolerate corrosion.

B4 Dimensional Limits and Considerations

Either the Effective Width Method or the Direct Strength Method shall be equally acceptable. When the Effective Width Method or the Direct Strength Method presented in Chapters E through H is used, the limitations detailed in Section B4.1 shall be met in order to use the safety and resistance factors provided in Chapters E through H. Members that do not meet the limits of B4.1 shall follow Section B4.2 for determination of the safety factor, $\Omega$, or resistance factor, $\phi$.

B4.1 Limitations for Use of the Effective Width Method or the Direct Strength Method

Members designed in accordance with the Effective Width Method or the Direct Strength Method and employing the safety factor, $\Omega$, or resistance factor, $\phi$, of Chapters E though H shall fall within the dimensional limitations of Table B4.1-1.


<table>
<thead>
<tr>
<th>Criteria</th>
<th>Limiting Variables(^a)</th>
<th>Effective Width Method</th>
<th>Direct Strength Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened element in compression</td>
<td>(w/t)</td>
<td>(\leq 500)</td>
<td>(\leq 500)</td>
</tr>
<tr>
<td>Edge-stiffened element in compression</td>
<td>(b/t)</td>
<td>(\leq 90) for (I_s \geq I_a)</td>
<td>(\leq 160)</td>
</tr>
<tr>
<td>Unstiffened element in compression</td>
<td>(d/t)</td>
<td>(\leq 60)</td>
<td>(\leq 60)</td>
</tr>
<tr>
<td>Stiffened element in bending (e.g. a web)</td>
<td>(h/t)</td>
<td>(\leq 200) for unstiffened web</td>
<td>(\leq 300)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\leq 260) for bearing stiffener(^c)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(\leq 300) for bearing and intermediate stiffener(^c)</td>
<td></td>
</tr>
<tr>
<td>Inside bend radius</td>
<td>(R/t)</td>
<td>(\leq 10^d)</td>
<td>(\leq 20)</td>
</tr>
<tr>
<td>Simple edge stiffener length/width ratio</td>
<td>(d_o/b_o)</td>
<td>(\leq 0.7)</td>
<td>(\leq 0.7)</td>
</tr>
<tr>
<td>Edge stiffener type</td>
<td></td>
<td>Simple only</td>
<td>Simple and complex</td>
</tr>
<tr>
<td>Maximum number of intermediate stiffeners in w</td>
<td>(n_f)</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Maximum number of intermediate stiffeners in b</td>
<td>(n_{fe})</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Number of intermediate stiffeners in h</td>
<td>(n_w)</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>Nominal yield stress</td>
<td>(F_y)</td>
<td>(&lt; 80) ksi (552 MPa)(^e)</td>
<td>(&lt; 95) ksi (655 MPa)(^e)</td>
</tr>
</tbody>
</table>

Note:

\(^a\) Variable definitions:
- \(w\) = Flat width of stiffened compression element (disregard intermediate stiffeners)
- \(t\) = Thickness of element
- \(b\) = Flat width of element with edge stiffeners (disregard intermediate stiffeners)
- \(b_o\) = Out-to-out width of element with edge stiffeners (disregard intermediate stiffeners)
- \(d\) = Flat width of unstiffened element (disregard intermediate stiffeners)
- \(d_o\) = Out-to-out width of unstiffened element (disregard intermediate stiffeners)
- \(h\) = Depth of flat portion of web measured along plane of web (disregard intermediate stiffeners)
- \(R\) = Inside bend radius
- \(n_f\) = Number of intermediate stiffeners in stiffened compression element
- \(n_{fe}\) = Number of intermediate stiffeners in edge-stiffened element
- \(n_w\) = Number of intermediate stiffeners in stiffened element under stress gradient (e.g. web)
- \(F_y\) = Nominal yield stress

\(^b\) Stiffened compression elements with \(w/t > 250\) and unstiffened compression elements with \(d/t > 30\) are likely to have noticeable deformations prior to developing their full strength.

\(^c\) Bearing and intermediate stiffener requirements in accordance with Section F5.1.

\(^d\) For inside bend \(R/t\) ratios larger than 10, rational engineering analysis is permitted.

\(^e\) See Section A3 for additional limitations.
B4.2 Members Falling Outside the Applicability Limits

Members that fall outside of the geometric and material limitations given in Section B4.1 shall be subjected to the provisions of Section A1.2, with the exception that members are permitted to be designed using the Direct Strength Method provided the safety factor, \( \Omega \), and resistance factor, \( f \), are determined using (a) or (b), as follows:

(a) Use the safety factor, \( \Omega \), or resistance factor, \( f \), determined by the rational engineering analysis clause of Section A1.2(c).

(b) Use the existing safety factor, \( \Omega \), or resistance factor, \( f \), in Chapters E through H if in an analysis of test data using Section K2, the predicted resistance factor, \( f \), from Section K2 provides an equal or higher \( f \) than that used in Chapters E through H.

In the provisions of Section K2, the professional factor, \( P \), shall be the test-to-predicted ratio, where the prediction is that of the Direct Strength Method; \( P_m \) is the mean of \( P \); and \( V_P \) is the coefficient of variation of \( P \). If \( V_P \) is less than or equal to 15 percent, \( C_p \) is permitted to be set to 1.0. At least three tests shall be conducted.

B4.3 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 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 B4.3-1.
Table B4.3-1
Short Span, Wide Flanges – Maximum Allowable Ratio of Effective Design Width (b) to Actual Width (w)

<table>
<thead>
<tr>
<th>L/w_f</th>
<th>Ratio b/w</th>
<th>L/w_f</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 the 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 the 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.

B5 Member Properties

Properties of cross-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 used in determining member strengths 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 determined in accordance with Appendix 1, is required.

The section properties used in design for serviceability shall be determined in accordance with Chapter L.

B6 Fabrication and Erection

(Reserved)

B7 Quality Control and Quality Assurance

B7.1 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold-formed steel 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 thickness is permitted at bends, such as corners, due to cold-forming effects.

B8 Evaluation of Existing Structures

(Reserved)
C. DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for stability. The chapter is organized as follows:

C1 Design for System Stability
C2 Member Bracing

C1 Design for System Stability

This chapter addresses requirements for the elastic design of structures for stability. System stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered:

(a) Flexural, shear, and axial member deformations, and all other component and connection deformations that contribute to displacements of the structure;

(b) Second-order effects (including P-Δ and P-δ effects);

(c) Geometric imperfections;

(d) Stiffness reductions due to inelasticity, including the effect of residual stresses and partial yielding of the cross-section;

(e) Stiffness reductions due to cross-section deformations or local and distortional buckling;

(f) Uncertainty in system, member, and connection stiffness and strength.

All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations, LSD load combinations, or 1.6 times ASD load combinations.

Any rational method of design for stability that considers all of the listed effects is permitted, including the methods identified in Section C1.1, C1.2, or C1.3 within the limitations stated therein.

C1.1 Direct Analysis Method Using Rigorous Second-Order Elastic Analysis

The direct analysis method of design, which consists of the calculation of required strengths [effects due to factored loads] in accordance with Section C1.1.1 and the calculation of available strengths [factored resistance] in accordance with Section C1.1.2, is permitted for all systems.

C1.1.1 Determination of Required Strengths

For the direct analysis method of design, the required strengths [effects due to factored loads] of components of the structure shall be determined from an analysis conforming to Section C1.1.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.1.1.2 and adjustments to stiffness in accordance with Section C1.1.1.3.

C1.1.1.1 Analysis

It is permitted to use any elastic analysis method capable of explicit consideration of the P-Δ and P-δ effects by capturing the effects of system and member displacements, respectively, on member forces.

Alternatively, it is permitted to use any elastic analysis method capable of explicit
consideration of the P-∆ effects by capturing the effects of system displacements on member forces. The required flexural strength [effect due to factored loads], $\overline{M}$, shall then be taken as the moment resulting from such an analysis amplified by $B_1$, where $B_1$ is determined in accordance with Section C1.2.1.1.

**C1.1.1.2 Consideration of Initial Imperfections**

Initial imperfections at the points of member intersection shall be considered as provided by either (a) or (b) below. Additionally, it is permitted, but not required, to consider imperfections in the initial position of points along members.

(a) Direct Geometric Consideration of Initial Imperfections:

In all cases, it is permitted to account for the effect of initial imperfections by including the imperfections directly in the analysis. The structure shall be analyzed with points of intersection of members displaced from their nominal locations. The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

In the analysis of structures that support gravity loads primarily through nominally vertical columns, walls, or frames, where the ratio of maximum second-order elastic analysis story drift to maximum first-order elastic analysis story drift (both determined for LRFD or LSD load combinations or 1.6 times ASD load combinations, with stiffnesses as specified in Section C1.1.1.3) in all stories is equal to or less than 1.7, it is permissible to include initial imperfections only in the analysis for gravity-only load combinations and not in the analysis for load combinations that include applied lateral loads.

(b) Consideration of Initial Imperfections Through Application of Notional Loads:

For structures that support gravity loads primarily through nominally vertical columns, walls, or frames, it is permitted to use notional loads to represent the effects of initial imperfections in accordance with the requirements of this section. The notional load shall be applied to a model of the structure based on its nominal geometry.

(1) **Notional loads** shall be applied as lateral loads at all levels. The notional loads shall be additive to other lateral loads and shall be applied in all load combinations, except as indicated in (3), below. The magnitude of the notional loads shall be:

$$N_i = \left(\frac{1}{240}\right)\alpha Y_i$$  \hspace{1cm} (Eq. C1.1.1.2-1)

where

- $\alpha = 1.0$ (LRFD or LSD)
- $\alpha = 1.6$ (ASD)

$N_i$ = Notional load applied at level $i$

$Y_i$ = Gravity load applied at level $i$ from LRFD, LSD, or ASD load combinations, as applicable

Where the applicable project or other quality assurance criteria stipulate a more stringent imperfection criteria, (1/240) in the above equation is permitted to be replaced by a lesser value.

(2) The notional load at any level, $N_i$, shall be distributed over that level in the same manner as the gravity load at the level. The notional loads shall be applied in the direction that provides the greatest destabilizing effect.
(3) For structures in which the ratio of maximum second-order elastic analysis story drift to maximum first-order elastic analysis story drift (both determined for LRFD load combinations or LSD load combinations, or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C1.1.3) in all stories is equal to or less than 1.7, it is permitted to apply the notional load, \( N_i \), only in gravity-only load combinations and not in combinations that include other lateral loads.

### C1.1.1.3 Modification of Section Stiffness

The analysis of the structure to determine the required strengths [effects due to factored loads] of components shall use reduced stiffnesses, as follows:

(a) A factor of 0.90 shall be applied to all stiffnesses considered to contribute to the stability of the structure. Additionally, it is permitted, but not required, to also apply the stiffness reduction to those members that are not part of the lateral force resisting system.

(b) An additional factor, \( \tau_b \), shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure.

For \( \alpha \frac{\bar{P}}{P_y} \leq 0.5 \),

\[
\tau_b = 1.0 \quad (Eq. \text{ C1.1.1.3-1})
\]

For \( \alpha \frac{\bar{P}}{P_y} > 0.5 \),

\[
\tau_b = 4(\alpha \frac{\bar{P}}{P_y})[1- (\alpha \frac{\bar{P}}{P_y})] \quad (Eq. \text{ C1.1.1.3-2})
\]

where

- \( \alpha = 1.0 \) (LRFD or LSD)
- \( \alpha = 1.6 \) (ASD)

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

\( P_y = \text{Axial yield strength} \)

\( = F_y A_g \quad (Eq. \text{ C1.1.1.3-3}) \)

where

- \( F_y = \text{Yield stress} \)
- \( A_g = \text{Gross area of cross-section} \)

(c) In lieu of using \( \tau_b < 1.0 \) where \( \alpha \frac{\bar{P}}{P_y} > 0.5 \), it is permitted to use \( \tau_b = 1.0 \) for all members if a notional load of \( (1/1000)\alpha Y_i \) is applied at all levels, in the direction specified in Section C1.1.2, in all load combinations. These notional loads shall be added to those stipulated in Section C1.1.2, except that C1.1.2(3) shall not apply.

(d) Where components comprised of materials other than cold-formed steel are considered to contribute to the stability of the structure, stiffness reductions shall be applied to those components as required by the codes and specifications governing their design.

### C1.1.2 Determination of Available Strengths [Factored Resistances]

The available strengths [factored resistances] of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J, and K, as applicable, with no further consideration of overall structure stability. The flexural buckling
**effective length factors**, \( K_Y \) and \( K_X \) of all members shall be taken as unity unless a smaller value can be justified by rational engineering analysis.

Bracing intended to define the unbraced lengths of members shall have enough stiffness and strength to control member movement at the braced points, and shall be designed in accordance with Section C2.

When initial imperfections in the position of points along a member are considered in the analysis in addition to imperfections at the points of intersection as stipulated in Section C1.1.1.2, it is permissible to take the flexural buckling strength of the member in the plane of the initial imperfection as the cross-section strength. The available strengths [factored resistances] due to torsional, flexural-torsional, local, and distortional buckling of compression members shall be as specified in Chapter E.

**C1.2 Direct Analysis Method Using Amplified First-Order Elastic Analysis**

The direct analysis method of design, which consists of the calculation of required strengths [effects due to factored loads] in accordance with Section C1.2.1 and the calculation of available strengths [factored resistance] in accordance with Section C1.2.2, shall be limited to structures that support gravity loads primarily through nominally vertical columns, walls, or frames.

**C1.2.1 Determination of Required Strengths [Effects due to Factored Loads]**

For the direct analysis method of design, the required strengths [effects due to factored loads] of components of the structure shall be determined from an analysis conforming to Section C1.2.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.2.1.2 and adjustments to stiffness in accordance with Section C1.2.1.3.

**C1.2.1.1 Analysis**

The required flexural strength [moment due to factored loads], \( \overline{M} \), and required axial strength [axial force due to factored loads], \( \overline{P} \), of all members shall be determined as follows:

\[
\overline{M} = B_1 M_{nt} + B_2 \overline{M}_{lt} \quad (Eq. \ C1.2.1.1-1)
\]

\[
\overline{P} = \overline{P}_{nt} + B_2 \overline{P}_{lt} \quad (Eq. \ C1.2.1.1-2)
\]

where

\( B_1 \) = Multiplier to account for \( P-\delta \) effects, determined for each member subject to compression and flexure, and each direction of bending of the member in accordance with Eq. C1.2.1.1-3, with \( B_1 \) taken as 1.0 for members not subject to compression

\( B_2 \) = Multiplier to account for \( P-\Delta \) effects, determined for each story of the structure and each direction of lateral translation of the story using Eq. C1.2.1.1-6

\( \overline{M}_{lt} \) = Moment from first-order elastic analysis using LRFD, LSD, or ASD load combinations, as applicable, due to lateral translation of the structure only

\( \overline{M}_{nt} \) = Moment from first-order elastic analysis using LRFD, LSD, or ASD load combinations
combinations, as applicable, with the structure restrained against lateral translation

\[ \overline{M} = \text{Required second-order flexural strength [moment due to factored loads] using LRFD, LSD or ASD load combinations, as applicable} \]

\[ \overline{P}_{ft} = \text{Axial force from first-order elastic analysis using LRFD, LSD or ASD load combinations, as applicable, due to lateral translation of the structure only} \]

\[ \overline{P}_{nt} = \text{Axial force from first-order elastic analysis using LRFD, LSD or ASD load combinations, as applicable, with the structure restrained against lateral translation} \]

\[ \overline{P} = \text{Required second-order axial strength [compressive force due to factored loads] using LRFD, LSD or ASD load combinations, as applicable} \]

The \( P-\delta \) effect amplifier \( B_1 \) shall be determined in accordance with Eq. C1.2.1.1-3, in which \( \overline{P} \) shall be determined by iteration or is permitted to be taken as \( \overline{P}_{nt} + \overline{P}_{ft} \).

\[ B_1 = \frac{C_m}{(1 - \alpha \overline{P} / P_{e1})} \geq 1.0 \]  \hspace{1cm} (Eq. C1.2.1.1-3)

where

\[ \alpha = \begin{cases} 1.00 & (LRFD \text{ or LSD}) \\ 1.60 & (ASD) \end{cases} \]

\[ C_m = \text{Coefficient assuming no lateral translation of the frame determined as follows:} \]

(a) For beam-columns not subject to transverse loading between supports in the plane of bending

\[ C_m = 0.6 - 0.4(M_1/M_2) \]  \hspace{1cm} (Eq. C1.2.1.1-4)

where

\[ M_1 \text{ and } M_2 = \text{Smaller and larger moments, respectively, at the ends of that} \]
\[ \text{portion of the member unbraced in the plane of bending under} \]
\[ \text{consideration. } M_1 \text{ and } M_2 \text{ are calculated from a first-order elastic} \]
\[ \text{analysis. } M_1/M_2 \text{ is positive when the member is bent in reverse} \]
\[ \text{curvature, negative when bent in single curvature.} \]

(b) For beam-columns subject to transverse loading between supports, \( C_m \) shall be determined either by analysis or conservatively taken as 1.0 for all cases.

\[ P_{e1} = \text{Elastic critical buckling strength of the member in the plane of bending,} \]
\[ \text{calculated based on the assumption of no lateral translation at member ends} \]
\[ = \pi k_f / (K_1 L)^2 \]  \hspace{1cm} (Eq. C1.2.1.1-5)

where

\[ k_f = \text{Flexural stiffness in the plane of bending as modified in Section C1.2.1.3} \]
\[ L = \text{Unbraced length of member} \]
\[ K_1 = \text{Effective length factor for flexural buckling in the plane of bending, } K_y \text{ or } K_x, \]
\[ \text{as applicable, calculated based on the assumption of no lateral translation} \]
\[ \text{at member ends} \]
\[ = 1.0 \text{ unless analysis justifies a smaller value} \]

The \( P-\Delta \) effect amplifier \( B_2 \) for each story and each direction of lateral translation shall be calculated as follows:
\[ B_2 = \frac{1}{1 - (\alpha \bar{P}_{\text{story}} / P_{e,\text{story}})} \geq 1.0 \]  
(Eq. C1.2.1.1-6)

where

\[ \bar{P}_{\text{story}} = \text{Total vertical load supported by the story using LRFD, LSD, or ASD load combinations, as applicable, including loads in columns that are not part of the lateral force-resisting system} \]

\[ P_{e,\text{story}} = \text{Elastic critical buckling strength for the story in the direction of translation being considered, determined by sidesway buckling analysis or taken as:} \]

\[ P_{e,\text{story}} = RMH \bar{F} / \Delta_F \]  
(Eq. C1.2.1.1-7)

where

\[ R_M = 1.0 - 0.15(P_{mf} / \bar{P}_{\text{story}}) \]  
(Eq. C1.2.1.1-8)

where

\[ P_{mf} = \text{Total vertical load in columns in the story that are part of moment frames, if any, in the direction of translation being considered} \]

\[ = 0 \text{ for braced frame systems} \]

\[ H = \text{Height of story} \]

\[ \Delta_F = \text{Inter-story drift from first-order elastic analysis in the direction of translation being considered, due to story shear, } \bar{F}, \text{computed using the stiffness as required by Section C1.2.1.3} \]

\[ \bar{F} = \text{Story shear, in the direction of translation being considered, produced by the lateral forces using LRFD, LSD, or 1.6 times ASD load combinations} \]

Where \( \Delta_F \) varies over the plan area of the structure in a three-dimensional system with rigid diaphragms, it shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift in the story. In two-dimensional systems with flexible and semi-rigid diaphragms, \( \Delta_F \) shall be evaluated at each independent frame (i.e., line of resistance), or alternatively taken as the maximum drift in the story.

**C1.2.1.2 Consideration of Initial Imperfections**

Initial imperfections shall be considered as provided by Sections C1.1.1.2(a) or C1.1.1.2(b).

**C1.2.1.3 Modification of Section Stiffness**

Section stiffness modifications shall be made as required by Section C1.1.1.3.

**C1.2.2 Determination of Available Strengths [Factored Resistances]**

The available strengths [factored resistances] of members and connections shall be calculated as provided by Section C1.1.2.

**C1.3 Effective Length Method**

The use of the effective length method shall be limited to the following conditions:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls, or frames.

(b) The ratio of maximum second-order drift to maximum first-order drift (both determined
for LRFD load combinations, LSD load combinations, or 1.6 times ASD load combinations) in all stories is equal to or less than 1.5, as determined based on nominal unreduced stiffness.

### C1.3.1 Determination of Required Strengths [Effects of Factored Loads]

For the design, the required strengths [effects due to factored loads] of components of the structure shall be determined from an analysis conforming to Section C1.3.1.1. The analysis shall include consideration of initial imperfections in accordance with Section C1.3.1.2.

#### C1.3.1.1 Analysis

The analysis shall be performed in accordance with the requirements of Section C1.2.1.1, except that nominal stiffnesses shall be used in the analysis and Section C1.2.1.3 shall not apply.

#### C1.3.1.2 Consideration of Initial Imperfections

*Notional loads* shall be applied in the analysis as required by Section C1.1.1.2(b).

### C1.3.2 Determination of Available Strengths [Factored Resistances]

The available strengths [factored resistances] of members and connections shall be calculated in accordance with the provisions of Chapters D, E, F, G, H, I, J, and K, as applicable.

The flexural buckling effective length factors, $K_x$ and $K_y$, of members subject to compression shall be taken as specified in (a) or (b), below, as applicable:

(a) In braced frame systems, shear wall systems, and other structural systems where lateral stability and resistance to lateral loads do not rely on the flexural stiffness of columns, $K_x$ and $K_y$ of members subject to compression shall be taken as 1.0, unless rational engineering analysis indicates that a lower value is appropriate.

(b) In moment frame systems and other structural systems in which the flexural stiffnesses of columns are considered to contribute to lateral stability and resistance to lateral loads, $K_x$ and $K_y$, or elastic critical buckling stress, $F_{cwe}$, of those columns whose flexural stiffnesses are considered to contribute to lateral stability and resistance to lateral loads shall be determined from a sideways buckling analysis of the structure; $K_x$ and $K_y$ shall be taken as 1.0 for columns whose flexural stiffnesses are not considered to contribute to lateral stability and resistance to lateral loads.

**Exception:** It is permitted to take $K_x$ or $K_y$, as applicable, as 1.0 in the design of all columns if the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD or LSD load combinations or 1.6 times ASD load combinations) in all stories is equal to or less than 1.1.

Bracing intended to define the unbraced lengths of members shall have enough stiffness and strength to control member movement at the braced points, and shall be designed in accordance with Section C2.
C2 Member Bracing

C2.1 Symmetrical Beams and Columns

The provisions of this section shall only apply to Canada. See Section C2.1 of Appendix B.

C2.2 C-Section and Z-Section Beams

The provisions of Section C2.2.1 for bracing to restrain twisting of C-sections and Z-sections used as beams loaded in the plane of the web shall apply only when neither flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange. When only the top flange is so connected, see Section C2.2.2. Also, see Appendix B for additional requirements applicable to Canada.

Where both flanges are so connected, no further bracing is required.

C2.2.1 Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z-Section

Each intermediate brace at the top and bottom flanges of C- or Z-section members shall be designed with resistance of $P_{L1}$ and $P_{L2}$, where $P_{L1}$ is the brace force required on the flange in the quadrant with both x and y axes positive, and $P_{L2}$ is the brace force on the other flange. The x-axis shall be designated as the centroidal axis perpendicular to the web, and the y-axis shall be designated as the centroidal axis parallel to the web. The x and y coordinates shall be oriented such that one of the flanges is located in the quadrant with both positive x and y axes. See Figure C2.2.1-1 for illustrations of coordinate systems and positive force directions.

(a) For uniform loads

$$P_{L1} = 1.5\left[\bar{W}_yK' - (\bar{W}_x/2) + (\bar{M}_z/d)\right]$$  \hspace{1cm} (Eq. C2.2.1-1)  

$$P_{L2} = 1.5\left[\bar{W}_yK' - (\bar{W}_x/2) - (\bar{M}_z/d)\right]$$  \hspace{1cm} (Eq. C2.2.1-2)

When the uniform load, $\bar{W}$, acts through the plane of the web, i.e., $\bar{W}_y = \bar{W}$ and $\bar{W}_x = 0$:

$$P_{L1} = -P_{L2} = 1.5(m/d)\bar{W}$$  \hspace{1cm} (Eq. C2.2.1-3)  

$$P_{L1} = P_{L2} = 1.5\left(\frac{1}{2I_x}\right)\bar{W}$$  \hspace{1cm} (Eq. C2.2.1-4)

where

$\bar{W}_x$, $\bar{W}_y$ = Components of design load [factored load] $\bar{W}$ parallel to the x- and y-axis, respectively. $\bar{W}_x$ and $\bar{W}_y$ are positive if pointing to the positive x- and y-direction, respectively

where

$\bar{W}$ = Design load [factored load] (applied load determined in accordance with the most critical ASD, LRFD, or LSD load combinations, depending on the design method used) within a distance of 0.5a on each side of the brace

where

a = Longitudinal distance between centerline of braces
\[ K' = 0 \quad \text{for C-sections} \]
\[ = \frac{I_{xy}}{2I_x} \quad \text{for Z-sections} \quad (Eq. \ C2.2.1-5) \]

where
\[ I_{xy} = \text{Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the purlin web} \]
\[ I_x = \text{Moment of inertia of full unreduced section about x-axis} \]
\[ M_z = -\overline{W_x e_{sy}} + \overline{W_y e_{sx}}, \text{torsional moment of } \overline{W} \text{ about shear center} \]

where
\[ e_{sx}, e_{sy} = \text{Eccentricities of load components measured from the shear center and in the x- and y-directions, respectively} \]
\[ d = \text{Depth of section} \]
\[ m = \text{Distance from shear center to mid-plane of web of C-section} \]

The diagram shows the coordinate systems and positive force directions.

(b) For concentrated loads,
\[ \overline{P}_{l1} = \overline{P}_y K' - (\overline{P}_x / 2) + (M_z / d) \quad (Eq. \ C2.2.1-6) \]
\[ \overline{P}_{l2} = \overline{P}_y K' - (\overline{P}_x / 2) - (M_z / d) \quad (Eq. \ C2.2.1-7) \]

When a design load [factored load] acts through the plane of the web, i.e., \[ \overline{P}_y = \overline{P} \text{ and } \overline{P}_x = 0: \]
\[ \overline{P}_{l1} = -\overline{P}_{l2} = (m / d) \overline{P} \quad \text{for C-sections} \quad (Eq. \ C2.2.1-8) \]
\[ \overline{P}_{l1} = -\overline{P}_{l2} = \left( \frac{I_{xy}}{2I_x} \right) \overline{P} \quad \text{for Z-sections} \quad (Eq. \ C2.2.1-9) \]

where
\[ \overline{P}_x, \overline{P}_y = \text{Components of design load [factored load] } \overline{P} \text{ parallel to the x- and y-axis, respectively. } \overline{P}_x \text{ and } \overline{P}_y \text{ are positive if pointing to the positive x- and y-direction, respectively.} \]
\[ M_z = -\overline{P}_x e_{sy} + \overline{P}_y e_{sx}, \text{torsional moment of } \overline{P} \text{ about shear center} \]
\[ \overline{P} = \text{Design concentrated load [factored load] within a distance of 0.3a on each side of the brace, plus 1.4(1-l/a) times each design concentrated load [factored load] located farther than 0.3a but not farther than 1.0a from the brace. The design concentrated load [factored load] is the applied load determined in accordance} \]
with the most critical ASD, LRFD, or LSD load combinations, depending on the
design method used

where

\[ l = \text{Distance from concentrated load to the brace} \]

See Section C2.2.1(a) for definitions of other variables.

The bracing force, \( P_{L1} \) or \( P_{L2} \), is positive where restraint is required to prevent the
movement of the corresponding flange in the negative x-direction.

Where braces are provided, they shall be attached in such a manner as 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 that 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 in accordance with Section F3.

C2.2.2 Flange Connected to Sheathing That Contributes to the Strength and Stability of the
C- or Z-Section

For members with sheathing attached to the compression flange, Section I6.4.1 shall be
followed provided the conditions specified in the section are met; for members with
sheathing attached to the tension flange, Section I6.4.2 shall be followed provided the
conditions specified in the section are met.

C2.3 Bracing of Axially Loaded Compression Members

The required brace strength [brace force due to factored loads] and stiffness are permitted to
be determined by a second-order analysis in accordance with the requirements of Section C1.

Alternatively, to provide an adequate intermediate brace (or braces) that will allow an
individual concentrically loaded compression member to develop its required axial strength
[compressive axial force due to factored loads], the required strength [brace force due to factored
loads] acting on the brace (or braces) shall be calculated in accordance with Eq. C2.3-1.

\[
P_{rb} = 0.01 \ P_{ra} \quad (\text{Eq. C2.3-1})
\]

where

\[
P_{rb} = \text{Required brace strength [brace force due to factored loads] to brace a single}
\text{compression member with an axial load } P_{ra}
\]

\[
P_{ra} = \text{Required compressive axial strength [compressive axial force due to factored loads] of}
\text{individual concentrically loaded compression member to be braced, which is}
\text{calculated in accordance with ASD, LRFD, or LSD load combinations depending on}
\text{the design method used}
\]

The stiffness of each brace shall equal or exceed \( \beta_{rb} \), as calculated in Eq. C2.3-2:

For ASD

\[
\beta_{rb} = \frac{2[4-(2/n)]}{L_b} (\Omega P_{ra})
\quad (\text{Eq. C2.3-2a})
\]

\[ \Omega = 2.00 \]

For LRFD and LSD
\[ \beta_{rb} = \frac{2[4 - (2/n)]}{L_b} \left( \frac{P_{ra}}{\phi} \right) \]  

(Eq. C2.3-2b)

\[ \phi = 0.75 \text{ for } LRFD \]
\[ = 0.70 \text{ for } LSD \]

where

\( \beta_{rb} = \) Minimum required brace stiffness to brace a single compression member
\( n = \) Number of equally spaced intermediate brace locations
\( L_b = \) Distance between braces on individual concentrically loaded compression member to be braced

For braces not oriented perpendicular to the braced member, the required brace strength [brace force due to factored loads] and stiffness shall be adjusted for the angle of inclination.
D. MEMBERS IN TENSION

This chapter addresses members subjected to axial tension caused by static forces acting through the centroidal axes.

The chapter is organized as follows:

D1 General Requirements
D2 Yielding of Gross Section
D3 Rupture of Net Section

D1 General Requirements

For axially loaded tension members, the available tensile strength [factored resistance] \( \phi_t T_n \) or \( T_n/\Omega_t \) shall be the lesser of the values obtained in accordance with Sections D2 and D3, where the nominal strengths [resistance] and the corresponding safety and resistance factors are provided.

The available strengths [factored resistance] shall be determined in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

The nominal tensile strength [resistance] shall also be limited by the connection strength of the tension members, which is determined in accordance with the provisions of Chapter J.

D2 Yielding of Gross Section

The nominal tensile strength [resistance], \( T_{nY} \) due to yielding of the gross section shall be determined as follows:

\[
T_n = A_g F_y \quad (Eq. \text{ D2-1})
\]

\( \Omega_t = 1.67 \quad (ASD) \)

\( \phi_t = 0.90 \quad (LRFD) \)

\( = 0.90 \quad (LSD) \)

where

\( A_g = \text{Gross area of cross-section} \)

\( F_y = \text{Design yield stress as determined in accordance with Section A3.3.1} \)

D3 Rupture of Net Section

The nominal tensile strength [resistance], \( T_{nR} \) due to rupture of the net section shall be determined as follows:

\[
T_n = A_n F_u \quad (Eq. \text{ D3-1})
\]

\( \Omega_t = 2.00 \quad (ASD) \)

\( \phi_t = 0.75 \quad (LRFD) \)

\( = 0.75 \quad (LSD) \)

where

\( A_n = \text{Net area of cross-section} \)

\( F_u = \text{Tensile strength as specified in Section A3.1} \)
E. MEMBERS IN COMPRESSION

This chapter addresses members subjected to concentric axial compression.

This chapter is organized as follows:
E1 General Requirements
E2 Yielding and Global (Flexural, Flexural-Torsional, and Torsional) Buckling
E3 Local Buckling Interacting With Yielding and Global Buckling
E4 Distortional Buckling

Additionally, built-up compression member provisions are provided in:
I1.2 Compression Members Composed of Two Sections in Contact

E1 General Requirements

The available axial strength [factored resistance] ($\phi_c P_n$ or $P_n/\Omega_c$) shall be the smallest of the values calculated in accordance with Sections E2 to E4 where applicable.

E2 Yielding and Global (Flexural, Flexural-Torsional, and Torsional) Buckling

The nominal axial strength [resistance], $P_{ne}$, for yielding, and global (flexural, torsional, or flexural-torsional) buckling shall be calculated in accordance with this section. The applicable safety factor and resistance factors given in this section shall be used to determine the available axial strength [factored resistance] ($\phi_c P_{ne}$ or $P_{ne}/\Omega_c$) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ P_{ne} = A_g F_n \]  \hspace{1cm} (Eq. E2-1)

\[ \Omega_c = 1.80 \hspace{0.5cm} (ASD) \]

\[ \phi_c = 0.85 \hspace{0.5cm} (LRFD) \]

\[ = 0.80 \hspace{0.5cm} (LSD) \]

where

\[ A_g = \text{Gross area} \]

\[ F_n = \text{Compressive stress} \] and shall be calculated as follows:

For $\lambda_c \leq 1.5$

\[ F_n = \left(\frac{0.658^{2/3} c^2}{\lambda_c}ight) F_y \]  \hspace{1cm} (Eq. E2-2)

For $\lambda_c > 1.5$

\[ F_n = \left(\frac{0.877}{\lambda_c^2}ight) F_y \]  \hspace{1cm} (Eq. E2-3)

where

\[ \lambda_c = \sqrt{\frac{F_y}{F_{cre}}} \]  \hspace{1cm} (Eq. E2-4)

where

\[ F_{cre} = \text{Least of the applicable elastic global (flexural, torsional, and flexural-torsional) buckling stresses determined in accordance with Sections E2.1 through E2.5 or Appendix 2} \]

\[ F_y = \text{Yield stress} \]
**E2.1 Sections Not Subject to Torsional or Flexural-Torsional Buckling**

For *doubly-symmetric sections*, closed cross-sections, and any other sections that can be shown not to be subjected to *torsional* or *flexural-torsional buckling*, the elastic flexural buckling stress, $F_{cre}$, shall be calculated as follows:

$$F_{cre} = \frac{\pi^2 E}{(KL/r)^2} \quad \text{(Eq. E2.1-1)}$$

where

- $E = \text{Modulus of elasticity of steel}$
- $K = \text{Effective length factor determined in accordance with Chapter C}$
- $L = \text{Laterally unbraced length of member}$
- $r = \text{Radius of gyration of full unreduced cross-section about axis of buckling}$

**E2.1.1 Closed-Box Sections**

For a concentrically loaded compression member with a closed-box section that is made of steel with a specified minimum elongation between three to ten percent, inclusive, a reduced radius of gyration $(R_r)(r)$ shall be used in Eq. E2.1-1 when the value of the *effective length* $KL$ is less than $1.1 L_0$, where $L_0$ is given by Eq. E2.1.1-1, and $R_r$ is given by Eq. E2.1.1-2.

$$L_0 = \frac{\pi r}{E} \sqrt{\frac{E}{F_{cr\ell}}} \quad \text{(Eq. E2.1.1-1)}$$

$$R_r = 0.65 + \frac{0.35(KL)}{1.1L_0} \quad \text{(Eq. E2.1.1-2)}$$

where

- $L_0 = \text{Length at which local buckling stress equals flexural buckling stress}$
- $r = \text{Radius of gyration of full unreduced cross-section about axis of buckling}$
- $E = \text{Modulus of elasticity of steel}$
- $F_{cr\ell} = \text{Minimum critical buckling stress for cross-section calculated by Eq. 1.1-4}$
- $R_r = \text{Reduction factor}$
- $KL = \text{Effective length determined in accordance with Chapter C}$

**E2.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling**

For *singly-symmetric sections* subject to *flexural-torsional buckling*, $F_{cre}$ shall be taken as the smaller of $F_{cre}$ calculated in accordance with Section E2.1 and $F_{cre}$ calculated as follows:

$$F_{cre} = \frac{1}{2\beta} \left[ (\sigma_{tx} + \sigma_t) - \sqrt{(\sigma_{tx} + \sigma_t)^2 - 4\sigma_{tx}\sigma_t} \right] \quad \text{(Eq. E2.2-1)}$$

Alternatively, a conservative estimate of $F_{cre}$ is permitted to be calculated as follows:

$$F_{cre} = \frac{\sigma_t\sigma_{ex}}{\sigma_t + \sigma_{ex}} \quad \text{(Eq. E2.2-2)}$$

where

- $\beta = 1 - (x_o/r_o)^2 \quad \text{(Eq. E2.2-3)}$
where

\[ r_o = \text{Polar radius of gyration of cross-section about shear center} \]

\[ = \sqrt{r_x^2 + r_y^2 + x_0^2} \quad (Eq. E2.2-4) \]

where

\[ r_x, r_y = \text{Radii of gyration of cross-section about centroidal principal axes} \]

\[ x_0 = \text{Distance from centroid to shear center in principal x-axis direction, taken as negative} \]

\[ \sigma_t = \frac{1}{A r_o^2} \left[ G J + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] \quad (Eq. E2.2-5) \]

where

\[ A = \text{Full unreduced cross-sectional area of member} \]

\[ G = \text{Shear modulus of steel} \]

\[ J = \text{Saint-Venant torsion constant of cross-section} \]

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

\[ C_w = \text{Torsional warping constant of cross-section} \]

\[ K_t = \text{Effective length factor for twisting determined in accordance with Chapter C} \]

\[ L_t = \text{Unbraced length of member for twisting} \]

\[ \sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \quad (Eq. E2.2-6) \]

where

\[ K_x = \text{Effective length factor for bending about x-axis determined in accordance with Chapter C} \]

\[ L_x = \text{Unbraced length of member for bending about x-axis} \]

For **singly-symmetric sections**, the x-axis shall be selected as the axis of symmetry.

For **doubly-symmetric sections subject to torsional buckling**, \( F_{cre} \) shall be taken as the smaller of \( F_{cre} \) calculated in accordance with Section E2.1 and \( F_{cre} = \sigma_t \), where \( \sigma_t \) is defined in accordance with Eq. E2.2-5.

For **singly-symmetric unstiffened angle sections not subject to local buckling at stress \( F_y \)**, \( F_{cre} \) shall be computed using Eq. E2.1-1, where \( r \) is the least radius of gyration.

### E2.3 Point-Symmetric Sections

For **point-symmetric sections**, \( F_{cre} \) shall be taken as the lesser of \( \sigma_t \) as defined in Section E2.2 and \( F_{cre} \) as calculated in Section E2.1 using the minor principal axis of the section.

### E2.4 Non-Symmetric Sections

For shapes whose cross-sections do not have any symmetry either about an axis or about a point, \( F_{cre} \) shall be determined by Appendix 2 or rational engineering analysis. Alternatively, compression members composed of such shapes are permitted to be tested in accordance with Section K2.
E2.5 Sections With Holes

For shapes whose cross-sections have holes, F_{cre} shall consider the influence of holes in accordance with Appendix 2. Alternatively, compression members with holes are permitted to be tested in accordance with Section K2.

Exception: For the Effective Width Method, where hole sizes meet the limitations of Appendix 1.1.1, the provisions of this section shall not be required.

E3 Local Buckling Interacting With Yielding and Global Buckling

The nominal axial strength [resistance], \( P_{n\ell} \), for local buckling interacting with yielding and global buckling shall be calculated in accordance with this section. All members shall be checked for potential reduction in available strength [factored resistance] due to interaction of the yielding or global buckling with local buckling. This reduction shall be considered through either the Effective Width Method of Section E3.1 or the Direct Strength Method of Section E3.2.

The applicable safety factors and resistance factors given in this section shall be used to determine the available axial strength [factored resistance] \( (\phi_c P_{n\ell} \text{ or } P_{n\ell}/\Omega_c) \) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_c = 1.80 \quad (ASD) \]
\[ \phi_c = 0.85 \quad (LRFD) \]
\[ = 0.80 \quad (LSD) \]

E3.1 Effective Width Method

For the Effective Width Method, the nominal axial strength [resistance], \( P_{n\ell} \), for local buckling shall be calculated in accordance with the following:

\[ P_{n\ell} = A_e F_n \leq P_{ne} \quad (Eq. E3.1-1) \]

where

\( F_n \) = Global column stress as defined in Section E2

\( A_e \) = Effective area calculated at stress \( F_n \), determined in accordance with Sections E3.1.1 and E3.1.2

\( P_{ne} \) = Nominal strength [resistance] considering yielding and global buckling, determined in accordance with Section E2

Concentrically loaded angle sections shall be designed for an additional bending moment as specified in the definitions of \( \overline{M}_X \) and \( \overline{M}_Y \) in Section H1.2.

E3.1.1 Members Without Holes

For members without holes, except closed cylindrical tubular members, \( A_e \) shall be determined from the summation of the thickness times the effective width of each element comprising the cross-section. The effective width of all elements shall be determined in accordance with Appendix 1 at stress \( F_n \).

E3.1.1.1 Closed Cylindrical Tubular Sections

For closed cylindrical tubular members having a ratio of outside diameter to wall
thickness, D/t, not greater than 0.441 E/F_y, the effective area, A_e, shall be calculated as follows:

\[ A_e = A_o + R(A - A_o) \]  
\( (Eq. \text{E3.1.1.1-1}) \)

where

\[ A_o = \begin{cases} 0.037 \left( \frac{DF_y}{(tE)} \right) + 0.667 & A \leq A \begin{cases} \leq 0.441 \frac{E}{F_y} \end{cases} \\ \end{cases} \]  
\( (Eq. \text{E3.1.1.1-2}) \)

where

D = Outside diameter of cylindrical tube

F_y = Yield stress

t = Thickness

E = Modulus of elasticity of steel

A = Area of full unreduced cross-section

R = \( \frac{F_y}{(2F_{cre})} \leq 1.0 \)  
\( (Eq. \text{E3.1.1.1-3}) \)

where

F_{cre} = Elastic flexural buckling stress, determined in accordance with Section E2.1

### E3.1.2 Members With Circular Holes

For members with circular holes, A_e shall be determined from the effective width in accordance with Appendix 1.1.1(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 is permitted to be determined by ignoring the holes, i.e., in accordance with Section E3.1.1.

### E3.2 Direct Strength Method

For the Direct Strength Method, the nominal axial strength [resistance], P_{n\ell}, for local buckling shall be calculated in accordance with Sections E3.2.1 and E3.2.2.

#### E3.2.1 Members Without Holes

For \( \lambda_{\ell} \leq 0.776; \) \[ P_{n\ell} = P_{ne} \]  
\( (Eq. \text{E3.2.1-1}) \)

For \( \lambda_{\ell} > 0.776; \) \[ P_{n\ell} = \left[ 1 - 0.15 \left( \frac{P_{cr\ell}}{P_{ne}} \right)^{0.4} \right] \left( \frac{P_{cr\ell}}{P_{ne}} \right)^{0.4} P_{ne} \]  
\( (Eq. \text{E3.2.1-2}) \)

where

\[ \lambda_{\ell} = \sqrt{\frac{P_{ne}}{P_{cr\ell}}} \]  
\( (Eq. \text{E3.2.1-3}) \)

P_{ne} = Global column strength as defined in Section E2

P_{cr\ell} = Critical elastic local column buckling load, determined in accordance with Appendix 2

#### E3.2.2 Members With Holes

The nominal axial strength [resistance], P_{n\ell}, for local buckling of columns with holes shall be calculated in accordance with Section E3.2.1, except P_{cr\ell} shall be determined including
the influence of holes and:
\[ P_{\text{net}} \leq P_{\text{ynet}} \]  \hspace{1cm} (Eq. E3.2.2-1)

where
\[ P_{\text{ynet}} = A_{\text{net}} F_y \]  \hspace{1cm} (Eq. E3.2.2-2)

where
\[ A_{\text{net}} = \text{Net area of cross-section at the location of a hole} \]
\[ F_y = \text{Yield stress} \]

E4 Distortional Buckling

The nominal axial strength [resistance], \( P_{\text{nd}} \), for distortional buckling shall be calculated in accordance with this section. The provisions of this section shall apply to I-, Z-, C-, Hat, and other open cross-section members that employ flanges with edge stiffeners.

The applicable safety factor and resistance factors given in this section shall be used to determine the available axial strength [factored resistance] \( (\phi_c P_{\text{nd}} \text{ or } P_{\text{nd}}/\Omega_c) \) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_c = 1.80 \hspace{1cm} (ASD) \]
\[ \phi_c = 0.85 \hspace{1cm} (LRFD) \]
\[ = 0.80 \hspace{1cm} (LSD) \]

E4.1 Members Without Holes

The nominal axial strength [resistance], \( P_{\text{nd}} \), for distortional buckling shall be calculated in accordance with the following:

For \( \lambda_d \leq 0.561 \); \( P_{\text{nd}} = P_y \)  \hspace{1cm} (Eq. E4.1-1)

For \( \lambda_d > 0.561 \); \( P_{\text{nd}} = \left[ 1 - 0.25 \left( \frac{P_{\text{crd}}}{P_y} \right) \right]^{0.6} \left( \frac{P_{\text{crd}}}{P_y} \right)^{0.6} P_y \)  \hspace{1cm} (Eq. E4.1-2)

where
\[ \lambda_d = \sqrt{\frac{P_y}{P_{\text{crd}}}} \]  \hspace{1cm} (Eq. E4.1-3)

where
\[ P_y = A_g F_y \]  \hspace{1cm} (Eq. E4.1-4)

where
\[ A_g = \text{Gross area of cross-section} \]
\[ F_y = \text{Yield stress} \]
\[ P_{\text{crd}} = \text{Critical elastic distortional column buckling load, determined in accordance with Appendix 2} \]

E4.2 Members With Holes

The nominal axial strength [resistance], \( P_{\text{nd}} \), for distortional buckling of columns with holes shall be calculated in accordance with Section E4.1, except \( P_{\text{crd}} \) shall be determined including the influence of holes, and if \( \lambda_d \leq \lambda_{d2} \) then:

For \( \lambda_d \leq \lambda_{d1} \); \( P_{\text{nd}} = P_{\text{ynet}} \)  \hspace{1cm} (Eq. E4.2-1)
For $\lambda_{d1} < \lambda_d \leq \lambda_{d2}$: \[ P_{nd} = P_{y\text{net}} - \left( \frac{P_{y\text{net}} - P_{d2}}{\lambda_{d2} - \lambda_{d1}} \right) (\lambda_d - \lambda_{d1}) \] (Eq. E4.2-2)

where
\[ \lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \] (Eq. E4.2-3)
\[ \lambda_{d1} = 0.561 \left( \frac{P_{y\text{net}}}{P_y} \right) \] (Eq. E4.2-4)
\[ \lambda_{d2} = 0.561 \left[ 14.0 \left( \frac{P_y}{P_{y\text{net}}} \right)^{0.4} - 13.0 \right] \] (Eq. E4.2-5)
\[ P_{d2} = \left[ 1 - 0.25 \left( \frac{1}{\lambda_{d2}} \right)^{1.2} \right] \left( \frac{1}{\lambda_{d2}} \right)^{1.2} P_y \] (Eq. E4.2-6)

\[ P_y = A_g F_y \] (Eq. E4.2-7)
\[ P_{y\text{net}} = A_{\text{net}} F_y \] (Eq. E4.2-8)

where
- $A_g = \text{Gross area}$
- $A_{\text{net}} = \text{Net area of cross-section at the location of a hole}$
- $F_y = \text{Yield stress}$
F. MEMBERS IN FLEXURE

This chapter addresses members subjected to bending about one principal axis, or Z-section members about centroidal axis passing through or perpendicular to the web. In addition, the member is loaded in a plane parallel to the axis that passes through the shear center, or is restrained against twisting.

This chapter is organized as follows:
F1 General Requirements
F2 Yielding and Global (Lateral-Torsional Buckling) Buckling
F3 Local Buckling Interacting with Yielding and Global Buckling
F4 Distortional Buckling
F5 Stiffeners

Additionally, built-up flexural member provisions are provided in:
II.1 Flexural Members Composed of Two Back-to-Back C-Sections

F1 General Requirements

The available flexural strength [factored resistance] \( \phi_b M_n \) or \( M_n / \Omega_b \) shall be the smallest of the values calculated in accordance with Sections F2 to F4, where applicable.

F2 Yielding and Global (Lateral-Torsional) Buckling

The nominal flexural strength [resistance], \( M_{ne} \), for yielding and global (lateral-torsional) buckling shall be calculated considering capacity up to first yield in accordance with Sections F2.1, F2.3 for tubular sections, or considering inelastic reserve capacity in accordance with Section F2.4.

The applicable safety factor and resistance factors given in this section, unless otherwise specified, shall be used to determine the available flexural strength [factored resistance] \( \phi_b M_{ne} \) or \( M_{ne} / \Omega_b \) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_b = 1.67 \quad (ASD) \]
\[ \phi_b = 0.90 \quad (LRFD) \]
\[ = 0.90 \quad (LSD) \]

F2.1 Initiation of Yielding Strength

The nominal flexural strength [resistance], \( M_{ne} \), for yielding and global (lateral-torsional) buckling considering capacity up to first yield shall be calculated in accordance with Eq. F2.1-1.

\[ M_{ne} = S_f F_n \leq M_y \quad (Eq. \, F2.1-1) \]

where

\[ M_{ne} = \text{Nominal flexural strength [resistance] for yielding and global buckling} \]
\[ S_f = \text{Elastic section modulus of full unreduced section relative to extreme compression fiber} \]
\[ M_y = S_f y F_y \quad (Eq. \, F2.1-2) \]
where
\[ \text{St} = \text{Elastic section modulus of full unreduced cross-section relative to extreme fiber in first yielding} \]
\[ F_y = \text{Yield stress} \]

\( F_n \) shall be determined as follows:

For \( F_{cre} \geq 2.78 F_y \)
\[ F_n = F_y \]  
(Eq. F2.1-3)

For \( 2.78 F_y > F_{cre} > 0.56 F_y \)
\[ F_n = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_{cre}}\right) \]  
(Eq. F2.1-4)

For \( F_{cre} \leq 0.56 F_y \)
\[ F_n = F_{cre} \]  
(Eq. F2.1-5)

where
\[ F_{cre} = \text{Critical elastic lateral-torsional buckling stress, determined in accordance with Section F2.1.1 to F2.1.5, as applicable, or Appendix 2} \]

### F2.1.1 Singly- or Doubly- Symmetric Sections Bending About Symmetric Axis

The elastic buckling stress for singly- or doubly-symmetric sections bending about the symmetric axis shall be calculated as follows:

\[ F_{cre} = \frac{C_b r_o A}{S_f} \frac{\sigma_y \sigma_t}{\sigma} \]  
(Eq. F2.1.1-1)

where
\[ C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} \]  
(Eq. F2.1.1-2)

where
\( M_{max} = \text{Absolute value of maximum moment in unbraced segment} \)
\( M_A = \text{Absolute value of moment at quarter point of unbraced segment} \)
\( M_B = \text{Absolute value of moment at centerline of unbraced segment} \)
\( M_C = \text{Absolute value of moment at three-quarter point of unbraced segment} \)

\( C_b \) 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.

\( r_o = \text{Polar radius of gyration of cross-section about shear center} \)
\[ = \sqrt{r_x^2 + r_y^2 + x_o^2} \]  
(Eq. F2.1.1-3)

where
\( r_x, r_y = \text{Radii of gyration of cross-section about centroidal principal axes} \)
\( x_o = \text{Distance from centroid to shear center in principal x-axis direction} \)
\( A = \text{Full unreduced cross-sectional area} \)
\( S_f = \text{Elastic section modulus of full unreduced cross-section relative to extreme compression fiber} \)
\[ \sigma_{ey} = \frac{\pi^2 E}{(K_y L_y/r_y)^2} \]  
(Eq. F2.1.1-4)

where

\( E \) = Modulus of elasticity of steel
\( K_y \) = *Effective length factor* for bending about y-axis
\( L_y \) = *Unbraced length* of member for bending about y-axis

\[ \sigma_t = \frac{1}{A r_y^2} \left[ G J + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] \]  
(Eq. F2.1.1-5)

where

\( G \) = Shear modulus of steel
\( J \) = Saint-Venant torsion constant of cross-section
\( C_w \) = Torsional warping constant of cross-section
\( K_t \) = *Effective length factor* for twisting
\( L_t \) = *Unbraced length* of member for twisting

For *singly-symmetric sections*, x-axis shall be the axis of symmetry.

Alternatively, for *doubly-symmetric I-sections*, \( F_{cre} \) is permitted to be calculated using the equation given

\[ F_{cre} = \frac{C_b \pi^2 E d I_{yc}}{S_t (K_y L_y)^2} \]  
(Eq. F2.1.1-6)

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

**F2.1.2 Singly-Symmetric Sections Bending About Centroidal Axis Perpendicular to Axis of Symmetry**

The elastic buckling stress, \( F_{cre} \), for *singly-symmetric sections* bending about the centroidal axis perpendicular to the axis of symmetry shall be calculated as follows, where x-axis is the symmetric axis of the cross-section oriented such that the shear center has a negative x-coordinate:

\[ F_{cre} = \frac{C_s A \sigma_{ex}}{C_{TF} S_t} \left[ j + C_s \sqrt{j^2 + r_o^2 (\sigma_t / \sigma_{ex})} \right] \]  
(Eq. F2.1.2-1)

where

\( C_s \) = +1 for moment causing compression on shear center side of centroid
  = -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. F2.1.2-2)

where

\( K_x \) = *Effective length factor* for bending about x-axis
L_x = Unbraced length of member for bending about x-axis

\[ C_{TF} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) \quad \text{(Eq. F2.1.2-3)} \]

where

\[ M_1 \text{ and } M_2 = \text{The smaller and the larger bending moment, respectively, at the} \]
ends of the unbraced length in the plane of bending; \( M_1/M_2 \), 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.

\[ j = \frac{1}{2I_y} \left[ \int_A x^3 dA + \int_A xy^2 dA \right] x_o \quad \text{(Eq. F2.1.2-4)} \]

where

\[ x_o = \text{Distance from centroid to shear center in principal x-axis direction, taken as negative} \]

Other variables are defined in Section F2.1.1.

**F2.1.3 Point-Symmetric Sections**

The elastic buckling stress, \( F_{cre} \), for point-symmetric Z-sections bending about x-axis that is perpendicular web and through the centroid is permitted to be calculated as follows:

\[ F_{cre} = \frac{C_b r_o A}{2S_f} \sqrt{\frac{\sigma_y}{\sigma_t}} \quad \text{(Eq. F2.1.3-1)} \]

Alternatively, \( F_{cre} \) is permitted to be calculated using Eq. F2.1.3-2:

\[ F_{cre} = \frac{C_b \pi^2 EdI_y c}{2S_f (K_y L_y)^2} \quad \text{(Eq. F2.1.3-2)} \]

Variables are defined in Section F2.1.1.

**F2.1.4 Closed-Box Sections**

For closed-box section members, if the laterally unbraced length of the member is less than or equal to \( L_u \), as calculated in Eq. F2.1.4-1, the global buckling does not need to be considered, and the nominal stress, \( F_n = F_y \).

\[ L_u = \frac{0.36C_b \pi}{F_y S_f} \sqrt{\frac{E G J}{I_y}} \quad \text{(Eq. F2.1.4-1)} \]

where

\[ J = \text{Torsional constant of closed-box section} \]
\[ I_y = \text{Moment of inertia of full unreduced section about centroidal axis parallel to web} \]
\[ F_y = \text{Yield stress} \]

Other variables are defined in Section F2.1.1.

If the laterally unbraced length of a member is larger than \( L_u \), as calculated in Eq. F2.1.4-
1, the elastic buckling stress, $F_{cre}$, for bending about the symmetric axis shall be calculated as follows:

$$F_{cre} = \frac{C_b\pi}{K_y L_y S_f} \sqrt{E} \|J_y$$  \hspace{1cm} (Eq. F2.1.4-2)

### F2.1.5 Other Cross-Sections

For cross-sections other than those defined in Sections F2.1.1 through F2.1.4, the elastic buckling stress is permitted to be determined in accordance with Section 2.2 of Appendix 2.

### F2.2 Beams With Holes

For shapes whose cross-sections have holes, $F_{cre}$ shall consider the influence of holes in accordance with Appendix 2.

**Exception**: For the Effective Width Method, where hole sizes meet the limitations of Appendix 1.1.3, the provisions of this section shall not be required.

### F2.3 Initiation of Yielding Strength for Closed Cylindrical Tubular Sections

For closed cylindrical tubular members having a ratio of outside diameter to wall thickness, $D/t$, not greater than 0.441 $E/F_y$, the nominal flexural strength [resistance], $M_{ne}$, shall be calculated in accordance with Eq. F2.3-1.

The safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$M_{ne} = S_f F_n$$  \hspace{1cm} (Eq. F2.3-1)

$$\Omega_b = 1.67 \quad (ASD)$$

$$\phi_b = 0.95 \quad (LRFD)$$

$$= 0.90 \quad (LSD)$$

where

- $M_{ne}$ = Nominal flexural strength [resistance] for yielding and global buckling
- $S_f$ = Elastic section modulus of full unreduced section relative to extreme compression fiber
- $F_n$ shall be determined as follows:

For $D/t \leq 0.0714 \ E/F_y$

$$F_n = 1.25 \ F_y$$  \hspace{1cm} (Eq. F2.3-2)

For $0.0714 \ E/F_y < D/t \leq 0.318 \ E/F_y$

$$F_n = \left[ 0.970 + 0.020 \left( \frac{E/F_y}{D/t} \right) \right] F_y$$  \hspace{1cm} (Eq. F2.3-3)

For $0.318 \ E/F_y < D/t \leq 0.441 \ E/F_y$

$$F_n = 0.328 E/(D/t)$$  \hspace{1cm} (Eq. F2.3-4)

where

- $F_y$ = Yield stress
- $D$ = Outside diameter of cylindrical tube
F2.4 Inelastic Reserve Strength

The nominal flexural strength [resistance], $M_{ne}$, for yielding and global (lateral-torsional) buckling considering inelastic reserve shall be calculated in accordance with this section. Inelastic reserve is permitted to be considered through either the Element-Based Method of Section F2.4.1 or the Direct Strength Method of Section F2.4.2.

F2.4.1 Element-Based Method

The inelastic flexural reserve capacity is permitted to be used provided the following conditions are met:

1. The member is not subject to twisting or to lateral, torsional, or flexural-torsional buckling.
2. The effect of cold work of forming is not included in determining the yield stress $F_Y$.
3. The ratio of the depth of the compressed portion of the web to its thickness does not exceed $\lambda_1$ as defined in Eq. F2.4.1-3.
4. The shear force does not exceed $0.35F_Y$ for ASD, and $0.6F_Y$ for LRFD and LSD times the web area (ht for stiffened elements or wt for unstiffened elements).
   where
   \[ h = \text{Flat depth of web} \]
   \[ t = \text{Base steel thickness of element} \]
   \[ w = \text{Element flat width} \]
5. The angle between any web and the vertical does not exceed 30.

The nominal flexural strength [resistance], $M_{ne}$, shall not exceed $1.25S_eF_Y$ or shall not cause a maximum compression strain of $C_ye_y$ (no limit is placed on the maximum tensile strain).

where
\[ S_e = \text{Effective section modulus calculated relative to extreme compression or tension fiber at } F_Y \]
\[ F_Y = \text{Yield stress} \]
\[ e_y = \text{Yield strain} \]
\[ = \frac{F_Y}{E} \] \hspace{1cm} (Eq. F2.4.1-1)

where
\[ E = \text{Modulus of elasticity of steel} \]
\[ C_y = \text{Compression strain factor calculated as follows:} \]

(a) Stiffened compression elements without intermediate stiffeners

For compression elements without intermediate stiffeners, $C_y$ shall be calculated as follows:

\[ C_y = 3 \text{ when } \frac{w}{t} \leq \lambda_1 \]

\[ C_y = 3 - 2 \left( \frac{w}{t} - \lambda_1 \right) \text{ when } \lambda_1 < \frac{w}{t} < \lambda_2 \] \hspace{1cm} (Eq. F2.4.1-2)

\[ C_y = 1 \text{ when } \frac{w}{t} \geq \lambda_2 \]
where
\[
\lambda_1 = \frac{1.11}{\sqrt{F_y / E}}
\]
\[(Eq. F2.4.1-3)\]
\[
\lambda_2 = \frac{1.28}{\sqrt{F_y / E}}
\]
\[(Eq. F2.4.1-4)\]

(b) Unstiffened compression elements

For unstiffened compression elements, \(C_y\) shall be calculated as follows:

(1) Unstiffened compression elements under stress gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:
\[
C_y = 3 \quad \text{when } \lambda \leq \lambda_3
\]
\[
C_y = 3 - 2[(\lambda - \lambda_3)/ (\lambda_4 - \lambda_3)] \quad \text{when } \lambda_3 < \lambda < \lambda_4
\]
\[
C_y = 1 \quad \text{when } \lambda \geq \lambda_4
\]

where
\[
\lambda = \text{Slenderness factor defined in Section 1.2.2}
\]
\[
\lambda_3 = 0.43
\]
\[
\lambda_4 = 0.673(1+\psi)
\]
\[(Eq. F2.4.1-5)\]

(2) Unstiffened compression elements under stress gradient causing compression at both longitudinal edges:
\[
C_y = 1
\]

(3) Unstiffened compression elements under uniform compression:
\[
C_y = 1
\]

(4) Multiple-stiffened compression elements and compression elements with edge stiffeners

For multiple-stiffened compression elements and compression elements with edge stiffeners, \(C_y\) shall be taken as follows:
\[
C_y = 1
\]

\(M_n\) 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 the provisions of Section H3.

F2.4.2 Direct Strength Method

The nominal strength [resistance], \(M_{ne}\), considering inelastic flexural reserve capacity is permitted to be considered in accordance with the provisions of this section:

For \(M_{cre} > 2.78 M_y\)
\[
M_{ne} = M_p - (M_p - M_y) \sqrt{M_y / M_{cre}} \frac{0.23}{0.37} \leq M_p
\]
\[(Eq. F2.4.2-1)\]
where
\[ M_{cre} = \text{Critical elastic lateral-torsional buckling moment} \]
\[ = S_F F_{cre} \quad \text{(Eq. F2.4.2-2)} \]
where
\[ S_F = \text{Elastic section modulus of full unreduced cross-section relative to extreme compression fiber} \]
\[ F_{cre} = \text{Critical elastic lateral-torsional buckling stress, determined in accordance with Appendix 2 or Section F2.1} \]
\[ M_y = \text{Member yield moment in accordance with Section F2.1} \]
\[ M_P = \text{Member plastic moment} \]
\[ = Z_F F_Y \quad \text{(Eq. F2.4.2-3)} \]
where
\[ Z_F = \text{Plastic section modulus} \]
\[ F_Y = \text{Yield stress} \]

F3 Local Buckling Interacting With Yielding and Global Buckling

All members shall be checked for potential reduction in available strength [factored resistance] due to interaction of the yielding or global buckling with local buckling. This reduction shall be considered through either the Effective Width Method of Section F3.1 or the Direct Strength Method of Section F3.2.

The applicable safety factor and resistance factors given in this section shall be used to determine the available flexural strength [factored resistance] \( \phi_b M_{nf} \) or \( M_{nf}/\Omega_b \) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_b = 1.67 \quad (ASD) \]
\[ \phi_b = 0.90 \quad (LRFD) \]
\[ = 0.90 \quad (LSD) \]

F3.1 Effective Width Method

For the Effective Width Method, the nominal flexural strength [resistance], \( M_{nf} \), for local buckling shall be calculated in accordance with the following:

\[ M_{nf} = S_F F_n \leq S_{et} F_Y \quad \text{(Eq. F3.1-1)} \]
where
\[ S_F = \text{Effective section modulus calculated at extreme fiber compressive stress of } F_n, \]
determined in accordance with Sections F3.1.1 through F3.1.3
\[ F_n = \text{Global flexural stress as defined in Section F2} \]
\[ S_{et} = \text{Effective section modulus calculated at extreme fiber tension stress of } F_Y \]
\[ F_Y = \text{Yield stress} \]

F3.1.1 Members Without Holes

For members without holes, \( S_F \) shall be determined from the effective width of each element comprising the cross-section. The effective width of all elements is determined in
accordance with Appendix 1 at extreme compressive stress $F_n$. 

For cylindrical tubular members having a ratio of outside diameter to wall thickness, $D/t$, not greater than 0.441E/$F_y$, local buckling does not need to be checked.

**F3.1.2 Members With Holes**

For members with holes, the elements adjacent to the hole shall be treated as unstiffened elements. $S_e$ shall be determined from the effective width in accordance with Appendix 1.

**F3.1.3 Members Considering Inelastic Reserve Strength**

The Element-Based Method of Section F2.4.1 shall be applied as given in this section. When applicable, effective design widths (Appendix 1) shall be used in calculating section properties.

**F3.2 Direct Strength Method**

For the Direct Strength Method, the nominal flexural strength [resistance], $M_{n/\ell}$, for local buckling shall be calculated in accordance with Sections F3.2.1 through F3.2.3.

**F3.2.1 Members Without Holes**

The nominal flexural strength [resistance], $M_{n/\ell}$, for considering interaction of local buckling and global buckling shall be determined as follows:

(a) For $\lambda_{\ell} \leq 0.776$

$$M_{n/\ell} = M_{ne} \quad \text{(Eq. F3.2.1-1)}$$

(b) For $\lambda_{\ell} > 0.776$

$$M_{n/\ell} = \left[1 - 0.15 \left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right]^{0.4} M_{ne} \quad \text{(Eq. F3.2.1-2)}$$

where

$$\lambda_{\ell} = \sqrt{M_{ne}/M_{cr\ell}} \quad \text{(Eq. F3.2.1-3)}$$

$M_{ne} = \text{Nominal flexural strength [resistance] for lateral-torsional buckling as defined in Section F2}$

$M_{cr\ell} = \text{Critical elastic local buckling moment, determined in accordance with Appendix 2}$

**F3.2.2 Members With Holes**

The nominal flexural strength [resistance], $M_{n/\ell}$, for local buckling of beams with holes shall be calculated in accordance with Section F3.2.1, except $M_{cr\ell}$ shall be determined including the influence of holes:

$$M_{n/\ell} \leq M_{y\text{net}} \quad \text{(Eq. F3.2.2-1)}$$
where
\[ M_{\text{ynet}} = \text{Member yield moment of net cross-section} \]
\[ = S_{\text{fnet}}F_y \]
\[ \text{(Eq. F3.2.2-2)} \]
where
\[ S_{\text{fnet}} = \text{Net section modulus referenced to the extreme fiber at first yield} \]
\[ F_y = \text{Yield stress} \]

**F3.2.3 Members Considering Local Inelastic Reserve Strength**

Inelastic reserve capacity is permitted to be considered as follows, provided \( \lambda \leq 0.776 \) and \( M_{\text{ne}} \geq M_y \):

(a) Sections symmetric about the axis of bending or sections with first yield in compression:
\[ M_{n\ell} = M_y + (1 - 1/C_y^2)(M_p - M_y) \]
\[ \text{(Eq. F3.2.3-1)} \]
(b) Sections with first yield in tension:
\[ M_{n\ell} = M_{yc} + (1 - 1/C_y^2)(M_p - M_y) \leq M_{yt3} \]
\[ \text{(Eq. F3.2.3-2)} \]
where
\[ \lambda = \sqrt{M_y/M_{ct\ell}} \]
\[ \text{(Eq. F3.2.3-3)} \]
\[ M_{\text{ne}} = \text{Nominal flexural strength [resistance] as defined in Section F2} \]
\[ C_y = \sqrt{0.776/\lambda} \leq 3 \]
\[ \text{(Eq. F3.2.3-4)} \]
\[ M_{ct\ell} = \text{Critical elastic local buckling moment, determined in accordance with Appendix 2} \]
\[ M_p = \text{Member plastic moment as given in Eq. F2.4.2-3} \]
\[ M_y = \text{Member yield moment in accordance with Section F2.1} \]
\[ M_{yc} = \text{Moment at which yielding initiates in compression (after yielding in tension).} \]
\[ M_{yt3} = M_y + (1 - 1/C_y^2)(M_p - M_y) \]
\[ \text{(Eq. F3.2.3-5)} \]
\[ C_{yt} = \text{Ratio of maximum tension strain to yield strain} \]
\[ = 3 \]

**F4 Distortional Buckling**

The provisions of this section shall apply to I-, Z-, C-, and other open cross-section members that employ compression flanges with edge stiffeners.

The applicable safety factor and resistance factors given in this section shall be used to determine the available flexural strength [factored resistance] \( \phi_b M_{nd} \) or \( M_{nd}/\Omega_b \) in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_b = 1.67 \quad \text{(ASD)} \]
\[ \phi_b = 0.90 \quad \text{(LRFD)} \]
\[ = 0.90 \quad \text{(LSD)} \]
**F4.1 Members Without Holes**

The nominal flexural strength [resistance], $M_{nd}$, shall be calculated in accordance with Eq. F4.1-1 or Eq. F4.1-2.

For $\lambda_d \leq 0.673$

$$M_{nd} = M_y$$  \hspace{1cm} (Eq. F4.1-1)

For $\lambda_d > 0.673$

$$M_{nd} = \left[ 1 - 0.22 \left( \frac{M_{crd}}{M_y} \right)^{0.5} \right] \left( \frac{M_{crd}}{M_y} \right)^{0.5} M_y$$  \hspace{1cm} (Eq. F4.1-2)

where

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}}$$  \hspace{1cm} (Eq. F4.1-3)

$$M_y = S_{fy} F_y$$ \hspace{1cm} (Eq. F4.1-4)

where

$S_{fy} =$ Elastic section modulus of full unreduced cross-section relative to extreme fiber in first yielding

$F_y =$ Yield stress

$M_{crd} = S_t F_{crd}$ \hspace{1cm} (Eq. F4.1-5)

where

$S_t =$ Elastic section modulus of full unreduced cross-section relative to extreme compression fiber

$F_{crd} =$ Elastic distortional buckling stress calculated in accordance with Appendix 2

**F4.2 Members With Holes**

The nominal flexural strength [resistance], $M_{nd}$, for distortional buckling shall be calculated in accordance with Section F4.1, except $M_{crd}$ shall be determined including the influence of holes, and when $\lambda_d \leq \lambda_{d2}$ then:

For $\lambda_d \leq \lambda_{d1}$

$$M_{nd} = M_{y_{net}}$$ \hspace{1cm} (Eq. F4.2-1)

For $\lambda_{d1} < \lambda_d \leq \lambda_{d2}$

$$M_{nd} = M_{y_{net}} - \left( \frac{M_{y_{net}} - M_{d2}}{\lambda_{d2} - \lambda_{d1}} \right) (\lambda_d - \lambda_{d1}) \leq \left[ 1 - 0.22 \left( \frac{M_{crd}}{M_y} \right)^{0.5} \right] \left( \frac{M_{crd}}{M_y} \right)^{0.5} M_y$$ \hspace{1cm} (Eq. F4.2-2)

where

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}}$$ \hspace{1cm} (Eq. F4.2-3)

where

$M_{crd} =$ Distortional buckling moment including influence of holes

$\lambda_{d1} = 0.673 \left( \frac{M_{y_{net}}}{M_y} \right)^3$ \hspace{1cm} (Eq. F4.2-4)
\[ \lambda_{d2} = \text{Limit of distortional slenderness transition} \]
\[ = 0.673[1.7(M_y / M_{y\text{net}})^{2.7} - 0.7] \quad (Eq. F4.2-5) \]
\[ M_{d2} = [1 - 0.22(1/\lambda_{d2})(1/\lambda_{d2})]M_y \quad (Eq. F4.2-6) \]
\[ M_y = \text{Member yield moment as given in Eq. F4.1-4} \]
\[ M_{y\text{net}} = \text{Member yield moment of net cross-section as given in Eq. F3.2.2-2} \]

### F4.3 Members Considering Distortional Inelastic Reserve Strength

Inelastic reserve capacity is permitted to be considered as follows, provided \( \lambda_d \leq 0.673 \):

(a) Sections symmetric about the axis of bending or sections with first yield in compression:

\[ M_{nd} = M_y + (1 - 1/C_{yd}^2)(M_p - M_y) \quad (Eq. F4.3-1) \]

(b) Sections with first yield in tension:

\[ M_{nd} = M_{yc} + (1 - 1/C_{yd}^2)(M_p - M_{yc}) \leq M_{yt3} \quad (Eq. F4.3-2) \]

where

\[ \lambda_d = \sqrt{M_y / M_{crd}} \quad (Eq. F4.3-3) \]
\[ C_{yd} = \sqrt{0.673 / \lambda_d} \leq 3 \quad (Eq. F4.3-4) \]

\( M_{crd} = \) Critical elastic distortional buckling moment, determined in accordance with Appendix 2

\( M_p = \) Member plastic moment as given in Eq. F2.4.2-3

\( M_y = \) Member yield moment in accordance with Section F2.1

\( M_{yc} = \) Moment for yield in compression as defined in Section F3.2.3

\( M_{yt3} = \) Maximum moment for yielding in tension as given in Eq. F3.2.3-5

---

### F5 Stiffeners

#### F5.1 Bearing Stiffeners

Bearing 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 in accordance with Chapter J. For concentrated loads or reactions, the nominal strength [resistance], \( P_n \), shall be the smaller value calculated by (a) and (b) of this section. The safety factor and resistance factors provided in this section shall be used to determine the available strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ \Omega_c = 2.00 \quad (ASD) \]
\[ \phi_c = 0.85 \quad (LRFD) \]
\[ = 0.80 \quad (LSD) \]

(a) \( P_n = F_{wy}A_c \quad (Eq. F5.1-1) \)
(b) \( P_n \) = *Nominal axial strength [resistance]* evaluated in accordance with Section E3.1, with \( A_e \) replaced by \( A_b \)

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

where
\( t \) = Base steel thickness of beam web
\( A_s \) = *Cross-sectional area* of bearing stiffener

\[
A_b = b_1 t + A_{sr} \text{ for bearing stiffener at interior support or under concentrated load}
\]
\[
= b_2 t + A_{sr} \text{ for bearing stiffener at end support}
\]

where
\[
b_1 = 25t \left[ 0.0024 \left( \frac{L_{st}}{t} \right) + 0.72 \right] \leq 25t \quad (Eq. F5.1-6)
\]
\[
b_2 = 12t \left[ 0.0044 \left( \frac{L_{st}}{t} \right) + 0.83 \right] \leq 12t \quad (Eq. F5.1-7)
\]

The \( w/t_s \) ratio for the stiffened and unstiffened elements of the bearing stiffener shall not exceed \( 1.28 \sqrt{E/F_{ys}} \) and \( 0.42 \sqrt{E/F_{ys}} \), respectively, where \( F_{ys} \) is the *yield stress* of the stiffener steel, and \( t_s \) is the *thickness* of the stiffener steel.

### F5.2 Bearing Stiffeners in C-Section Flexural Members

For two-flange loading of C-section flexural members with bearing stiffeners that do not meet the requirements of Section F5.1, the *nominal strength [resistance]*, \( P_n \), shall be calculated in accordance with Eq. F5.2-1. The *safety factor* and *resistance factors* in this section shall be used to determine the *available strength [factored resistance]* in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
P_n = 0.7(P_{wc} + A_e F_{ys}) \geq P_{wc} \quad (Eq. F5.2-1)
\]

\( \Omega_c = 1.70 \) \hspace{0.5cm} (ASD)

\( \phi_c = 0.90 \) \hspace{0.5cm} (LRFD)

\( = 0.80 \) \hspace{0.5cm} (LSD)

where
\( P_{wc} = \text{Nominal web crippling strength [resistance] for C-section flexural member, calculated in accordance with Eq. G5-1 for single web members, at end or interior locations} \)

\( A_e = \text{Effective area of bearing stiffener subjected to uniform compressive stress, calculated at yield stress} \)

\( F_{ys} = \text{Yield stress of bearing stiffener steel} \)

Eq. F5.2-1 shall apply within the following limits:

(a) Full bearing of the stiffener is required. If the bearing width is narrower than the stiffener such that one of the stiffener flanges is unsupported, \( P_n \) is reduced by 50 percent.
(b) Stiffeners are C-section stud or track members with a minimum web depth of 3-1/2 in. (88.9 mm) and a minimum base steel thickness of 0.0329 in. (0.836 mm).

(c) The stiffener is attached to the flexural member web with at least three fasteners (screws or bolts).

(d) The distance from the flexural member flanges to the first fastener(s) is not less than d/8, where d is the overall depth of the flexural member.

(e) The length of the stiffener is not less than the depth of the flexural member minus 3/8 in. (9.53 mm).

(f) The bearing width is not less than 1-1/2 in. (38.1 mm).

F5.3 Nonconforming Stiffeners

The available strength [factored resistance] of members with stiffeners that do not meet the requirements of Sections F5.1 and F5.2, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Section K2 or rational engineering analysis in accordance with Section A1.2.
G. MEMBERS IN SHEAR AND WEB CRIPPLING

This chapter addresses webs of singly-, doubly-, or point symmetric cross-section members subject to shear in the plane of web, or web crippling due to high intensity of load or reaction on the web. The webs may contain holes or transverse web reinforcement. The design of transverse web stiffeners is considered as well.

This chapter is organized as follows:
G1 General Requirements
G2 Shear Strength of Webs Without Holes
G3 Shear Strength of C-Section Webs With Holes
G4 Transverse Web Stiffeners
G5 Web Crippling Strength of Webs Without Holes
G6 Web Crippling Strength of C-Section Webs With Holes

G1 General Requirements

The available shear strength [factored resistance] shall be determined in accordance with Section G2 for webs without holes and Section G3 for webs with holes, as applicable. Transverse web stiffeners shall be designed in accordance with Section G4, as applicable. Webs subjected to concentrated loads shall be checked for web crippling in accordance with Sections G5 or G6, as applicable.

G2 Shear Strength of Webs Without Holes

The nominal shear strength [resistance], \( V_n \), of flexural members without holes in the web(s) shall be calculated in accordance with this section, as applicable. For flexural members meeting the geometric and material criteria of Section B4, \( \Omega_v \) and \( \phi_v \) shall be as follows:

\[
\Omega_v = 1.60 \text{ (ASD)}
\]

\[
\phi_v = 0.95 \text{ (LRFD)}
\]

\[
= 0.80 \text{ (LSD)}
\]

For all other flexural members, \( \Omega \) and \( \phi \) of the Specification, Section A1.2(c), shall apply. The available strength [factored resistance] shall be determined in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3 of the Specification.

G2.1 Flexural Members Without Transverse Web Stiffeners

The nominal shear strength [resistance], \( V_{nv} \), of flexural members without transverse web stiffeners shall be calculated as follows:

For \( \lambda_v \leq 0.815 \),

\[
V_n = V_y \quad \text{(Eq. G2.1-1)}
\]

For \( 0.815 < \lambda_v \leq 1.227 \)

\[
V_n = 0.815 \sqrt{V_{cr} V_y} \quad \text{(Eq. G2.1-2a)}
\]
\[ V_n = V_{cr} = 0.904E_k v t^3/h \]  
\[ \lambda_v = \sqrt{\frac{V_y}{V_{cr}}} \]  
\[ V_y = \text{Yield shear force of cross-section} = 0.6 A_w F_y \]  
\[ A_w = \text{Area of web element} = ht \]  
\[ h = \text{Depth of flat portion of web measured along plane of web} \]  
\[ t = \text{Web thickness} \]  
\[ F_y = \text{Design yield stress as determined in accordance with Section A3.3.1} \]  
\[ V_{cr} = \text{Elastic shear buckling force as defined in Section G2.3 for flat web alone, or} \]  
\[ \text{determined in accordance with Appendix 2 for full cross-section of prequalified (Table B4.1-1) members} \]  
\[ E = \text{Modulus of elasticity of steel} \]  
\[ k_v = \text{Shear buckling coefficient, determined in accordance with Section G2.3} \]  

\section*{G2.2 Flexural Members With Transverse Web Stiffeners}

For a reinforced web with transverse web stiffeners meeting the criteria of Section G4, and spacing not exceeding twice the web depth, this section is permitted to be used to determine the nominal shear strength [resistance], \( V_{n} \), in lieu of Section G2.1.

For \( \lambda_v \leq 0.776 \),
\[ V_n = V_y \]  
\[ (Eq. G2.2-1) \]

For \( \lambda_v > 0.776 \),
\[ V_n = \left[ 1 - 0.15 \left( \frac{V_{cr}}{V_y} \right)^{0.4} \right]^{0.4} \left( \frac{V_{cr}}{V_y} \right)^{0.4} V_y \]  
\[ (Eq. G2.2-2) \]

where
\[ V_{cr} = \text{Elastic shear buckling force as defined in Section G2.3 for flat web alone, or} \]  
\[ \text{determined in accordance with Appendix 2 for full cross-section of prequalified (Table B4.1-1) members} \]  
Other variables are defined in Section G2.1.

\section*{G2.3 Web Elastic Critical Shear Buckling Force, \( V_{cr} \)}

The shear buckling force, \( V_{cr} \), of a web is permitted to be determined in accordance with this section:
\[ V_{cr} = A_w F_{cr} \]  
(Eq. G2.3-1)

where
\[ A_w = \text{Web area as given in Eq. G2.1-6} \]
\[ F_{cr} = \text{Elastic shear buckling stress} \]
\[ = \frac{\pi^2 E k_v}{12(1 - \mu^2)(h/t)^2} \]  
(Eq. G2.3-2)

where
\[ E = \text{Modulus of elasticity of steel} \]
\[ k_v = \text{Shear buckling coefficient calculated in accordance with (a) or (b) as follows:} \]
(a) For unreinforced webs, \( k_v = 5.34 \)
(b) For webs with transverse stiffeners satisfying the requirements of Section G4
when \( a/h \leq 1.0 \)
\[ k_v = 4.00 + \frac{5.34}{(a/h)^2} \]  
(Eq. G2.3-3)
when \( a/h > 1.0 \)
\[ k_v = 5.34 + \frac{4.00}{(a/h)^2} \]  
(Eq. G2.3-4)

where
\[ a = \text{Shear panel length of unreinforced web element} \]
\[ = \text{Clear distance between transverse stiffeners of reinforced web elements} \]
Other variables are defined in Section G2.1.

**G3 Shear Strength of C-Section Webs With Holes**

The provisions of this section shall apply within the following limits:
(a) \( d_h/h \leq 0.7 \),
(b) \( h/t \leq 200 \),
(c) Holes centered at mid-depth of web,
(d) Clear distance between holes \( \geq 18 \) in. (457 mm),
(e) Noncircular holes, corner radii \( \geq 2t \),
(f) Noncircular holes, \( d_h \leq 2.5 \) in. (63.5 mm) and \( L_h \leq 4.5 \) in. (114 mm),
(g) Circular holes, diameter \( \leq 6 \) in. (152 mm), and
(h) \( d_h > 9/16 \) in. (14.3 mm).

where
\[ d_h = \text{Depth of web hole} \]
\[ h = \text{Depth of flat portion of web measured along plane of web} \]
\[ t = \text{Web thickness} \]
\[ L_h = \text{Length of web hole} \]

For C-section webs with holes, the shear strength shall be calculated in accordance with Section G2, with \( V_{cr} \) computed using G2.3, multiplied by the reduction factor, \( q_s \), as defined in this section.
Chapter G, Members in Shear and Web Crippling

When \( c/t \geq 54 \)
\[
q_s = 1.0
\]
When \( 5 \leq c/t < 54 \)
\[
q_s = c/(54t)
\]  \( (Eq. \ G3-1) \)

where
\[
c = h/2 - dh/2.83 \quad \text{for circular holes} \]  \( (Eq. \ G3-2) \)
\[
c = h/2 - dh/2 \quad \text{for noncircular holes} \]  \( (Eq. \ G3-3) \)

G4 Transverse Web Stiffeners

G4.1 Conforming Transverse Web Stiffeners

Where transverse web stiffeners are required for shear, the spacing shall be based on the nominal shear strength [resistance], \( V_{n_r} \), permitted by Section G2.2, 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 transverse web stiffeners, or of a single transverse web stiffener, with reference to an axis in the plane of the web, shall have a minimum value calculated in accordance with Eq. G4.1-1 as follows:
\[
I_{s\min} = 5ht^3 \left[ h/a - 0.7(a/h) \right] \geq (h/50)^4 \]  \( (Eq. \ G4.1-1) \)

where
\[
h \text{ and } t = \text{Values as defined in Section G2.1} \]
\[
a = \text{Distance between transverse web stiffeners} \]

The gross area of transverse web stiffeners shall not be less than:
\[
A_{st} = \frac{1 - C_v}{2} \left[ \frac{a}{h} - \frac{(a/h)^2}{(a/h) + \sqrt{1 + (a/h)^2}} \right] Y D h t \]  \( (Eq. \ G4.1-2) \)

where
\[
C_v = \frac{1.53E_k v}{F_y (h/t)^2} \quad \text{when } C_v \leq 0.8 \]  \( (Eq. \ G4.1-3) \)
\[
= \frac{1.11}{h/t} \sqrt{\frac{E_k v}{F_y}} \quad \text{when } C_v > 0.8 \]  \( (Eq. \ G4.1-4) \)

where
\[
k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \leq 1.0 \]  \( (Eq. \ G4.1-5) \)
\[
= 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h > 1.0 \]  \( (Eq. \ G4.1-6) \)

\[
Y = \frac{\text{Yield stress of web steel}}{\text{Yield stress of stiffener steel}}
\]
\[
D = 1.0 \text{ for stiffeners furnished in pairs}
\]
\[
= 1.8 \text{ for single-angle stiffeners}
\]
\[
= 2.4 \text{ for single-plate stiffeners}
\]

Other variables are defined in Section G2.1.
G4.2 Nonconforming Transverse Web Stiffeners

The available strength [factored resistance] of members with transverse web stiffeners that do not meet the requirements of Section G4.1, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Section K2 or rational engineering analysis in accordance with Section A1.2(c).

G5 Web Crippling Strength of Webs Without Holes

The nominal web crippling strength [resistance], \( P_n \), shall be determined in accordance with Eq. G5-1 or Eq. G5-2, as applicable. The safety factors and resistance factors in Tables G5-1 to G5-5 shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

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

(Eq. G5-1)

where:
- \( P_n \) = Nominal web crippling strength [resistance]
- \( C \) = Coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
- \( t \) = Web thickness
- \( F_y \) = Design yield stress as determined in accordance with Section A3.3.1
- \( \theta \) = Angle between plane of web and plane of bearing surface, \( 45^\circ \leq \theta \leq 90^\circ \)
- \( C_R \) = Inside bend radius coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
- \( R \) = Inside bend radius
- \( C_N \) = Bearing length coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
- \( N \) = Bearing length (3/4 in. (19 mm) minimum)
- \( C_h \) = Web slenderness coefficient from Table G5-1, G5-2, G5-3, G5-4, or G5-5
- \( h \) = Flat dimension of web measured in plane of web

Alternatively, for an end one-flange loading condition on a C- or Z-section, the nominal web crippling strength [resistance], \( P_{nc} \), with an overhang on one side, is permitted to be calculated as follows, except that \( P_{nc} \) shall not be larger than the interior one-flange loading condition:

\[
P_{nc} = \alpha P_n
\]  

(Eq. G5-2)

where
- \( P_{nc} \) = Nominal web crippling strength [resistance] of C- and Z-sections with overhang(s)
- \( \alpha \) = \[
0.13(0.09 + \frac{0.009(h/t) + 0.3}{L_0/h})^{0.26} \geq 1.0
\]  

(Eq. G5-3)

where
- \( L_0 \) = Overhang length measured from edge of bearing to the end of the member
- \( P_n \) = Nominal web crippling strength [resistance] with end one-flange loading as calculated by Eq. G5-1 and Tables G5-2 and G5-3

Eq. G5-2 shall be limited to \( 0.5 \leq L_0/h \leq 1.5 \) and \( h/t \leq 154 \). For \( L_0/h \) or \( h/t \) outside these limits, \( \alpha = 1 \).

Webs of members in bending for which \( h/t \) is greater than 200 shall be provided with means of transmitting concentrated loads or reactions directly into the web(s).
P_n and P_{nc} shall represent the nominal strengths [resistances] for load or reaction for one solid web connecting top and bottom flanges. For hat, multi-web sections and C- or Z-sections, P_n or P_{nc} shall be the nominal strength [resistance] for a single web, and the total nominal strength [resistance] shall be computed by multiplying P_n or P_{nc} by the number of webs at the considered cross-section.

One-flange loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or greater than 1.5h.

Two-flange loading or reaction shall be defined as the condition where the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is less than 1.5h.

End loading or reaction shall be defined as the condition where 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 shall be defined as the condition where the distance from the edge of the bearing to the end of the member is greater than 1.5h, except as otherwise noted herein.

Table G5-1 shall apply to I-beams made from two channels connected back-to-back where h/t ≤ 200, N/t ≤ 210, N/h ≤ 1.0, and θ = 90°. See Section G5 of Commentary for further explanation.

### TABLE G5-1

<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</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>20.5</td>
<td>0.17</td>
<td>0.11</td>
<td>0.001</td>
<td>1.75</td>
<td>0.85</td>
</tr>
<tr>
<td>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>Two-Flange Loading or Reaction</td>
<td>End</td>
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<td>0.08</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
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<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>
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<td></td>
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<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>
Table G5-2 shall apply to single web channel and C-section members where \( h/t \leq 200 \), \( N/t \leq 210 \), \( N/h \leq 2.0 \), and \( \theta = 90^\circ \). In Table G5-2, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of the bearing to the end of the member shall be extended at least 2.5\( h \). For unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5\( h \).

### TABLE G5-2
Safety Factors, Resistance Factors, and Coefficients for 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>Ch</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>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.75</td>
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<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>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>
<td>0.85</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></td>
<td>Unfastened</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>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>13</td>
<td>0.32</td>
<td>0.05</td>
<td>0.04</td>
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<td>0.90</td>
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<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></td>
<td>Unstiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.40</td>
<td>0.60</td>
<td>0.03</td>
<td>1.80</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>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>
</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: \( d = \) Out-to-out depth of section in the plane of the web
Table G5-3 shall apply to single web Z-section members where \( h/t \leq 200 \), \( N/t \leq 210 \), \( N/h \leq 2.0 \), and \( \theta = 90^\circ \). In Table G5-3, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of the bearing to the end of the member shall be extended at least 2.5\( h \); for unfastened cases, the distance from the edge of the bearing to the end of the member shall be extended at least 1.5\( h \).

**TABLE G5-3**

<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</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td><strong>Stiffened or Partially Stiffened Flanges</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \Omega_w )</td>
<td>LRFD ( \phi_w )</td>
<td></td>
</tr>
<tr>
<td><strong>One-Flange Loading or Reaction</strong></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></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>
<td>0.80</td>
</tr>
<tr>
<td><strong>Two-Flange Loading or Reaction</strong></td>
<td>End</td>
<td>9</td>
<td>0.05</td>
<td>0.16</td>
<td>0.052</td>
<td>1.75</td>
<td>0.85</td>
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</tr>
<tr>
<td></td>
<td>Interior</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><strong>Stiffened or Partially Stiffened Flanges</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \Omega_w )</td>
<td>LRFD ( \phi_w )</td>
<td></td>
</tr>
<tr>
<td><strong>One-Flange Loading or Reaction</strong></td>
<td>End</td>
<td>5</td>
<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></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>
<td>0.80</td>
</tr>
<tr>
<td><strong>Two-Flange Loading or Reaction</strong></td>
<td>End</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></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>
<td>0.65</td>
</tr>
<tr>
<td>Unstiffened</td>
<td><strong>Flanges</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( \Omega_w )</td>
<td>LRFD ( \phi_w )</td>
<td></td>
</tr>
<tr>
<td><strong>One-Flange Loading or Reaction</strong></td>
<td>End</td>
<td>4</td>
<td>0.40</td>
<td>0.60</td>
<td>0.03</td>
<td>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<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>
<td>0.70</td>
</tr>
<tr>
<td><strong>Two-Flange Loading or Reaction</strong></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></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>
Table G5-4 shall apply to single hat section members where \( h/t \leq 200, N/t \leq 200, N/h \leq 2, \) and \( \theta = 90^\circ. \)

**TABLE G5-4**

*Safety Factors, Resistance Factors, and Coefficients for Single Hat Sections per Web*

<table>
<thead>
<tr>
<th>Support 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 ASD ( \Omega_w )</th>
<th>LRFD ( \phi_w )</th>
<th>Canada LSD ( \phi_w )</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<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>
<td>0.65</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>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>9</td>
<td>0.10</td>
<td>0.07</td>
<td>0.03</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</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>
<td>0.75</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>
<td>0.65</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>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Table G5-5 shall apply to multi-web section members where \( h/t \leq 200, N/t \leq 210, N/h \leq 3, \) and \( 45^\circ \leq \theta \leq 90^\circ. \)

**TABLE G5-5**

*Safety Factors, Resistance Factors, and Coefficients for Multi-Web Deck Sections per Web*

<table>
<thead>
<tr>
<th>Support 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 ASD ( \Omega_w )</th>
<th>LRFD ( \phi_w )</th>
<th>Canada LSD ( \phi_w )</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.04</td>
<td>0.25</td>
<td>0.025</td>
<td>1.70</td>
<td>0.90</td>
<td>0.80</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>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>9</td>
<td>0.12</td>
<td>0.14</td>
<td>0.040</td>
<td>1.80</td>
<td>0.85</td>
<td>0.70</td>
</tr>
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<td>0.020</td>
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<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>Unfastened</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>3</td>
<td>0.04</td>
<td>0.29</td>
<td>0.028</td>
<td>2.45</td>
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<td>0.50</td>
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<tr>
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<td></td>
<td>Interior</td>
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<td>0.10</td>
<td>0.17</td>
<td>0.004</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</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>
<td>0.80</td>
</tr>
<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>
<td>0.80</td>
</tr>
</tbody>
</table>

Note: Multi-web deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm).
G6 Web Crippling Strength of C-Section Webs With Holes

Where a web hole is within the bearing length, a bearing stiffener shall be used. For beam webs with holes, the available web crippling strength [factored resistance] shall be calculated in accordance with Section G5, multiplied by the reduction factor, \( R_c \), given in this section.

The provisions of this section shall apply within the following limits:
(a) \( \frac{d_h}{h} \leq 0.7 \),
(b) \( h/t \leq 200 \),
(c) Hole centered at mid-depth of web,
(d) Clear distance between holes \( \geq 18 \) in. (457 mm),
(e) Distance between end of member and edge of hole \( \geq d \),
(f) Noncircular holes, corner radii \( \geq 2t \),
(g) Noncircular holes, \( d_h \leq 2.5 \) in. (63.5 mm) and \( L_h \leq 4.5 \) in. (114 mm),
(h) Circular holes, diameters \( \leq 6 \) in. (152 mm), and
(i) \( d_h > 9/16 \) in. (14.3 mm).

where
\( d_h = \) Depth of web hole
\( h = \) Depth of flat portion of web measured along plane of web
\( t = \) Web thickness
\( d = \) Depth of cross-section
\( L_h = \) Length of web hole

For end one-flange reaction (Equation G5-1 with Table G5-2) where a web hole is not within the bearing length, the reduction factor, \( R_c \), shall be calculated as follows:
\[
R_c = 1.01 - 0.325 \frac{d_h}{h} + 0.083 \frac{x}{h} \leq 1.0 \quad (Eq. G6-1)
\]
\( N \geq 1 \) in. (25.4 mm)

For interior one-flange reaction (Equation G5-1 with Table G5-2) where any portion of a web hole is not within the bearing length, the reduction factor, \( R_c \), shall be calculated as follows:
\[
R_c = 0.90 - 0.047 \frac{d_h}{h} + 0.053 \frac{x}{h} \leq 1.0 \quad (Eq. G6-2)
\]
\( N \geq 3 \) in. (76.2 mm)

where
\( x = \) Nearest distance between web hole and edge of bearing
\( N = \) Bearing length
H. MEMBERS UNDER COMBINED FORCES

This chapter addresses members subjected to axial force and flexure about one or both axes, flexure and torsion, flexure and shear, and flexure and web crippling.

The chapter is organized as follows:
H1 Combined Axial Load and Bending
H2 Combined Bending and Shear
H3 Combined Bending and Web Crippling
H4 Combined Bending and Torsional Loading

H1 Combined Axial Load and Bending

H1.1 Combined Tensile Axial Load and Bending

The required strengths [effects of factored loads] $\bar{T}$, $\overline{M_x}$, and $\overline{M_y}$ shall satisfy the following interaction equations:

$$
\frac{\overline{M_x}}{M_{ax}} + \frac{\overline{M_y}}{M_{ay}} + \frac{\bar{T}}{T_a} \leq 1.0
$$

(Eq. H1.1-1)

$$
\frac{\overline{M_x}}{M_{ax}} + \frac{\overline{M_y}}{M_{ay}} - \frac{\bar{T}}{T_a} \leq 1.0
$$

(Eq. H1.1-2)

where

$\overline{M_x}, \overline{M_y}$ = Required flexural strengths [moment due to factored loads] with respect to centroidal axes in accordance with ASD, LRFD, or LSD load combinations

$\bar{T}$ = Required tensile axial strength [tensile axial force due to factored loads] in accordance with ASD, LRFD, or LSD load combinations

$M_{ax}, M_{ay}$ = Available flexural strengths [factored resistances] with respect to centroidal axes in considering tension yielding

$$
= S_{ft} F_y / \Omega_b \quad (ASD)
$$

(Eq. H1.1-3a)

$$
= \phi_b S_{ft} F_y \quad (LRFD, LSD)
$$

(Eq. H1.1-3b)

where

$S_{ft}$ = Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis

$F_y$ = Design yield stress determined in accordance with Section A3.3.1

$\Omega_b = 1.67$

$\phi_b = 0.90$ (LRFD and LSD)

$M_{ax}, M_{ay}$ = Available flexural strengths [factored resistances] about centroidal axes in considering compression buckling, as determined in accordance with Chapter F

$T_a$ = Available tensile axial strength [factored resistance], determined in accordance with Chapter D
**H1.2 Combined Compressive Axial Load and Bending**

The *required strengths* [effects due to *factored loads*] $P$, $M_x$, and $M_y$ shall be determined in accordance with Section C1. Each individual ratio in Eq. H1.2-1 shall not exceed unity.

For singly-symmetric unstiffened angle sections not subject to *local buckling* at stress $F_y$, $M_y$ is permitted to be taken as the *required flexural strength* [moment due to *factored loads*] only. For other angle sections or singly-symmetric unstiffened angles subject to *local buckling* at stress $F_y$, $M_y$ shall be taken either as the *required flexural strength* [moment due to *factored loads*] or the *required flexural strength* [moment due to *factored loads*] plus $(\bar{P})L/1000$, whichever results in a lower permissible value of $\bar{P}$.

\[
\frac{\bar{P}}{P_a} + \frac{M_x}{M_{ax}} + \frac{M_y}{M_{ay}} \leq 1.0 \quad \text{(Eq. H1.2-1)}
\]

where

- $\bar{P}$ = *Required compressive axial strength* [compressive axial force due to *factored loads*] determined as required in Section C1, in accordance with ASD, LRFD, or LSD load combinations
- $P_a$ = *Available axial strength* [factored resistance], determined in accordance with Chapter E
- $M_x, M_y$ = *Required flexural strengths* [moment due to *factored loads*], determined as required in Section C1, in accordance with ASD, LRFD, or LSD load combinations
- $M_{ax}, M_{ay}$ = *Available flexural strengths* [factored resistances] about centroidal axes, determined in accordance with Chapter F
- $P_{n\ell}$ = *Nominal axial strength* [resistance] for *local buckling* defined in Section E3.2
- $P_{ne}$ = *Nominal axial strength* [resistance] for yielding and global *buckling* defined in Section E2

**H2 Combined Bending and Shear**

For beams subjected to combined bending and shear, the *required flexural strength* [moment due to *factored loads*], $\bar{M}$, and the *required shear strength* [shear force due to *factored loads*], $\bar{V}$, shall not exceed $M_a$ and $V_a$, respectively.

For beams without shear stiffeners as defined in Section G4, the *required flexural strength* [moment due to *factored loads*], $\bar{M}$, and the *required shear strength* [shear force due to *factored loads*], $\bar{V}$, shall also satisfy the following interaction equation:

\[
\sqrt{\left(\frac{\bar{M}}{M_{a(0)}}\right)^2 + \left(\frac{\bar{V}}{V_a}\right)^2} \leq 1.0 \quad \text{(Eq. H2-1)}
\]

For beams with shear stiffeners as defined in Section G4, when $\bar{M}/M_{a(0)} > 0.5$ and $\bar{V}/V_a > 0.7$, $\bar{M}$ and $\bar{V}$ shall also satisfy the following interaction equation:
0.6 \left( \frac{M}{M_{a/f_0}} \right) + \left( \frac{V}{V_a} \right) \leq 1.3 \quad (Eq. H2-2)

where:

\( \overline{M} \) = Required flexural strength [moment due to factored loads] in accordance with ASD, LRFD, or LSD load combinations

\( \overline{V} \) = Required shear strength [shear force due to factored loads] in accordance with ASD, LRFD or LSD load combinations

\( M_a \) = Available flexural strength [factored resistance] when bending alone is considered, determined in accordance with Chapter F

\( V_a \) = Available shear strength [factored resistance] when shear alone is considered, determined in accordance with Sections G2 to G4

\( M_{a/f_0} \) = Available flexural strength [factored resistance] for globally braced member determined as follows:

(a) For members without transverse web stiffeners, \( M_{a/f_0} \) is determined in accordance with Section F3 with \( F_n = F_y \) or \( M_{ne} = M_y \), and

(b) For members with transverse web stiffeners, \( M_{a/f_0} \) is the lesser of

(1) Available strength [factored resistance] determined in accordance with Section F3 with \( F_n = F_y \) or \( M_{ne} = M_y \), and

(2) Available strength [factored resistance] determined in accordance with Section F4.

\( F_n \) = Global flexural buckling stress as defined in Section F2

\( F_y \) = Yield stress

\( M_{ne} \) = Nominal flexural strength [resistance] considering yielding and global buckling, determined in accordance with Section F2

\( M_y \) = Member yield moment in accordance with Section F2.1

**H3 Combined Bending and Web Crippling**

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed such that the moment, \( \overline{M} \), and the concentrated load or reaction, \( \overline{P} \), satisfy \( \overline{M} \leq M_{a/f_0} \) and \( \overline{P} \leq P_a \). In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced webs, Eq. H3-1 shall be satisfied as follows:

\[
0.91 \left( \frac{\overline{P}}{P_n} \right) + \left( \frac{\overline{M}}{M_{n/f_0}} \right) \leq \frac{1.33}{\Omega} \quad (ASD) \quad (Eq. H3-1a)
\]

\[
0.91 \left( \frac{\overline{P}}{P_n} \right) + \left( \frac{\overline{M}}{M_{n/f_0}} \right) \leq 1.33 \phi \quad (LRFD \text{ and } LSD) \quad (Eq. H3-1b)
\]

where

\( \Omega = 1.70 \ (ASD) \)

\( \phi = 0.90 \ (LRFD) \)

\( \phi = 0.75 \ (LSD) \)

**Exception:** At the interior supports of continuous spans, Eq. H3-1 shall not apply 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 that provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a C-section), Eq. H3-2 shall be satisfied as follows:

\[ 0.88 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n/o}} \right) \leq \frac{1.46}{\Omega} \quad (ASD) \]  
\[ 0.88 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n/o}} \right) \leq 1.46\phi \quad (LRFD \text{ and LSD}) \]  

where
\[ \Omega = 1.70 \ (ASD) \]
\[ \phi = 0.90 \ (LRFD) \]
\[ = 0.75 \ (LSD) \]

(c) For two nested Z-shapes, Eq. H3-3 shall be satisfied as follows:

\[ 0.86 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n/o}} \right) \leq \frac{1.65}{\Omega} \quad (ASD) \]  
\[ 0.86 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n/o}} \right) \leq 1.65\phi \quad (LRFD \text{ and LSD}) \]  

where
\[ \Omega = 1.70 \ (ASD) \]
\[ \phi = 0.90 \ (LRFD) \]
\[ = 0.80 \ (LSD) \]

Eq. H3-3 shall apply to shapes that meet the following limits:
1. \( h/t \leq 150, \)
2. \( N/t \leq 140, \)
3. \( F_y \leq 70 \text{ ksi} \) (483 MPa or 4920 kg/cm²), and
4. \( R/t \leq 5.5 \)

where
\( h = \) Depth of flat portion of web measured along plane of web
\( t = \) Web thickness
\( N = \) Bearing length
\( F_y = \) Yield stress
\( R = \) Inside bend radius

The following conditions shall also be satisfied:
1. The ends of each section are 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 is 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 are in contact.
(iv) The ratio of the thicker to the thinner part does not exceed 1.3.

The following notations shall apply in this section:

\[ \bar{P} = \text{Required strength [force due to factored loads] for concentrated load or reaction in presence of bending moment, determined in accordance with ASD, LRFD, or LSD load combinations} \]

\[ \bar{M} = \text{Required flexural strength [moment due to factored loads] at, or immediately adjacent to, the point of application of the concentrated load or reaction \( \bar{P} \), determined in accordance with ASD, LRFD, or LSD load combinations} \]

\[ P_a = \text{Available strength [factored resistance] for concentrated load or reaction in absence of bending moment, determined in accordance with Sections G5 and G6, as applicable} \]

\[ M_{a/o} = \text{Available flexural strength [factored resistance] about centroidal x-axis in absence of axial load, determined in accordance with Section F3 with } F_n = F_y \text{ or } M_{ne} = M_y \]

\[ M_{n/o} = \text{Nominal flexural strength [resistance] about centroidal x-axis in absence of axial load, determined in accordance with Section F3 with } F_n = F_y \text{ or } M_{ne} = M_y \]

\[ P_n = \text{Nominal strength [resistance] for concentrated load or reaction in absence of bending moment, determined in accordance with Sections G5 and G6, as applicable} \]

\[ F_n = \text{Global flexural buckling stress as defined in Section F2} \]

\[ M_{ne} = \text{Nominal flexural strength [resistance] considering yielding and global buckling, determined in accordance with Section F2} \]

\[ M_y = \text{Member yield moment in accordance with Section F2.1} \]

**H4 Combined Bending and Torsional Loading**

For torsionally unrestrained flexural members subjected to both bending and torsional loading, the available flexural strength [factored resistance] calculated in accordance with Section F3 with \( F_n = F_y \) or \( M_{ne} = M_y \) shall be multiplied by a reduction factor, R.

As specified in Eq. H4-1, the reduction factor, R, shall be equal to the ratio of the maximum normal stresses due to bending alone divided by the combined stresses due to both bending and torsional warping at the point of maximum combined stress on the cross-section. Eq. H4-1 is limited to singly- or doubly-symmetric sections subject to bending about an axis of symmetry and not subject to biaxial bending. The torsional effect for other sections shall be considered using rational engineering analysis.

\[ R = \frac{f_{bending \_max}}{f_{bending} + f_{torsion}} \leq 1 \]  

\( (Eq. \ H4-1) \)

where

\[ f_{bending \_max} = \text{Bending stress at extreme fiber, taken on the same side of the neutral axis as } f_{bending} \]

\[ f_{bending} = \text{Bending stress at location in cross-section where combined bending and torsion stress is maximum} \]

\[ f_{torsion} = \text{Torsional warping stress at location in cross-section where combined bending and torsion stress is maximum} \]

\[ F_n = \text{Global flexural buckling stress as defined in Section F2} \]
\( F_y \) = Yield stress
\( M_{ne} \) = Nominal flexural strength [resistance] considering yielding and global buckling, determined in accordance with Section F2
\( M_y \) = Member yield moment in accordance with Section F2.1

Stresses shall be calculated using full unreduced section properties. For C-sections with edge-stiffened flanges, if the maximum combined stresses occur at the junction of the web and flange, the R factor is permitted to be increased by 15 percent, but the R factor shall not be greater than 1.0.

The provisions of this section shall not apply if the provisions of Sections I6.2.1 and I6.2.2 are used.
I. ASSEMBLIES AND SYSTEMS

This chapter addresses design provisions related to cold-formed steel assemblies and systems.

The chapter is organized as follows:

I1 Built-Up Sections
I2 Floor, Roof, or Wall Steel Diaphragm Construction
I3 Mixed Systems
I4 Cold-Formed Steel Light-Frame Construction
I5 Special Bolted Moment Frame Systems
I6 Metal Roof and Wall Systems
I7 Rack Systems

I1.1 Flexural Members Composed of Two Back-to-Back C-Sections

The maximum longitudinal spacing of connections (one or more welds or other connectors), \( s_{\text{max}} \), joining two C-sections to form an I-section shall be:

\[
\frac{q}{m} = \frac{L}{6} \quad \text{or} \quad \frac{2gT_s}{m}, \quad \text{whichever is smaller} \quad (\text{Eq. I1.1-1})
\]

where

\( L \) = Span of beam

\( g \) = Vertical distance between two rows of connections nearest to top and bottom flanges

\( T_s \) = Available strength [factored resistance] of connection in tension (Chapter J)

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

\( q \) = Design load [factored load] on beam for determining longitudinal spacing of connections (See below for methods of determination.)

The load, \( q \), shall be 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 as equal to three times the uniformly distributed load, based on the critical load combinations for ASD, LRFD, and LSD. If the length of bearing of a concentrated load or reaction is smaller than the longitudinal connection spacing, \( s \), the required strength [force due to factored loads] of the connections closest to the load or reaction shall be calculated as follows:

\[
T_r = \frac{P_s m}{2g} \quad (\text{Eq. I1.1-2})
\]

where

\( P_s \) = Concentrated load [factored load] or reaction based on critical load combinations for ASD, LRFD, and LSD

\( T_r \) = Required strength [force due to factored loads] of connection in tension

The allowable maximum spacing of connections, \( s_{\text{max}} \), shall depend 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 is permitted to be adopted:
(a) The connection spacing varies along the beam according to the variation of the load intensity, or
(b) Reinforcing cover plates are welded to the flanges at points where concentrated loads occur. The available shear strength [factored resistance] of the connections joining these plates to the flanges is then used for $T_{gs}$ and $g$ is taken as the depth of the beam.

### I1.2 Compression Members Composed of Two Sections in Contact

For compression members composed of two sections in contact, the available axial strength [factored resistance] shall be determined in accordance with Section E2 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 $(KL/r)_m$ calculated as follows:

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

(Eq. I1.2-1)

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

See Section E2.1 for definition of other symbols.

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

(a) The intermediate fastener or spot weld spacing, $a$, is limited such that $a/r_i$ does not exceed one-half the governing slenderness ratio of the built-up member.

(b) The ends of a built-up compression member are 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.

(c) The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting the required strength [force due to factored loads] in any direction of 2.5 percent of the available axial strength [factored resistance] of the built-up member.

### I1.3 Spacing of Connections in Cover-Plated Sections

To develop the strength required of the compression element, 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), (b), and (c) as follows:

(a) That which is required to transmit the shear between the connected parts on the basis of the available strength [factored resistance] per connection specified elsewhere herein,

(b) $1.5t\sqrt{E/\alpha f_c}$

where

- $t =$ Thickness of the cover plate or sheet
- $E =$ Modulus of elasticity of steel
- $f_c =$ Compressive stress in the cover plate or sheet based on ASD, LRFD, or LSD load combinations
\[ \alpha = \begin{align*} 
&= \text{Coefficient} \\
&= 1.67 \text{ for ASD load combinations, and} \\
&= 1.0 \text{ for LRFD or LSD load combinations} 
\end{align*} \]

(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 that act only as sheathing material and are not considered load-carrying elements.

When any of the limits in (a), (b), or (c) in this section are exceeded, the effective width shall be determined in accordance with Section 1.1.4.

I2 Floor, Roof, or Wall Steel Diaphragm Construction

The following AISI standards shall be applied, as applicable, for diaphragm design: AISI S310, AISI S240, and AISI S400.

User Note:
AISI S310 is for diaphragms and wall diaphragms constructed with profiled steel panels or decks.
AISI S240 is for diaphragms constructed with wood structural panel sheathing; shear walls constructed with flat steel sheet sheathing; wood structural panel sheathing, gypsum board panel sheathing or fiberboard panel sheathing; and strap braced walls utilized in cold-formed steel light-frame construction applications.
AISI S400 includes additional seismic design requirements for diaphragms, shear walls, and strap braced walls covered in AISI S240.

I3 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.

I4 Cold-Formed Steel Light-Frame Construction

The design and installation of structural members utilized in cold-formed steel repetitive framing applications shall be in accordance with AISI S240 and, as applicable, the seismic requirements of AISI S400.

I4.1 All-Steel Design of Wall Stud Assemblies

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 Chapters D through H.
I5 Special Bolted Moment Frame Systems

The design of special bolted moment frame systems shall be in accordance with the requirements of AISI S400.

I6 Metal Roof and Wall Systems

The provisions of Sections I6.1 through I6.4 shall apply to metal roof and wall systems that include cold-formed steel members (girts and purlins), through-fastened wall or roof panels, or standing seam roof panels, as applicable. Members shall be designed in accordance with Section I6.1 or I6.2, as applicable; standing seam roof panel systems shall be designed in accordance with Section I6.3, and roof system bracing and anchorage shall be designed in accordance with Section I6.4.

I6.1 Member Strength: General Cross-Sections and System Connectivity

I6.1.1 Compression Member Design

The nominal axial strength \([\text{resistance}]\), \(P_{n}\), shall be the minimum of \(P_{ne}\), \(P_{n/t}\), and \(P_{nd}\) as given in Sections I6.1.1.1 to I6.1.1.3. For members meeting the geometric and material limits of Section B4, the safety and resistance factors shall be as follows:

\[
\Omega_c = 1.80 \quad (ASD)
\]

\[
\phi_c = 0.85 \quad (LRFD)
\]

\[
= 0.80 \quad (LSD)
\]

For all other members, the safety and resistance factors in Section A1.2(c) shall apply. The available strength \([\text{factored resistance}]\) shall be determined in accordance with the applicable method in Section B3.2.1, B3.2.2 or B3.2.3.

I6.1.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

The nominal compressive strength \([\text{resistance}]\), \(P_{ne}\), for flexural, torsional, or flexural-torsional buckling shall be calculated in accordance with Section E2, except \(F_{cre}\) or \(P_{cre}\) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing, bridging and bracing, and span continuity.

I6.1.1.2 Local Buckling

The nominal compressive strength \([\text{resistance}]\), \(P_{n/t}\), for local buckling shall be calculated in accordance with Section E3, except \(F_n\) or \(P_{cr}\) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing.

I6.1.1.3 Distortional Buckling

The nominal compressive strength \([\text{resistance}]\), \(P_{nd}\), for distortional buckling shall be calculated in accordance with Section E4, except \(P_{crd}\) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing.
I6.1.2 Flexural Member Design

The nominal flexural strength [resistance], \( M_{n} \), shall be the minimum of \( M_{ne} \), \( M_{nl} \), and \( M_{nd} \) as given in Sections I6.1.2.1 to I6.1.2.3. For members meeting the geometric and material limits of Section B4, the safety and resistance factors shall be as follows:

\[
\Omega_b = 1.67 \quad (ASD)
\]
\[\phi_b = 0.90 \quad (LRFD)\]
\[= 0.85 \quad (LSD)\]

For all other members, the safety and resistance factors in Section A1.2(c) shall apply. The available strength [factored resistance] shall be determined in accordance with the applicable method in Section B3.2.1, B3.2.2 or B3.2.3.

I6.1.2.1 Lateral-Torsional Buckling

The nominal flexural strength [resistance], \( M_{ne} \), for lateral-torsional buckling shall be calculated in accordance with Section F2, except \( F_{cre} \) or \( M_{cre} \) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing, bridging and bracing, and span continuity.

I6.1.2.2 Local Buckling

The nominal flexural strength [resistance], \( M_{nl} \), for local buckling shall be calculated in accordance with Section F3, except \( F_{n} \) or \( M_{cr} \) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing.

I6.1.2.3 Distortional Buckling

The nominal flexural strength [resistance], \( M_{nd} \), for distortional buckling of girts and purlins shall be calculated in accordance with Section F4, except \( M_{crd} \) shall be determined including lateral, rotational, and composite stiffness provided by the deck or sheathing.

I6.1.3 Member Design for Combined Flexure and Torsion

The nominal flexural strength [resistance], \( M_{n} \), for members in combined flexure and torsion shall be reduced by applying the reduction factor, \( R \), determined in accordance with Eq. H4-1.

I6.2 Member Strength: Specific Cross-Sections and System Connectivity

I6.2.1 Flexural Members Having One Flange Through-Fastened to Deck or Sheathing

This section shall 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 [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 in accordance with Eq. I6.2.1-1.
Consideration of *distortional buckling* in accordance with Section F4 shall be excluded. The *safety factor* and *resistance factors* given in this section shall be used to determine the *allowable flexural strength* or *design flexural strength [factored resistance]* in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
M_n = R M_{n/o} \quad (Eq. I6.2.1-1)
\]

\[
\Omega_b = 1.67 \quad (ASD)
\]

\[
\phi_b = 0.90 \quad (LRFD) = 0.90 \quad (LSD)
\]

where

- \( R \) = A value obtained from Table I6.2.1-1 for C- or Z-sections
- \( M_{n/o} \) = *Nominal flexural strength with consideration of local buckling only*, as determined from Section F3 with \( F_n = F_y \) or \( M_{ne} = M_y \)

### TABLE I6.2.1-1

<table>
<thead>
<tr>
<th>C- or Z-Section R Values</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Simple Span</strong></td>
</tr>
<tr>
<td>Member Depth Range, in. (mm)</td>
</tr>
<tr>
<td>( d \leq 6.5 ) (165)</td>
</tr>
<tr>
<td>( 6.5 ) (165) (&lt; d \leq 8.5 ) (216)</td>
</tr>
<tr>
<td>( 8.5 ) (216) (&lt; d \leq 12 ) (305)</td>
</tr>
<tr>
<td>( 8.5 ) (216) (&lt; d \leq 12 ) (305)</td>
</tr>
<tr>
<td><strong>Continuous Span</strong></td>
</tr>
<tr>
<td>Profile</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>Z</td>
</tr>
</tbody>
</table>

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

- (a) Member depth \( \leq 12 \) in. (305 mm),
- (b) Member *flanges* with edge stiffeners,
- (c) \( 60 \leq \text{depth/thickness} \leq 170 \),
- (d) \( 2.8 \leq \text{depth/flange width} \leq 5.5 \),
- (e) *Flange width* \( \geq 2.125 \) in. (54.0 mm),
- (f) \( 16 \leq \text{flat width/thickness of flange} \leq 43 \),
- (g) For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than 1.5d,
- (h) Member span length is not greater than 33 feet (10 m),
- (i) Both *flanges* are prevented from moving laterally at the supports,
- (j) Roof or wall panels are steel sheets with 50 ksi (340 MPa or 3520 kg/cm²) minimum *yield stress*, and a minimum of 0.018 in. (0.46 mm) base metal *thickness*, having a minimum rib depth of 1-1/8 in. (29 mm), spaced at a maximum of 12 in. (305 mm) on
centers and attached in a manner to effectively inhibit relative movement between
the panel and member flange,

(k) Insulation is 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,

(l) Fastener type is, at minimum, No. 12 self-drilling or self-tapping sheet metal screws
or 3/16 in. (4.76 mm) rivets, having washers with 1/2 in. (12.7 mm) diameter,

(m) Fasteners are not standoff type screws,

(n) Fasteners are spaced not greater than 12 in. (305 mm) on centers and placed near the
center of the member flange, and adjacent to the panel high rib, and

(o) The ratio of tensile strength to design yield stress shall not be less than 1.08.

If variables fall outside any of the above-stated limits, the user shall perform full-scale
tests in accordance with Section K2.1 of this Specification or apply a rational engineering
analysis procedure. For continuous purlin and girt systems in which adjacent bay span
lengths vary by more than 20 percent, the R values for the adjacent bays shall be taken
from the simple-span values in Table I6.2.1-1. The user is permitted to perform tests in
accordance with Section K2.1 as an alternative to the procedure described in this section.

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 I6.2.1-1 by the following correction factor, r:

\[ r = 1.00 - 0.01 t_i \quad \text{when} \ t_i \text{ is in inches} \]
\[ r = 1.00 - 0.0004 t_i \quad \text{when} \ t_i \text{ is in millimeters} \]

where

\[ t_i = \text{Thickness of uncompressed glass fiber blanket insulation} \]

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

See Section I6.2.2 of Appendix A or B for the provisions of this section.

I6.2.3 Compression Members Having One Flange Through-Fastened to Deck or Sheathing

These provisions shall apply 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 in accordance with (a) and (b). Consideration of distortional buckling in
accordance with Section E4 shall be excluded.

(a) The weak axis nominal strength [resistance], \( P_n \), shall be calculated in accordance with
Eq. I6.2.3-1. The safety factor and resistance factors given in this section shall be used to
determine the allowable axial strength or design axial strength [factored resistance] in
accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[ P_n = C_1 C_2 C_3 A E / 29500 \]
\[ \Omega = 1.80 \quad (ASD) \]
\[ \phi = 0.85 \quad (LRFD) \]
\[ = 0.80 \quad (LSD) \]
where

\[ C_1 = (0.79x + 0.54) \]  
\[ C_2 = (1.17\alpha t + 0.93) \]  
\[ C_3 = \alpha(2.5b - 1.63d) + 22.8 \]

(Eq. I6.2.3-2)  
(Eq. I6.2.3-3)  
(Eq. I6.2.3-4)

where

\[ x = \text{For Z-sections, fastener distance from outside \textit{web} edge divided by \textit{flange} width, as shown in Figure I6.2.3-1} \]
\[ = \text{For C-sections, \textit{flange} width minus fastener distance from outside \textit{web} edge divided by \textit{flange} width, as shown in Figure I6.2.3-1} \]

\[ \alpha = \text{Coefficient for conversion of units} \]
\[ = 1 \quad \text{when} \ t, \ b, \ \text{and} \ d \ \text{are in inches} \]
\[ = 0.0394 \quad \text{when} \ t, \ b, \ \text{and} \ d \ \text{are in mm} \]
\[ = 0.394 \quad \text{when} \ t, \ b, \ \text{and} \ d \ \text{are in cm} \]
\[ t = \text{C- or Z-section thickness} \]
\[ b = \text{C- or Z-section \textit{flange} width} \]
\[ d = \text{C- or Z-section depth} \]

\[ A = \text{Full unreduced cross-sectional area of C- or Z-section} \]
\[ E = \text{Modulus of elasticity of steel} \]
\[ = 29,500 \text{ ksi for U.S. customary units} \]
\[ = 203,000 \text{ MPa for SI units} \]
\[ = 2,070,000 \text{ kg/cm}^2 \text{ for MKS units} \]

Figure I6.2.3-1 Definition of \( x \)

Eq. I6.2.3-1 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. \textit{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 \textit{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 or 0.105 kg/cm/cm) (fastener at mid-flange width for stiffness determination) determined in accordance with AISI S901,
9. C- and Z-sections having a minimum \textit{yield stress} of 33 ksi (228 MPa or 2320
kg/cm²), and
(10) Span length not exceeding 33 feet (10.1 m).

(b) The strong axis available strength [factored resistance] shall be determined in accordance with Sections E2 and E3.

### I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

The provisions of this section shall apply only to the United States and Mexico. See Section I6.2.4 of Appendix A.

### I6.3 Standing Seam Roof Panel Systems

#### I6.3.1 Strength of Standing Seam Roof Panel Systems

Under gravity loading, the nominal strength [resistance] of standing seam roof panels shall be determined in accordance with Chapter F of this Specification or shall be tested in accordance with AISI S906. Under uplift loading, the nominal strength [resistance] of standing seam roof panel systems shall be determined in accordance with AISI S906. Tests shall be performed in accordance with AISI S906 with the following exceptions:

(a) The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 is permitted.

(b) Existing tests conducted in accordance with CEGS 07416 Uplift Test Procedure prior to the adoption of these provisions are permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 Test Procedure, is permitted provided the tested end conditions represent the installed condition, and the test follows the requirements given in AISI S906. All test results shall be evaluated in accordance with this section.

For load combinations that include wind uplift, additional provisions are provided in Section I6.3.1a of Appendix A.

When the number of physical test assemblies is three (3) or more, safety factor and resistance factors shall be determined in accordance with the procedures of Section K2.1.1(c) with the following definitions for the variables:

- $\beta_0$ = 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 the same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)

The safety factor, \( \Omega \), shall not be less than 1.67, and the resistance factor, \( \phi \), shall not be greater than 0.9 (LRFD and LSD).

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

### 16.4 Roof System Bracing and Anchorage

#### 16.4.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load With Top Flange Connected to Metal Sheathing

Anchorage, in the form of a device capable of transferring force from the roof diaphragm to a support, shall be provided for roof systems with C-sections or Z-sections, designed in accordance with Chapter F, Section 16.1 or 16.2, having through-fastened or standing seam sheathing attached to the top flanges. Each anchorage device shall be designed to resist the force, \( P_L \), determined by Eq. I6.4.1-1 and shall satisfy the minimum stiffness requirement of Eq. I6.4.1-7. In addition, purlins shall be restrained laterally by the sheathing so that the maximum top flange lateral displacements between lines of lateral anchorage resulting from ASD load combinations (specified loads for LSD) do not exceed the span length divided by 360.

Anchorage devices shall be located in each purlin bay and shall connect to the purlin at or near the purlin top flange. If anchorage devices are not directly connected to all purlin lines of each purlin bay, provision shall be made to transmit the forces from other purlin lines to the anchorage devices. It shall be demonstrated that the required force, \( P_L \), can be transferred to the anchorage device through the roof sheathing and its fastening system. The lateral stiffness of the anchorage device shall be determined by analysis or testing. This analysis or testing shall account for the flexibility of the purlin web above the attachment of the anchorage device connection.

\[
P_{Lj} = \sum_{i=1}^{N_p} \left( K_{eff,i} \cdot \frac{P_i}{K_{total,i}} \right) \tag{Eq. I6.4.1-1}
\]

where

- \( P_{Lj} \) = Lateral force to be resisted by the \( j \)th anchorage device (positive when restraint is required to prevent purlins from translating in the upward roof slope direction)
- \( N_p \) = Number of purlin lines on roof slope
- \( i \) = Index for each purlin line (\( i=1, 2, \ldots, N_p \))
- \( j \) = Index for each anchorage device (\( j=1, 2, \ldots, N_a \))
- \( N_a \) = Number of anchorage devices along a line of anchorage
- \( P_i \) = Lateral force introduced into the system at the \( i \)th purlin

\[
P_{Lj} = (C1)W_{p_j} \left[ \frac{C2}{1000} \frac{I_{xy}}{L_x} + (C3) \frac{(m + 0.25b)t}{d^2} \right] \alpha \cos \theta - (C4) \sin \theta \tag{Eq. I6.4.1-2}
\]
where
C1, C2, C3, and C4 = Coefficients tabulated in Tables I6.4.1-1 to I6.4.1-3

\[ W_{pi} = \text{Total required vertical load supported by the } i^{th} \text{ purlin in a single bay} \]

\[ = w_i L \]  \hspace{1cm} (Eq. I6.4.1-3)

where
\[ w_i = \text{Required distributed gravity load supported by the } i^{th} \text{ purlin per unit length} \]
\( \text{(determined from the critical ASD, LRFD, or LSD load combination depending on the design method used)} \)

\[ I_{xy} = \text{Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the purlin web} \]
\( (I_{xy} = 0 \text{ for C-sections}) \)

\[ L = \text{Purlin span length} \]

\[ m = \text{Distance from shear center to mid-plane of web} \]
\( (m = 0 \text{ for Z-sections}) \)

\[ b = \text{Top flange width of purlin} \]

\[ t = \text{Purlin thickness} \]

\[ I_x = \text{Moment of inertia of full unreduced section about centroidal axis perpendicular to the purlin web} \]

\[ d = \text{Depth of purlin} \]

\[ \alpha = \begin{cases} +1 & \text{for top flange facing in the up-slope direction} \\ -1 & \text{for top flange facing in the down-slope direction} \end{cases} \]

\[ \theta = \text{Angle between vertical and plane of purlin web} \]

\[ K_{eff,i,j} = \text{Effective lateral stiffness of the } j^{th} \text{ anchorage device with respect to the } i^{th} \text{ purlin} \]

\[ = \left[ \frac{1}{K_a} + \frac{d_{p,i,j}}{(C6)A_pE} \right]^{-1} \]  \hspace{1cm} (Eq. I6.4.1-4)

where
\[ d_{p,i,j} = \text{Distance along roof slope between the } i^{th} \text{ purlin line and the } j^{th} \text{ anchorage device} \]

\[ K_a = \text{Lateral stiffness of the anchorage device} \]

\[ C6 = \text{Coefficient tabulated in Tables I6.4.1-1 to I6.4.1-3} \]

\[ A_p = \text{Gross cross-sectional area of roof panel per unit width} \]

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

\[ K_{total,i} = \text{Effective lateral stiffness of all elements resisting force } P_i \]

\[ = \sum_{j=1}^{N_p} (K_{eff,i,j}) + K_{sys} \]  \hspace{1cm} (Eq. I6.4.1-5)

where
\[ K_{sys} = \text{Lateral stiffness of the roof system, neglecting anchorage devices} \]

\[ = \left( \frac{C5}{1000} \right)(N_p)^{\frac{ELt^2}{d^2}} \]  \hspace{1cm} (Eq. I6.4.1-6)

where
\[ C5 = \text{Coefficient tabulated in Tables I6.4.1-1 to I6.4.1-3} \]
For multi-span systems, force $P_i$, calculated in accordance with Eq. I6.4.1-2 and coefficients C1 to C4 from Tables I6.4.1-1 to I6.4.1-3 for the “Exterior Frame Line,” “End Bay,” or “End Bay Exterior Anchor” cases, shall not be taken as less than 80 percent of the force determined using the coefficients C2 to C4 for the corresponding “All Other Locations” case.

For systems with multiple spans and anchorage devices at supports (support restraints), where the two adjacent bays have different section properties or span lengths, the following procedures are to be used:

(a) The values for $P_i$ in Eq. I6.4.1-1 and Eq. I6.4.1-8 shall be taken as the average of the values found from Eq. I6.4.1-2 evaluated separately for each of the two bays, and

(b) The values of $K_{sys}$ used in Eq. I6.4.1-5 and $K_{eff,i,j}$ in Eq. I6.4.1-1 and Eq. I6.4.1-5 shall be calculated using Eq. I6.4.1-4 and Eq. I6.4.1-6, with $L$, $t$, and $d$ taken as the average of the values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or mid-points, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures are to be used to account for the influence of the adjacent bays:

(a) The values for $P_i$ in Eq. I6.4.1-1 and Eq. I6.4.1-8 shall be taken as the average of the values found from Eq. I6.4.1-2 evaluated separately for each of the three bays,

(b) The value of $K_{sys}$ in Eq. I6.4.1-5 shall be calculated using Eq. I6.4.1-6, with $L$, $t$, and $d$ taken as the average of the values from the three bays,

(c) The values of $K_{eff,i,j}$ shall be calculated using Eq. I6.4.1-4, with $L$ taken as the span length of the bay under consideration, and

(d) At an end bay, when computing the average values for $P_i$ or averaging the properties for computing $K_{sys}$, the averages shall be found by adding the value from the first interior bay and two times the value from the end bay and then dividing the sum by three.

The total effective stiffness at each purlin shall satisfy the following equation:

$$K_{total,i} \geq K_{req} \quad (Eq. \ I6.4.1-7)$$

where

$$K_{req} = \Omega \frac{20}{d} \sum_{i=1}^{N_p} P_i \quad (ASD) \quad (Eq. \ I6.4.1-8a)$$

$$K_{req} = \frac{1}{\phi} \frac{20}{d} \sum_{i=1}^{N_p} P_i \quad (LRFD, LSD) \quad (Eq. \ I6.4.1-8b)$$

$$\Omega = 2.00 \quad (ASD)$$

$$\phi = 0.75 \quad (LRFD)$$

$$\phi = 0.70 \quad (LSD)$$

In lieu of Eqs. I6.4.1-1 through I6.4.1-6, lateral restraint forces are permitted to be determined from alternative analysis. Alternative analysis shall include the first- or
second-order effect and account for the effects of roof slope, torsion resulting from applied loads eccentric to shear center, torsion resulting from the lateral resistance provided by the sheathing, and load applied oblique to the principal axes. Alternative analysis shall also include the effects of the lateral and rotational restraint provided by sheathing attached to the top flange. Stiffness of the anchorage device shall be considered and shall account for flexibility of the purlin web above the attachment of the anchorage device connection.

When lateral restraint forces are determined from rational engineering analysis, the maximum top flange lateral displacement of the purlin between lines of lateral bracing resulting from ASD load combinations (specified loads for LSD) shall not exceed the span length divided by 360. The lateral displacement of the purlin top flange at the line of restraint, $\Delta_{tf}$, shall satisfy Eq. I6.4.1-9a for ASD load combinations and Eq. I6.4.1-9b for LRFD or LSD load combinations:

$$\Delta_{tf} \leq \frac{1}{\Omega} \frac{d}{20} \quad (ASD) \quad (Eq. \ I6.4.1-9a)$$

$$\Delta_{tf} \leq \phi \frac{d}{20} \quad (LRFD, LSD) \quad (Eq. \ I6.4.1-9b)$$

<table>
<thead>
<tr>
<th>Simple Span</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>C6</th>
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<th>C4</th>
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<td>0.33</td>
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<tr>
<td>First Interior Bay</td>
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<td>0.95</td>
<td>3.1</td>
<td>0.33</td>
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<td>0.46</td>
<td>2.7</td>
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Table I6.4.1-1  
Coefficients for Support Restraints

Table I6.4.1-2  
Coefficients for Mid-Point Restraints
Table I6.4.1-3
Coefficients for One-Third Point Restraints

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<td>0.82</td>
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<td>and 1st Int. Bay Ext.</td>
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</tr>
<tr>
<td></td>
<td>Anchor</td>
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<tr>
<td></td>
<td>All Other Locations</td>
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<td>6.1</td>
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<td>0.96</td>
<td>0.69</td>
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<td>End Bay Exterior Anchor</td>
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<td>13</td>
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<td>0.59</td>
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<td>and 1st Int. Bay Ext.</td>
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<td></td>
<td>Anchor</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>All Other Locations</td>
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<td>3.8</td>
<td>45</td>
<td>0.65</td>
<td>0.10</td>
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I6.4.2 Alternate Lateral and Stability Bracing for Purlin Roof Systems

Torsional bracing that prevents twist about the longitudinal axis of a member in combination with lateral restraints that resist lateral displacement of the top flange at the frame line is permitted in lieu of the requirements of Section I6.4.1. A torsional brace shall prevent torsional rotation of the cross-section at a discrete location along the span of the member. Connection of braces shall be made at or near both flanges of ordinary open sections, including C- and Z-sections. The effectiveness of torsional braces in preventing torsional rotation of the cross-section and the required strength [brace force due to factored loads] of lateral restraints at the frame line shall be determined by rational engineering analysis or testing. The lateral displacement of the top flange of the C- or Z-section at the frame line shall be limited to \( \frac{d}{20\Omega} \) for ASD load combinations or \( \phi d/20 \) for LRFD and LSD load combinations, where \( d \) is the depth of the C- or Z-section member, \( \Omega \) is the safety factor for ASD, and \( \phi \) is the resistance factor for LRFD and LSD. Lateral displacement between frame lines resulting from ASD load combinations (specified loads for LSD) shall be limited to \( L/180 \), where \( L \) is the span length of the member. For pairs of adjacent purlins that provide bracing against twist to each other, external anchorage of torsional brace forces shall not be required.

where
\[
\begin{align*}
\Omega &= 2.0 \quad (ASD) \\
\phi &= 0.75 \quad (LRFD) \\
&= 0.70 \quad (LSD)
\end{align*}
\]

I7 Rack Systems

Steel rack systems shall be designed and constructed in accordance with ANSI MH16.1.
J. CONNECTIONS AND JOINTS

This chapter addresses cold-formed steel-to-steel welded, bolted, screw, and power-actuated fastener connections, as well as connections of cold-formed steel structural members to other materials.

This chapter is organized as follows:
J1 General Provisions
J2 Welded Connections
J3 Bolted Connections
J4 Screw Connections
J5 Power-Actuated Fastener (PAF) Connections
J6 Rupture
J7 Connections to Other Materials

J1 General Provisions

Connections shall be designed to transmit the required strength [force due to factored loads] acting on the connected members with consideration of eccentricity where applicable.

J2 Welded Connections

The design of welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 3/16 in. (4.76 mm) or less shall be based on the following sub-sections. Additionally, the following specifications or standards shall apply:
For the United States and Mexico:
(a) AWS D1.3, and
(b) AWS C1.1 or AWS C1.3 for resistance welds.
For Canada:
(a) CSA W59, and
(b) CSA W55.3 for resistance welds.

For the design of welded connections in which the thickness of the thinnest connected part is greater than 3/16 in. (4.76 mm), the following specifications or standards shall apply:
(a) ANSI/AISC 360 for the United States and Mexico, and
(b) CSA S16 for Canada.

For diaphragm applications, Section I2 shall apply.
See Appendix A or B for additional requirements.

J2.1 Groove Welds in Butt Joints

The nominal strength [resistance], \( P_{\text{nr}} \), of a groove weld in a butt joint, welded from one or both sides, shall be determined in accordance with (a) or (b), as applicable. The corresponding safety factor and resistance factors shall be used to determine the available strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.
(a) For tension or compression normal to the effective area, the nominal strength [resistance], \( P_{\text{nr}} \), shall be calculated in accordance with Eq. J2.1-1:
\[ P_n = L t_e F_y \quad (Eq. \, J2.1-1) \]
\[ \Omega = 1.70 \quad (ASD) \]
\[ \phi = 0.90 \quad (LRFD) \]
\[ = 0.80 \quad (LSD) \]

(b) For shear on the effective area, the nominal strength [resistance], \( P_n \), shall be the smaller value calculated in accordance with Eqs. J2.1-2 and J2.1-3:

\[ P_n = L t_e 0.6F_{xx} \quad (Eq. \, J2.1-2) \]
\[ \Omega = 1.90 \quad (ASD) \]
\[ \phi = 0.80 \quad (LRFD) \]
\[ = 0.70 \quad (LSD) \]

\[ P_n = L t_e F_y / \sqrt{3} \quad (Eq. \, J2.1-3) \]
\[ \Omega = 1.70 \quad (ASD) \]
\[ \phi = 0.90 \quad (LRFD) \]
\[ = 0.80 \quad (LSD) \]

where

- \( P_n \) = Nominal strength [resistance] of groove weld
- \( L \) = Length of weld
- \( t_e \) = Effective throat dimension of groove weld
- \( F_y \) = Yield stress of lowest strength base steel
- \( F_{xx} \) = Tensile strength of electrode classification

### J2.2 Arc Spot Welds

Arc spot welds, where permitted by this Specification, shall be for welding sheet steel to thicker supporting members or sheet to sheet in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest sheet 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, as shown in Figures J2.2-1 and J2.2-2, shall be used where the thickness of the sheet is less than 0.028 in. (0.711 mm). Weld washers shall have a thickness between 0.05 in. (1.27 mm) and 0.08 in. (2.03 mm), with a minimum pre-punched hole of 3/8 in. (9.53 mm) in diameter. Sheet-to-sheet welds shall not require weld washers.

Arc spot welds shall be specified by a minimum effective diameter of fused area, \( d_e \). The minimum allowable effective diameter shall be 3/8 in. (9.53 mm).
J2.2.1 Minimum Edge and End Distance

The distance from the centerline of an arc spot weld to the end or edge of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where d is the visible diameter of the outer surface of the arc spot weld. See Figures J2.2.1-1 and J2.2.1-2 for details.
J2.2.2 Shear

J2.2.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength \( P_{nv} \), of each arc spot weld between the sheet or sheets and a thicker supporting member shall be determined by using the smaller of either (a) or (b). The corresponding safety factor and resistance factors shall be used to determine the available strength \( P_{av} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

(a) \( P_{nv} = \frac{\pi d_e^2}{4} 0.75 F_{xx} \)  
\[ \Omega = 2.55 \quad (ASD) \]  
\[ \phi = 0.60 \quad (LRFD) \]  
\[ = 0.50 \quad (LSD) \]  

(b) For \( (d_a/t) \leq 0.815 \sqrt{E/F_u} \)
\( P_{nv} = 2.20 t d_a F_u \)  
\[ \Omega = 2.20 \quad (ASD) \]  
\[ \phi = 0.70 \quad (LRFD) \]  
\[ = 0.60 \quad (LSD) \]  

For \( 0.815 \sqrt{E/F_u} < (d_a/t) < 1.397 \sqrt{E/F_u} \)
\( P_{nv} = 0.280 \left[ 1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] t d_a F_u \)  
\[ \Omega = 2.80 \quad (ASD) \]  
\[ \phi = 0.55 \quad (LRFD) \]  
\[ = 0.45 \quad (LSD) \]  

For \( (d_a/t) \geq 1.397 \sqrt{E/F_u} \)
\( P_{nv} = 1.40 t d_a F_u \)  
\( (Eq. J2.2.2.1-4) \)
\[ \Omega = 3.05 \quad (ASD) \]
\[ \phi = 0.50 \quad (LRFD) \]
\[ = 0.40 \quad (LSD) \]

where

\[ P_{nv} = \text{Nominal shear strength [resistance] of arc spot weld} \]
\[ d_e = \text{Effective diameter of fused area at plane of maximum shear transfer} \]
\[ = 0.7d - 1.5t \leq 0.55d \quad (Eq. J2.2.2.1-5) \]

where

\[ d = \text{Visible diameter of outer surface of arc spot weld} \]
\[ t = \text{Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer} \]

\[ F_{xx} = \text{Tensile strength of electrode classification} \]

---

**J2.2.2.2 Shear Strength for Sheet-to-Sheet Connections**

The *nominal shear strength [resistance], P_{nv},* for each weld between two sheets of equal...
thickness shall be determined in accordance with Eq. J2.2.2.2-1. The safety factor and resistance factors in this section shall be used to determine the available strength [factored resistance], \( P_{av} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
P_{nv} = 1.65td_aF_u
\]

(\text{Eq. J2.2.2.2-1})

\[
\Omega = 2.20 \quad (\text{ASD})
\]

\[
\phi = 0.70 \quad (\text{LRFD})
\]

\[
= 0.60 \quad (\text{LSD})
\]

where

\[ P_{nv} = \text{Nominal shear strength [resistance] of sheet-to-sheet connection} \]

\( t \) = Base steel \( \text{thickness (exclusive of coatings) of single welded sheet} \)

\( d_a \) = Average diameter of arc spot weld at mid-thickness of \( t \). See Figure J2.2.2.2-1 for diameter definitions

\[
= (d - t)
\]

(\text{Eq. J2.2.2.2-2})

where

\( d \) = Visible diameter of the outer surface of arc spot weld

\( F_u \) = Tensile strength of sheet as determined in accordance with Section A3.1 or A3.2

In addition, the following limits shall apply:
(a) \( F_u \leq 59 \text{ ksi (407 MPa or 4150 kg/cm}^2) \)
(b) \( F_{xx} > F_u \), and
(c) \( 0.028 \text{ in. (0.71 mm) \leq t \leq 0.0635 \text{ in. (1.61 mm)}} \).

See Section J2.2.2.1 for definition of \( F_{xx} \).

**Figure J2.2.2.2-1 Arc Spot Weld – Sheet to Sheet**

**J2.2.3 Tension**

The uplift nominal tensile strength [resistance], \( P_{nt} \), of each concentrically loaded arc spot weld connecting sheet(s) and supporting member shall be computed as the smaller of either Eq. J2.2.3-1 or Eq. J2.2.3-2, as follows. The safety factors and resistance factors shall be used to determine the available strength [factored resistance], \( P_{ab} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
P_{nt} = \frac{\pi d_c^2}{4} F_{xx}
\]

(\text{Eq. J2.2.3-1})
\[ P_{nt} = 0.8(F_u/F_y)^2 t d_a F_u \]  
(Eq. J2.2.3-2)

For panel and deck applications:
- \( \Omega = 2.50 \) (ASD)
- \( \phi = 0.60 \) (LRFD)
- \( = 0.50 \) (LSD)

For all other applications:
- \( \Omega = 3.00 \) (ASD)
- \( \phi = 0.50 \) (LRFD)
- \( = 0.40 \) (LSD)

The following limits shall apply:
(a) \( t d_a F_u \leq 3 \) kips (13.3 kN or 1360 kg),
(b) \( F_{xx} \geq 60 \) ksi (410 MPa or 4220 kg/cm²),
(c) \( F_u \leq 82 \) ksi (565 MPa or 5770 kg/cm²) (of connecting sheets), and
(d) \( F_{xx} > F_u \).

See Section J2.2.2.1 for definitions of variables.

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 shall be determined by using the sum of the sheet thicknesses as given by Eq. J2.2.3-2.

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

Where it is shown by measurement that a given weld procedure consistently gives a larger effective diameter, \( d_e \), or average diameter, \( d_{av} \) as applicable, this larger diameter is permitted to be used provided the particular welding procedure used for making those welds is followed.

**J2.2.4 Combined Shear and Tension on an Arc Spot Weld**

For arc spot weld connections subjected to a combination of shear and tension, the following interaction check shall be applied:

If \[ \left( \frac{T}{P_{at}} \right)^{1.5} \leq 0.15, \] no interaction check is required.

If \[ \left( \frac{T}{P_{at}} \right)^{1.5} > 0.15, \]

\[ \left( \frac{\bar{V}}{P_{av}} \right)^{1.5} + \left( \frac{T}{P_{at}} \right)^{1.5} \leq 1 \]  
(Eq. J2.2.4-1)

where
- \( \bar{T} \) = Required tensile strength [tensile force due to factored loads] per connection fastener determined in accordance with ASD, LRFD, or LSD load combinations
- \( \bar{V} \) = Required shear strength [shear force due to factored loads] per connection fastener, determined in accordance with ASD, LRFD, or LSD load combinations
\( P_{at} = \text{Available tension strength [factored resistance]} \) as given by Section J2.2.3

\( P_{av} = \text{Available shear strength [factored resistance]} \) as given by Section J2.2.2

In addition, the following limitations shall be satisfied:

(a) \( F_u \leq 105 \text{ ksi (724 MPa or 7380 kg/cm}^2) \),

(b) \( F_{xx} \geq 60 \text{ ksi (414 MPa or 4220 kg/cm}^2) \),

(c) \( t_d F_u \leq 3 \text{ kips (13.3 kN or 1360 kg)} \),

(d) \( F_u/F_y \geq 1.02 \), and

(e) \( 0.47 \text{ in. (11.9 mm)} \leq d \leq 1.02 \text{ in. (25.9 mm)} \).

See Section J2.2.2.1 for definition of variables.

### J2.3 Arc Seam Welds

Arc seam welds covered by this Specification shall apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position (See Figure J2.3-1), and

(b) Sheet to sheet in the horizontal or flat position.

![Figure J2.3-1 Arc Seam Welds – Sheet to Supporting Member in Flat Position](image)

#### J2.3.1 Minimum Edge and End Distance

The distance from the centerline of an arc seam weld to the end or edge of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end or edge of the member be less than 1.0d, where \( d \) is the visible width of the arc seam weld. See Figure J2.3.1-1 for details.
J2.3.2 Shear

J2.3.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength [resistance], \( P_{nv} \), of arc seam welds shall be determined by using the smaller of either Eq. J2.3.2.1-1 or Eq. J2.3.2.1-2. The safety factor and resistance factors in this section shall be used to determine the available strength [factored resistance], \( P_{av} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
P_{nv} = \left( \frac{\pi d_e^2}{4} + L d_e \right) \Omega F_{xx} \quad \text{(Eq. J2.3.2.1-1)}
\]

\[
P_{nv} = 2.5F_u (0.25L + 0.96d_a) \quad \text{(Eq. J2.3.2.1-2)}
\]

\[
\Omega = 2.55 \quad \text{(ASD)}
\]

\[
\phi = 0.60 \quad \text{(LRFD)}
\]

\[
\phi = 0.50 \quad \text{(LSD)}
\]

where

\( P_{nv} \) = Nominal shear strength [resistance] of arc seam weld

\( d_e \) = Effective width of arc seam weld at fused surfaces

\[
= 0.7d - 1.5t \quad \text{(Eq. J2.3.2.1-3)}
\]

where

\( d \) = Visible width of arc seam weld

\( L \) = Length of seam weld not including circular ends

(For computation purposes, \( L \) shall not exceed 3d)

\( d_a \) = Average width of arc seam weld

\[
= (d - t) \text{ for single or double sheets} \quad \text{(Eq. J2.3.2.1-4)}
\]

\( F_u, F_{xx}, \) and \( t \) = Values as defined in Section J2.2.2.1

J2.3.2.2 Shear Strength for Sheet-to-Sheet Connections

The nominal shear strength [resistance], \( P_{nv} \), for each weld between two sheets of equal thickness shall be determined in accordance with Eq. J2.3.2.2-1. The safety factor and resistance factors in this section shall be used to determine the available strength [factored resistance], \( P_{av} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2.
or B3.2.3.

\[ P_{nv} = 1.65t_dF_u \]  
\[ \Omega = 2.20 \quad (ASD) \]
\[ \phi = 0.70 \quad (LRFD) \]
\[ = 0.60 \quad (LSD) \]

where

\[ P_{nv} = \text{Nominal shear strength [resistance] of sheet-to-sheet connection} \]
\[ d_a = \text{Average width of arc seam weld at mid-thickness. See Figure J2.3.2.2-1 for width definitions.} \]
\[ d_a = (d - t) \]  
\[ = \text{Visible width of the outer surface of arc seam weld} \]
\[ t = \text{Base steel thickness (exclusive of coatings) of single welded sheet} \]
\[ F_u = \text{Tensile strength of sheet as determined in accordance with Section A3.1 or A3.2} \]

In addition, the following limits shall apply:
(a) \( F_u \leq 59 \) ksi (407 MPa or 4150 kg/cm²),
(b) \( F_{xx} > F_u \), and
(c) \( 0.028 \) in. (0.711 mm) \( \leq t \leq 0.0635 \) in. (1.61 mm).

![Figure J2.3.2.2-1 Arc Seam Weld – Sheet to Sheet](image)

\[ d_a = 0.7d - 1.5t \leq 0.55d \]
\[ d_a = d - t \]

**J2.4 Top Arc Seam Sidelap Welds**

**J2.4.1 Shear Strength of Top Arc Seam Sidelap Welds**

The *nominal shear strength [resistance]*, \( P_{nv} \), for longitudinal loading of *top arc seam sideload* welds shall be determined in accordance with Eq. J2.4.1-1. The following limits shall apply:
(a) \( h_{st} \leq 1.25 \) in. (31.8 mm),
(b) \( F_{xx} \geq 60 \) ksi (414 MPa),
(c) \( 0.028 \) in. (0.711 mm) \( \leq t \leq 0.064 \) in. (1.63 mm), and
(d) \( 1.0 \) in. (25.4 mm) \( \leq L_w \leq 2.5 \) in. (63.5 mm).

where

\[ h_{st} = \text{Nominal seam height. See Figure J2.4.1-1} \]
\[ F_{xx} = \text{Tensile strength of electrode classification} \]
\[ \begin{align*}
L_w &= \text{Length of top arc seam sidelap weld} \\
t &= \text{Base steel thickness (exclusive of coatings) of thinner connected sheet} \\
P_{nv} &= [4.0(F_u/F_{sy})-1.52](t/L_w)^{0.33}L_w t F_u \\
\Omega &= 2.60 \text{ (ASD)} \\
\phi &= 0.60 \text{ (LRFD)} \\
&= 0.55 \text{ (LSD)} \\
\text{where} \\
P_{nv} &= \text{Nominal shear strength [resistance] of top arc seam sidelap weld} \\
F_u &= \text{Specified minimum tensile strength of connected sheets as determined in accordance with Section A3.1.1, A3.1.2, or A3.1.3} \\
F_{sy} &= \text{Specified minimum yield stress of connected sheets as determined in accordance with Section A3.1.1, A3.1.2, or A3.1.3}
\end{align*} \] (Eq. J2.4.1-1)

(a) Vertical Leg and Overlapping Hem Joint

(b) Back-to-Back Vertical leg Joint

Figure J2.4.1-1 Top Arc Seam Sidelap Weld

It is permitted to exclude the connection design reduction specified in Sections A3.1.2, A3.1.3(b), and A3.1.3(c) for top arc seam welds provided the arc seam welds meet minimum spacing requirements along steel deck diaphragm side laps.

The minimum end distance and the weld spacing shall satisfy the shear rupture requirements in Section J6.
The top arc seam sidelpad weld connection shall be made as follows:
(a) Vertical legs in either vertical leg and overlapping hem joints or vertical leg joints fit snugly, and
(b) In hem joints, the overlapping hem is crimped onto the vertical leg and the crimp length shall be longer than the specified weld length, Lw.
Holes or openings in the hem at either one or both ends of the weld are permitted.

J2.5 Fillet Welds

Fillet welds covered by this Specification shall apply to the welding of joints in any position, either:
1. Sheet to sheet, or
2. Sheet to thicker steel member.

The nominal shear strength [resistance], Pnv, of a fillet weld shall be the lesser of Pnv1 and Pnv2 as determined in accordance with this section. The corresponding safety factors and resistance factors given in this section shall be used to determine the available strength [factored resistance], Pav, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

(a) For longitudinal loading:
For L/t < 25

\[ P_{nv1} = \left( 1 - \frac{0.01L}{t_1} \right) t_1 F_{u1} \]  
(Eq. J2.5-1)

\[ P_{nv2} = \left( 1 - \frac{0.01L}{t_2} \right) t_2 F_{u2} \]  
(Eq. J2.5-2)

\[ \Omega = 2.55 \ (ASD) \]
\[ \phi = 0.60 \ (LRFD) \]
\[ = 0.50 \ (LSD) \]

For L/t ≥ 25

\[ P_{nv1} = 0.75 t_1 F_{u1} \]  
(Eq. J2.5-3)

\[ P_{nv2} = 0.75 t_2 F_{u2} \]  
(Eq. J2.5-4)

\[ \Omega = 3.05 \ (ASD) \]
\[ \phi = 0.50 \ (LRFD) \]
\[ = 0.40 \ (LSD) \]

(b) For transverse loading:

\[ P_{nv1} = t_1 F_{u1} \]  
(Eq. J2.5-5)

\[ P_{nv2} = t_2 F_{u2} \]  
(Eq. J2.5-6)

\[ \Omega = 2.35 \ (ASD) \]
\[ \phi = 0.65 \ (LRFD) \]
\[ = 0.60 \ (LSD) \]

where
\[ t_1, t_2 = \text{Thickness of connected parts, as shown in Figures J2.5-1 and J2.5-2} \]
\[ t = \text{Lesser value of } t_1 \text{ and } t_2 \]
\[ F_{u1}, F_{u2} = \text{Tensile strength of connected parts corresponding to thicknesses } t_1 \text{ and } t_2 \]
\[ P_{nv1}, P_{nv2} = \text{Nominal shear strength [resistance] corresponding to connected thicknesses } t_1 \text{ and } t_2 \]

In addition, for \( t > 0.10 \text{ in. (2.54 mm)} \), the nominal strength [resistance] determined in accordance with (1) and (2) shall not exceed the following value of \( P_n \):
\[ P_n = 0.75 \ t_w L F_{xx} \]  
\( (Eq. J2.5-7) \)
\[ \Omega = 2.55 \ (ASD) \]
\[ \phi = 0.60 \ (LRFD) \]
\[ = 0.50 \ (LSD) \]

where
\[ P_n = \text{Nominal fillet weld strength [resistance]} \]
\( L = \text{Length of fillet weld} \)
\( F_{xx} = \text{Tensile strength of electrode classification} \)
\( t_w = \text{Effective throat} \)
\[ = 0.707 \ w_1 \text{ or } 0.707 \ w_2, \text{ whichever is smaller. A larger effective throat is permitted if measurement shows that the welding procedure to be used consistently yields a larger value of } t_w. \]

where
\( w_1, w_2 = \text{leg of weld (see Figures J2.5-1 and J2.5-2) and } w_1 \leq t_1 \text{ in lap joints} \)

### J2.6 Flare Groove Welds

Flare groove welds covered by this Specification shall apply to welding of joints in any position, either sheet to sheet for flare V-groove welds, sheet to sheet for flare bevel groove welds, or sheet to thicker steel member for flare bevel groove welds.

The nominal shear strength [resistance], \( P_{nv} \), of a flare groove weld shall be determined in accordance with this section. The corresponding safety factors and resistance factors given in this section shall be used to determine the available strength [factored resistance], \( P_{av} \), in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

Larger effective throat thicknesses, \( t_w \), than those determined by Eq. J2.6-5 or Eq. J2.6-7, as appropriate, are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.
(a) For flare bevel groove welds, transverse loading (see Figure J2.6-1):

\[ P_{nv} = 0.833tLF_u \]  \hspace{1cm} (Eq. J2.6-1)

\[ \Omega = 2.55 \hspace{1cm} (ASD) \]

\[ \phi = 0.60 \hspace{1cm} (LRFD) \]

\[ = 0.50 \hspace{1cm} (LSD) \]

(b) For flare groove welds, longitudinal loading (see Figures J2.6-2 and J2.6-3):

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

\[ P_{nv} = 0.75tLF_u \]  \hspace{1cm} (Eq. J2.6-2)

\[ \Omega = 2.80 \hspace{1cm} (ASD) \]

\[ \phi = 0.55 \hspace{1cm} (LRFD) \]

\[ = 0.45 \hspace{1cm} (LSD) \]

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

\[ P_{nv} = 1.50tLF_u \]  \hspace{1cm} (Eq. J2.6-3)

\[ \Omega = 2.80 \hspace{1cm} (ASD) \]

\[ \phi = 0.55 \hspace{1cm} (LRFD) \]

\[ = 0.45 \hspace{1cm} (LSD) \]

(c) For \( t > 0.10 \text{ in.} \ (2.54 \text{ mm}) \), the nominal strength [resistance] determined in accordance with (a) or (b) shall not exceed the value of \( P_n \) calculated in accordance with Eq. J2.6-4:

\[ P_n = 0.75t_wLF_{xx} \]  \hspace{1cm} (Eq. J2.6-4)

\[ \Omega = 2.55 \hspace{1cm} (ASD) \]
\[ \phi = 0.60 \quad (LRFD) \]
\[ = 0.50 \quad (LSD) \]

where

- \( P_n \) = Nominal flare groove weld strength [resistance]
- \( t \) = Thickness of welded member as illustrated in Figures J2.6-1 to J2.6-3
- \( L \) = Length of weld
- \( F_u \) and \( F_{xx} \) = Values as defined in Section J2.2.2.1
- \( h \) = Height of lip
- \( t_w \) = Effective throat of flare groove weld determined using Eq. J2.6-5 or J2.6-7

(1) For a flare-bevel groove weld

\[
t_w = \left( w_2 + t_w - R + \sqrt{2Rw_1 - w_1^2} \right) \left( \frac{w_1}{w_f} \right) - R \eta \left( \frac{w_2}{w_f} \right) \quad (Eq. J2.6-5)
\]

where

- \( w_1, w_2 \) = Leg of weld (see Figure J2.6-4)
- \( t_{wf} \) = Effective throat of groove weld that is filled flush to the surface, \( w_1 = R \), determined in accordance with Table J2.6-1
- \( R \) = Radius of outside bend surface
- \( \eta \) = \([1 - \cos(\text{equivalent angle})]\) determined in accordance with Table J2.6-1
- \( w_f \) = Face width of weld

\[
t_{wf} = \sqrt{w_1^2 + w_2^2} \quad (Eq. J2.6-6)
\]

---

**Figure J2.6-4 Flare-Bevel Groove Weld**
### Table J2.6-1
**Flare Bevel Groove Welds**

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Throat Depth ($t_{wf}$)</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMAW, FCAW-S[1]</td>
<td>5/16 R</td>
<td>0.274</td>
</tr>
<tr>
<td>GMAW, FCAW-G[2]</td>
<td>5/8 R</td>
<td>0.073</td>
</tr>
<tr>
<td>SAW</td>
<td>5/16 R</td>
<td>0.274</td>
</tr>
</tbody>
</table>

**Notes:**
[1] In Canada, FCAW-S is known as FCAW (self-shielded).
[2] In Canada, FCAW-G is known as FCAW (gas-shielded).

(2) For a flare V-groove weld

$$t_w = \text{smaller of } (t_{wf} - d_1) \text{ and } (t_{wf} - d_2) \quad (Eq. \ J2.6-7)$$

where

- $d_1$ and $d_2$ = Weld offset from flush condition (see Figure J2.6-5)
- $t_{wf}$ = Effective throat of groove weld that is filled flush to the surface (i.e. $d_1 = d_2 = 0$), determined in accordance with Table J2.6-2
- $R_1$ and $R_2$ = Radius of outside bend surface as illustrated in Figure J2.6-5

![Figure J2.6-5 Flare V-Groove Weld](image)

### Table J2.6-2
**Flare V-Groove Welds**

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Throat Depth ($t_{wf}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SMAW, FCAW-S[1]</td>
<td>5/8 R</td>
</tr>
<tr>
<td>SAW</td>
<td>1/2 R</td>
</tr>
</tbody>
</table>

where

- $R$ is the lesser of $R_1$ and $R_2$

**Notes:**
[1] In Canada, FCAW-S is known as FCAW (self-shielded).
[2] In Canada, FCAW-G is known as FCAW (gas-shielded).

### J2.7 Resistance Welds

The *nominal shear strength [resistance]*, $P_{nv}$, of resistance (spot) welds shall be determined in accordance with this section. The *safety factor* and *resistance factors* given in this section shall...
be used to determine the available strength [factored resistance], $P_{av}$, in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$\Omega = 2.35 \text{ (ASD)}$$

$$\phi = 0.65 \text{ (LRFD)}$$

$$\phi = 0.55 \text{ (LSD)}$$

When $t$ is in inches and $P_{nv}$ is in kips:

For $0.01 \text{ in.} \leq t < 0.14 \text{ in.}$

$$P_{nv} = 144t^{1.47} \text{ (Eq. J2.7-1)}$$

For $0.14 \text{ in.} \leq t \leq 0.18 \text{ in.}$

$$P_{nv} = 43.4t + 1.93 \text{ (Eq. J2.7-2)}$$

When $t$ is in millimeters and $P_{nv}$ is in kN:

For $0.25 \text{ mm} \leq t < 3.56 \text{ mm}$

$$P_{nv} = 5.11t^{1.47} \text{ (Eq. J2.7-3)}$$

For $3.56 \text{ mm} \leq t \leq 4.57 \text{ mm}$

$$P_{nv} = 7.6t + 8.57 \text{ (Eq. J2.7-4)}$$

When $t$ is in centimeters and $P_{nv}$ is in kg:

For $0.025 \text{ cm} \leq t < 0.356 \text{ cm}$

$$P_{nv} = 16600t^{1.47} \text{ (Eq. J2.7-5)}$$

For $0.356 \text{ cm} \leq t \leq 0.457 \text{ cm}$

$$P_{nv} = 7750t + 875 \text{ (Eq. J2.7-6)}$$

where

$P_{nv} = \text{Nominal resistance weld shear strength [resistance]}$

$t = \text{Thickness of thinnest outside sheet}$

J3 Bolted Connections

The following design criteria shall apply to steel-to-steel bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is $3/16$ in. (4.76 mm) or less. For bolted connections in which the thickness of the thinnest connected part is greater than $3/16$ in. (4.76 mm), the following specifications and standards shall apply:

(a) ANSI/AISC 360 for the United States and Mexico, and
(b) CSA S16 for Canada.

Bolts, nuts, and washers conforming to one of the following ASTM specifications are approved for use under this Specification:

- ASTM A194/A194M, Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both
- ASTM A307 (Type A), Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength
- ASTM A354 (Grade BD), Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 in.)
ASTM A563, *Standard Specification for Carbon and Alloy Steel Nuts*
ASTM F436, *Standard Specification for Hardened Steel Washers*
ASTM F844, *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*
ASTM F959, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners*
ASTM F959M, *Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use With Structural Fasteners [Metric]*
ASTM F3125, *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions* (for Grades A325, A325M, A490, and A490M only)

When bolts, nuts, and washers other than the above are used, drawings shall clearly indicate 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. The holes for bolts shall not exceed the sizes specified in Table J3-1 (J3-1M), except that larger holes are permitted to be used in column base details or structural systems connected to concrete walls.

### TABLE J3-1
Maximum Size of Bolt Holes, in Inches

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>d &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 (21/2 d)</td>
<td></td>
</tr>
<tr>
<td>1/2 ≤ d &lt; 1</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 (21/2 d)</td>
<td>9/16 by 7/8</td>
</tr>
<tr>
<td>d = 1</td>
<td>11/8</td>
<td>11/4</td>
<td>(11/8) by (15/16)</td>
<td>(11/8) by (21/2)</td>
<td></td>
</tr>
<tr>
<td>d ≥ 1</td>
<td>d + 1/8</td>
<td>d + 5/16</td>
<td>(d + 1/8) by (d + 3/8)</td>
<td>(d + 1/8) by (21/2 d)</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1 The alternative short-slotted hole is only applicable for d=1/2 in.
(a) For Hole Deformation Considered

When the bolt hole deformation is considered in design in accordance with Eq. J3.3.2 -1, the following restrictions shall be applied:

(1) Standard holes are used in bolted connections, except that oversized and slotted holes are permitted to be used as approved by the designer,

(2) The length of slotted holes is normal to the direction of the shear load, and

(3) Washers or backup plates are installed over oversized or slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Section K2.

(b) For Hole Deformation Not Considered

When the bolt hole deformation is not considered in design, oversized holes and short-slotted holes are permitted. The holes for bolts shall not exceed the sizes specified in Table J3-1 (J3-1M).

Slotted or oversized holes shall be taken as standard holes when the holes occur within the lap of lapped or nested Z-members, subject to the following restrictions:

(1) 1/2 in. (12 mm)-diameter bolts only with or without washers or backup plates,

(2) Maximum slot size is 9/16 in. × 7/8 in. (15 mm × 23 mm), slotted vertically,

(3) Maximum oversize hole is 5/8 in. (16 mm) diameter,

(4) Minimum member thickness is 0.060 in. (1.52 mm) nominal,

(5) Maximum member yield stress is 60 ksi (414 MPa, and 4220 kg/cm²), and

(6) Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

### J3.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d, where d is the nominal bolt diameter. In addition, the minimum distance between centers of bolt holes shall provide clearance for bolt heads, nuts, washers and the wrench. For oversized and slotted holes, the clear distance between the edges of two adjacent holes shall not be less than 2d.
J3.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5d, where d is the nominal bolt diameter. For oversized and slotted holes, the distance between the edge of the hole and the edge or end of the member shall not be less than d.

J3.3 Bearing

The available bearing strength [factored resistance] of bolted connections shall be determined in accordance with Sections J3.3.1 and J3.3.2. For conditions not shown, the available bearing strength [factored resistance] of bolted connections shall be determined by tests.

J3.3.1 Bearing Strength Without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the nominal bearing strength [resistance], $P_{nb}$, of the connected sheet for each loaded bolt shall be determined in accordance with Eq. J3.3.1-1. The safety factor and resistance factors given in this section shall be used to determine the available strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

$$P_{nb} = C m_f d t F_u \quad (Eq. J3.3.1-1)$$

$$\Omega = 2.50 \quad (ASD)$$

$$\phi = 0.60 \quad (LRFD)$$

$$\phi = 0.50 \quad (LSD)$$

where

$C$ = Bearing factor, determined in accordance with Table J3.3.1-1

$m_f$ = Modification factor for type of bearing connection, which is determined according to Table J3.3.1-2

$d$ = Nominal bolt diameter

$t$ = Uncoated sheet thickness

$F_u$ = Tensile strength of sheet as defined in Section A3.1 or A3.2

<table>
<thead>
<tr>
<th>Thickness of Connected Part, t, in. (mm)</th>
<th>Connections With Standard Holes</th>
<th>Connections With Oversized or Short-Slotted Holes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ratio of Fastener Diameter to Member Thickness, $d/t$</td>
<td>$C$</td>
<td>Ratio of Fastener Diameter to Member Thickness, $d/t$</td>
</tr>
<tr>
<td>$0.024 \leq t &lt; 0.1875 \quad (0.61 \leq t &lt; 4.76)$</td>
<td>$d/t &lt; 10$</td>
<td>$3.0$</td>
</tr>
<tr>
<td>$10 \leq d/t \leq 22$</td>
<td>$4 - 0.1(d/t)$</td>
<td>$7 \leq d/t \leq 18$</td>
</tr>
<tr>
<td>$d/t &gt; 22$</td>
<td>$1.8$</td>
<td>$d/t &gt; 18$</td>
</tr>
</tbody>
</table>

Note: Oversized or short-slotted holes within the lap of lapped or nested Z-members as defined in Section J3 are permitted to be considered as standard holes.
J3.3.2 Bearing Strength With Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the nominal bearing strength [resistance], \( P_{nb} \), shall be calculated in accordance with Eq. J3.3.2-1. The safety factor and resistance factors given in this section shall be used to determine the available strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3. In addition, the available strength [factored resistance] shall not exceed the available strength [factored resistance] obtained in accordance with Section J3.3.1.

\[
P_{nb} = (4.64\alpha t + 1.53)dtF_u
\]

\[\Omega = 2.22 \quad (ASD)\]
\[\phi = 0.65 \quad (LRFD)\]
\[= 0.55 \quad (LSD)\]

where
\( \alpha \) = Coefficient for conversion of units
\( = 1 \) for U.S. customary units (with \( t \) in inches)
\( = 0.0394 \) for SI units (with \( t \) in mm)
\( = 0.394 \) for MKS units (with \( t \) in cm)

See Section J3.3.1 for definitions of other variables.
J3.4 Shear and Tension in Bolts

See Section J3.4 of Appendix A or B for provisions provided in this section.

J4 Screw Connections

The provisions of this section shall apply to steel-to-steel screw connections within specified limitations used for cold-formed steel structural members. All provisions in Section J4 shall apply to screws with 0.08 in. \((2.03 \text{ mm}) \leq d \leq 0.25 \text{ in.} \ (6.35 \text{ mm})\). 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 \([\text{resistances}]\) shall also be limited by Chapter D.

For diaphragm applications, Section I2 shall be used.

Except where otherwise indicated, the following safety factor or resistance factor shall be used to determine the available strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

\[
\Omega = 3.00 \quad (\text{ASD}) \\
\phi = 0.50 \quad (LRFD) \\
= 0.40 \quad (\text{LSD})
\]

Alternatively, design values for a particular application are permitted to be based on tests, with the safety factor, \(\Omega\), and the resistance factor, \(\phi\), determined in accordance with Section K2.

The following notation shall apply to Section J4:

- \(d\) = Nominal screw diameter
- \(d_h\) = Screw head diameter or hex washer head integral washer diameter
- \(d_w\) = Steel washer diameter
- \(d'_w\) = Effective pull-over resistance diameter
- \(P_{nv}\) = Nominal shear strength \([\text{resistance}]\) of sheet per screw
- \(P_{nvs}\) = Nominal shear strength \([\text{resistance}]\) of screw as reported by manufacturer or determined by independent laboratory testing
- \(P_{not}\) = Nominal pull-out strength \([\text{resistance}]\) of sheet per screw
- \(P_{nov}\) = Nominal pull-over strength \([\text{resistance}]\) of sheet per screw
- \(P_{nts}\) = Nominal tension strength \([\text{resistance}]\) of screw as reported by manufacturer or determined by independent laboratory testing
- \(t_1\) = Thickness of member in contact with screw head or washer
- \(t_2\) = Thickness of member not in contact with screw head or washer
- \(t_c\) = Lesser of depth of penetration and thickness \(t_2\)
- \(F_{u1}\) = Tensile strength of member in contact with screw head or washer
- \(F_{u2}\) = Tensile strength of member not in contact with screw head or washer

J4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

J4.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5d.
J4.3 Shear

J4.3.1 Shear Strength Limited by Tilting and Bearing

The nominal shear strength \[\text{resistance}\] of sheet per screw, \(P_{nv}\), shall be determined in accordance with this section.

For \(t_2/t_1 \leq 1.0\), \(P_{nv}\) shall be taken as the smallest of

\[
P_{nv} = 4.2 \left(t_2^3d\right)^{1/2}F_{u2} \quad \text{(Eq. J4.3.1-1)}
\]
\[
P_{nv} = 2.7 \ t_1 \ d \ F_{u1} \quad \text{(Eq. J4.3.1-2)}
\]
\[
P_{nv} = 2.7 \ t_2 \ d \ F_{u2} \quad \text{(Eq. J4.3.1-3)}
\]

For \(t_2/t_1 \geq 2.5\), \(P_{nv}\) shall be taken as the smaller of

\[
P_{nv} = 2.7 \ t_1 \ d \ F_{u1} \quad \text{(Eq. J4.3.1-4)}
\]
\[
P_{nv} = 2.7 \ t_2 \ d \ F_{u2} \quad \text{(Eq. J4.3.1-5)}
\]

For \(1.0 < t_2/t_1 < 2.5\), \(P_{nv}\) shall be calculated by linear interpolation between the above two cases.

J4.3.2 Shear in Screws

The nominal shear strength \[\text{resistance}\] of the screw shall be taken as \(P_{nvs}\).

In lieu of the value provided in Section J4, the safety factor or the resistance factor is permitted to be determined in accordance with Section K2.1 and shall be taken as \(1.25\Omega \leq 3.0\) (ASD), \(\phi/1.25 \geq 0.5\) (LRFD), or \(\phi/1.25 \geq 0.4\) (LSD).

J4.4 Tension

For screws that carry tension, the head of the screw or washer, if a washer is provided, shall have a diameter \(d_h\) or \(d_w\) not less than \(5/16\) in. (7.94 mm). The nominal washer thickness shall be at least \(0.050\) in. (1.27 mm) for \(t_1\) greater than \(0.027\) in. (0.686 mm) and at least \(0.024\) in. (0.610 mm) for \(t_1\) equal to or less than \(0.027\) in. (0.686 mm). The washer shall be at least \(0.063\) in. (1.60 mm) thick when \(5/8\) in. (15.9 mm) \(< d_w \leq 3/4\) in. (19.1 mm).

J4.4.1 Pull-Out Strength

The nominal pull-out strength \[\text{resistance}\] of sheet per screw, \(P_{not}\), shall be calculated as follows:

\[
P_{not} = 0.85 \ t_c \ d \ F_{u2} \quad \text{(Eq. J4.4.1-1)}
\]

J4.4.2 Pull-Over Strength

The nominal pull-over strength \[\text{resistance}\] of sheet per screw, \(P_{nov}\), shall be calculated as follows:

\[
P_{nov} = 1.5t_1d'_w \ F_{u1} \quad \text{(Eq. J4.4.2-1)}
\]

where

\(d'_w\) = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

(a) For a round head, hex head (Figure J4.4.2-1(1)), pancake screw washer head (Figure J4.4.2-1(2)), or hex washer head (Figure J4.4.2-1(3)) screw with an independent and solid steel washer beneath the screw head:
\[ d'_w = d_h + 2t_w + t_1 \leq d_w \]  

(Eq. J4.4.2-2)

where  
\[ t_w = \text{Steel washer thickness} \]

(b) For a round head, a hex head, or a hex washer head screw without an independent washer beneath the screw head:

\[ d'_w = d_h \] but not larger than 3/4 in. (19.1 mm)

(c) For a domed (non-solid and either independent or integral) washer beneath the screw head (Figure J4.4.2-1(4)), it is permitted to use \( d'_w \) as calculated in Eq. J4.4.2-2, where \( t_w \) is the thickness of the domed washer. In the equation, \( d'_w \) shall not exceed 3/4 in. (19.1 mm).

Figure J4.4.2-1 Screw Pull-Over With Washer
J4.4.3 Tension in Screws

The *nominal tension strength* [resistance] of the screw shall be taken as $P_{nts}$.

In lieu of the value provided in Section J4, the *safety factor* or the *resistance factor* is permitted to be determined in accordance with Section K2.1 and shall be taken as $1.25 \Omega \leq 3.0$ (ASD), $\phi/1.25 \geq 0.5$ (LRFD), or $\phi/1.25 \geq 0.4$ (LSD).

J4.5 Combined Shear and Tension

J4.5.1 Combined Shear and Pull-Over

For a screw *connection* subjected to combined shear and pull-over, the *required shear strength* [shear due to factored loads], $\overline{V}$, and *required tension strength* [tension due to factored loads], $\overline{T}$, shall not exceed the corresponding *available strength* [factored resistance] determined by Sections J4.3 and J4.4, respectively.

In addition, the following requirements shall be met:

\[
\frac{\overline{V}}{P_{nv}} + 0.71 \frac{\overline{T}}{P_{nov}} \leq \frac{1.10}{\Omega} \quad (ASD) \tag{Eq. J4.5.1-1a}
\]

\[
\frac{\overline{V}}{P_{nv}} + 0.71 \frac{\overline{T}}{P_{nov}} \leq 1.10\phi \quad (LRFD, LSD) \tag{Eq. J4.5.1-1b}
\]

where

- $\overline{V}$ = *Required shear strength* [shear force due to factored loads] per connection screw, determined in accordance with ASD, LRFD, or LSD load combinations
- $\overline{T}$ = *Required tension strength* [tensile force due to factored loads] per connection screw, determined in accordance with ASD, LRFD, or LSD load combinations
- $P_{nv} = \text{Nominal shear strength} [\text{resistance}]$ of sheet per screw
  \[
  P_{nv} = 2.7t_1d_F u_1 \quad (Eq. J4.5.1-2)
  \]
- $P_{nov} = \text{Nominal pull-over strength} [\text{resistance}]$ of sheet per screw
  \[
  P_{nov} = 1.5t_1d_w F_{u1} \quad (Eq. J4.5.1-3)
  \]

where

- $d_w = \text{Larger of screw head diameter or washer diameter}$
- $\Omega = 2.35$ (ASD)
- $\phi = 0.65$ (LRFD)
- $\phi = 0.55$ (LSD)

Eq. J4.5.1-1 shall be valid for *connections* that meet the following limits:

(a) $0.0285 \text{ in. (0.724 mm)} \leq t_1 \leq 0.0445 \text{ in. (1.13 mm),}$
(b) No. 12 and No. 14 self-drilling screws with or without washers,
(c) $d_w \leq 0.75 \text{ in. (19.1 mm),}$
(d) Washer dimension limitations of Section J4.4 apply,
(e) $F_{u1} \leq 70 \text{ ksi (483 MPa or 4920 kg/cm}^2),$ and
(f) $t_2/t_1 \geq 2.5.$

For eccentrically loaded *connections* that produce a nonuniform pull-over force on the screw, the *nominal pull-over strength* [resistance] shall be taken as 50 percent of $P_{nov}$. 
J4.5.2 Combined Shear and Pull-Out

For a screw connection subjected to combined shear and pull-over, the required shear strength [shear due to factored loads], $V$, and required tension strength [tension due to factored loads], $T$, shall not exceed the corresponding available strength [factored resistance] determined by Sections J4.3 and J4.4, respectively.

In addition, the following requirement shall be met:

$$\frac{V}{P_{nvs}} + \frac{T}{P_{nts}} \leq 1.15 \frac{\Omega}{\phi}$$

(ASD) (Eq. J4.5.2-1a)

$$\frac{V}{P_{nvs}} + \frac{T}{P_{nts}} \leq 1.15\phi$$

(LRFD, LSD) (Eq. J4.5.2-1b)

where

$P_{nvs} = \text{Nominal shear strength [resistance] of sheet per screw}$

$P_{nts} = \text{Nominal pull-out strength [resistance] of sheet per screw}$

$\Omega = 2.55 \text{ (ASD)}$

$\phi = 0.60 \text{ (LRFD)}$

$\phi = 0.50 \text{ (LSD)}$

Other variables are as defined in Section J4.5.1.

Eq. J4.5.2-1 shall be valid for connections that meet the following limits:

(a) $0.0297$ in. $(0.754 \text{ mm}) \leq t_2 \leq 0.0724$ in. $(1.84 \text{ mm})$,

(b) No. 8, 10, 12, or 14 self-drilling screws with or without washers,

(c) $F_{u2} \leq 121 \text{ ksi} (834 \text{ MPa or 8510 kg/cm}^2)$, and

(d) $1.0 \leq F_u/F_y \leq 1.62$.

J4.5.3 Combined Shear and Tension in Screws

For screws subjected to a combination of shear and tension forces, the required shear strength [shear due to factored loads], $V$, and required tension strength [tension due to factored loads], $T$, shall not exceed the corresponding available strength [factored resistance] determined by Sections J4.3.2 and J4.4.3, respectively.

In addition, the following requirement shall be met:

$$\frac{V}{P_{nvs}} + \frac{T}{P_{nts}} \leq 1.3 \frac{\Omega}{\phi}$$

(ASD) (Eq. J4.5.3-1a)

$$\frac{V}{P_{nvs}} + \frac{T}{P_{nts}} \leq 1.3\phi$$

(LRFD, LSD) (Eq. J4.5.3-1b)

where

$V$ = Required shear strength [shear force due to factored loads], determined in accordance with ASD, LRFD, or LSD load combinations

$T$ = Required tension strength [tensile force due to factored loads], determined in accordance with ASD, LRFD, or LSD load combinations

$P_{nvs} = \text{Nominal shear strength [resistance] of screw as reported by manufacturer or}$
determined by independent laboratory testing
\[ P_{nts} = \text{Nominal tension strength [resistance]} \] of screw as reported by manufacturer or
determined by independent laboratory testing
\[ \Omega = \text{Safety factor in accordance with Section J4} \]
\[ \phi = \text{Resistance factor in accordance with Section J4} \]

**J5 Power-Actuated Fastener (PAF) Connections**

The provisions of this section shall apply to steel-to-steel PAF connections within specified limitations. The steel thickness of the substrate not in contact with the PAF head shall be limited to a maximum of 0.75 in. (19.1 mm). The steel thickness of the substrate in contact with the PAF head shall be limited to a maximum of 0.06 in. (1.52 mm). The washer diameter shall not exceed 0.6 in. (15.2 mm) in computations, although the actual diameter may be larger. PAF diameter shall be limited to a range of 0.106 in. (2.69 mm) to 0.206 in. (5.23 mm).

For diaphragm applications, the provisions of Section I2 shall be used.

Alternatively, the available strengths [factored resistances] for any particular application are permitted to be determined through independent laboratory testing, with the resistance factors, \( \phi \), and safety factors, \( \Omega \), determined in accordance with Section K2. The values of \( P_{ntp} \) and \( P_{nvp} \) are permitted to be reported by the manufacturer.

The following notation shall apply to Section J5:
- \( a \) = Major diameter of tapered PAF head
- \( d \) = Fastener diameter measured at near side of embedment
  = \( d_s \) for PAF installed such that entire point is located behind far side of embedment material
- \( d_{ae} \) = Average embedded diameter, computed as average of installed fastener diameters measured at near side and far side of embedment material
  = \( d_s \) for PAF installed such that entire point is located behind far side of embedment material
- \( d_s \) = Nominal shank diameter
- \( d'_{w} \) = Actual diameter of washer or fastener head in contact with retained substrate
  \( \leq 0.60 \) in. (15.2 mm) in computation
- \( E \) = Modulus of elasticity of steel
- \( \Gamma_{bs} \) = Base stress parameter
  = 66,000 psi (455 MPa or 4640 kg/cm²)
- \( F_{u1} \) = Tensile strength of member in contact with PAF head or washer
- \( F_{u2} \) = Tensile strength of member not in contact with PAF head or washer
- \( F_{uh} \) = Tensile strength of hardened PAF steel
- \( F_{ut} \) = Tensile strength of non-hardened PAF steel
- \( F_{y2} \) = Yield stress of member not in contact with PAF head or washer
- \( HRC_p \) = Rockwell C hardness of PAF steel
- \( d_{dp} \) = PAF point length. See Figure J5-1
- \( P_{nb} \) = Nominal bearing and tilting strength [resistance] per PAF
- \( P_{nos} \) = Nominal pull-out strength [resistance] in shear per PAF
- \( P_{not} \) = Nominal pull-out strength [resistance] in tension per PAF
P_{nov} = \textit{Nominal pull-over strength [resistance]} per PAF

P_{nt} = \textit{Nominal tensile strength [resistance]} per PAF

P_{ntp} = \textit{Nominal tensile strength [resistance]} of PAF

P_{nv} = \textit{Nominal shear strength [resistance]} per PAF

P_{nvp} = \textit{Nominal shear strength [resistance]} of PAF

t_1 = \textit{Thickness of member in contact with PAF head or washer}

 t_2 = \textit{Thickness of member not in contact with PAF head or washer}

 t_w = \textit{Steel washer thickness}

Various fastener dimensions used throughout Section J5 are shown in Figure J5-1.

\[\text{Figure J5-1 Geometric Variables in Power-Actuated Fasteners (PAFs)}\]

\textbf{J5.1 Minimum Spacing, Edge and End Distances}

The minimum center-to-center spacing of the \textit{power-actuated fasteners (PAFs)} and the minimum distance from the center of the fastener to any edge of the connected part, regardless of the direction of the force, shall be as provided by Table J5.1-1.
Table J5.1-1
Minimum Required Edge and Spacing Distances in Steel

<table>
<thead>
<tr>
<th>PAF Shank Diameter, ( d_s ) in. (mm)</th>
<th>Minimum PAF Spacing in. (mm)</th>
<th>Minimum Edge Distance in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 0.106 ) (2.69) ( \leq ) ( d_s ) ( &lt; ) 0.200 (5.08)</td>
<td>1.00 (25.4)</td>
<td>0.50 (12.7)</td>
</tr>
<tr>
<td>( 0.200 ) (5.08) ( \leq ) ( d_s ) ( &lt; ) 0.206 (5.23)</td>
<td>1.60 (40.6)</td>
<td>1.00 (25.4)</td>
</tr>
</tbody>
</table>

J5.2 Power-Actuated Fasteners (PAFs) in Tension

The available tensile strength [factored resistance] per PAF shall be the minimum of the available strengths [factored resistance] determined by the applicable Sections J5.2.1 through J5.2.3. The washer thickness, \( t_w \), limitations of Section J4 shall apply, except that for tapered head fasteners, the minimum thickness, \( t_w \), shall not be less than 0.039 in. (0.991 mm). The thickness of collapsible pre-mounted top-hat washers shall not exceed 0.020 in. (0.508 mm).

J5.2.1 Tension Strength of Power-Actuated Fasteners (PAFs)

The nominal tension strength [resistance] of PAFs, \( P_{ntp} \), is permitted to be calculated in accordance with Eq. J5.2.1-1, and the following safety factor or resistance factors shall be applied to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[
P_{ntp} = \left( \frac{d}{2} \right)^2 \pi F_{uh} \\
\Omega = 2.65 \text{ (ASD)} \\
\phi = 0.60 \text{ (LRFD)} \\
\phi = 0.50 \text{ (LSD)}
\]

\( F_{uh} \) in Eq. J5.2.1-1 shall be calculated with Eq. J5.2.1-2. Alternatively, for fasteners with HRC\( P \) of 52 or more, \( F_{uh} \) is permitted to be taken as 260,000 psi (1790 MPa).

\[
F_{uh} = F_{bs} e^{(HRC_p / 40)} \\
\text{where} \\
e = 2.718
\]

J5.2.2 Pull-Out Strength

The nominal pull-out strength [resistance], \( P_{nov} \), shall be determined through independent laboratory testing with the safety factor or the resistance factor determined in accordance with Section K2. Alternatively, for connections with the entire PAF point length, \( \ell_{dp} \), below \( t_2 \), the following safety factor or resistance factors are permitted to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[
\Omega = 4.00 \text{ (ASD)} \\
\phi = 0.40 \text{ (LRFD)} \\
\phi = 0.30 \text{ (LSD)}
\]

J5.2.3 Pull-Over Strength

The nominal pull-over strength [resistance], \( P_{nov} \), is permitted to be computed in
accordance with Eq. J5.2.3-1, and the following safety factor or resistance factors shall be applied to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[ P_{nov} = \alpha_w \tau_1 d_w F_{u1} \]

\[ (Eq. \ J5.2.3-1) \]

\[ \Omega = 3.00 \ (ASD) \]

\[ \phi = 0.50 \ (LRFD) \]

\[ = 0.40 \ (LSD) \]

where

\[ \alpha_w = 1.5 \] for screw-, bolt-, nail-like flat heads or simple PAF, with or without head washers (see Figures J5-1(a) and J5-1(b))

\[ = 1.5 \] for threaded stud PAFs and for PAFs with tapered standoff heads that achieve pull-over by friction and locking of the pre-mounted washer (see Figure J5-1(c)), with \( a/d_s \) ratio of no less than 1.6 and \( a - d_s \) of no less than 0.12 in. (3.1 mm)

\[ = 1.25 \] for threaded stud PAFs and for PAFs with tapered standoff heads that achieve pull-over by friction and locking of pre-mounted washer (see Figure J5-1(c)), with \( a/d_s \) ratio of no less than 1.4 and \( a - d_s \) of no less than 0.08 in. (2.0 mm)

\[ = 2.0 \] for PAFs with collapsible spring washer (see Figure J5-1(d))

### J5.3 Power-Actuated Fasteners (PAFs) in Shear

The available shear strength [factored resistance] shall be the minimum of the available strengths [factored resistances] determined by the applicable Sections J5.3.1 through J5.3.5.

#### J5.3.1 Shear Strength of Power-Actuated Fasteners (PAFs)

The nominal shear strength [resistance] of PAFs, \( P_{nvp} \), is permitted to be computed in accordance with Eq. J5.3.1-1, and the safety factor and resistance factors shall be applied to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[ P_{nvp} = 0.6(d/2)^2 \pi F_{uh} \]

\[ (Eq. \ J5.3.1-1) \]

\[ \Omega = 2.65 \ (ASD) \]

\[ \phi = 0.60 \ (LRFD) \]

\[ = 0.55 \ (LSD) \]

where

\( F_{uh} \) is determined in accordance with Section J5.2.1.

#### J5.3.2 Bearing and Tilting Strength

For PAFs embedded such that the entire length of PAF point length, \( \ell_{dp} \), is below \( t_2 \), the nominal bearing and tilting strength [resistance], \( P_{nb} \), is permitted to be computed in accordance with Eq. J5.3.2-1, and the following safety factor or resistance factors shall be applied to determine the available strength [factored resistance] in steel in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[ P_{nb} = \alpha_b d_{s1} t_1 F_{u1} \]

\[ (Eq. \ J5.3.2-1) \]
\[ \Omega = 2.05 \text{ (ASD)} \]
\[ \phi = 0.80 \text{ (LRFD)} \]
\[ = 0.65 \text{ (LSD)} \]

where

\[ \alpha_b = 3.7 \text{ for connections with PAF types as shown in Figures J5-1(c) and J5-1(d)} \]
\[ = 3.2 \text{ for other types of PAFs} \]

Eq. J5.3.2-1 shall apply for connections within the following limits:

(a) \( \frac{t_2}{t_1} \geq 2 \),
(b) \( t_2 \geq 1/8 \text{ in. (3.18 mm)} \), and
(c) \( 0.146 \text{ in. (3.71 mm)} \leq d_s \leq 0.177 \text{ in. (4.50 mm)} \).

### J5.3.3 Pull-Out Strength in Shear

For PAFs driven in steel through a depth of at least 0.6\( t_2 \), the nominal pull-out strength [resistance], \( P_{nos} \), in shear is permitted to be computed in accordance with Eq. J5.3.3-1, and the following safety factor and the resistance factors shall be applied to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3:

\[
P_{nos} = \frac{d_{ae}^{1.8} t_2 0.2 F_y E^2}{30} \left( \frac{\Omega}{\phi} \right)^{1/3} \quad (Eq. \ J5.3.3-1)
\]

\[ \Omega = 2.55 \text{ (ASD)} \]
\[ \phi = 0.60 \text{ (LRFD)} \]
\[ = 0.50 \text{ (LSD)} \]

Eq. J5.3.3-1 shall apply for connections within the following limits:

(a) \( 0.113 \text{ in. (2.87 mm)} \leq t_2 \leq 3/4 \text{ in. (19.1 mm)} \), and
(b) \( 0.106 \text{ in. (2.69 mm)} \leq d_s \leq 0.206 \text{ in. (5.23 mm)} \).

### J5.3.4 Net Section Rupture Strength

The available strength [factored resistance] due to net cross-section rupture and block shear shall be determined in accordance with Section J6. In computations of net section rupture and block shear limit states, the hole size shall be taken as 1.10 times the nominal PAF shank diameter, \( d_s \).

### J5.3.5 Shear Strength Limited by Edge Distance

The available shear strength [factored resistance] limited by edge distance shall be computed in accordance with Section J6.1 and the applicable safety factor or the resistance factors provided in Table J6-1 shall be applied to determine the available strength [factored resistance] in accordance with Section B3.2.1, B3.2.2, or B3.2.3. The consideration of edge distance shall be based upon nominal shank diameter, \( d_s \).

### J5.4 Combined Shear and Tension

Effects of combined shear and tension on the PAF connection, including the interaction due to combined shear and pull-out, combined shear and pull-over, and combined shear and tension on the PAF, shall be considered in design.
J6 Rupture

The provisions of this section shall apply to steel-to-steel welded, bolted, screw, and power-actuated fastener (PAF) connections within specified limitations. The design criteria of this section shall apply where the thickness of the thinnest connected part is 3/16 in. (4.76 mm) or less. For connections where the thickness of the thinnest connected part is greater than 3/16 in. (4.76 mm), the following specifications and standards shall apply:

(a) ANSI/AISC 360 for the United States and Mexico, and
(b) CSA S16 for Canada

For connection types utilizing welds or bolts, the nominal rupture strength [resistance], $R_n$, shall be the smallest of the values obtained in accordance with Sections J6.1, J6.2, and J6.3, as applicable. For connection types utilizing screws and PAFs, the nominal rupture strength [resistance], $R_n$, shall be the lesser of the values obtained in accordance with Sections J6.1 and J6.2, as applicable. See Section J6a of Appendix B for additional requirements.

The corresponding safety factor and resistance factors given in Table J6-1 shall be applied to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section B3.2.1, B3.2.2, or B3.2.3.

<table>
<thead>
<tr>
<th>Connection Type</th>
<th>$\Omega$ (ASD)</th>
<th>$\phi$ (LRFD)</th>
<th>$\phi$ (LSD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welds</td>
<td>2.50</td>
<td>0.60</td>
<td>0.75</td>
</tr>
<tr>
<td>Bolts</td>
<td>2.22</td>
<td>0.65</td>
<td>0.75</td>
</tr>
<tr>
<td>Screws and Power-Actuated Fasteners</td>
<td>3.00</td>
<td>0.50</td>
<td>0.75</td>
</tr>
</tbody>
</table>

J6.1 Shear Rupture

The nominal shear rupture strength [resistance], $P_{nv}$, shall be calculated in accordance with Eq. J6.1-1.

$$P_{nv} = 0.6 F_u A_{nv}$$  \hspace{1cm} (Eq. J6.1-1)

where

- $F_u = Tensile strength$ of connected part as specified in Section A3.1 or A3.2
- $A_{nv} = Net area$ subject to shear (parallel to force):
- For a connection where each individual fastener pulls through the material towards the limiting edge individually:
  $$A_{nv} = 2n t e_{net}$$  \hspace{1cm} (Eq. J6.1-2)

  where
  - $n = Number$ of fasteners on critical cross-section
  - $t = Base$ steel thickness of section
  - $e_{net} = Clear$ distance between end of material and edge of fastener hole or weld
- For a beam-end connection where one or more of the flanges are coped:
  $$A_{nv} = (h_{wc} - n_{bdh})t$$  \hspace{1cm} (Eq. J6.1-3)
where
\( h_{wc} \) = Coped flat web depth
\( n_b \) = Number of fasteners along failure path being analyzed
\( d_h \) = Diameter of hole
\( t \) = Thickness of coped web

### J6.2 Tension Rupture

The nominal tensile rupture strength [resistance], \( P_{nt} \), shall be calculated in accordance with Eq. J6.2-1.

\[
P_{nt} = F_u A_e \quad \text{(Eq. J6.2-1)}
\]

where
\( A_e \) = Effective net area subject to tension
\[
A_e = U_{sl} A_{nt} \quad \text{(Eq. J6.2-2)}
\]

where
\( U_{sl} \) = Shear lag factor determined in Table J6.2-1
\( A_{nt} \) = Net area subject to tension (perpendicular to force), except as noted in Table J6.2-1
\[
A_{nt} = A_g - n_b d_h t + t \Sigma [s'/(4g + 2d_h)] \quad \text{(Eq. J6.2-3)}
\]

where
\( A_g \) = 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 fasteners along failure path being analyzed
\( d_h \) = Diameter of a standard hole
\( t \) = Base steel thickness of section
\( F_u \) = Tensile strength of connected part as specified in Section A3.1 or A3.2
The variables in Table J6.2-1 shall be defined as follows:

- $\bar{x}$ = Distance from shear plane to centroid of cross-section
- $L$ = Length of longitudinal weld or length of connection
- $s$ = Sheet width divided by number of bolt holes in cross-section being analyzed
- $d$ = Nominal bolt diameter
- $b_1$ = Out-to-out width of angle leg not connected
- $b_2$ = Out-to-out width of angle leg connected
- $b_f$ = Out-to-out width of flange not connected
- $b_w$ = Out-to-out width of web connected

### J6.3 Block Shear Rupture

The nominal block shear rupture strength [resistance], $P_{nr}$, shall be determined as the lesser of the following:

\[
P_{nr} = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \quad (Eq. J6.3-1)
\]

\[
P_{nr} = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \quad (Eq. J6.3-2)
\]

where
\[ A_{gv} = \text{Gross area subject to shear (parallel to force)} \]
\[ A_{nv} = \text{Net area subject to shear (parallel to force)} \]
\[ A_{nt} = \text{Net area subject to tension (perpendicular to force), except as noted in Table J6.2-1} \]
\[ U_{bs} = \text{Nonuniform block shear factor} \]
\[ = 0.5 \text{ for coped beam shear conditions with more than one vertical row of connectors} \]
\[ = 1.0 \text{ for all other cases} \]
\[ F_y = \text{Yield stress of connected part as specified in Section A3.1 or A3.2} \]
\[ F_u = \text{Tensile strength of connected part as specified in Section A3.1 or A3.2} \]

### J7 Connections to Other Materials

In bolted, screw, and power-actuated fastener connections, the available strength [factored resistance] of the connection to other materials shall be determined in accordance with Section J7.1.

#### J7.1 Strength of Connection to Other Materials

**J7.1.1 Bearing**

Provisions shall be made to transfer bearing forces from steel components covered by this Specification to adjacent structural components made of other materials.

**J7.1.2 Tension**

The pull-over shear or 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, product specifications, product literature, or combination thereof.

**J7.1.3 Shear**

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 [shear or bearing force due to factored loads] on the steel components shall not exceed that allowed by this Specification. The available shear strength [factored resistance] on the fasteners and other material shall not be exceeded. Embedment requirements shall be met. Provisions shall also be made for shearing forces in combination with other forces.
K. STRENGTH FOR SPECIAL CASES

This chapter addresses determination of member and connection strengths through testing. The chapter is organized follows:

K1 Test Standards
K2 Tests for Special Cases

K1 Test Standards

The following test standards are permitted to be used to determine the strength, flexibility, or stiffness of cold-formed steel members and connections via testing:

- AISI S901, Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies
- AISI S902, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns
- AISI S903, Standard Methods for Determination of Uniform and Local Ductility
- AISI S904, Standard Test Methods for Determining the Tensile and Shear Strength of Screws
- AISI S905, Test Standard for Cold-Formed Steel Connections
- AISI S906, Standard Procedures for Panel and Anchor Structural Tests
- AISI S907, Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms
- AISI S908, Base Test Method for Purlins Supporting a Standing Seam Roof System
- AISI S909, Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Beams
- AISI S910, Test Method for Distortional Buckling of Cold-Formed Steel Hat-Shaped Columns
- AISI S911, Method for Flexural Testing of Cold-Formed Steel Hat-Shaped Beams
- AISI S912, Test Procedure for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection
- AISI S913, Test Standard for Hold-Downs Attached to Cold-Formed Steel Structural Framing
- AISI S914, Test Standard for Connectors Attached to Cold-Formed Steel Structural Framing
- AISI S915, Test Standard for Through-the-Web Punchout Cold-Formed Steel Wall Stud Bridging Connectors
- AISI S916, Test Standard for Cold-Formed Steel Framing – Nonstructural Interior Partition Walls With Gypsum Board

K2 Tests for Special Cases

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

K2.1 Tests for Determining Structural Performance

K2.1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance that is required to be established by tests in accordance with Section A1.2(a) or by rational engineering analysis with confirmatory tests in accordance with Section A1.2(b) shall be evaluated with the following performance procedure:

(a) Evaluation of the test results for use with Section A1.2(a) 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 is 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) Evaluation of a rational engineering analysis model by confirmatory tests for use with Section A1.2(b): The correlation coefficient, \( C_C \), between the tested strength [resistance] \( (R_t) \) and the nominal strength [resistance] \( (R_n) \) predicted from the rational engineering analysis model shall be greater than or equal to 0.80. Only one limit state is permitted for evaluation of the rational engineering analysis model being verified, and the test result shall reflect the limit state under consideration.

The rational engineering analysis model is only verified within parameters varied in the testing. Extrapolation outside of the tested parameters is not permitted. For each parameter being evaluated:

1. All other parameters shall be held constant,
2. The nominally selected values of the parameter to be tested shall not bias the study to a specific region of the parameter, and
3. A minimum of three tests shall be performed. No test results shall be eliminated unless a rationale for their exclusion is given.

Dimensions and material properties shall be measured for all test specimens. The as-measured dimensions and properties shall be used in determination of the calculated nominal strength [resistance] \( (R_{n,i}) \) as employed in determining the resistance factor or safety factor in accordance with (c). The specified dimensions and properties shall be used in the determination of the calculated nominal strength [resistance] for design. The bias and variance between the as-measured dimensions and properties and the nominally specified dimensions and properties shall be reflected in the selected material (\( M_m, V_M \)) and fabrication (\( F_m, V_F \)) factors per Table K2.1.1-1. Otherwise, the selected values of \( M_m \) and \( F_m \) shall not be greater than in Table K2.1.1-1, and the values of \( V_M \) and \( V_F \) shall not be less than the values given in Table K2.1.1-1.

(c) The strength of the tested elements, assemblies, connections, or members shall satisfy Eq. K2.1.1-1a or Eq. K2.1.1-1b as applicable.

\[
\sum \gamma_i Q_i \leq \phi R_n \quad \text{for LRFD} \tag{Eq. K2.1.1-1a}
\]
\[
\phi R_n \geq \sum \gamma_i Q_i \quad \text{for LSD} \tag{Eq. K2.1.1-1b}
\]

where

\[
\sum \gamma_i Q_i = \text{Required strength [effect of factored loads] based on the most critical load combination, determined in accordance with Section B2. } \gamma_i \text{ and } Q_i \text{ are load factors and load effects, respectively.}
\]

\[
\phi = \text{Resistance factor}
\]

\[
= C_\phi (M_m F_m P_m) e^{-\beta \phi \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \tag{Eq. K2.1.1-2}
\]

where

\[
C_\phi = \text{Calibration coefficient}
\]
\[ \begin{align*}
&= 1.52 \text{ for LRFD} \\
&= 1.42 \text{ for LSD} \\
&= 1.6 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \\
&= 1.42 \text{ for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \\
M_m &= \text{Mean value of material factor, } M, \text{ determined by statistical analysis or, where applicable, as limited by Table K2.1.1-1 for type of component involved} \\
F_m &= \text{Mean value of fabrication factor, } F, \text{ determined by statistical analysis or, where applicable, as limited by Table K2.1.1-1 for type of component involved} \\
P_m &= \text{Mean value of professional factor, } P, \text{ for tested component} \\
&= 1.0, \text{ if the available strength [factored resistance] is determined in accordance with Section K2.1.1(a); or} \\
&= \frac{\sum_{i=1}^{n} R_{t,i}}{n}, \text{ when the available strength [factored resistance]} \quad (\text{Eq. K2.1.1-3}) \\
&\text{is determined in accordance with Section K2.1.1(b)} \\
\text{where} \\
i &= \text{Index of tests} \\
&= 1 \text{ to } n \\
n &= \text{Total number of tests} \\
R_{t,i} &= \text{Tested strength [resistance] of test } i \\
R_{n,i} &= \text{Calculated nominal strength [resistance] of test } i \text{ per rational engineering analysis model} \\
e &= \text{Natural logarithmic base} \\
&= 2.718 \\
\beta_o &= \text{Target reliability index} \\
&= 2.5 \text{ for structural members and 3.5 for connections for LRFD} \\
&= 3.0 \text{ for structural members and 4.0 for connections for LSD} \\
&= 1.5 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \\
&= 3.0 \text{ for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \\
V_M &= \text{Coefficient of variation of material factor listed in Table K2.1.1-1 for type of component involved} \\
V_F &= \text{Coefficient of variation of fabrication factor listed in Table K2.1.1-1 for type of component involved} \\
C_p &= \text{Correction factor} \\
&= \frac{(1+1/n)m/(m-2)}{\text{ for } n \geq 4} \\
&= 5.7 \text{ for } n = 3 \quad (\text{Eq. K2.1.1-4}) \\
\text{where} \\
n &= \text{Number of tests} \\
m &= \text{Degrees of freedom} \\
&= n - 1
\end{align*} \]
\[ V_P = \text{Coefficient of variation of test results, but not less than 0.065} \]

\[ = \frac{s_t}{R_n}, \text{if the available strength [factored resistance] is determined in accordance with Section K2.1.1(a)} \]

\[ = \frac{s_{C}}{P_{m}}, \text{if the available strength [factored resistance] is determined in accordance with Section K2.1.1 (b)} \]

where

- \( s_t \) = Standard deviation of all of the test results
- \( s_{C} \) = Standard deviation of \( R_{t,i} \) divided by \( R_{n,i} \) for all of the test results
- \( V_Q \) = Coefficient of variation of load effect

\[ V_Q = 0.21 \text{ for LRFD and LSD} \]

\[ = 0.43 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]

\[ = 0.21 \text{ for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]

\[ C_C = \text{Correlation coefficient} \]

\[ = \frac{n \sum R_{t,i} R_{n,i} - (\sum R_{t,i}) (\sum R_{n,i})}{\sqrt{n \sum R_{t,i}^2 - (\sum R_{t,i})^2} \sqrt{n \sum R_{n,i}^2 - (\sum R_{n,i})^2}} \]  

\[ \text{Eq. K2.1.1-7} \]

\[ R_n = \text{Average value of all test results} \]

The listing in Table K2.1.1-1 shall not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.

For steels not listed in Section A3.1, the values of \( M_m \) and \( V_M \) shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. K2.1.1-1a or Eq. K2.1.1-1b, as applicable, except that the resistance factor, \( \phi \), shall be taken as unity and the load factor for dead load shall be taken as 1.0.

(d) For strength determined in accordance with Section K2.1.1(a) or K2.1.1(b), the mechanical properties of the steel sheet shall be determined based on representative samples of the material taken from the test specimen or the flat sheet used to form the test specimen. Alternatively, for connectors or devices that are too small to obtain standard size or sub-size tensile specimens per ASTM A370, and are produced from steel sheet coils that have not undergone a secondary process to alter the mechanical or chemical properties, mechanical properties are permitted to be determined based on mill certificates, and the mean value of the material factor, \( M_m \), shall be equal to 0.85. If the yield stress of the steel is larger than the specified value, the test results shall be adjusted down to the specified minimum yield stress of the steel that the manufacturer intends to use. The test results shall not be adjusted upward if the yield stress of the test specimen is less than the specified minimum yield stress. Similar adjustments shall be made on the basis of tensile strength instead of yield stress where tensile strength is the critical factor.

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

### TABLE K2.1.1-1

Statistical Data for the Determination of Resistance Factor

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>M&lt;sub&gt;m&lt;/sub&gt;</th>
<th>V&lt;sub&gt;M&lt;/sub&gt;</th>
<th>F&lt;sub&gt;m&lt;/sub&gt;</th>
<th>V&lt;sub&gt;F&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Members</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Compression</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Flexure</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Shear and Web Crippling</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Under Combined Forces</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Other Member Limit States&lt;sup&gt;1&lt;/sup&gt;</td>
<td>1.00</td>
<td>0.10</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td><strong>Connections and Joints</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded Connections</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Bolted Connections</td>
<td>1.10</td>
<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Screw Connections</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Power-Actuated Fasteners</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Other Connectors or Fasteners&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.15</td>
</tr>
<tr>
<td>Connections to Structural Concrete</td>
<td>1.10</td>
<td>0.10</td>
<td>0.90</td>
<td>0.10</td>
</tr>
<tr>
<td>Connections to Wood</td>
<td>1.10</td>
<td>0.15</td>
<td>1.00</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Notes:

1 For member limit states captured in testing but not covered in AISI S100.
2 For steel-to-steel connectors and fasteners not already listed in the table.

### K2.1.2 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 K2.1.1, except as modified in this section for allowable strength design.

The allowable strength shall be calculated as follows:

\[
R_d = \frac{R_n}{\Omega}
\]  
\[
\Omega = 6.1
\]

where

\( R_n = \) Average value of all test results

\( \Omega = \) Safety factor

\[
\Omega = \frac{1.6}{\phi}
\]

where

\( \phi = \) A value evaluated in accordance with Section K2.1.1
The required strength shall be determined from ASD load combinations as described in Section B2.

K2.2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies for which the nominal strength [resistance] is computed in accordance with this Specification or its specific references, confirmatory tests are permitted to be made to demonstrate the strength is not less than the nominal strength [resistance], \( R_{nv} \) specified in this Specification or its specific references for the type of behavior involved.

K2.3 Tests for Determining Mechanical Properties

K2.3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A3.3.2 shall be conducted in accordance with this section:

(a) Tensile testing procedures shall conform to the requirements of ASTM A370.

(b) Compressive yield stress determinations shall be made by means of compression tests of short specimens of the section. See AISI S902.

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

(1) For sharp-yielding steel, the yield stress is determined by the autographic diagram method or by the total strain under load method.

(2) For gradual-yielding steel, the yield stress is 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 stress so determined is within five (5) percent with the yield stress that 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 stress shall be determined for the flanges only. In determining such yield stress, 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 are permitted to be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield stress of the section when subjected to the kind of stress under which the member is to be used.

K2.3.2 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 A3.3.2 shall be made in accordance with this section.

The yield stress of flats, \( F_{yf} \), shall be established by means of a weighted average of the
yield stresses 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 stress for each flat portion times its cross-sectional area, divided by the total area of flats in the cross-section. Although 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 stress exceeds the specified minimum yield stress, the yield stress of the flats, $F_{yt}$, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield stress to the actual virgin yield stress.

K2.3.3 Virgin Steel

The following provisions shall apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield stress of the steel after cold forming is computed from the virgin steel properties in accordance with Section A3.3.2. 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 stress and tensile strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.
L. DESIGN FOR SERVICEABILITY

This chapter addresses the serviceability determination using the Effective Width Method and Direct Strength Method, and flange curling.

The chapter is organized as follows:
L1 Serviceability Determination for the Effective Width Method
L2 Serviceability Determination for the Direct Strength Method
L3 Flange Curling

Reduced stiffness values used in the direct analysis method, described in Chapter C, are not intended for use with the provisions of this chapter.

L1 Serviceability Determination for the Effective Width Method

The bending deflection at any moment, M, due to service loads is permitted to be determined by using the effective moment of inertia, $I_{eff}$, determined in accordance with Appendix 1.

L2 Serviceability Determination for the Direct Strength Method

The bending deflection at any moment, M, due to service loads is permitted to be determined by reducing the gross moment of inertia, $I_g$, to an effective moment of inertia for deflection, as given in Eq. L2-1:

$$I_{eff} = I_g \left( \frac{M_d}{M} \right) \leq I_g \quad (Eq. \ L2-1)$$

where
- $M_d$ = Nominal flexural strength [resistance], $M_n$, defined in Chapter F with Direct Strength Method, but with $M_y$ replaced by M in all equations
- $M$ = Moment due to service loads on member to be considered ($M \leq M_y$)

L3 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, Eq. L3-1 is permitted to be applied to compression and tension flanges, either stiffened or unstiffened, as follows:

$$w_f = \sqrt{0.061tdE/\bar{f}_{av}} \cdot \frac{4}{100} \cdot \frac{c_f}{d} \quad (Eq. \ L3-1)$$

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
- $E$ = Modulus of elasticity of steel
- $f_{av}$ = 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.)
- $c_f$ = Amount of curling displacement
M. DESIGN FOR FATIGUE

This chapter addresses cold-formed steel structural members and connections subject to cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the limit state of fatigue.

This chapter is organized as follows:

M1 General
M2 Calculation of Maximum Stresses and Stress Ranges
M3 Design Stress Range
M4 Bolts and Threaded Parts
M5 Special Fabrication Requirements

M1 General

When cyclic loading is a design consideration, the provisions of this chapter shall apply to stresses calculated on the basis of ASD load combinations [specified loads]. The maximum permitted tensile stress shall be 0.6 Fy.

Stress range shall be defined as the magnitude of the change in stress due to the application or removal of the live load [specified 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.

Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components. Fatigue need not be considered when the live load [specified live load] stress range is less than the threshold stress range, FTH, given in Table M1-1.

Evaluation of fatigue strength [resistance] shall not be required if the number of cycles of application of live load [specified live load] is less than 20,000.

The fatigue strength [resistance] determined by the provisions of this chapter shall be applicable to structures with corrosion protection or subject only to non-aggressive atmospheres.

The fatigue strength [resistance] determined by the provisions of this chapter shall be applicable only to structures subject to temperatures not exceeding 300°F (149°C).

The contract documents shall either provide complete details including weld sizes, or specify the planned cycle life and the maximum range of moments, shears, and reactions for the connections.
### Table M1-1

**Fatigue Design Parameters for Cold-Formed Steel Structures**

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant $C_f$</th>
<th>Threshold $F_{TH}$, ksi (MPa)</th>
<th>Reference</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>$3.2 \times 10^{10}$</td>
<td>25 (172) [1760]</td>
<td>M1-1</td>
</tr>
<tr>
<td>As-received base metal and weld metal in members connected by continuous longitudinal welds</td>
<td>II</td>
<td>$1.0 \times 10^{10}$</td>
<td>15 (103) [1050]</td>
<td>M1-2</td>
</tr>
<tr>
<td>Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than or equal to 2 in. (50.8 mm), bolt and screw connections, and spot welds</td>
<td>III</td>
<td>$3.2 \times 10^{9}$</td>
<td>16 (110) [1120]</td>
<td>M1-3, M1-4</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>$1.0 \times 10^{9}$</td>
<td>9 (62) [633]</td>
<td>M1-4</td>
</tr>
</tbody>
</table>

---

**Figure M1-1 Typical Detail for Stress Category I**

**Cold-Formed Steel Channels, Stress Category I**

**Figure M1-2 Typical Detail for Stress Category II**

**Welded I Beam, Stress Category II**

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**Shear Edges**

**Cold-Formed Corner**

---
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.

**M3 Design Stress Range**

The range of stress shall not exceed the design stress range computed using Eq. M3-1 for all stress categories as follows:

\[ F_{SR} = (\alpha C_f / N)^{0.333} \geq F_{TH} \]  

(Eq. M3-1)

where

- \( F_{SR} \) = Design stress range
- \( \alpha \) = Coefficient for conversion of units
  - 1 for U.S. customary units
  - 327 for SI units
  - 352,000 for MKS units
- \( C_f \) = Constant from Table M1-1
- \( N \) = Number of stress range fluctuations in design life
- \( N = Number of stress range fluctuations per day \times 365 \times years of design life \)
- \( F_{TH} \) = Threshold fatigue stress range, maximum stress range for indefinite design life from Table M1-1

**M4 Bolts and Threaded Parts**

For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material shall not exceed the design stress range computed using Equation M3-1. 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/cm\(^2\)).

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 Eq. M3-1. 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/cm\(^2\)). The net tensile area shall be calculated by Eq. M4-1a or M4-1b as applicable.

\[ A_t = (\pi/4) [d_b - (0.9743/n)]^2 \]  

for U.S. Customary units  

(Eq. M4-1a)

\[ A_t = (\pi/4) [d_b - (0.9382p)]^2 \]  

for SI or MKS units  

(Eq. M4-1b)

where:

- \( A_t \) = Net tensile area
- \( d_b \) = Nominal diameter (body or shank diameter)
- \( n \) = Number of threads per inch
- \( p \) = Pitch (mm per thread for SI units and cm per thread for MKS units)

**M5 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 μin. (25 μm) in accordance with 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 μin. (25 μm) in accordance with ASME B46.1 or other equivalent approved standards.

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 shall not be required for sheet material if the welding procedures used result in smooth, flush edges.
APPENDIX 1, EFFECTIVE WIDTH OF ELEMENTS

This appendix addresses the Effective Width Method for elements on cold-formed steel cross-sections subject to compression stress. The effective section properties are used to determine the member strengths and deflections.

This appendix is organized as follows:

1.1 Effective Width of Uniformly Compressed Stiffened Elements
1.2 Effective Width of Unstiffened Elements
1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener
1.4 Effective Width of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

1.1 Effective Width of Uniformly Compressed Stiffened Elements

(a) Strength Determination

The effective width, b, shall be calculated as follows:

\[ b = \rho w \]  
\[ (Eq. 1.1-1) \]

where

\[ w = \text{Flat width as shown in Figure 1.1-1} \]
\[ \rho = \text{Local reduction factor} \]
\[ = 1 \quad \text{when } \lambda \leq 0.673 \]
\[ = (1 - 0.22/\lambda )/\lambda \quad \text{when } \lambda > 0.673 \]  
\[ (Eq. 1.1-2) \]

where

\[ \lambda = \text{Slenderness factor} \]
\[ = \sqrt[3]{\frac{f}{F_{cr}/\mu}} \]  
\[ (Eq. 1.1-3) \]

where

\[ f = \text{Compressive stress in element considered, which is computed as follows:} \]

For flexural members:

(1) In considering global, distortional, and local buckling, \( f = F_n \) as determined in accordance with Chapter F.

(2) In considering inelastic reserve, \( f \) is the stress in the compression element.

(3) If Section F2.4.1 is used, \( f \) is the stress in the element considered at \( M_n \) determined on the basis of the effective section.

For compression members, \( f \) is equal to \( F_n \) as determined in accordance with Chapter E.

\[ F_{cr} = \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{w} \right)^2 \]  
\[ (Eq. 1.1-4) \]

where

\[ k = \text{Plate buckling coefficient} \]
\[ = 4 \text{ for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.} \]
\[ E = \text{Modulus of elasticity of steel} \]
(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated as follows:

\[
b_d = \rho w
\]

where

\( w = \text{Flat width} \)

\( \rho = \text{Local reduction factor determined by either of the following two procedures:} \)

(1) Procedure I:

A conservative estimate of the effective width is obtained from Section 1.1(a) by substituting \( f_d \) 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 is obtained by calculating \( \rho \) as follows:

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

\( \rho \leq 1 \) for all cases.

where

\( \lambda = \text{Slenderness factor as defined by Eq. 1.1-3, except that } f_d \text{ is substituted for } f \)

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

\( F_y = \text{Yield stress} \)

1.1.1 Uniformly Compressed Stiffened Elements With Circular or Noncircular Holes

(a) Strength Determination

For circular holes:

The effective width, \( b \), shall be calculated by either Eq. 1.1.1-1 or Eq. 1.1.1-2 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 $\geq 3d_h$

$$b = w - d_h$$

when $\lambda \leq 0.673$  \hspace{1cm} \text{(Eq. 1.1.1-1)}

$$b = \frac{w}{\lambda} \left[ 1 - \frac{0.22}{\lambda} - \frac{(0.8d_h)}{w} + \frac{(0.085d_h)}{w\lambda} \right]$$

when $\lambda > 0.673$  \hspace{1cm} \text{(Eq. 1.1.1-2)}

In all cases, $b \leq w - d_h$

where

$w$ = Flat width

$t$ = Thickness of element

$d_h$ = Diameter of holes

$\lambda$ = Slenderness factor as defined in Section 1.1 with $k = 4.0$

For noncircular holes:

A uniformly compressed stiffened element with noncircular holes shall be assumed to consist of two unstiffened strips of flat width, $c$, adjacent to the holes (see Figure 1.1.1-1). The effective width, $b$, of each unstiffened strip adjacent to the hole shall be determined in accordance with Section 1.1(a), except that the plate buckling coefficient, $k$, shall be taken as 0.43 and $w$ as $c$. These provisions shall be applicable within the following limits:

1. Center-to-center hole spacing, $s \geq 24$ in. (610 mm),
2. Clear distance from the hole at ends, $s_{end} \geq 10$ in. (254 mm),
3. Depth of hole, $d_h \leq 2.5$ in. (63.5 mm),
4. Length of hole, $L_h \leq 4.5$ in. (114 mm), and
5. Ratio of the depth of hole, $d_h$, to the out-to-out width, $w$, $d_h/w_o \leq 0.5$.

Alternatively, the effective width, $b_d$, is permitted to be determined by stub-column tests in accordance with the test procedure, AISI S902.

\(b\) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be equal to $b$ calculated in accordance with Procedure I of Section 1.1(b), except that $f_d$ is substituted for $f$, where $f_d$ is the computed compressive stress in the element being considered.

\[\text{Figure 1.1.1-1 Uniformly Compressed Stiffened Elements With Noncircular Holes}\]
1.1.2 Webs and Other Stiffened Elements Under Stress Gradient

The following notation shall apply in this section:

- \( b_1 \) = Effective width, dimension defined in Figure 1.1.2-1
- \( b_2 \) = Effective width, dimension defined in Figure 1.1.2-1
- \( b_e \) = Effective width, \( b \), determined in accordance with Section 1.1, 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 1.1.2-2
- \( f_1, f_2 \) = Stresses shown in Figure 1.1.2-1 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 1.1.2-2
- \( k \) = Plate buckling coefficient
- \( \psi \) = \( |f_2/f_1| \) (absolute value) \quad (Eq. 1.1.2-1)

(a) Strength Determination

(1) For webs under stress gradient (\( f_1 \) in compression and \( f_2 \) in tension as shown in Figure 1.1.2-1(a)), the effective widths and plate buckling coefficient shall be calculated as follows:

\[
k = 4 + 2(1 + \psi)^3 + 2(1 + \psi) \quad (Eq. 1.1.2-2)
\]

For \( h_o/b_o \leq 4 \)

- \( b_1 = b_e/(3 + \psi) \) \quad (Eq. 1.1.2-3)
- \( b_2 = b_e/2 \) when \( \psi > 0.236 \) \quad (Eq. 1.1.2-4)
- \( b_2 = b_e - b_1 \) when \( \psi \leq 0.236 \) \quad (Eq. 1.1.2-5)

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

For \( h_o/b_o > 4 \)

- \( b_1 = b_e/(3 + \psi) \) \quad (Eq. 1.1.2-6)
- \( b_2 = b_e/(1 + \psi) - b_1 \) \quad (Eq. 1.1.2-7)

(2) For other stiffened elements under stress gradient (\( f_1 \) and \( f_2 \) in compression as shown in Figure 1.1.2-1(b)):

\[
k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad (Eq. 1.1.2-8)
\]

- \( b_1 = b_e/(3 - \psi) \) \quad (Eq. 1.1.2-9)
- \( b_2 = b_e - b_1 \) \quad (Eq. 1.1.2-10)

(b) Serviceability Determination

The effective widths used in determining serviceability shall be calculated in accordance with Section 1.1.2(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 1.1.2-1 Web and Other Stiffened Elements Under Stress Gradient

(a) Webs Under Stress Gradient

(b) Other Stiffened Elements Under Stress Gradient

Figure 1.1.2-2 Out-to-Out Dimensions of Webs and Stiffened Elements Under Stress Gradient
**1.1.3 C-Section Webs With Holes Under Stress Gradient**

The provisions of Section 1.1.3 shall apply within the following limits:

1. \( \frac{d_h}{h} \leq 0.7 \),
2. \( \frac{h}{t} \leq 200 \),
3. Holes centered at mid-depth of web,
4. Clear distance between holes \( \geq 18 \) in. (457 mm),
5. Noncircular holes, corner radii \( \geq 2t \),
6. Noncircular holes, \( d_h \leq 2.5 \) in. (63.5 mm) and \( L_h \leq 4.5 \) in. (114 mm),
7. Circular holes, diameter \( \leq 6 \) in. (152 mm), and
8. \( d_h > 9/16 \) in. (14.3 mm).

where
\[ d_h = \text{Depth of web hole} \]
\[ h = \text{Depth of flat portion of web measured along plane of web} \]
\[ t = \text{Thickness of web} \]
\[ L_h = \text{Length of web hole} \]

(a) Strength Determination

When \( \frac{d_h}{h} < 0.38 \), the effective widths, \( b_1 \) and \( b_2 \), as illustrated in Figure 1.1.2-1, shall be determined in accordance with Section 1.1.2(a) by assuming no hole exists in the web.

When \( \frac{d_h}{h} \geq 0.38 \), the effective width shall be determined in accordance with Section 1.2.1(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 1.1.2-1.

(b) Serviceability Determination

The effective widths shall be determined in accordance with Section 1.1.2(b) by assuming no hole exists in the web.

**1.1.4 Uniformly Compressed Elements Restrained by Intermittent Connections**

The provisions of this section shall apply to compressed elements of flexural members only. The provisions shall be limited to multiple flute built-up members having edge-stiffened cover plates. When the spacing of fasteners, \( s \), of a uniformly compressed element restrained by intermittent connections is not greater than the limits specified in Section II.3, the effective width shall be calculated in accordance with Section 1.1. When the spacing of fasteners is greater than the limits specified in Section II.3, the effective width shall be determined in accordance with (a) and (b) below.

(a) Strength Determination

The effective width of the uniformly compressed element restrained by intermittent connections shall be determined as follows:

1. When \( f < F_c \), the effective width of the compression element between connection lines shall be calculated in accordance with Section 1.1(a).
2. When \( f \geq F_c \), the effective width of the compression element between connection lines shall be calculated in accordance with Section 1.1(a), except that the reduction factor, \( \rho \), shall be the lesser of the value determined in accordance with Section 1.1 and the value
determined by Eq. 1.1.4-1:
\[ \rho = \rho_t \rho_m \quad \text{(Eq. 1.1.4-1)} \]

where
\[ \rho_t = 1.0 \quad \text{for } \lambda_t \leq 0.673 \]
\[ \rho_t = (1.0 - 0.22 / \lambda_t) / \lambda_t \quad \text{for } \lambda_t > 0.673 \quad \text{(Eq. 1.1.4-2)} \]

where
\[ \lambda_t = \sqrt{\frac{F_c}{F_{crt}}} \quad \text{(Eq. 1.1.4-3)} \]

where
\[ F_c = \text{Critical column buckling stress of compression element} \]
\[ = 3.29 E/(s/t)^2 \quad \text{(Eq. 1.1.4-4)} \]

where
\[ s = \text{Center-to-center spacing of connectors in line of compression stress} \]
\[ E = \text{Modulus of elasticity of steel} \]
\[ t = \text{Thickness of cover plate in compression} \]
\[ F_{crt} = \text{Critical buckling stress defined in Eq. 1.1-4 where } w \text{ is the transverse spacing of connectors} \]
\[ \rho_m = 8 \left( \frac{F_y}{f} \right) \sqrt{\frac{f F_{crt}}{d}} \leq 1.0 \quad \text{(Eq. 1.1.4-5)} \]

where
\[ F_y = \text{Design yield stress of the compression element restrained by intermittent connections} \]
\[ d = \text{Overall depth of the built-up member} \]
\[ f = \text{Stress in compression element restrained by intermittent connections when the controlling extreme fiber stress is } F_y \]

The provisions of this section shall apply to shapes that meet the following limits:

(1) 1.5 in. (38.1 mm) \( \leq d \leq 7.5 \text{ in. (191 mm)}, \)
(2) 0.035 in. (0.889 mm) \( \leq t \leq 0.060 \text{ in. (1.52 mm)}, \)
(3) 2.0 in. (50.8 mm) \( \leq s \leq 8.0 \text{ in. (203 mm)}, \)
(4) 33 ksi (228 MPa or 2320 kg/cm²) \( \leq F_y \leq 60 \text{ ksi (414 MPa or 4220 kg/cm²)}, \)
(5) 100 \( \leq w/t \leq 350. \)

The effective width of the edge stiffener and the flat portion, e, shall be determined in accordance with Section 1.3(a) with modifications as follows:

For \( f < F_c \)
\[ w = e \quad \text{(Eq. 1.1.4-6)} \]

For \( f \geq F_c \)

For the flat portion, e, the effective width, b, in Eqs. 1.3-4 and 1.3-5 shall be calculated in accordance with Section 1.1(a) with

(i) \( w \) taken as \( e \),
(ii) if \( D/e \leq 0.8 \)
k is determined in accordance with Table 1.3-1
if \( D/e > 0.8 \)
\[ k = 1.25, \text{ and} \]

(iii) \( \rho \) calculated using Eq. 1.1.4-1 in lieu of Eq. 1.1-2.
where
\[ w = \text{Flat width of element measured between longitudinal connection lines and exclusive of radii at stiffeners} \]
\[ e = \text{Flat width between the first line of connector and the edge stiffener. See Figure 1.1.4-1} \]
\[ D = \text{Overall length of stiffener as defined in Section 1.3} \]

For the edge stiffener, \( d_s \) and \( I_a \) shall be determined using \( w' \) and \( f' \) in lieu of \( w \) and \( f \), respectively.

\[ w' = 2e + \text{minimum of (0.75s and } w_1) \]  \hspace{1cm} (Eq. 1.1.4-7)
\[ f' = \text{Maximum of (} \rho_m f \text{ and } F_c) \]  \hspace{1cm} (Eq. 1.1.4-8)
where
\[ f' = \text{Stress used in Section 1.3(a) for determining effective width of edge stiffener} \]
\[ F_c = \text{Buckling stress of cover plate determined in accordance with Eq. 1.1.4-4} \]
\[ w' = \text{Equivalent flat width for determining the effective width of edge stiffener} \]
\[ w_1 = \text{Transverse spacing between the first and the second line of connectors in the compression element. See Figure 1.1.4-1.} \]

(b) Serviceability Determination

The provisions of this section shall not apply to single flute members having compression plates with edge stiffeners.

The effective width of the uniformly compressed element restrained by intermittent connections used for computing deflection shall be determined in accordance with Section 1.1.4(a) except that:

(1) \( f_d \) shall be substituted for \( f \), where \( f_d \) is the computed compression stress in the element being considered at service load, and

(2) The maximum extreme fiber stress in the built-up member shall be substituted for \( F_y \).
1.2 Effective Width of Unstiffened Elements

1.2.1 Uniformly Compressed Unstiffened Elements

(a) Strength Determination

The effective width, \( b \), shall be determined in accordance with Section 1.1(a), except that the plate buckling coefficient, \( k \), shall be taken as 0.43 and \( w \) as defined in Figure 1.2.1-1.

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated in accordance with Procedure I of Section 1.1(b), except that \( f_d \) is substituted for \( f \) and \( k = 0.43 \).

1.2.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

The following notation shall apply in this section:

- \( b \) = Effective width measured from the supported edge, determined in accordance with Section 1.1(a), with \( f \) equal to the maximum compressive stress on the effective element and with \( k \) and \( \rho \) being determined in accordance with this section
- \( b_o \) = Overall width of unstiffened element of unstiffened C-section member as defined in Fig. 1.2.2-3
- \( f_1, f_2 \) = Stresses, shown in Figures 1.2.2-1, 1.2.2-2, and 1.2.2-3. Where \( f_1 \) and \( f_2 \) are both compression, \( f_1 \geq f_2 \).
- \( h_o \) = Overall depth of unstiffened C-section member. See Figure 1.2.2-3
- \( k \) = Plate buckling coefficient defined in this section or, otherwise, as defined in Section 1.1(a)
- \( t \) = Thickness of element
- \( w \) = Flat width of unstiffened element, where \( w/t \leq 60 \)
- \( \psi = |f_2/f_1| \) (absolute value) \( \quad \) (Eq. 1.2.2-1)
- \( \lambda \) = Slenderness factor defined in Section 1.1(a) with \( f \) equal to the maximum compressive stress on the effective element
- \( \rho \) = Reduction factor defined in this section or, otherwise, as defined in Section 1.1(a)

(a) Strength Determination

The effective width, \( b \), of an unstiffened element under stress gradient shall be determined in accordance with Section 1.1(a) with stress, \( f \), equal to the maximum compressive stress on the effective element and the plate buckling coefficient, \( k \), determined in accordance with this
section, unless otherwise noted. For the cases where \( f_1 \) is in compression and \( f_2 \) is in tension, \( \rho \) in Section 1.1(a) shall be determined in accordance with this section.

(1) When both \( f_1 \) and \( f_2 \) are in compression (Figure 1.2.2-1), the plate buckling coefficient shall be calculated in accordance with either Eq. 1.2.2-2 or Eq. 1.2.2-3 as follows:

If the stress decreases toward the unsupported edge (Figure 1.2.2-1(a)):

\[
k = \frac{0.578}{\psi + 0.34} \quad (Eq. \ 1.2.2-2)
\]

If the stress increases toward the unsupported edge (Figure 1.2.2-1(b)):

\[
k = 0.57 - 0.21\psi + 0.07\psi^2 \quad (Eq. \ 1.2.2-3)
\]

(2) When \( f_1 \) is in compression and \( f_2 \) in tension (Fig. 1.2.2-2), the reduction factor and plate buckling coefficient shall be calculated as follows:

(i) If the unsupported edge is in compression (Figure 1.2.2-2(a)):

\[
\rho = 1 \quad \text{when } \lambda \leq 0.673(1 + \psi)
\]

\[
\rho = (1 + \psi) \left(1 - \frac{0.22(1 + \psi)}{\lambda} \right) \quad \text{when } \lambda > 0.673(1 + \psi) \quad (Eq. \ 1.2.2-4)
\]

\[
k = 0.57 + 0.21\psi + 0.07\psi^2 \quad (Eq. \ 1.2.2-5)
\]
(ii) If the supported edge is in compression (Fig. 1.2.2-2(b)):

For $\psi < 1$

$\rho = 1$ when $\lambda \leq 0.673$

$\rho = (1 - \psi) \left(1 - \frac{0.22}{\lambda}\right) + \psi$ when $\lambda > 0.673$ (Eq. 1.2.2-6)

$k = 1.70 + 5\psi + 17.1\psi^2$ (Eq. 1.2.2-7)

For $\psi \geq 1$,

$\rho = 1$

The effective width, $b$, of the unstiffened elements of an unstiffened C-section member is permitted to be determined using the following alternative methods, as applicable:

Alternative 1 for unstiffened C-sections: When the unsupported edge is in compression and the supported edge is in tension (Figure 1.2.2-3(a)):

$b = w$ when $\lambda \leq 0.856$ (Eq. 1.2.2-8)

$b = \rho w$ when $\lambda > 0.856$ (Eq. 1.2.2-9)

where

$\rho = 0.925/\sqrt{\lambda}$ (Eq. 1.2.2-10)

$k = 0.145(b_o/h_o) + 1.256$ (Eq. 1.2.2-11)

$0.1 \leq b_o/h_o \leq 1.0$

Alternative 2 for unstiffened C-sections: When the supported edge is in compression and the unsupported edge is in tension (Figure 1.2.2-3(b)), the effective width is determined in accordance with Section 1.1.2.

Where stress, $f_1$, occurs at the unsupported edge as in Figures 1.2.2-1(b), 1.2.2-2(a), and 1.2.2-3(a), the design stress, $f$, shall be taken at the extreme fiber of the effective section, and $f_1$ is the calculated stress, based on the effective section, at the edge of the gross section. If the only elements not fully effective are unstiffened elements with stress gradient, as in Figure 1.2.2-3(a), the stresses $f_1$ and $f_2$ are permitted to be based on the gross section, $f$ taken equal to $f_1$, and iteration is not required.

In calculating the effective section modulus, $S_e$, in Section F3.1, the extreme compression fiber in Figures 1.2.2-1(b), 1.2.2-2(a), and 1.2.2-3(a) shall be taken as the edge of the effective section closer to the unsupported edge, and the extreme tension fiber in Figures 1.2.2-2(b) and 1.2.2-3(b) shall be taken as the edge of the effective section closer to the unsupported edge.
(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated in accordance with Section 1.2.2(a), except that \( f_{d1} \) and \( f_{d2} \) are substituted for \( f_1 \) and \( f_2 \), respectively, where \( f_{d1} \) and \( f_{d2} \) are the computed stresses \( f_1 \) and \( f_2 \) as shown in Figures 1.2.2-1, 1.2.2-2, and 1.2.2-3, respectively, at the load for which serviceability is determined.

1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener

The effective widths of uniformly compressed elements with a simple lip edge stiffener shall be calculated in accordance with (a) for strength determination and (b) for serviceability determination.

(a) Strength Determination

For \( w/t \leq 0.328\)S:

\[
I_a = 0 \quad \text{(no edge stiffener needed)}
\]

\[
b = w \quad \text{(Eq. 1.3-1)}
\]

\[
b_1 = b_2 = w/2 \quad \text{(see Figure 1.3-1)} \quad \text{(Eq. 1.3-2)}
\]

\[
d_s = d'_s \quad \text{(Eq. 1.3-3)}
\]

For \( w/t > 0.328\)S:

\[
b_1 = (b/2) (R_I) \quad \text{(see Figure 1.3-1)} \quad \text{(Eq. 1.3-4)}
\]

\[
b_2 = b - b_1 \quad \text{(see Figure 1.3-1)} \quad \text{(Eq. 1.3-5)}
\]

\[
d_s = d'_s (R_I) \quad \text{(Eq. 1.3-6)}
\]

where

\[
S = 1.28\sqrt{E/f} \quad \text{(Eq. 1.3-7)}
\]

where

- \( E \) = Modulus of elasticity of steel
- \( f \) = Stress in compression flange
- \( w \) = Flat dimension of flange (see Figure 1.3-1)
- \( t \) = Thickness of section
- \( I_a \) = Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element

\[
= 399t^4 \left( \frac{w/t}{S} - 0.328 \right)^3 \leq t^4 \left( 115 \frac{w/t}{S} + 5 \right) \quad \text{(Eq. 1.3-8)}
\]

\( b \) = Effective design width

\( b_1, b_2 \) = Portions of effective design width (see Figure 1.3-1)

\( d_s \) = Reduced effective width of stiffener (see Figure 1.3-1), which is used in computing overall effective section properties

\( d'_s \) = Effective width of stiffener calculated in accordance with Section 1.2.1 or 1.2.2 (see Figure 1.3-1)

\( (R_I) = I_s/I_a \leq 1 \quad \text{(Eq. 1.3-9)} \)

where

\( I_s \) = Unreduced moment of inertia of stiffener about its own centroidal axis parallel to element to be stiffened. For edge stiffeners, the round corner between stiffener and element to be stiffened is not considered a part of the stiffener.
= (d^3t \sin^2 \theta)/12 \quad (Eq. 1.3-10)

See Figure 1.3-1 for definitions of other dimensional variables.

The effective width, b, in Eqs. 1.3-4 and 1.3-5 shall be calculated in accordance with Section 1.1.1 with the plate buckling coefficient, k, as given in Table 1.3-1 below:

<table>
<thead>
<tr>
<th>Simple Lip Edge Stiffener (140° ≥ θ ≥ 40°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D/w ≤ 0.25</td>
</tr>
<tr>
<td>3.57(R_1)^n + 0.43 ≤ 4</td>
</tr>
</tbody>
</table>

where

n = \left( \frac{0.582 - \frac{w}{t}}{4S} \right) \geq \frac{1}{3} \quad (Eq. 1.3-11)

(b) Serviceability Determination

The effective width, b_d, used in determining serviceability shall be calculated as in Section 1.3(a), except that f_d is substituted for f, where f_d is computed compressive stress in the effective section at the load for which serviceability is determined.

Figure 1.3-1 Element With Simple Lip Edge Stiffener
1.4 Effective Width of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

1.4.1 Effective Width of Uniformly Compressed Stiffened Elements With Single or Multiple Intermediate Stiffeners

The following notations shall apply 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 1.4.1-2
- \( b_o \) = Total flat width of stiffened element; see Figure 1.4.1-1
- \( b_p \) = Largest sub-element flat width; see Figure 1.4.1-1
- \( c_i \) = Horizontal distance from edge of element to centerline(s) of stiffener(s); see Figure 1.4.1-1
- \( E \) = Modulus of elasticity of steel
- \( F_{cr} \) = 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 that connect the stiffener to the flat can be included.
- \( k \) = Plate buckling coefficient of element
- \( k_d \) = Plate buckling coefficient for distortional buckling
- \( k_{loc} \) = Plate buckling coefficient for local sub-element buckling
- \( L_{br} \) = Unsupported length between brace points or other restraints that 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
- \( \mu \) = Poisson’s ratio of steel
- \( \rho \) = Reduction factor

The effective width shall be calculated in accordance with Eq. 1.4.1-1 as follows:

\[
be = \rho \left( \frac{A_g}{t} \right)
\]  

\( (Eq. \ 1.4.1-1) \)

where

- \( \rho = 1 \) when \( \lambda \leq 0.673 \)
- \( \rho = \left( 1 - 0.22 / \lambda \right) / \lambda \) when \( \lambda > 0.673 \)  

\( (Eq. \ 1.4.1-2) \)
where
\[ \lambda = \frac{f}{\sqrt{F_{cr}}} \]  
(Eq. 1.4.1-3)

where
\[ F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left( \frac{t}{b_o} \right)^2 \]  
(Eq. 1.4.1-4)

The plate buckling coefficient, \( k \), shall be determined from the minimum of \( Rk_d \) and \( k_{loc} \), as determined in accordance with Section 1.4.1.1 or 1.4.1.2, as applicable.

\[ k = \text{the minimum of } Rk_d \text{ and } k_{loc} \]  
(Eq. 1.4.1-5)

\[ R = 2 \quad \text{when } b_o/h < 1 \]
\[ R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \quad \text{when } b_o/h \geq 1 \]  
(Eq. 1.4.1-6)

**1.4.1.1 Specific Case: Single or \( n \) Identical Stiffeners, Equally Spaced**

For uniformly compressed elements with single or multiple identical and equally spaced stiffeners, the plate buckling coefficients and effective widths shall be calculated as follows:

(a) Strength Determination

\[ k_{loc} = 4 \left( \frac{b_o}{b_p} \right)^2 \]  
(Eq. 1.4.1.1-1)
Appendix 1, Effective Width of Elements

\[ k_d = \frac{(1 + \beta^2)^2 + \gamma(1 + n)}{\beta^2 (1 + \delta(n + 1))} \]  
(Eq. 1.4.1.1-2)

where

\[ \beta = (1 + \gamma(n + 1))^{1/4} \]  
(Eq. 1.4.1.1-3)

where

\[ \gamma = \frac{10.92I_{sp}}{b_o t^3} \]  
(Eq. 1.4.1.1-4)

\[ \delta = \frac{A_s}{b_o t} \]  
(Eq. 1.4.1.1-5)

If \( L_{br} < b_{bo} \), \( L_{br}/b_o \) is permitted to be substituted for \( \beta \) to account for increased capacity due to bracing.

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated as in Section 1.4.1.1(a), except that \( f_d \) is 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.

1.4.1.2 General Case: Arbitrary Stiffener Size, Location, and Number

For uniformly compressed stiffened elements with stiffeners of arbitrary size, location, and number, the plate buckling coefficients and effective widths shall be calculated as follows:

(a) Strength Determination

\[ k_{loc} = 4\left(\frac{b_o}{b_p}\right)^2 \]  
(Eq. 1.4.1.2-1)

\[ k_d = \frac{(1 + \beta^2)^2 + 2 \sum_{i=1}^{n} \gamma_i \omega_i}{\beta^2 \left(1 + 2 \sum_{i=1}^{n} \delta_i \omega_i\right)} \]  
(Eq. 1.4.1.2-2)

where

\[ \beta = \left(2 \sum_{i=1}^{n} \gamma_i \omega_i + 1\right)^{1/4} \]  
(Eq. 1.4.1.2-3)

where

\[ \gamma_i = \frac{10.92(I_{sp})_i}{b_o t^3} \]  
(Eq. 1.4.1.2-4)

\[ \omega_i = \sin^2\left(\frac{\pi c_i}{b_o}\right) \]  
(Eq. 1.4.1.2-5)

\[ \delta_i = \frac{(A_s)_i}{b_o t} \]  
(Eq. 1.4.1.2-6)

If \( L_{br} < \beta b_{bo} \), \( L_{br}/b_o \) is permitted to be substituted for \( \beta \) to account for increased
capacity due to bracing.

(b) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be calculated as in Section 1.4.1.2(a), except that $f_d$ is 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.

1.4.2 Edge-Stiffened Elements With Intermediate Stiffener(s)

(a) Strength Determination

For edge-stiffened elements with intermediate stiffener(s), the effective width, $b_e$, shall be determined as follows:

If $b_o/t \leq 0.328S$, the element is fully effective and no local buckling reduction is required.

If $b_o/t > 0.328S$, the plate buckling coefficient, $k$, is determined in accordance with Section 1.3, but with $b_o$ replacing $w$ in all expressions:

If $k$ calculated from Section 1.3 is less than 4.0 ($k < 4$), the intermediate stiffener(s) is ignored and the provisions of Section 1.3 are followed for calculation of the effective width.

If $k$ calculated from Section 1.3 is equal to 4.0 ($k = 4$), the effective width of the edge-stiffened element is calculated from the provisions of Section 1.4.1, with the following exception:

$R$ calculated in accordance with Section 1.4.1 is less than or equal to 1.

where

$b_o =$ Total flat width of edge-stiffened element

See Sections 1.3 and 1.4.1 for definitions of other variables.

(b) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be calculated as in Section 1.4.2(a), except that $f_d$ is 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.
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APPENDIX 2, ELASTIC BUCKLING ANALYSIS OF MEMBERS

This appendix addresses the elastic buckling stress and stress resultant (force or moment) that are used for the determination of member strength in the Specification.

Elastic buckling occurs at a load in which the equilibrium of the member (approximated with linear elastic material) is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members may have at least three relevant elastic buckling modes: local, distortional, and global. The global buckling mode includes flexural, torsional, or flexural-torsional buckling for columns, and lateral-torsional buckling for beams. This appendix provides a means to determine all three relevant buckling modes for use in the design process.

This appendix is organized as follows:
2.1 General Provisions
2.2 Numerical Solutions
2.3 Analytical Solutions

2.1 General Provisions

The elastic buckling stresses or elastic buckling stress resultants (forces or moments) that are used in the Specification Chapters D through H are permitted to be calculated numerically in accordance with Section 2.2, analytically in accordance with Section 2.3, or in any combination.

In compression, global, local, and distortional buckling conversion between force and stress shall use the gross area, except where a reduced (e.g., net or effective) area is explicitly required by the Specification. Therefore:

\[ P_{cr} = A_g F_{cr} \]  \hspace{1cm} (Eq. 2.1-1)

where

\[ P_{cr} = P_{cre}^{\text{global}} - \text{flexural, torsional, or flexural-torsional}, P_{cr^L}^{\text{local}}, \text{or } P_{cr^D}^{\text{distortional}} \]

\[ F_{cr} = F_{cre}^{\text{global}} - \text{flexural, torsional, or flexural-torsional}, F_{cr^L}^{\text{local}}, \text{or } F_{cr^D}^{\text{distortional}} \]

\[ A_g = \text{Gross cross-sectional area} \]

In flexure, global, local, and distortional buckling conversion between moment and stress at the extreme compression fiber shall use the gross section modulus, except where a reduced (e.g., net or effective) section modulus is explicitly required by the Specification. Therefore:

\[ M_{cr} = S_f F_{cr} \]  \hspace{1cm} (Eq. 2.1-2)

where

\[ M_{cr} = M_{cre}^{\text{global}} - \text{lateral-torsional}, M_{cr^L}^{\text{local}}, \text{or } M_{cr^D}^{\text{distortional}} \]

\[ F_{cr} = F_{cre}^{\text{global}} - \text{lateral-torsional}, F_{cr^L}^{\text{local}}, \text{or } F_{cr^D}^{\text{distortional}} \]

\[ S_f = \text{Gross elastic section modulus referenced to the extreme compression fiber} \]

In shear, shear buckling conversion between force and stress shall use the web gross area, except where a reduced area is explicitly required by the Specification. Therefore:

\[ V_{cr} = F_{cr} A_w \]  \hspace{1cm} (Eq. 2.1-3)
where

\[ V_{cr} = \text{Shear elastic buckling force} \]
\[ F_{cr} = \text{Shear elastic buckling stress} \]
\[ A_w = \text{Web gross area} \]

**User Note:**
The *Specification* uses both stress and stress resultants (force, moment, etc.) in elastic buckling analysis. In particular, Effective Width Method calculations (e.g., Section E3.1) and traditional column and beam buckling formulas use stress \( F_{cr} \), while the Direct Strength Method (e.g., Section E3.2) uses stress resultants \( P_{cr} \). Numerical solutions are also performed as stress or stress resultants; either is adequate, but conversion of results between stress and stress resultant may be needed in order to use Specification equations.

### 2.2 Numerical Solutions

Any numerical elastic buckling solution that includes the relevant mechanics for the buckling mode under consideration is permitted to be utilized.

**User Note:**
A number of numerical methods, and related software programs, are known to be accurate for local, distortional, and global buckling, including the finite strip method utilizing plate bending strips for discretizing the cross-section, the finite element method utilizing plate or shell finite elements for discretizing the cross-section, and generalized beam theory with appropriate cross-section modes added for local and distortional buckling. See the Commentary for greater elaboration on the application of these numerical methods, including methods for members with holes, members with bracing, etc.

For local buckling, the impact of plate bending and cross-sectional distortion on the elastic buckling mode shall be considered.

For distortional buckling, the impact of plate bending and cross-sectional distortion, including distortion resulting from longitudinal strains, shall be considered.

For shear buckling (a specialized case of local or distortional buckling or both), the interaction of shear and longitudinal stresses on plate bending and cross-sectional distortion shall be considered.

For global buckling, the interaction of bending and torsion (i.e., flexural-torsional buckling or lateral-torsional buckling), particularly for cross-sections that are not doubly symmetric, shall be considered.

**User Note:**
Most conventional beam finite elements used in structural analysis software do not include the interaction of bending and torsion and should be used with care for global elastic buckling determination.

### 2.3 Analytical Solutions

The analytical solutions described in this section are permitted to be used for the given boundary conditions and cross-section geometry. For other boundary conditions or cross-section geometry, numerical analysis as detailed in Section 2.2 shall be used.
2.3.1 Members Subject to Compression

The buckling loads of cold-formed steel structural members subject to a concentric load are permitted to be determined analytically in accordance with this section.

2.3.1.1 Global Buckling ($F_{cre}$, $P_{cre}$)

The global buckling force, $P_{cre}$, shall be determined as follows:

$$P_{cre} = A_g F_{cre}$$  \hspace{1cm} (Eq. 2.3.1.1-1)

where

$P_{cre} = $ Global (flexural, flexural-torsional, or torsional) buckling force

$A_g = $ Gross cross-sectional area

$F_{cre} = $ Global buckling stress of a member subjected to a concentric load determined in accordance with Sections E2.1 through E2.4, as applicable; or for any cross-section, including non-symmetric sections, $F_{cre}$ is determined as the smallest root of the following cubic equation:

$$(F_{cre} - \sigma_{ex})(F_{cre} - \sigma_{ey})(F_{cre} - \sigma_t) - F_{cre}^2(F_{cre} - \sigma_{ex}) - \frac{x_o}{r_o} \left( \frac{F_{cre} - \sigma_{ex}}{\sigma_{ex}} \right)^2 - \frac{y_o}{r_o} \left( \frac{F_{cre} - \sigma_{ex}}{\sigma_{ex}} \right)^2 = 0$$  \hspace{1cm} (Eq. 2.3.1.1-2)

where

$x$ and $y$ are the principal axes of the cross-section; and

$$\sigma_{ex} = \frac{\pi^2E}{(K_x L_x / r_x)^2}$$  \hspace{1cm} (Eq. 2.3.1.1-3)

$$\sigma_{ey} = \frac{\pi^2E}{(K_y L_y / r_y)^2}$$  \hspace{1cm} (Eq. 2.3.1.1-4)

$$\sigma_t = \frac{1}{A_g r_o^2} \left[ G J + \frac{\pi^2E C_w}{(K_t L_t)^2} \right]$$  \hspace{1cm} (Eq. 2.3.1.1-5)

where

$K_x = $ Effective length factor for bending about x-axis in accordance with Chapter C

$K_y = $ Effective length factor for bending about y-axis in accordance with Chapter C

$K_t = $ Effective length factor for twisting determined in accordance with Chapter C

$L_x = $ Unbraced length of member for bending about x-axis

$L_y = $ Unbraced length of member for bending about y-axis

$L_t = $ Unbraced length of member for torsion

$r_x = $ Radius of gyration of full unreduced cross-section about x-axis

$r_y = $ Radius of gyration of full unreduced cross-section about y-axis

$J = $ St. Venant torsion constant of cross-section

$G = $ Shear modulus of steel

$E = $ Modulus of elasticity of steel

$C_w = $ Torsional warping constant of cross-section

$x_o = $ Distance from centroid to shear center in principal x-axis direction
\[ y_0 = \text{Distance from centroid to shear center in principal y-axis direction} \]
\[ r_0 = \text{Polar radius of gyration about shear center} \]
\[ = \sqrt{\frac{x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g}}}{A_g} \quad \text{(Eq. 2.3.1.1-6)} \]

where
\[ I_x = \text{Gross moment of inertia about x-axis} \]
\[ I_y = \text{Gross moment of inertia about y-axis} \]

### 2.3.1.2 Local Buckling (\(F_{crL}, P_{crL}\))

The local buckling force, \(P_{crL}\), of a member shall be based on the lowest buckling stress among elements in the cross-section as follows:
\[ P_{crL} = A_g F_{crL} \quad \text{(Eq. 2.3.1.2-1)} \]

where
\[ A_g = \text{Gross cross-sectional area} \]
\[ F_{crL} = \text{Smallest local buckling stress of all elements in cross-section} \]
\[ = k \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{w} \right)^2 \quad \text{(Eq. 2.3.1.2-2)} \]

where
\[ k = \text{Plate buckling coefficient provided in Appendix 1 for different types of elements and supporting conditions} \]
\[ E = \text{Modulus of elasticity of steel} \]
\[ t = \text{Element thickness} \]
\[ \mu = \text{Poisson’s ratio of steel} \]
\[ w = \text{Element flat width} \]

**User Note:**
Determining the local buckling force by using the smallest of the element (flange, web, lip, etc.) local buckling stresses can be very conservative if one element is much more slender than the rest of the elements in the cross-section. Numerical solutions or more advanced analytical solutions are recommended in this case.

### 2.3.1.3 Distortional Buckling (\(F_{crD}, P_{crD}\))

The provisions of this section shall apply to any open cross-section with stiffened flanges of equal dimension where the stiffener is either a simple lip or a complex edge stiffener. The elastic distortional buckling load, \(P_{crD}\), shall be calculated as follows:
\[ P_{crD} = A_g F_{crD} \quad \text{(Eq. 2.3.1.3-1)} \]

where
\[ A_g = \text{Gross cross-sectional area} \]
\[ F_{crD} = \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{k_{\phi g} + k_{\phi wg}} \quad \text{(Eq. 2.3.1.3-2)} \]
where

\[ k_{\phi fc} = \text{Elastic rotational stiffness provided by the flange to the flange/web juncture} \]

\[ = \left( \frac{\pi}{L} \right)^4 \left[ EI_{xf} (x_{of} - h_{xf})^2 + EC_{wf} - E \frac{I_{xyf}^2}{I_{yf}} (x_{of} - h_{xf})^2 \right] + \left( \frac{\pi}{L} \right)^2 GJ_f \quad (Eq. 2.3.1.3-3) \]

\[ k_{\phi we} = \text{Elastic rotational stiffness provided by the web to flange/web juncture} \]

\[ = \frac{Et^3}{6h_o(1-\mu^2)} \quad (Eq. 2.3.1.3-4) \]

where

\[ h_o = \text{Out-to-out web depth (See Figure 1.1.2-2)} \]

\[ t = \text{Base steel thickness} \]

\[ k_f = \text{Rotational stiffness provided by restraining elements (brace, panel, sheathing) to flange/web juncture of member (zero if the flange is unrestrained). If rotational stiffness provided to the two flanges is dissimilar, the smaller rotational stiffness is used.} \]

\[ \tilde{k}_{\phi f} = \text{Geometric rotational stiffness demanded by flange from flange/web juncture} \]

\[ = \left( \frac{\pi}{L} \right)^2 \left[ A_f \left( x_{of} - h_{xf} \right)^2 \left( \frac{I_{xyf}^2}{I_{yf}} \right)^2 - 2y_{of} \left( x_{of} - h_{xf} \right) \left( \frac{I_{xyf}}{I_{yf}} \right) + h_{xf}^2 + y_{of}^2 \right] + I_{xf} + I_{yf} \quad (Eq. 2.3.1.3-5) \]

\[ \tilde{k}_{\phi wg} = \text{Geometric rotational stiffness demanded by web from flange/web juncture} \]

\[ = \left( \frac{\pi}{L} \right)^2 \frac{th_o^3}{60} \quad (Eq. 2.3.1.3-6) \]

where

\[ L = \text{Minimum of } L_{crd} \text{ and } L_m \]

where

\[ L_{crd} = \left[ \frac{6\pi^4 h_o (1-\mu^2)}{t^3} \left[ I_{xf} (x_{of} - h_{xf})^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}} (x_{of} - h_{xf})^2 \right] \right]^{1/4} \quad (Eq. 2.3.1.3-7) \]

\[ L_m = \text{Distance between discrete restraints that restrict distortional buckling} \]

(for continuously restrained members \( L_m = L_{crd} \))

Variables \( A_f, J_f, I_{xf}, I_{yf}, I_{xyf}, C_{wf}, x_{of}, y_{of}, \) and \( h_{xf} \) are defined in Table 2.3.1.3-1, and variables \( L_x, L_y, L_t, E, G, \mu, \) and \( A_g \) are defined in Sections 2.3.1.1 and 2.3.1.2.
2.3.2 Members With Holes Subject to Compression

2.3.2.1 Global Buckling \((F_{cre}, P_{cre})\) for Members With Holes

The global buckling force, \(P_{cre}\), shall be calculated as follows:

\[
P_{cre} = A_g F_{cre} \quad (Eq. 2.3.2.1-1)
\]
where
\( A_g \) = Gross cross-sectional area
\( P_{cre} \) = Global (flexural, flexural-torsional, or torsional) buckling force
\( F_{cre} \) = Smallest global buckling stress of member as determined in accordance with Sections 2.3.2.1.1 to 2.3.2.1.4, as applicable

### 2.3.2.1.1 Sections With Holes Not Subject to Torsional or Flexural-Torsional Buckling

For doubly-symmetric sections, closed cross-sections, and any other sections that can be shown not to be subjected to torsional or flexural-torsional buckling, the elastic flexural buckling stress, \( F_{cre} \), of a member with holes shall be calculated as follows:

\[
F_{cre} = \frac{\pi^2 EI_{avg}}{A_g(KL)^2}
\]

(Eq. 2.3.2.1.1-1)

where
\( K \) = Effective length factor determined in accordance with Chapter C
\( L \) = Unbraced length about the axis of buckling
\( A_g \) = Gross cross-sectional area
\( I_{avg} \) = Weighted average moment inertia about axis of buckling as defined in Table 2.3.2-1

**User Note:**
The gross cross-sectional area, \( A_g \), in Eq. 2.3.2.1.1-1 is for converting the uniform compressive stress at the ends of the column to a force and should not be confused with \( A_{avg} \). Formulas for members with holes not symmetric to the longitudinal mid-height are referenced in the Commentary.

### Table 2.3.2-1
Weighted Average Cross-Sectional Properties for Symmetric Hole Distribution About Mid-Length of Member

<table>
<thead>
<tr>
<th>Average Properties</th>
<th>Formulas</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-sectional area</td>
<td>( A_{avg} = \frac{A_g L_g + A_{net} L_{net}}{L} )</td>
</tr>
<tr>
<td>Moment of inertia about axis of buckling</td>
<td>( I_{avg} = \frac{I_g L_g + I_{net} L_{net}}{L} )</td>
</tr>
<tr>
<td>Saint-Venant Torsion constant</td>
<td>( J_{avg} = \frac{J_g L_g + J_{net} L_{net}}{L} )</td>
</tr>
<tr>
<td>Distance from centroid to shear center in principal x-axis direction</td>
<td>( x_{o,avg} = \frac{x_{o,g} L_g + x_{o,net} L_{net}}{L} )</td>
</tr>
<tr>
<td>Distance from centroid to shear center in principal y-axis direction</td>
<td>( y_{o,avg} = \frac{y_{o,g} L_g + y_{o,net} L_{net}}{L} )</td>
</tr>
<tr>
<td>Polar radius gyration about shear center</td>
<td>( r_{o,avg} = \sqrt{x_{o,avg}^2 + y_{o,avg}^2 + \frac{I_{x,avg} + I_{y,avg}}{A_{avg}}} )</td>
</tr>
</tbody>
</table>

1. Definition of variables:
\( A_g, A_{net} \) = Gross and net area, respectively
Lg = Segment length without holes
Lnet = Length of holes or net section regions
L = Unbraced length about the axis of buckling = Lg + Lnet
Ig, Inet = Moment of inertia of gross or net cross-section about axis of buckling, respectively
Jg, Jnet = Saint-Venant torsion constant of gross or net cross-section, respectively
xog, xo, net = Distance from gross or net cross-section centroid to shear center in principal x-axis direction, respectively
yog, yo, net = Distance from gross or net cross-section centroid to shear center in principal y-axis direction, respectively
ro, g, ro, net = Polar radius gyration about shear center of gross or net cross-section, respectively

2.3.2.1.2 Doubly- or Singly-Symmetric Sections (With Holes) Subject to Torsional or Flexural-Torsional Buckling

For singly-symmetric sections, including the influence of holes that are symmetric about the longitudinal mid-height, subject to flexural-torsional buckling stress, Fcre shall be taken as the smaller of Fcre calculated in accordance with Section 2.3.2.1.1 and Fcre calculated as follows:

\[ F_{cre} = \frac{1}{2\beta} \left[ (\sigma_{ex} + \sigma_t) - \sqrt{\left(\sigma_{ex} + \sigma_t\right)^2 - 4\beta \sigma_{ex} \sigma_t} \right] \]  
(Eq. 2.3.2.1.2-1)

where

\[ \sigma_{ex} = \frac{\pi^2 EI_{x,avg}}{A_g (K_x L_x)^2} \]  
(Eq. 2.3.2.1.2-2)

\[ \sigma_t = \frac{1}{A_g r_{o,avg}^2} \left[ GJ_{avg} + \frac{\pi^2 E C_w, net}{(K_t L_t)^2} \right] \]  
(Eq. 2.3.2.1.2-3)

where

\[ C_{w, net} = \text{Net warping constant assuming the cross-section thickness is zero at hole} \]

\[ \beta = 1 - \left( \frac{x_{o,avg}}{r_{o,avg}} \right)^2 \]  
(Eq. 2.3.2.1.2-4)

Variables A_g, Ix,avg, Javg, ro,avg and xo,avg are weighted average cross-sectional properties with holes as defined in Table 2.3.2-1, and E and G are defined in Section 2.3.1.1.

User Note:
The gross cross-sectional area, A_g, in Eqs. 2.3.2.1.2-2 and 2.3.2.1.2-3 is for converting the uniform compressive stress at the ends of the column to a force and should not be confused with the average of the area, Aavg. Formulas for members with holes not symmetric to the longitudinal mid-height are referenced in the Commentary.

2.3.2.1.3 Point Symmetric Sections With Holes

For point-symmetric sections with holes, Fcre shall be taken as the smaller of σt as
defined in Section 2.3.2.1.2 and $F_{cre}$ as calculated in Section 2.3.2.1.1 using the minor principal axis of the cross-section.

### 2.3.2.1.4 Non-Symmetric Sections With Holes

For any cross-section, including non-symmetric sections, it shall be permitted to determine the global buckling stress, $F_{cre}$, for a member with holes as the smallest positive root of the following cubic equation:

\[
(F_{cre} - \sigma_{ex})(F_{cre} - \sigma_{ey})(F_{cre} - \sigma_{t}) - F_{cre}^2\left(F_{cre} - \sigma_{ex}\right)\left(\frac{x_{o,avg}}{r_{o,avg}}\right)^2 - F_{cre}^2\left(F_{cre} - \sigma_{ex}\right)\left(\frac{y_{o,avg}}{r_{o,avg}}\right)^2 = 0
\]

(Eq. 2.3.2.1.4-1)

where

\[
A_g = \text{Gross cross-sectional area}
\]

\[
\sigma_{ex} = \frac{\pi^2EI_{x,avg}}{A_g(K_xL_x)^2}
\]

(Eq. 2.3.2.1.4-2)

\[
\sigma_{ey} = \frac{\pi^2EI_{y,avg}}{A_g(K_yL_y)^2}
\]

(Eq. 2.3.2.1.4-3)

\[
\sigma_{t} = \frac{1}{A_g}r_{o,avg}^2\left[GJ_{avg} + \frac{\pi^2EC_{w,net}}{(K_tL_t)^2}\right]
\]

(Eq. 2.3.2.1.4-4)

where

- $K_x, K_y = \text{Effective length factor}$ for bending about principal x- and y-axes, respectively, in accordance with Chapter C
- $K_t = \text{Effective length factor}$ for torsion determined in accordance with Chapter C
- $L_x, L_y = \text{Unbraced length}$ of member for bending about principal x- and y-axes, respectively
- $L_t = \text{Unbraced length}$ of member for twisting
- $J_{avg} = \text{Weighted average Saint-Venant torsion constant}$ as defined in Table 2.3.2-1
- $C_{w,net} = \text{Net warping constant}$ assuming the cross-section thickness is zero at location of hole(s)

Variables $I_{x,avg}$, $I_{y,avg}$, $x_{o,avg}$, $y_{o,avg}$ and $r_{o,avg}$ are defined in Table 2.3.2-1, and $E$ and $G$ are defined in Section 2.3.1.1.

### 2.3.2.2 Local Buckling ($F_{cr}$, $P_{cr}$) for Members With Holes

Local buckling of members with holes shall be computed in accordance with Section 2.3.1.2. When determining $F_{cr}$ for all elements, elements with holes shall be calculated as both unstiffened elements at the hole location and as a separate element where the hole is not located. For the unstiffened elements at the hole location, the buckling stress shall be modified to account for the net section by multiplying by the ratio $A_{net}/A_g$. 

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2.3.2.3 Distortional Buckling (F<sub>crd</sub>, P<sub>crd</sub>) for Members With Holes

For members meeting the geometric criteria of Section 2.3.1.3 and having hole(s) in the web, the distortional buckling force shall be determined in accordance with Section 2.3.1.3 provided that thickness, t, in Eqs. 2.3.1.3-4 and 2.3.1.3-6 be replaced by modified thickness, t<sub>r</sub>, as follows:

\[
t_r = t \left(1 - \frac{L_h}{L_{crd}}\right)^{1/3}
\]

(Eq. 2.3.2.3-1)

where

- \(t\) = Thickness of web
- \(L_h\) = Hole length
- \(L_{crd}\) = Distortional buckling half-wavelength of member with gross cross-section, determined numerically or using Eq. 2.3.1.3-7

For members meeting the geometric criteria of Section 2.3.1.3 and having patterned hole(s) along the web, the distortional buckling force shall be determined in accordance with Section 2.3.1.3 provided that thickness, t, in Eqs. 2.3.1.3-4, 2.3.1.3-6, and 2.3.1.3-7 are replaced by modified thickness, t<sub>r</sub>, as follows:

\[
t_r = t \left(\frac{A_{web,net}}{A_{web,gross}}\right)^{1/3}
\]

(Eq. 2.3.2.3-2)

where

- \(A_{web,net}\) = Web surface area along member length subtracting the hole areas
- \(A_{web,gross}\) = Web surface area along member length

2.3.3 Members Subject to Flexure

The buckling moments of cold-formed steel structural members subject to bending are permitted to be determined analytically in accordance with this section.

2.3.3.1 Global Buckling (F<sub>cre</sub>, M<sub>cre</sub>)

The global (lateral-torsional) buckling moment of a member subject to bending shall be determined in accordance with Section F2.1.1 through F2.1.5, as applicable.

2.3.3.2 Local Buckling (F<sub>cr</sub>, M<sub>cr</sub>)

The local buckling moment, M<sub>cr</sub>, of a member shall be based on the smallest buckling stress among elements in the cross-section, referenced to the extreme compression fiber, as follows:

\[
M_{cr} = S_f F_{cr}
\]

(Eq. 2.3.3.2.-1)

where

- \(S_f\) = Gross elastic cross-sectional modulus referenced to the extreme compression fiber
\[ F_{cr} = Local \ buckling \ stress \ at \ extreme \ compression \ fiber \]

\[ = k \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{w} \right)^2 \]  

\textit{(Eq. 2.3.3.2-2)}

where

\[ k = \text{Plate buckling coefficient, provided in Appendix 1 for different types of elements and supporting conditions} \]

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

\[ t = \text{Element thickness} \]

\[ \mu = \text{Poisson’s ratio of steel} \]

\[ w = \text{Element flat width} \]

\textbf{User Note:}
The first step in the application of this method is the determination of the local buckling stress of all the elements (flange, web, lip, etc.). The local buckling moment or stress is controlled by the element local buckling stress that results in the smallest stress level when linearly extrapolated to the extreme compression fiber.

\section*{2.3.3.3 Distortional Buckling \((F_{crd}, M_{crd})\)}

The provisions of this section are permitted to apply to any open cross-section with a single web and single edge-stiffened compression flange extending to one side of the web where the stiffener is either a simple lip or a complex edge stiffener. The elastic distortional buckling moment, \(M_{crd}\), shall be calculated as follows:

\[ M_{crd} = S_I F_{crd} \]  

\textit{(Eq. 2.3.3.3-1)}

where

\[ F_{crd} = \beta \frac{k_{\text{flg}} + k_{\text{we}} + k_{\phi}}{k_{\text{fg}} + k_{\text{wg}}} \]  

\textit{(Eq. 2.3.3.3-2)}

where

\[ \beta = \text{A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0} \]

\[ = 1.0 + 1.0 \left( \frac{L}{L_m} \right)^{0.7} \left( 1 + \frac{M_1}{M_2} \right)^{0.7} \leq 1.3 \]  

\textit{(Eq. 2.3.3.3-3)}

where

\[ L = \text{Minimum of } L_{crd} \text{ and } L_m \]

where

\[ L_{crd} = \left[ \frac{4\pi^4 h_o \left(1-\mu^2\right)}{t^3} \left( \frac{I_{xf}}{I_{yf}} \right) \left( x_{of} - h_{xf} \right)^2 + C_{wf} \frac{I_{xyf}^2}{I_{yf}} \left( x_{of} - h_{xf} \right)^2 \right] + \frac{\pi^4 h_o^4}{720} \]  

\textit{(Eq. 2.3.3.3-4)}

\[ L_m = \text{Distance between discrete restraints that restrict distortional buckling (for continuously restrained members } L_m = L_{crd} \]

\(M_1\) and \(M_2\) = Smaller and larger end moments, respectively, in the unbraced segment \((L_m)\) of the beam; \(M_1/M_2\) is positive when the moments cause reverse curvature and negative when bent in single curvature

\[ k_{\text{flg}} = \text{Elastic rotational stiffness provided by the flange to the flange/web juncture,} \]
Appendix 2, Elastic Buckling Analysis of Members

k_{\phi we} = \text{Elastic rotational stiffness provided by the web to the flange/web juncture}
\[ k_{\phi we} = \frac{E t^3}{12(1-\mu^2)} \left[ \frac{3}{h_o} + \left( \frac{\pi}{L} \right)^2 \frac{19h_o}{60} + \left( \frac{\pi}{L} \right)^4 \frac{h_o^3}{240} \right] \quad (Eq. 2.3.3.3-5) \]

k_\phi = \text{Rotational stiffness provided by a restraining element (brace, panel, sheathing) to the flange/web juncture of a member (zero if the compression flange is unrestrained)}

\tilde{k}_{\phi fg} = \text{Geometric rotational stiffness demanded by the flange from the flange/web juncture, given in Eq. 2.3.1.3-5}

\tilde{k}_{\phi wg} = \text{Geometric rotational stiffness demanded by the web from the flange/web juncture}
\[ \tilde{k}_{\phi wg} = \frac{h_o \pi^2}{13440} \left\{ \frac{45360(1 - \xi_{web}) + 62160}{L} \left( \frac{L}{h_o} \right)^2 + 448\pi^2 + \left( \frac{h_o}{L} \right)^2 \left[ 53 + 3(1 - \xi_{web}) \right] \right\} \pi^4 + 28\pi^2 \left( \frac{L}{h_o} \right)^2 + 420 \left( \frac{L}{h_o} \right)^4 \] \quad (Eq. 2.3.3.3-6)

where
\[ \xi_{web} = \frac{f_1 - f_2}{f_1}, \text{ stress gradient in the web, where } f_1 \text{ and } f_2 \text{ are the stresses at the opposite ends of the web, } f_1 > f_2, \text{ compression is positive, tension is negative, and the stresses are calculated on the basis of the gross section (e.g., pure symmetrical bending, } f_1 = -f_2, \xi_{web} = 2) \]

All other variables are defined in Section 2.3.1.3.

2.3.4 Members With Holes Subject to Flexure

2.3.4.1 Global Buckling (F_{cre}, M_{cre}) for Members With Holes

The global buckling moment, M_{cre}, shall be calculated as follows:
\[ M_{cre} = S_f F_{cre} \quad (Eq. 2.3.4.1-1) \]

where
\[ S_f = \text{Gross elastic section modulus referenced to the extreme compression fiber} \]
\[ M_{cre} = \text{Global (lateral-torsional) buckling moment} \]
\[ F_{cre} = \text{Smallest global buckling stress of member as determined in accordance with Sections 2.3.4.1.1 to 2.3.4.1.3, as applicable} \]

2.3.4.1.1 Singly- or Doubly- Symmetric Sections (With Holes) Bending About Symmetric Axis

The global (lateral-torsional) buckling stress, F_{cre}, for singly- or doubly-symmetric sections bending about the symmetric axis, with holes spaced symmetrically along the length, shall be calculated as follows:
\[ F_{\text{cre}} = C_b \frac{r_{0,\text{avg}} A_g}{S_f} \sqrt{\sigma_{\text{ey}} \sigma_t} \tag{Eq. 2.3.4.1.1-1} \]

where

- \( C_b \) = Moment gradient factor, as defined in F2.1.1
- \( S_f \) is defined in Section 2.3.4.1, and other variables are defined in Section 2.3.2.1.4.

**User Note:**
An alternate format for Eq. 2.3.4.1.1-1 that more directly shows the impact of the hole on the lateral-torsional buckling moment is:

\[ M_{\text{cre}} = C_b \frac{\pi}{(K_y L_y)} \sqrt{\frac{EI_{y,\text{avg}} GJ_{\text{avg}} + E C_{w,\text{net}} \pi^2}{(K_l L_t)^2}} \]

where \( K_y L_y \) is the unbraced length of member that the lateral-torsional buckling moment is considered. Other variables are defined in Section 2.3.2.1.4.

---

### 2.3.4.1.2 Point-Symmetric Sections (With Holes)

The global (lateral-torsional) buckling stress, \( F_{\text{cre}} \) for point-symmetric Z-sections bending about an x-axis that is perpendicular to the web and through the centroid, with holes spaced symmetrically along the length, shall be calculated as follows:

\[ F_{\text{cre}} = C_b \frac{r_{0,\text{avg}} A_g}{2S_f} \sqrt{\sigma_{\text{ey}} \sigma_t} \tag{Eq. 2.3.4.1.2-1} \]

All the variables are as defined in Section 2.3.4.1.1.

### 2.3.4.1.3 Closed-Boxed Section (With Holes)

The global (lateral-torsional) buckling stress, \( F_{\text{cre}} \) for closed-boxed sections, with holes spaced symmetrically along the length, shall be calculated as follows:

\[ F_{\text{cre}} = C_b \frac{\pi}{S_f K_y L_y} \sqrt{\frac{EI_{y,\text{avg}} GJ_{\text{avg}}}{K_l L_t}} \tag{Eq. 2.3.4.1.3-1} \]

All the variables are as defined in Sections 2.3.4.1.1 and 2.3.2.1.4.

### 2.3.4.2 Local Buckling (\( F_{\text{cr}l}, M_{\text{cr}l} \)) for Members With Holes

The local buckling of members with holes shall be computed in accordance with Section 2.3.3.2. When determining \( F_{\text{cr}l} \) for all elements, elements with holes shall be calculated as both unstiffened elements at the hole location and as a separate element where the hole is not located. For the unstiffened elements at the hole location, the buckling stress shall be modified to account for the net section by multiplying the buckling stress times the ratio \( S_{\text{fnet}}/S_t \).

### 2.3.4.3 Distortional Buckling (\( F_{\text{cr}d}, M_{\text{cr}d} \)) for Members With Holes

For members meeting the geometric criteria of Section 2.3.3.3 and having hole(s) in the
web, the distortional buckling moment shall be determined in accordance with Section 2.3.3.3 provided that thickness, t, in Eqs. 2.3.3.3-5 and 2.3.3.3-6 is replaced by modified thickness, \( t_r \), as follows:

\[
t_r = t \left(1 - \frac{L_h}{L_{crd}}\right)^{1/3}
\]

(Eq. 2.3.4.3-1)

where

- \( t \) = Thickness of web
- \( L_h \) = Hole length
- \( L_{crd} \) = Distortional buckling half-wavelength of the member with gross cross-section, determined numerically or using Eq. 2.3.3.3-4

For members meeting the geometric criteria of Section 2.3.3.3 and having patterned hole(s) along the web, the distortional buckling moment shall be determined in accordance with Section 2.3.3.3 provided that thickness, t, in Eqs. 2.3.3.3-4, 2.3.3.3-5, and 2.3.3.3-6 is replaced by modified thickness, \( t_r \), as follows:

\[
t_r = t \left(\frac{A_{web,net}}{A_{web,gross}}\right)^{1/3}
\]

(Eq. 2.3.4.3-2)

where

- \( t \) = Thickness of web
- \( A_{web,net} \) = Web surface area along member length subtracting hole areas
- \( A_{web,gross} \) = Web surface area along member length

### 2.3.5 Shear Buckling \((V_{cr})\)

The elastic shear buckling force, \( V_{cr} \), is permitted to be determined as follows:

\[
V_{cr} = k_v \frac{\pi^2 E}{12(1-\mu^2)(h/t)^2} A_w
\]

(Eq. 2.3.5-1)

where

- \( V_{cr} \) = Elastic shear buckling force of the web
- \( E \) = Modulus of elasticity of steel
- \( A_w \) = Web area
- \( \mu \) = Poisson’s ratio of steel
- \( h \) = Depth of the flat portion of web measured along the plane of the web
- \( t \) = Thickness of web
- \( k_v \) = Shear buckling coefficient calculated in accordance with Section G2.3, or for any open cross-section with a single web and single edge-stiffened compression flange extending to one side of the web where the stiffener is either a simple lip or a complex edge stiffener, the shear buckling coefficient may be calculated as follows:

\[
k_v = \frac{0.9}{\sin 2\phi} \left[ \frac{1}{(L/h)^2 \cos^2 \varphi} + C_1 (L/h)^2 \cos^2 \varphi + C_2 (1 + 2 \sin^2 \varphi) \right]
\]

(Eq. 2.3.5-2)

where
\[ \varphi = \arccos \left( \sqrt{C_3 + \sqrt{C_3^2 + C_4}} \right) \]  

(Eq. 2.3.5-3)

\[ C_1 = \frac{5.143\varepsilon^2 + 64.58\varepsilon + 108.6}{\varepsilon^2 + 20.57\varepsilon + 108.6} \]  

(Eq. 2.3.5-4)

\[ C_2 = \frac{2.472\varepsilon^2 + 41.14\varepsilon + 217.2}{\varepsilon^2 + 20.57\varepsilon + 108.6} \]  

(Eq. 2.3.5-5)

\[ C_3 = \frac{1.5C_2 - \frac{2}{(L/h)^2}}{4C_2 + C_1(L/h)^2} \]  

(Eq. 2.3.5-6)

\[ C_4 = \frac{(L/h)^2}{4C_2 + C_1(L/h)^2} \]  

(Eq. 2.3.5-7)

L = Minimum of L_{crd} and L_{m}

where

L_{crd} = 0.85h  

(Eq. 2.3.5-8)

L_{m} = Distance between discrete restraints that restrict shear buckling

\[ \varepsilon = \frac{k_{\phi fc}h}{Et^3/[12(1-\mu^2)]} \]  

(Eq. 2.3.5-9)

where

k_{\phi fc} = Elastic rotational stiffness provided by flange to flange/web juncture, as given in Eq. 2.3.1.3-3
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Appendix A

Provisions Applicable to the United States and Mexico

2016 EDITION WITH SUPPLEMENT 1
PREFACE TO APPENDIX A

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. Appendix A provides Specification provisions that apply only to the United States and Mexico.

Also included in Appendix A are technical items where full agreement between countries was not reached. Such items include certain provisions pertaining to the design of:

(a) Beams and compression members (C- and Z-sections) for standing seam roofs, and
(b) Bolted and welded connections.

Efforts are being made to minimize these differences in future editions of the Specification.
APPENDIX A, PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix addresses design provisions or supplements to Chapters A through M that specifically apply to the United States and Mexico. This appendix is considered mandatory for applications in the United States and Mexico.

A section number ending with the letter “a” indicates that the provisions herein supplement the corresponding section in Chapters A through M of the Specification. A section number not ending with the letter “a” indicates that the section gives the entire design provision.

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

The available flexural strength 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 F3, or shall be calculated in accordance with this section, where consideration of distortional buckling in accordance with Section F4 is permitted to be excluded.

The safety factor and the resistance factor provided in this section shall be applied to the nominal strength, $M_{n/o}$, calculated by Eq. I6.2.2-1 to determine the available strengths in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

$$M_n = RM_{n/o} \quad (Eq. \ I6.2.2-1)$$

$$\Omega_b = 1.67 \ (ASD)$$

$$\phi_b = 0.90 \ (LRFD)$$

where

$R$ = Reduction factor determined in accordance with AISI S908

$M_{n/o}$ = Nominal flexural strength with consideration of local buckling only, as determined from Section F3 with $F_n = F_y$ or $M_{ne} = M_y$

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

These provisions shall apply to Z-sections concentrically loaded along their longitudinal axis, with only one flange attached to standing seam roof panels. Alternatively, design values for a particular system are permitted to be based on discrete point bracing locations, or on tests in accordance with Section K2.

The nominal axial strength, $P_{nv}$, of simple span or continuous Z-sections shall be calculated in accordance with (a) and (b). Consideration of distortional buckling in accordance with Section E4 is permitted to be excluded.

Unless otherwise specified, the safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

(a) For weak axis available strength

$$P_n = k_{af} RF_y A \quad (Eq. \ I6.2.4-1)$$
\[ \Omega = 1.80 \quad (ASD) \]
\[ \phi = 0.85 \quad (LRFD) \]

where

For \( \frac{d}{t} \leq 90 \)
\[ k_{af} = 0.36 \]

For \( 90 < \frac{d}{t} \leq 130 \)
\[ k_{af} = 0.72 - \frac{d}{250t} \quad (Eq. I6.2.4-2) \]

For \( \frac{d}{t} > 130 \)
\[ k_{af} = 0.20 \]

\( R \) = Reduction factor determined from uplift tests performed using AISI S908
\( A \) = Full unreduced cross-sectional area of Z-section
\( d \) = Z-section depth
\( t \) = Z-section thickness
\( F_y \) = Design yield stress determined in accordance with Section A3.3.1

Eq. I6.2.4-1 shall be limited to roof systems meeting the following conditions:

1. Purlin thickness, 0.054 in. (1.37 mm) \( \leq t \leq 0.125 \) in. (3.22 mm),
2. 6 in. (152 mm) \( \leq d \leq 12 \) in. (305 mm),
3. Flanges are edge-stiffened compression elements,
4. \( 70 \leq \frac{d}{t} \leq 170 \),
5. \( 2.8 \leq \frac{d}{b} < 5 \), where \( b \) = Z-section flange width,
6. \( 16 \leq \frac{t}{\text{flange flat width}} < 50 \),
7. Both flanges are prevented from moving laterally at the supports, and
8. Yield stress, \( F_y \leq 70 \) ksi (483 MPa or 4920 kg/cm²).

(b) The available strength about the strong axis shall be determined in accordance with Sections E2 and E3.

**I6.3.1a Strength of Standing Seam Roof Panel Systems**

In addition to the provisions provided in Section I6.3.1, for load combinations that include wind uplift, the nominal wind load, to be applied to the standing seam roof panel, clips and fasteners, is permitted to be multiplied by 0.67 provided the tested system and wind load evaluation satisfy the following conditions:

(a) The roof system is tested in accordance with AISI S906.
(b) The wind load is calculated using ASCE/SEI 7 for components and cladding.
(c) The area of the roof being evaluated is in Zone 2 (edge zone) or Zone 3 (corner zone), as defined in ASCE/SEI 7; i.e., the 0.67 factor does not apply to the field of the roof (Zone 1). The nominal wind load applied to Zone 2 or Zone 3, after the 0.67 multiplier is applied, shall not be less than the nominal wind load applied to the field of the roof (Zone 1).
(d) The base metal thickness of the standing seam roof panel is greater than or equal to 0.023 in. (0.59 mm) and less than or equal to 0.030 in. (0.77 mm).
(e) For trapezoidal profile standing seam roof panels, the distance between sidelaps is no
greater than 24 in. (610 mm).
(f) For vertical rib profile standing seam roof panels, the distance between sidelaps is no greater than 18 in. (460 mm).
(g) The observed failure mode of the tested system is one of the following:
   (1) The standing seam roof clip mechanically fails by separating from the panel sidelap.
   (2) The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base.

**J2a  Welded Connections**

Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions shall apply to the welding positions as listed in Table J2a.

**J3.4  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 in accordance with this section. The safety factor and the resistance factor given in this section shall be used to determine the available strengths in accordance with the applicable design method in Section B3.2.1 or B3.2.2.

\[
P_n = A_B F_n \quad (Eq. J3.4-1)
\]

\[
\Omega = 2.00 \quad (ASD)
\]

\[
\phi = 0.75 \quad (LRFD)
\]

where

- $A_B = \text{Gross cross-sectional area of bolt}$
- $F_n = \text{Nominal strength, ksi (MPa), determined in accordance with (a) or (b) as follows:}$
- (a) When bolts are subjected to shear only or tension only, $F_n$ shall be given by $F_{nv}$ or $F_{nt}$ in Table J3.4-1.

The pull-over strength of the connected sheet at the bolt head, nut, or washer shall be
considered where bolt tension is involved. See Section J6.2.

(b) When bolts are subjected to a combination of shear and tension, $F_n$ is given by $F'_{nt}$ in Eq. J3.4-2 or J3.4-3 as follows:

For ASD

$$F'_{nt} = 1.3 F_{nt} - \frac{\Omega F_{nt} f_v}{F_{nv}} \leq F_{nt}$$

(Eq. J3.4-2)

For LRFD

$$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt} f_v}{\phi F_{nv}} \leq F_{nt}$$

(Eq. J3.4-3)

where

$F'_{nt} = \text{Nominal tensile strength modified to include the effects of required shear strength, ksi (MPa)}$

$F_{nt} = \text{Nominal tensile strength from Table J3.4-1}$

$F_{nv} = \text{Nominal shear strength from Table J3.4-1}$

$f_v = \text{Required shear strength, ksi (MPa)}$

In addition, the required shear strength, $f_v$, shall not exceed the allowable shear strength, $F_{nv} / \Omega$ (ASD), or the design shear strength, $\phi F_{nv}$ (LRFD), of the fastener.

In Table J3.4-1, the nominal shear strength shall apply to bolts in holes as limited by Table J3-1 (J3-1M). Washers or back-up plates shall be installed over long-slotted holes, and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Section K2.
TABLE J3.4-1
Nominal Tensile and Shear Strengths for Bolts

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Nominal Tensile Strength ( F_{nt} ), ksi (MPa)</th>
<th>Nominal Shear Strength ( F_{nv} ), ksi (MPa) (^a)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1/4 in. ( \leq d &lt; 1/2 ) in. ((6.4 \text{ mm} \leq d &lt; 12 \text{ mm}))</td>
<td>1/4 in. ( \leq d &lt; 1/2 ) in. ((6.4 \text{ mm} \leq d &lt; 12 \text{ mm}))</td>
</tr>
<tr>
<td>ASTM A307 Grade A Bolts</td>
<td>40 (280)</td>
<td>24 (169) (^b)</td>
</tr>
<tr>
<td>ASTM F3125 Grade A325/A325M Bolts:</td>
<td>NA</td>
<td>54 (372)</td>
</tr>
<tr>
<td>- When threads are not excluded from shear planes</td>
<td>90 (620)</td>
<td>NA</td>
</tr>
<tr>
<td>- When threads are excluded from shear planes</td>
<td>61 (411)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>ASTM A354 Grade BD Bolts:</td>
<td>101 (700)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>- When threads are not excluded from shear planes</td>
<td>113 (780)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>- When threads are excluded from shear planes</td>
<td>84 (579)</td>
<td>84 (579)</td>
</tr>
<tr>
<td>ASTM A449 Bolts:</td>
<td>81 (560)</td>
<td>54 (372)</td>
</tr>
<tr>
<td>- When threads are not excluded from shear planes</td>
<td>90 (620)</td>
<td>54 (372)</td>
</tr>
<tr>
<td>- When threads are excluded from shear planes</td>
<td>48 (334)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>ASTM F3125 Grade A490/A490M Bolts:</td>
<td>NA</td>
<td>68 (457)</td>
</tr>
<tr>
<td>- When threads are not excluded from shear planes</td>
<td>113 (780)</td>
<td>68 (457)</td>
</tr>
<tr>
<td>- When threads are excluded from shear planes</td>
<td>NA</td>
<td>84 (579)</td>
</tr>
<tr>
<td>Threaded Parts:</td>
<td>0.675 ( F_u ) (^c)</td>
<td>0.400 ( F_u )</td>
</tr>
<tr>
<td>- When threads are not excluded from shear planes</td>
<td>0.75 ( F_u )</td>
<td>0.450 ( F_u )</td>
</tr>
<tr>
<td>- When threads are excluded from shear planes</td>
<td>0.563 ( F_u )</td>
<td>0.563 ( F_u )</td>
</tr>
</tbody>
</table>

Notes:
- a. For end-loaded connections with a fastener pattern length greater than 38 in. (965 mm), \( F_{nv} \) should be reduced to 83.3 percent of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.
- b. Threads permitted in shear planes.
- c. Tensile strength of bolt.
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Appendix B

Provisions Applicable to Canada

2016 EDITION WITH SUPPLEMENT 1
PREFACE TO APPENDIX B

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix addresses Specification provisions that are applicable only to 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 the following:

(a) Beams (C- and Z-sections) for standing seam roofs,
(b) Bolted and welded connections, and
(c) Lateral and stability bracing.

Efforts will be made to minimize these differences in future editions of the Specification.

In Canada, SI units are the units of record for the purpose of this Specification.
APPENDIX B, PROVISIONS APPLICABLE TO CANADA

Specification Chapters A through M contain design provisions that are applicable to Canada, Mexico, and the United States, and accommodate those provisions that may be partially applicable to certain countries. This appendix is considered mandatory for applications in Canada.

A section number ending with the letter “a” indicates that the provisions herein supplement the corresponding section in Chapters A through M of the Specification. A section number not ending with the letter “a” indicates that the section gives the entire design provision.

C2a Lateral and Stability 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 ensure 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.

C2.1 Symmetrical Beams and Columns

Discrete bracing of axially loaded compression members shall meet the requirements specified in Section C2.3 of the Specification. In addition, the provisions of Sections C2.1.1 and C2.1.2 of this appendix shall apply to symmetric sections in compression or bending in which the applied load does not induce twist.

C2.1.1 Discrete Bracing for Beams

The factored resistance of braces shall be at least 2 percent of the factored compressive force in the compressive flange of a member in bending at the braced location. 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.

C2.1.2 Bracing by Deck, Slab, or Sheathing for Beams and Columns

The factored resistance of the attachments along the entire length of the braced member shall be at least 5 percent 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.

C2.2a C-Section and Z-Section Beams

The provisions of Sections C2.2.2, C2.2.3, and C2.2.4 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.
C2.2.2 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 engineering analysis, testing, or Section I6.2.1 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 engineering 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 ≤ 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 centre of this loaded length.

C2.2.3 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 C2.1.2 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 C2.2.2 of this appendix.

C2.2.4 Both Flanges Braced by Deck, Slab, or Sheathing

The factored resistance of the attachment shall be as given by Section C2.1.2 of this appendix.

I6.2.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

This type of member shall have discrete bracing in accordance with Section C2.2.2 of this appendix.

J2a Welded Connections

Fabricators and erectors performing arc welding shall comply with the requirements of CSA W47.1 (Division 1 or Division 2). The work may be sublet to a Division 3 fabricator or erector; however, the Division 1 or Division 2 fabricator or erector shall retain responsibility for the sublet work. Fabricators and erectors performing resistance welding shall comply with the requirements of CSA W55.3.

Note: In Canada, accreditation of welding inspection bodies is provided by the Standards Council of Canada.

Where at least one of the connected parts is between 0.70 mm and 4.76 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 W59. Except as provided in Section J2.2 of the Specification, 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. For arc spot welds connecting sheets to a thicker supporting member, the applicable base steel thickness limits shall be 0.70 mm to 5.84 mm.
The resistance in tension or compression of butt welds shall be the same as that prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the joint.

### J3.4 Shear and Tension in Bolts

For ASTM A307 bolts less than 12.7 mm in diameter, refer to Tables J3.4-1 and J3.4-2 of this appendix. For all other bolts, refer to CSA S16.

#### TABLE J3.4-1
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 6.4 mm ≤ $d$ &lt; 12.7 mm</td>
<td>279</td>
<td>0.65</td>
<td>165</td>
<td>0.55</td>
</tr>
</tbody>
</table>

#### TABLE J3.4-2
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 When 6.4 mm ≤ $d$ &lt; 12.7 mm</td>
<td>$324 - 2.4f_v \leq 279$</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Note: The actual shear stress, $f_v$, shall also satisfy Table J3.4-1 of this appendix.

The nominal bolt resistance, $P_n$, resulting from shear, tension, or a combination of shear and tension shall be calculated as follows:

$P_n = A_b F_n$  

(Eq. J3.4-1)

where

$A_b = \text{Gross cross-sectional area of bolt}$

$F_n = \text{A value determined in accordance with Items (a) and (b) below, as applicable:}$

(a) When bolts are subjected to shear or tension, $F_n$ is given by $F_{nt}$ or $F_{nv}$ in Table J3.4-1, as well as the $\phi$ values.

(b) When bolts are subjected to a combination of shear and tension, $F_n$ is given by $F'_{nt}$ in Table J3.4-2, 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 J6.2 of the Specification.

### K2.1.1a Load and Resistance Factor Design and Limit States Design

To calculate the resistance factor of an interior partition wall stud that is in a composite steel-framed wall system with gypsum sheathing attached to both flanges and that is
limited to a transverse (out-of-plane) specified load of not more than 0.5 kPa, a superimposed specified axial load, exclusive of sheathing materials, of not more than 1.46 kN/m, or a superimposed specified axial load not more than 0.89 kN, the following shall apply:

(a) \( C_\phi = 1.42 \),
(b) \( M_m = 1.10 \),
(c) \( F_m = 1.00 \),
(d) \( V_M = 0.10 \),
(e) \( V_F = 0.05 \), and
(f) \( \beta_o = 1.82 \).
Commentary on the
North American Specification
for the Design of Cold-Formed
Steel Structural Members

2016 EDITION WITH SUPPLEMENT 1
DISCLAIMER

The material contained herein has been developed by a joint effort of the American Iron and Steel Institute (AISI) Committee on Specifications, the CSA Group 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 – October 2016
2nd Printing – March 2018
3rd Printing – December 2018
PREFACE

This document provides a commentary on the 2016 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Members*.

The purpose of the *Commentary* is: (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.

Consistent with the *Specification*, the *Commentary* contains a main document, Chapters A through M, Appendices 1 and 2, and the country-specific provisions Appendices A and B. A symbol $\Rightarrow_{A,B}$ is used in the main document to point out that additional discussions are provided in the corresponding country-specific provisions in Appendices A or B.

AISI appreciates the tremendous efforts of the committee members in reorganizing the whole *Specification* and the *Commentary*. Special thanks go to Mr. Richard Kaehler, who developed the outline; and Dr. Benjamin Schafer, who provided leadership in reorganizing the *Specification* and *Commentary*. Appreciation is extended to Chairman Roger Brockenbrough and Vice Chairman Richard Haws for their lasting contributions to the AISI Committee of Specifications.

In the third printing, the changes included in Supplement 1 are incorporated.

American Iron and Steel Institute
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INTRODUCTION

Cold-formed steel members have been used economically for building construction and other applications (Winter, 1959a, 1959b; Yu and LaBoube, 2010). 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 strips 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 and LaBoube, 2010): (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 around 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 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 into Spanish in 1965 (Camara, 1965).

The first edition of the North American Specification (AISI, 2001), applicable to the United States, Canada and Mexico, was published in 2001. This 2001 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 the North American Specification, the ASD and LRFD methods are used in the United States and Mexico, while the LSD method is used in Canada. The North American Specification was revised and updated in 2004, 2007, 2010, and 2012 (AISI, 2004; AISI, 2007; AISI, 2010; and AISI, 2012) as new technology was adopted. The Direct Strength Method was introduced in 2004 (AISI, 2004) as an alternative design method. The second-order analysis of structural systems was added in 2007 (AISI, 2007).
In the 2012 edition of the Specification, the added design provisions included the design of power-actuated fasteners and the Direct Strength Method for determining compression and flexural strength of perforated members, shear strength for non-perforated members, and member reserve capacities.

In 2016, the North American Specification for the Design of Cold-Formed Steel Structural Members was reorganized—The Direct Strength Method was moved into Chapters D through H of the Specification and is considered as an equivalent design method to the Effective Width Method; the provisions for determining the effective width of elements were moved into Appendix 1; and the determination of buckling loads was moved into Appendix 2. The provisions were reorganized to be consistent, where possible, with the layout of the AISC Specification. Accordingly, the Commentary on the Specification has also been revised and reorganized.


During the period from 1958 through 1983, AISI published Commentaries on several editions of the AISI design Specifications, which were prepared by Professor George Winter of Cornell University in 1958, 1961, 1962, and 1970. Since 1983, the format used for the AISI Commentary has been such that the same section numbers are used for the Commentary as for 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 2001 edition of the Commentary (AISI, 2001) was based on the Commentary for the 1996 AISI Specification.

The current edition of the Commentary is updated for the 2016 edition of the North American Specification. It contains Chapters A through M, Appendices 1 and 2, and Appendices A and B, where commentary on provisions that are only applicable to a specific country is included in the corresponding lettered appendix.

As in previous editions of the Commentary, this document contains a brief presentation of the characteristics and performance of cold-formed steel structural members, connections and assemblies. 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 edition of the Commentary, the majority of the technical contents in the 2012 edition of the Commentary have been retained. However, due to the content reorganization, readers may refer back to the 2012 edition of the Commentary for some specific changes made prior to the 2016 edition of the Specification.

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 Scope, Applicability, and Definitions

A1.1 Scope

The cross-sectional configurations, manufacturing processes and fabrication practices of cold-formed steel structural members differ in several respects from those 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 stress 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) different characteristics of stress-strain curves that can be either the sharp-yielding type or gradual-yielding type.

The Specification is applicable only to cold-formed sections not more than 1 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 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.

A1.2 Applicability

The Specification (AISI, 2012a) 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 (ASD) method or the Load and Resistance Factor Design (LRFD) method for the United States and Mexico. Only the Limit States Design (LSD) method is permitted in Canada.

In this Commentary, the bracketed terms are equivalent terms that apply particularly to LSD. A symbol $\equiv x$ is used to point out that additional provisions are provided in the country-specific appendices as indicated by the letter, x.

Because of the diverse forms of cold-formed steel structural members and connections, it is not possible to cover all design configurations by the design rules presented in the Specification. For those special cases where the available strength [factored resistance] and/or stiffness cannot be determined, it can be established by:

(a) Testing in accordance with the provisions of Section K2.1.1(a),

(b) Rational engineering analysis and confirmatory testing evaluated in accordance with the provisions of Section K2.1.1(b), or

(c) Rational engineering analysis only in accordance with the provisions of Section A1.2(c).

Prior to 2001, the only option in such cases was testing. Since 2001, in recognition of the fact that this was not always practical or necessary, the rational engineering analysis options were added. It is essential that such analysis be based on theory that is appropriate for the situation and sound engineering judgment. Specification Section A1.2(b) was added for
components that have significant geometric variations such that it becomes impractical to
test each variation in accordance with Specification Section A1.2(a). This is particularly
useful when the following applies:

(1) A form of cold-formed steel component is being evaluated that is outside the scope
of the Specification,

(2) The member or assembly being evaluated has a degree of variation, such as
variations in cross-sectional dimensions, that makes it impractical to test each
individual variation,

(3) More accurate safety and resistance factors than those prescribed by Section A1.2(c) are
desired, and

(4) A test program can be conducted in accordance with Section K2.

In any case, safety and resistance factors given in Specification Section A1.2(c) should not be
used if applicable safety factors or resistance factors in Specification Chapters A through M,
Appendices 1 and 2, and Appendices A and B are more conservative. These provisions must
not be used to circumvent the intent of the Specification. Where the provisions of Chapters B
through J and L through M of the Specification and Appendices A and B apply, those
provisions must be used and cannot be avoided by testing or rational engineering analysis.

In order to provide better alignment between the testing provisions of Section K2 and the
provisions for rational engineering analysis, the safety factor for the rational engineering analysis of
connections was adjusted upward from $\Omega = 2.5$ to $\Omega = 3.0$ in 2016. Compatible adjustments were
also made to the accompanying resistance factors, $\phi$, for rational engineering analysis.

A1.3 Definitions

Many of the definitions in Specification Section A1.3 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
post-buckling 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 Appendix 1, and the corresponding effective widths are discussed in the
Commentary on that appendix.

Multiple-Stiffened Elements

Multiple-stiffened elements of two cross-sections are shown in Figure C-A1.3-1. Each of the
two outer sub-elements of cross-section (1) is stiffened by a web and an intermediate
stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The
two sub-elements of cross-section (2) are stiffened by a web and the attached intermediate
middle stiffener.

Stiffened or Partially Stiffened Compression Elements

Stiffened compression elements of various cross-sections are shown in Figure C-A1.3-2 in
which Cross-sections (1) through (5) are for flexural members, and Cross-sections (6)
through (9) are for compression members. Cross-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 Appendix 1. Cross-sections (3), (4), and (5) show compression elements stiffened by two webs. Cross-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. Cross-section (7) has four compression elements stiffening each other, and cross-section (9) has each stiffened element stiffened by a lip and the other stiffened element.

![Multiple-Stiffened Compression Elements](image-url)

*Figure C-A1.3-1 Multiple-Stiffened Compression Elements*
Figure C-A1.3-2 Stiffened Compression Elements

- (1) Lipped Channel
- (2) I-Beam Made of Two Lipped Channels Back-to-Back
- (3) Hat Section
- (4) Box-Type Section
- (5) Inverted "U" Type Section
- (6) Lipped Channel
- (7) Box-Type Section
- (8) Section Made of Two Lipped Channels Back-to-Back
- (9) Lipped Angle

Flexural Members, Such as Beams (Top Flange in Compression)

Compression Members, Such as Columns
**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.

**Flexural-Torsional Buckling**

The 1968 edition of the *Specification* pioneered methods for computing column loads of cold-formed steel cross-sections prone to buckling by simultaneous twisting and bending. This complex behavior may result in lower column loads than would result from primary buckling by flexure alone.

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**Figure C-A1.3-3 Unstiffened Compression Elements**
Unstiffened Compression Elements

Unstiffened elements of various cross-sections are shown in Figure C-A1.3-3, in which Cross-sections (1) through (4) are for flexural members and cross-sections (5) through (8) are for compression members. Cross-sections (1), (2), and (3) have only a web to stiffen the compression flange element. The legs of Cross-section (4) provide mutual stiffening action to each other along their common edges. Cross-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 Cross-section (8) provide mutual stiffening action to each other.

ASD and LRFD Terms (United States and Mexico)

ASD (Allowable Strength Design, formerly referred to as Allowable Stress Design)

Allowable Strength Design (ASD) is a method of designing structural components such that the allowable strength (force or moment) permitted by various sections of the Specification is not exceeded when the structure is subjected to all appropriate loads and load combinations in accordance with Specification Section B2. See also Specification Section B3.2.1 for ASD requirements.

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 loads and load combinations in accordance with Specification Section B2. See also Specification Section B3.2.2 for LRFD 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 loads and load combinations in accordance with Specification Section B2. See also Specification Section B3.2.3 for LSD requirements.

In the 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 load ratios, and target reliability indexes are slightly different. In most cases, same nominal strength [resistance] equations are used for ASD, LRFD, and LSD approaches.
**A1.4 Units of Symbols and Terms**

The nondimensional 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-A1.4-1 is a conversion table for these three different units.

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**A2 Referenced Specifications, Codes, and Standards**

Other specifications and standards to which the *Specification* makes references have been listed and updated in *Specification* Section A2 to provide the effective dates of these standards at the time of approval of this *Specification*. References for country-specific provisions are provided in *Specification* Section A2.1 for the U.S. and Mexico and A2.2 for Canada.

Additional references which the designer may use for related information are listed in the *Commentary* section, References.
A3 Material

A3.1 Applicable Steels

ASTM International and CSA Group are the basic sources of steel designations for use with the Specification. Specification Section A3.1 contains the complete list of steel standards that are accepted by the Specification. Dates of issue are included in Specification Section A2.

In 2012, the list of applicable steels was enhanced by categorizing them into three groups based on the specified minimum elongation in a 2-inch (50-mm) gage length: ten (10) percent or greater elongation, three (3) percent to ten (10) percent elongation, and less than three (3) percent elongation. This eliminated the need to identify specific steel grades in subsequent sections.

In the 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 the applicable steels.

In 2007, the ASTM A1039 Standard was added to the list of the applicable steels. For all grades of steel, ASTM A1039 complies with the minimum required $F_u/F_y$ ratio of 1.08. Thicknesses equal to or greater than 0.064 in. (1.6 mm) and less than or equal to 0.078 in. (2.0 mm) also meet the minimum elongation requirements of Specification Section A3.1.1 and no reduction in the specified minimum yield stress is required. However, steel thicknesses less than 0.064 in. (1.6 mm) with yield stresses greater than 55 ksi (380 MPa) do not meet the requirements of Specification Section A3.1.1 and are subject to the limitations of Specification Section A3.1.2.

In 2012, the ASTM A1063/A1063M Standard was added to the list of the applicable steels. The ASTM A1063/A1063M Standard is intended to be a match to ASTM A653/A653M, but the materials are produced using a “twin-roll casting process,” which is also used to produce materials conforming to the ASTM A1039/A1039M Standard.

The important material properties for the design of cold-formed steel members are yield stress, tensile strength, and ductility. Ductility is the ability of 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 (50-mm) gage length. The ratio of the tensile strength to the yield stress is also an important material property; this is an indication of strain hardening and the ability of the material to redistribute stress.

A3.1.1 Steels With a Specified Minimum Elongation of Ten Percent or Greater

(Elongation $\geq 10\%$)

Material specifications for low-carbon sheet and strip steels with specified minimum yield stress from 24 to 50 ksi (165 to 345 MPa or 1690 to 3520 kg/cm²) provide specified minimum elongations in a 2-inch (50-mm) gage length of 11 to 30 percent, thus easily meeting the 10-percent minimum requirement for this category. Steels with yield stresses higher than 50 ksi (345 MPa or 3520 kg/cm²) are often produced as low-alloy steels in order to meet these ductility requirements. Elongations are determined in accordance with
ASTM test method A370 (A1058).

For the listed standards, the yield stresses of steels range from 24 to 80 ksi (165 to 550 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 not less than 1.13, and the elongations are not less than 10 percent. The conditions for use of steels that have a defined ductility of at least three percent (3%) are outlined in Specification Section A3.1.2. The conditions for use of steels that have a defined ductility of less than three percent (3%) are outlined in Specification Section A3.1.3.

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 stress 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. In 2004, the Specification listing of ASTM A1003/A1003M steel was revised to list only the grades designated Type H, because this is the only grade that satisfies the criterion for unrestricted usage. Grades designated Type L can still be used but are subject to the restrictions of Specification Section A3.1.3.

Certain grades of ASTM A653, A792, and A1039 have elongations that vary based upon the thickness of the material. Exceptions are provided for those steels that do not belong to the designated group.

### A3.1.2 Steels With a Specified Minimum Elongation From Three Percent to Less Than Ten Percent (3% ≤ Elongation < 10%)

Steels listed in this section have specified minimum elongations less than the 10 percent limitation for unlimited utilization within the Specification. However, they do have some defined ductility.

For the determination of the tension strength of members and connections in Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/A792M, tension tests on sheet steels and shear tests on connections using steel produced to Australian Standard AS1397 G550 (Standards Australia, 2001), which is similar in minimum ductility (2%) to ASTM A792 Grade 80 (550) Class 3 (minimum ductility 3%), were performed at the University of Sydney by Rogers and Hancock. These included sheet steels in tension with and without perforations (Rogers and Hancock, 1997), bolted connections in shear (Rogers and Hancock, 1998; Rogers and Hancock, 1999b), screw connections in shear (Rogers and Hancock, 1999a), and sheet steel fracture toughness tests (Rogers and Hancock, 2001).

### A3.1.3 Steels With a Specified Minimum Elongation of Less Than Three Percent (Elongation < 3%)

ASTM A653/A653M SS Grade 80 (550) Class 1 and 2; ASTM A792/A792M Grade 80 (550) Class 1 and 2; ASTM A875 SS Grade 80 (550); and ASTM A1008/A1008M SS Grade 80 (550) steels have a specified minimum yield stress of 80 ksi (550 MPa or 5620 kg/cm²), a specified minimum tensile strength of 82 ksi (565 MPa or 5770 kg/cm²), and no stipulated minimum elongation in a 2-inch (50-mm) gage length. These steels do not have adequate ductility as defined by Specification Section A3.1.1. 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. Their use has been limited in Specification Section A3.1.3 to particular multiple-web configurations such as roofing, siding, and floor decking.

In the past, the yield stress used in design was limited to 75 percent of the specified minimum yield stress, or 60 ksi (414 MPa or 4220 kg/cm$^2$), and the tensile strength used in design was limited to 75 percent of the specified minimum tensile strength, or 62 ksi (427 MPa or 4360 kg/cm$^2$), whichever was lower. This introduced a higher safety factor, but still made low-ductility steels, such as SS Grade 80 and Grade E, useful for the named applications.

Based on the UMR research findings (Wu, Yu, and LaBoube, 1996), Equation A3.1.3-1 was added in Specification Section A3.1.3 to determine the reduced yield stress, $R_bF_{sy}$, for the calculation of the nominal flexural strength [resistance] of multiple-web sections such as roofing, siding and floor decking (AISI, 1999). For the unstiffened compression flange, Equation A3.1.3-2 was 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 Specification. When the multiple-web section is composed of both stiffened and unstiffened compression flange elements, the smallest $R_b$ should be used to determine the reduced yield stress for use on the entire section. Different values of the reduced yield stress could be used for positive and negative moments.

The equations provided in Specification Section A3.1.3 can also be used for calculating the nominal flexural strength [resistance] when the available strengths [factored resistances] are determined on the basis of tests as permitted by rational engineering analysis.

It should be noted that the exception clause in Specification Section A3.1 should be followed for steel deck used for composite slabs when the deck is used as the tensile reinforcement.

For the calculation of web crippling strength of deck panels, although the UMR study (Wu, Yu, and LaBoube, 1997) shows that the specified minimum yield stress can be used to calculate the web crippling strength of deck panels, the Specification provides a more conservative approach. The lesser of 0.75 $F_{sy}$ and 60 ksi (414 MPa or 4220 kg/cm$^2$) is used to determine both the web crippling strength (Specification Section G5) and the shear strength (Specification Section G2) for the low-ductility steels. This is consistent with the previous edition of the 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/cm$^2$), 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 D through M.
A3.2 Other Steels

Although the use of the steel standards listed in Specification Section A3.1 is encouraged, other steels may also be used in cold-formed steel structures, provided they satisfy the requirements stipulated in Specification Section A3.2.

ASTM and CSA Group material standards include references to general requirements standards that cover information such as dimensional tolerances and testing protocols that are similar across a set of material standards to minimize duplication and inconsistencies. For sheet steel used for cold-formed products, the typical general requirements standards are as follows:

(a) For coated sheets, ASTM A924/A924M-14 or CSA G40.20-13, as applicable;
(b) For hot-rolled or cold-rolled sheet and strip, ASTM A568/A568M-15 or CSA G40.20-13, as applicable;
(c) For plate and bar, ASTM A6/A6M-14 or CSA G40.20-13, as applicable;
(d) For hollow structural sections (carbon steel), ASTM A500/A500M-13 or CSA G40.20-13, as applicable;
(e) For hollow structural sections (HSLAS steel), ASTM A847/A847M-14 or CSA G40.20-13, as applicable.

In 2004, these requirements were clarified and revised. The Specification has long required that such “other steels” conform to the chemical and mechanical requirements of one of the listed specifications or “other published specification.” Specific requirements for a published specification have been detailed in the definitions under Specification Section A1.3, General Terms. It is important to note that, by this definition, published requirements must be established before the steel is ordered, not by a post-order screening process. The requirements must include minimum tensile properties, chemical composition limits, and for coated sheets, coating properties. Testing procedures must be in accordance with the referenced ASTM or CSA Group specifications. A proprietary specification of a manufacturer, purchaser, or producer could qualify as a published specification if it meets the definition requirements.

As an example of these Specification provisions, it would not be permissible to establish a minimum yield stress or minimum tensile strength greater than that ordered to a standard ASTM grade by reviewing mill test reports or conducting additional tests. However, it would be permissible to publish a manufacturer’s or producer’s specification before the steel is ordered requiring that such enhanced properties be furnished as a minimum. Testing to verify that the minimum properties are achieved could be done by the manufacturer or the producer. The intent of these provisions is to ensure that the material factor, $M_m$ (see Specification Section K2), will be maintained at about 1.10, corresponding to an assumed typical 10 percent overrun in tensile properties for ASTM grades.

Where the material is used for fabrication by welding, care must be exercised in selection of chemical composition or mechanical properties to ensure compatibility with the welding process and its potential effect on altering the tensile properties.

Special additional requirements have been added to qualify unidentified material. In such a case, the manufacturer must run tensile tests sufficient to establish that the yield stress and tensile strength of each master coil are at least 10 percent greater than the applicable published specification. As used here, master coil refers to the coil being processed by the manufacturer. Of course, the testing must always be adequate to ensure that specified minimum properties
are met, as well as the ductility requirements of Specification Section A3.1.1, A3.1.2, or A3.1.3 as desired.

**A3.2.1 Ductility Requirements of Other Steels**

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 (50-mm) ranging from 50 to 1.3 percent, and tensile strength-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 strength-to-yield strength ratio shall not be less than 1.08, and (2) The total elongation in a 2-inch (50-mm) gage length shall not be less than 10 percent, or not less than 7 percent in an 8-inch (200-mm) gage length. Also, the Specification limits the use of Chapters D through J to adequately ductile steels. In lieu of the tensile strength-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 and LaBoube, 2010). Further information on the test procedure should be obtained from AISI S903, Standard Methods for Determination of Uniform and Local Ductility (AISI, 2013b). Because of limited experimental verification of the structural performance of members using materials having a tensile strength-to-yield strength ratio less than 1.08 (Macadam et al., 1988), the Specification limits the use of this material to purlins, girts, and curtain wall studs meeting the elastic design requirements of Sections F2, F3, I6.2.1, I6.2.2, I6.3.1, and additional country-specific requirements given in the appendices. Thus, the use of such steels in other applications is prohibited. However, in purlins, girts, and curtain wall studs (with special country-specific requirements given in the appendices), concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Specification Section H1.2 are met and \( \Omega_c P / P_n \) does not exceed 0.15 for allowable strength design, \( P_u / \phi_c P_n \) does not exceed 0.15 for the Load and Resistance Factor Design, and \( P_f / \phi_c P_n \) does not exceed 0.15 for the Limit States Design.

In 2007, curtain wall studs were added to the applications for materials having a tensile strength-to-yield strength ratio less than 1.08. Curtain wall studs are repetitive framing members that are typically spaced more closely than purlins and girts. Curtain wall studs are analogous to vertical girts; as such, they are not subjected to snow or other significant sustained gravity loads.

With the addition of the provisions of Specification Section A3.1.2 in 2012, the use of the alternative approach for the limited range of structural usage is largely superseded by the provisions of Specification Section A3.1.2.

**A3.2.1.1 Restrictions for Curtain Wall Studs**

Pending future research regarding the cyclic performance of connections, an exception is noted on use of lower ductility steels as defined in Section A3.2.1 for curtain wall studs supporting heavyweight exterior walls in high seismic areas.
A3.3 Yield Stress and Strength Increase From Cold Work of Forming

A3.3.1 Yield Stress

The strength of cold-formed steel structural members depends on the yield stress, 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 the sharp-yielding type (Figure C-A3.3.1-1(a)) or gradual-yielding type (Figure C-A3.3.1-1(b)), the method for determining the yield point for sharp-yielding steel and the yield strength for gradual-yielding steel are based on ASTM Standard A370 (ASTM, 2015). As shown in Figure C-A3.3.1-2(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-A3.3.1-2(b)) or the extension under the load method (Figure C-A3.3.1-2(c)). The term yield stress used in the Specification applies to either yield point or yield strength. Section 1.2 of the AISI Design Manual (AISI, 2013) lists the minimum mechanical properties specified by the ASTM specifications for various steels.

![Stress-Strain Curves of Carbon Steel Sheet or Strip](image)

Figure C-A3.3.1-1 Stress-Strain Curves of Carbon Steel Sheet or Strip
(a) Sharp Yielding, (b) Gradual Yielding

The strength of members that are governed by buckling depends not only on the yield stress but also on the modulus of elasticity of steel, E, and the tangent modulus of steel, Et. The modulus of elasticity is defined by the slope of the initial straight portion of the stress-strain curve (Figure C-A3.3.1-1). 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 2.0x10^6 to 2.1x10^6 kg/cm^2). A value of 29,500 ksi (203 GPa or 2.07x10^6 kg/cm^2) 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-A3.3.1-1(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 stress.

Determination of proportional limits for informational purposes can be done simply by using the offset method shown in Figure C-A3.3.1-2(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.

![Figure C-A3.3.1-2 Stress-Strain Diagrams Showing Methods of Yield Point and Yield Strength Determination](image)

**A3.3.2 Strength Increase From Cold Work of Forming**

The mechanical properties of the flat steel sheet, strip, plate or bar, such as yield stress, tensile strength, and elongation may be substantially different from the properties exhibited by the cold-formed steel sections. Figure C-A3.3.2-1 illustrates the increase of yield stress 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-A3.3.2-2 (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 stresses 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

Figure C-A3.3.2-1 Effect of Cold Work on Mechanical Properties in Cold-Formed Steel Sections. (a) Channel Section, (b) Joist Chord
(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 stress 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 stress.

Investigating the influence of cold work, Karren derived the following equations for the ratio of corner yield stress-to-virgin yield stress (Karren, 1967):

$$\frac{F_{yc}}{F_{yv}} = \frac{B_c}{(R/t)^m}$$  \hspace{1cm} (C-A3.3.2-1)

where

$$B_c = 3.69 \frac{F_{uv}}{F_{yv}} - 0.819 \left( \frac{F_{uv}}{F_{yv}} \right)^2 - 1.79$$

and

$$m = 0.192 \frac{F_{uv}}{F_{yv}} - 0.068$$

$F_{yc}$ = Corner yield stress  
$F_{yv}$ = Virgin yield stress  
$F_{uv}$ = Virgin ultimate tensile strength  
$R$ = Inside bend radius  
$t$ = Sheet thickness

With regard to the full-section properties, the tensile yield stress of the full section may be approximated by using a weighted average as follows:

$$F_{ya} = CF_{yc} + (1 - C)F_{yf}$$  \hspace{1cm} (C-A3.3.2-2)

where

$F_{ya}$ = Full-section tensile yield stress  
$F_{yc}$ = Average tensile yield stress of corners $= B_c F_{yv}/(R/t)^m$  
$F_{yf}$ = Average tensile yield stress 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).

The limitation $F_{ya} \leq F_{uv}$ places an upper bound on the average yield stress. The intent of the upper bound is to limit stresses in flat elements that may not see significant increases in yield stress and tensile strength as compared to the virgin steel properties.
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 thick materials. They also considered residual stress distribution, simplification of design methods, and other related subjects. For details, see Yu and LaBoube (2010).

In 1962, the Specification permitted the utilization of cold work of forming on the basis of full section tests. Since 1968, the Specification has allowed the use of the increased average yield stress 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-A3.3.2-2. However, such a strength increase is limited only to relatively compact sections designed according to Specification Chapter D (tension members), Chapter F (bending strength excluding the use of inelastic reserve capacity), Chapter E (concentrically loaded compression members), Section H1 (combined axial load and bending), Section I4 (cold-formed steel light-frame construction), and Sections I6.1 and I6.2 (purlins, girts and other members). A design example in the Cold-Formed Steel Design Manual (AISI, 2013) demonstrates the use of strength increase from cold work of forming for a channel section to be used as a beam.

Prior to 2016, the requirements for applying the provisions of strength increase from cold work of forming were written for using the Effective Width Method. The requirements were revised in 2016 to make the provisions also applicable to the Direct Strength Method. The strength increase from cold work of forming is applicable to sections that are not subject to strength reduction from local buckling. This requires the cross-section to be fully effective when using the Effective Width Method, or $\lambda_{t} \leq 0.776$ in Specification Section E3.2 or F3.2 when using the Direct Strength Method.

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 $\phi$:

\[
(F_y)_m = 1.10F_y \\
(F_{ya})_m = 1.10F_{ya} \\
(F_u)_m = 1.10F_u \\
M_m = 1.10 \\
V_{fy} = V_M = 0.10 \\
V_{Fya} = V_M = 0.11 \\
V_{Fu} = V_M = 0.08
\]
\[
F_m = 1.00 \quad V_F = 0.05
\]

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 \( F_y \), \( F_{y_A} \) and \( F_u \) are, respectively, the specified minimum yield stress, the average yield stress 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 are representative properties of materials and cross-sections used in the industrial application of cold-formed steel structures.
B. DESIGN REQUIREMENTS

B1 General Provisions

This Specification provides design provisions for cold-formed steel members and structural assemblies. Specification Section B1 provides the essential design requirements: the design of members and their connections should be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2 Loads and Load Combinations

Loads and load combinations should be determined in accordance with applicable building code. In the absence of an applicable building code, ASCE/SEI 7, Minimum Design Loads for Buildings and Other Structures, should be followed in the United States and Mexico, and the National Building Code of Canada (NBCC) should be followed in Canada.

When steel decks are used for roof and floor composite construction, they should be designed to carry the concrete dead load, the steel dead load, and the construction live load. When the ASD or LRFD method is used, the construction loads and load combinations should be based on the sequential loading of concrete as specified in ANSI/SDI C-2011 (SDI, 2011) or in ANSI/SDI NC-2010 (SDI, 2010).

These loads and load combinations with proper care during construction provide safe construction practices for cold-formed steel decks and panels which otherwise may be damaged.

When the LSD method is used, the NBCC should be followed.

B3 Design Basis

As stated in Specification Section B3, design should be based on the principle that no applicable strength or serviceability limit state is exceeded when the structure is subjected to load effects corresponding to the applicable load combinations.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented (Winter, 1970; Peköz, 1986b; and Yu and LaBoube, 2010), and a continuing research effort provides further improvement in understanding them.

Three design methods are provided in the Specification for strength: Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD). Both Allowable Strength Design (ASD) and Load and Resistance Factor Design (LRFD) are applicable only in the United States and Mexico, while the Limit States Design (LSD) is applicable in Canada. ASD and LRFD are distinct methods. They are not identical and not interchangeable. Indiscriminate use of combinations of the ASD and LRFD methods could result in unpredictable performance or unsafe design. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflict, such as providing
modifications to a structural floor system of an older building after assessing the as-built conditions.

In the ASD, LRFD and LSD methods, two types of limit states are considered. 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 ASD, LRFD and LSD methods focus on the limit state of strength in Specification Sections B3.2.1, B3.2.2, and B3.2.3, respectively; and the limit state of serviceability in Specification Section B3.7.

B3.1 Required Strength [Effect Due to Factored Loads]

Generally, design is performed by elastic analysis. The required strength [effect due to factored loads] is determined by the appropriate methods of structural analysis. In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces, the required strength [effect due to factored loads] is explicitly stated in the Specification.

B3.2 Design for Strength

The Allowable Strength Design method has been featured in AISI Specifications beginning with the 1946 edition. It is included in the Specification along with the LRFD and the LSD methods for use in the United States, Mexico, and Canada since the 2001 edition.

B3.2.1 Allowable Strength Design (ASD) Requirements

In the Allowable Strength Design approach, the required 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 determined according to Specification Section B2. These required strengths are not to exceed the allowable strengths permitted by the Specification. According to Specification Section B3.2.1, the allowable strength is determined by dividing the nominal strength by a safety factor as follows:

\[
R \leq \frac{R_n}{\Omega}
\]  
(C-B3.2.1-1)

where

\[
R = \text{Required strength}
\]

\[
R_n = \text{Nominal strength}
\]

\[
\Omega = \text{Safety factor}
\]

The fundamental nature of the safety factor 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 safety factors are explicitly specified in various sections of the Specification. Through experience, it has been established that the present safety factors provide satisfactory design. It should be noted that the ASD method employs only one safety factor for a given condition regardless of the type of load. Serviceability is addressed in Specification Section B3.7.

B3.2.2 Load and Resistance Factor Design (LRFD) Requirements

For the limit state of strength, the general format of the LRFD method is expressed by the following equation:
\[ \sum_{i} \gamma_i Q_i \leq \phi R_n \]  
\[ \text{or} \]  
\[ R_u \leq \phi R_n \]

where

- \( R_u = \sum_{i} \gamma_i Q_i \) = 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 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

Safety factors 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-B3.2.2-1). 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-B3.2.2-2. 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-B3.2.2-2), since \( \beta \sigma_{\ln(R/Q)} = \ln(R/Q)_m \), from which approximately

\[ \beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \]  

(C-B3.2.2-2)
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 a simply supported beam, braced against distortional buckling and lateral-torsional buckling, subjected to dead and live loading and designed considering local buckling using the Effective Width Method.

The ASD design requirement of the Specification for such a beam is

\[
S_e F_y / \Omega = (L_s^2 s / 8)(D + L)
\]

where

- \( S_e \) = Elastic section modulus based on the effective section
- \( \Omega = 1.67 = Safety factor \) for bending

\( (C-B3.2.2-3) \)
F_y = Specified yield stress
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):
\[ R_m = R_n(P_mM_mF_m) \]  
(C-B3.2.2-4)
In the above equation, R_n is the nominal resistance, which in this case is
\[ R_n = S_eF_y \]  
(C-B3.2.2-5)
that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange and the web using the Effective Width Method. 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 stress 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
\[ V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \]  
(C-B3.2.2-6)
The values of these data were obtained from examining available tests prior to 1990 on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield stress 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 = 1.22R_n and V_R = 0.14.
The mean load effect is equal to
\[ Q_m = \left( L_s^2 s / 8 \right)(D_m + L_m) \]  
(C-B3.2.2-7)
and
\[ V_Q = \sqrt{(D_mV_D)^2 + (L_mV_L)^2} \]  
\[ /D_m + L_m \]  
(C-B3.2.2-8)
where D_m and L_m are the mean dead and live load intensities, respectively, and V_D and V_L are the corresponding coefficients of variation.
Load statistics have been analyzed in 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-B3.2.2-7 and C-B3.2.2-8 gives:
\[ Q_m = \frac{L_s^2 s (1.05D)}{8} (\frac{1}{L} + 1)L \]  
(C-B3.2.2-9)
\[ V_Q = \sqrt{(1.05D/L)^2 V_D^2 + V_L^2} \]
\[ (1.05D/L + 1) \]

\( Q_m \) and \( V_Q \) thus depend on the dead-to-live load ratio. Cold-formed steel beams typically have small \( D/L \) ratios, which may vary for different applications. Different \( D/L \) ratio may be assumed by different countries for developing design criteria. The impact of \( D/L \) ratio on the reliability is also provided in Meimand and Schafer (2014). 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-B3.2.2-3 and C-B3.2.2-5, the nominal resistance, \( R_n \), can be obtained for \( D/L = 1/5 \) and \( \Omega = 1.67 \) as follows:
\[ R_n = 2L(L_s^2 s/8) \]

In order to determine the reliability index, \( \beta \), from Equation C-B3.2.2-2, 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-B3.2.2-2,
\[ \beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79 \]

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 ANSI/AISC 360 for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI 318 Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized by Raviendra and Galambos (1978), where many other papers are also 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 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 reanalyzed 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 prior designs for all materials of construction. Ellingwood’s recommended target reliability indices, $\beta_o$, were for members with gravity loading: $\beta_o = 3.0$, for connections with gravity loading: $\beta_o = 4.5$, and for wind loading: $\beta_o = 2.5$. These target reliability indices are the ones inherent in the load factors first 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 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 $\beta_o = 2.5$ is recommended as a lower limit in the United States for members with gravity loads.

The resistance factors, $\phi$, were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual $\beta$’s for members supporting gravity loads. In order to ensure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners 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 the basis for the AISC LRFD Specification (AISC, 1999).

For wind loading, the same ASCE target reliability index of $\beta_o = 2.5$ is used for connections in the U.S. 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 reliability index, $\beta_o$, used in the United States is reduced to 1.5. With this reduced target reliability index, the design based on the U.S. LRFD method is comparable to the U.S. 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 D through J. These $\phi$ factors are determined in conformance with the ASCE/SEI 7 load factors to provide approximately a target reliability index $\beta_o$ 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:

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL$$

(C-B3.2.2-11)
where c is the deterministic influence coefficient translating load intensities to load effects.

By assuming D/L = 1/5, Equations C-B3.2.2-11 and C-B3.2.2-9 can be rewritten as follows:

\[ R_n = 1.84(\frac{cL}{\phi}) \]  \hspace{1cm} (C-B3.2.2-12)
\[ Q_m = (1.05D/L+1)cL = 1.21cL \]  \hspace{1cm} (C-B3.2.2-13)

Therefore,

\[ \frac{R_m}{Q_m} = \left( \frac{1.521}{\phi} \right) \frac{R_m}{R_n} \]  \hspace{1cm} (C-B3.2.2-14)

The \( \phi \) factor can be computed from Equation C-B3.2.2-15 on the basis of Equations C-B3.2.2-2, C-B3.2.2-4 and C-B3.2.2-14 (Hsiao, Yu and Galambos, 1988b, AISI 1996):

\[ \phi = 1.521 \left( P_m M_m F_m \right) \exp(-\beta_o \sqrt{V_R^2 + V_Q^2}) \]  \hspace{1cm} (C-B3.2.2-15)

in which \( \beta_o \) is the target reliability index. Other symbols were defined previously. For other load combinations and load ratios, corrected values for the 1.521 pre-factor (known as \( C_\phi \)) and \( V_Q \) are provided in Meimand and Schafer (2014).

By knowing the \( \phi \) factor, the corresponding safety factor, \( \Omega \), for Allowable Strength Design can be computed for the load combination 1.2D+1.6L as follows:

\[ \Omega = \frac{(1.2D/L + 1.6)}{[\phi(D/L + 1)]} \]  \hspace{1cm} (C-B3.2.2-16)

where D/L is the dead-to-live load ratio for the given condition.

### B3.2.3 Limit States Design (LSD) Requirements

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 (NBCC). Examples of serviceability requirements include deflections and the possibility of vibrations.

For the limit state of strength, the general format of the LSD method is expressed by the following equation:

\[ \phi R_n \geq \Sigma \gamma_i Q_i \]  \hspace{1cm} (C-B3.2.3-1)

or

\[ \phi R_n \geq R_f \]

where

\[ R_f = \Sigma \gamma_i Q_i = \text{Effect of factored loads} \]
\[ R_n = \text{Nominal resistance} \]
\[ \phi = \text{Resistance factor} \]
\[ \gamma_i = \text{Load factors} \]
\[ Q_i = \text{Load effects} \]
\[ \phi R_n = \text{Factored resistance} \]

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 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, as well as 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.

The NBCC 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.

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.

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.

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 Group 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 percent of cold-formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20 percent 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.

Since the design basis for the LSD and the LRFD is the same, further discussions on how to obtain a resistance factor using probability analysis can be obtained from Section B3.2.2(c) of the Commentary. However, attention should be paid to the fact 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 differences in resistance factors. The dead-to-live load ratio used in Canada is assumed to be 1:3 (or 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 Group design standards.

### B3.3 Design of Structural Members

For the design of cold-formed steel axial or flexural members, consideration should be given to several design features: (a) axial or bending strength and combined axial and bending, (b) shear strength of webs and combined bending and shear, (c) web crippling strength and combined bending and web crippling, (d) bracing requirements, and (e) serviceability. 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 Chapters D, E, F, G and H, and Sections I6.1, I6.2, and I6.3; while Item (d), the requirements for lateral and stability bracing, is given in Specification Sections C2 and I6.4; and Item (e) is covered in Chapter L. The treatments for flange curling and shear lag are discussed in Sections L3 and B4.3 of the Commentary, respectively.

Rational engineering analysis, such as the Direct Strength Method, is permitted to be used if the section geometry or material properties are outside the limitations given in Specification Section B4.

Example problems are given in Parts II and III of the AISI Cold-Formed Steel Design Manual (AISI, 2013) for the design of flexural and axial members.

B3.4 Design of Connections

Specification Section B3.4 provides the charging language for Chapter J on the design of connections. Chapter J covers the proportioning of the individual elements of a connection (welds, bolts, screws, and power-actuated fasteners, etc.) once the load effects on the connection are known. Section B3.4 establishes that the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

B3.5 Design for Stability

Design for stability needs to consider the stability of the structural system and also the stability of its individual members. Design provisions are provided in Specification Chapter C.

B3.6 Design of Structural Assemblies and Systems

Specification Section B3.6 provides charging language on the design of cold-formed steel assemblies and systems included in Specification Chapter I. Chapter I provides design provisions for cold-formed steel built-up members and metal roof and wall systems; and references design standards for diaphragm, light-frame construction, and rack systems.

B3.7 Design for Serviceability

Serviceability limit states are conditions under which a structure can no longer perform its intended functions. Safety and strength 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:

(a) 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.

(b) Excessive vibrations which may cause occupant discomfort or equipment malfunctions.

(c) 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 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 (2010a) and ATC (1999).

B3.8 Design for Ponding

Ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of accumulated water is dependent on the stiffness of the framing. Unbounded incremental deflections due to the incremental increase in retained water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses.

The Specification requires that design for ponding be considered if water is impounded on the roof. Camber and deflections due to loads acting concurrently with rain or snow meltwater loads can be considered in establishing the initial conditions.

Determination of ponding stability is typically done by structural analysis where the rain loads are increased commensurate with the incremental deflections of the framing system under the accumulated rainwater assuming the primary roof drains are blocked.

ANSI/AISC 360 Appendix 2 (AISC, 2010) can be used for considering ponding stability, except the effective section properties as defined in the Specification should be used. The effective section properties should be calculated based on the load cases and combinations consistent with the requirements of ANSI/AISC 360 Appendix 2 for checking the ponding stability and the specific circumstances of the roof configuration considered.

For Canada, Commentary H of the User's Guide - NBC 2010, Structural Commentary (Part 4 of Division B) (NBC, 2010) can be used to determine the stiffness when ponding instability will occur. When calculating the stiffness, the effective section properties as defined in the Specification must be used.

B3.9 Design for Fatigue

Section B3.9 provides the charging language for Chapter M on the design of fatigue for cold-formed steel structural members and connections. Fatigue may occur when the structure is subjected to cyclic or repetitive load, which results in repetitive tensile stresses in the connections and the members. Fatigue, however, does not need to be considered for seismic load or wind load due to either infrequent load cycle or infrequent high load magnitude that would cause fatigue.

B3.10 Design for Corrosion Effects

Steel members may deteriorate in some service environments. This deterioration may appear either as external corrosion, which would be visible upon inspection, or as undetected
changes that would reduce member strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design, or providing adequate protection (for example, coatings or cathodic protection) or planned maintenance programs or both so that such problems do not occur.

**B4 Dimensional Limits and Considerations**

The *Specification* permits two methods for the basic design of members in Chapters E through H, including the *Effective Width Method* and the *Direct Strength Method*. The *Specification* indicates no preference between the two methods as either provides consistent levels of reliability even though they may not result in numerically equal answers.

**B4.1 Limitations for Use of the Effective Width Method or Direct Strength Method**

In 2016, the applicability limitations of the *Effective Width Method* and the *Direct Strength Method* were merged into this section and simplified. To some extent, these limitations are arbitrary; however, the provided limitations give practical limits on the applicability of the design methods and reflect serviceability limitations, limitations borne from practice, and in some cases limitations of available testing or other verification methods.

The *Effective Width Method* limitations originate with the work of Winter (1970). The limits for stiffened elements in bending were updated in 1980 based on the studies conducted at the University of Missouri-Rolla in the 1970s (LaBoube and Yu, 1978a, 1978b, and 1982b; Hetrakul and Yu, 1978 and 1980; Nguyen and Yu, 1978a and 1978b) and aligned with the AISC Specification (AISC, 1989) at that time.

The *effective width* provisions of Appendix 1 provide no reductions for corners. For inside bend radius-to-thickness ratios (R/t) in excess of 10, this is shown to be unconservative based on the studies of Sarawit (2003), and Zeinoddini and Schafer (2010). For members with large radius-to-thickness, the *Direct Strength Method* may be employed, which is applicable for radius-to-thickness ratios R/t less than 20. Alternatively, the *Specification* specifically allows for rational engineering analysis. Using an equivalent centerline model to determine the *effective width* of the flats or appropriately reducing the plate buckling coefficient are examples of such rational engineering analyses.

In Zeinoddini and Schafer (2010), the following method is shown to provide a rational reduction for 10 < R/t ≤ 20. A reduced plate buckling coefficient, kR, is determined by applying reduction factors based on the R/t value at each edge of the element. For unstiffened elements, only one reduction factor is applied. The plate buckling coefficient, kR, which replaces k in Appendix 1, is determined as follows:

\[
k_R = k \cdot R_{R1} \cdot R_{R2}
\]

where

\[
k = \text{Plate buckling coefficient determined in accordance with Specification Appendix 1, as applicable}
\]

\[
R_{R1} = 1.08 - (R_1/t)/50 \leq 1.0
\]

\[
R_{R2} = 1.08 - (R_2/t)/50 \leq 1.0
\]

where

\[
R_1, R_2 = \text{Inside bend radius. See Figure C-B4.1-1}
\]

\[
t = \text{Thickness of element. See Figure C-B4.1-1}
\]

Engineers are reminded that when rational engineering analysis methods are employed,
such as presented here for r/t>10, the safety and resistance factors of Section A1.2 apply.

Prior to 2016, the Specification provided detailed dimensional limits for all cross-sections using the Direct Strength Method (DSM). This approach was simplified and made parallel to the Effective Width Method limitations in 2016. The limits employed in Table B4.1-1 are based on the limits of available testing and judgment. The reliability of the Direct Strength Method within these limitations is detailed in Schäfer (2008) and were based on testing of concentrically loaded, pin-ended cold-formed steel columns (Kwon and Hancock, 1992; Lau and Hancock, 1987; Loughlan, 1979; Miller and Peköz, 1994; Mulligan, 1983; Polyzois et al., 1993; Thomasson, 1978); laterally braced beams (Cohen, 1987; Ellifritt et al., 1997; LaBoube and Yu, 1978; Moreyara, 1993; Phung and Yu, 1978; Rogers, 1995; Schardt and Schrade, 1982; Schuster, 1992; Shan et al., 1994; Willis and Wallace, 1990) and laterally braced hats and decks (Acharya and Schuster, 1998; Bernard, 1993; Desmond, 1977; Höglund, 1980; König, 1978; Papazian et al., 1994). Application to complex lip stiffeners was verified in Schäfer et al. (2006) and application to inside bend radius-to-thickness ratio limits up to 20 was verified in Zeinoddini and Schäfer (2010). Application of the DSM to sections with multiple stiffeners in the web for bending is given in Pham and Hancock, 2013; with multiple stiffeners in the web for shear in Pham and Hancock, 2012a; and with a single large intermediate stiffener in the web for shear in Pham and Hancock, 2015.

B4.2 Members Falling Outside the Application Limits

In general, members that are outside the applicability limits of Section B4.1 default to the general criteria in Section A1.2; however, the Direct Strength Method provides a general approach to design that is often applicable outside of the provided limits. Recognizing this, the Specification provides specific guidance when the Direct Strength Method is applied outside of Table B4.1-1. For example, companies with proprietary sections may wish to perform their own testing and follow Section K2 of the Specification to justify the use of the Ω and φ factors for a particular cross-section in Specification Chapters D through I. When such testing is performed, the provisions of Specification Sections B4.2 provide some relief from the sample size correction factor, C_p, of Specification Section K2. Based on the existing data, the largest observed V_p for the categories within Specification Table B4.1-1 is 15 percent (AISI, 2006; Schäfer, 2008). Therefore, as long as the tested section, over at least three tests, exhibits a V_p < 15 percent, then the section is assumed to be similar to the much larger database of tested sections used to calibrate the Direct Strength Method and the correction for small sample sizes is not required, and, therefore, C_p is set to 1.0. If the φ generated from Specification Section K2 is higher than that of Chapters E and F, this is evidence that the section behaves as a section that satisfies Specification Table B4.1-1.
It is not anticipated that member testing is necessarily required for all relevant limit states: local, distortional and global buckling. An engineer may only require testing to reflect a single common condition for the member, with a minimum of three tests in that condition. However, beams and columns should be treated as separate entities. A manufacturer who cannot establish a common condition for a product may choose to perform testing in each of the limit states to ensure reliable performance in any condition. Engineering judgment is required. Note that for the purposes of this section, the test results in Specification Section K2 are replaced by test-to-predicted ratios. The prediction is that of the Direct Strength Method using the actual material and cross-sectional properties from the tests. The $P_m$ parameter, taken as equal to one in Specification Section K2, is taken instead as the mean of the test-to-predicted ratios, and $V_P$ is the accompanying coefficient of variation.

Users of the Direct Strength Method should be aware that beams within the limits of Table B4.1-1 with large flat width-to-thickness ratios in the compression flange will be conservatively predicted by the Direct Strength Method when compared to the Effective Width Method (Schafer and Peköz, 1998). However, the same beam with small longitudinal stiffeners in the compression flange will be well-predicted using the Direct Strength Method.

Alternatively, member geometries that are outside the limits of Specification Table B4.1-1 may still use provisions given in Chapters E and F, but with the increased $\Omega$ and reduced $\phi$ factors consistent with any rational engineering analysis method as prescribed in Section A1.2 of the Specification.

### B4.3 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 nonuniform stress distribution across the width of the flange, similar to that in stiffened compression elements (see Section 1.1 of the Commentary), though for entirely different reasons. The simplest way of accounting for this stress variation in design is to replace the nonuniformly stressed flange of actual width, $w_f$, by one of reduced, effective width subject to uniform stress (Winter, 1970).

![Figure C-B4.3-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams](image-url)

Figure C-B4.3-1 Analytical Curves for Determining Effective Width of Flange of Short Span Beams
Theoretical analyses by various investigators have arrived at results which differ numerically (Roark, 1965). The provisions of Specification Section B4.3 are based on the analysis and supporting experimental evidence obtained by detailed stress measurements on 11 beams (Winter, 1940). In fact, the values of effective widths in Specification Table B4.3-1 are taken directly from Curve A of Figure 4 of Winter (1940).

It will be noted that according to Specification Section B4.3, the use of a reduced width for stable, wide flanges is required only for concentrated load as shown in Figure C-B4.3-1. For uniform load, it is seen from Curve B of the figure that the width reduction due to shear lag for any unrealistically large span-width ratios is so small as to be practically negligible.

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 B4.3. For design purpose, see the example in the AISI Design Manual (AISI, 2013).

For beams designed by the Direct Strength Method, the shear lag check of Section B4.3 may be reasonably applied by assuming that the member strength (M_n/M_y) reduces proportional to the reduced flange effectiveness (b/w). For short spans under concentrated loads, web crippling (not shear lag) is typically the controlling limit state for members with an unstiffened web.

**B5 Member Properties**

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 full cross-section dimensions, effective widths, or net section, as applicable.

*Effective Width Method*

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 cross-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. Peköz (1986a and 1986b) discussed this concept in more detail.

Section 3 of Part I of the AISI Design Manual (AISI, 2013) deals with the calculation of cross-sectional properties for C-sections, Z-sections, angles, hat sections, and decks.

*Direct Strength Method*

The Direct Strength Method uses the gross or net cross-section properties in member design. It considers local buckling through the whole cross-section and takes the interaction of the elements into consideration.

**B6 Fabrication and Erection**

(Reserved)
B7 Quality Control and Quality Assurance

In this edition of the Specification, only the delivered minimum thickness is addressed under this section. Other quality control and quality assurance issues may be considered in future editions.

B7.1 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 a similar material thickness applicable to both 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 safety factor 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.

B8 Evaluation of Existing Structures

(Reserved)
C. DESIGN FOR STABILITY

C1 Design for System Stability

In previous editions of the Specification, concluding with its 2012 edition, the primary technique for considering system stability was the effective length method, mainly structured as first introduced in the 1961 AISC Specification (AISC, 1961). Characteristic for this approach was that the member strength calculation models employing an effective length factor, K, were relied upon for considering the effects of residual stresses and geometric imperfections. Consideration of various sources of deformation, such as those at connections and those resulting from member shear, were not previously prescribed and the manner in which they were captured was largely dependent on the standard practice of various constituent industries and the judgment of individual professionals. The structure of the Specification implied the usage of first-order elastic analysis of a geometrically undisturbed structure, where the second-order effects were crudely captured through the approximate amplifiers embedded in the interaction equations. In 2006, based on the study by Sarawit and Peköz (2006) and the similar methodology in ANSI/AISC 360-05 (AISC, 2005), Appendix 2 of the Specification (AISI, 2007a and 2012a) incorporating a notional load approach was added. Supplied as an alternative to the effective length method, the notional load approach required that the member and system second-order effects be considered directly through an elastic analysis capable of establishing equilibrium on a deformed structure. In this analysis, initial imperfections were captured through the application of notional forces while stiffnesses used in such an analysis were reduced to model the effect of section softening due to inelastic deformations, including residual stresses, and to account for the strength reduction factor applied to column strength. For further background on AISI S100-12 (AISI, 2010a), the user is referred to the Commentary to AISI S100-12 (AISI, 2012b).

Similar to ANSI/AISC 360-10 (AISC, 2010a), recognizing the interrelated roles of analysis and member proportioning in assessing and assuring the overall system stability, the Specification introduced the concept of “method of design.” Therein, the term “design” refers to the comprehensive process of determining the required and available member strength [effects due to factored loads and member factored resistance], thus incorporating analysis, definition of imperfections, identifying sources of deformation, and determining the member strength. As described above, it is possible to capture many such effects either through determination of required forces (analysis) or through determination of available strength [factored resistance] (member proportioning). It is, therefore, crucial that the processes of determining the required and available member strengths [effects of factored loads and factored resistance] within any particular method of design are compatible.

In 2016, the Specification was reorganized whereby the interaction equations were decoupled from the analysis requirements and specific effects affecting system stability. In addition, the Specification relaxed the requirement that the bending moment (M) should be defined with respect to the centroidal axis of the effective section. For ideally pin-ended beam-columns, when determining applied bending from a compressive force, eccentricity from the line of compressive action may be increased if the effective centroid (accounting for local buckling) is considered. However, for continuous members or members with end restraint or members with support restrained in a manner that reduces the neutral axis eccentricity between gross and effective sections, this phenomenon is minor as the line of action of the force moves with the buckling deformations due to the continuity of the structure, and calculation of the required
bending moment [moment due to factored loads] about the gross centroidal axes is appropriate.

The Specification permits the usage of any method of design capable of assessing the stability of both the system and each of its individual members, provided it considers items (a) through (f) from Specification Section C1. The Specification offers three such design methods, subject to the limitations stipulated within each of the methods. However, it is not the intention of the Specification to prefer any of the methods of design enclosed therein, including approaches incorporating inelastic analyses, or to prevent the usage of any methods of design not stated therein, provided such a method considers the above items.

The load-displacement response resulting from a second-order elastic analysis is nonlinear. For this reason, and to assure that consistent reliability can be achieved through deployment of LRFD, LSD, or ASD, all load-dependent effects must be determined using either LRFD or LSD load combinations or 1.6 times the ASD load combinations. Subsequently, if ASD is used in the design, such effect should be divided by 1.6 to arrive at required member forces. Consequently, application of ASD in this regard may be conservative in systems for which the live-to-dead load ratio is relatively low.

Unbraced length, as used in Specification Section C1, is considered to occur between distinct bracing points possessing adequate strength and stiffness to restrain their translation and/or rotation, as applicable. Methods of satisfying the bracing requirement are provided in Specification Section C2. The requirements of Specification Section C2 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall force-resisting system.

Stiffness modification requirements of Specification Section C1.1 and C1.2 are intended only for the strength and stability checks under factored load combinations, as prescribed in those sections. An analysis utilizing such stiffnesses may not be suitable for many displacement-related design considerations. Unreduced (nominal) stiffness is considered appropriate for considering serviceability, such as deflection, drift and vibrations, or for calculating many other stiffness-based properties or design checks, including period, seismic drift, and seismic stability factor.

**C1.1 Direct Analysis Method Using Rigorous Second-Order Elastic Analysis**

The provisions of this section are based on Sarawit (2003), Sarawit and Peköz (2006), and ANSI/AISC 360-10 (AISC, 2010a). This method of design effectively incorporates the notional load approach, previously included in Appendix 2 of AISI S100-12. The study by Sarawit and Peköz on industrial steel storage racks at Cornell University (Sarawit, 2003) was sponsored by the Rack Manufacturers Institute and the American Iron and Steel Institute. The subject of notional loads is discussed fully in the Commentary to Chapter C of ANSI/AISC 360-10 (2010a). The application of the direct analysis method to cold-formed steel structures has to consider the items (a) through (f) listed in Specification Section C1, including frequently encountered flexural-torsional buckling, semi-rigid joints and local instabilities. In Sarawit (2003), and Sarawit and Peköz (2006), it was shown that the direct analysis method gives more accurate results than the effective length method.

Required strengths [effects due to factored loads] are determined by analysis according to Specification Section C1.1.1 and the members have to satisfy the provisions of Section H1 of the Specification. The work by Sarawit and Peköz is based on a linear moment-axial interaction equation, as depicted in Specification Section H1.2. It is the position of the committee that such a model adequately captures the interaction of a wide variety of cold-
formed steel shapes subject to different forms of buckling, axes of bending and buckling modes for the design methods proposed herein, including the direct analysis method. Further background on interaction equations is provided in the commentary to Specification Section H1.2.

Since the frame stability is considered by the direct analysis method, nominal axial strength [resistance] in Specification Chapters D and E should be determined considering the flexural buckling effective length equal to the unbraced length (i.e., $K_x = K_y = 1.0$). It is important to recognize that the application of the direct analysis method does not alter the torsional effective length factor, $K_t$, which could be larger or smaller than 1.0, depending on the member boundary conditions. As an example, one can consider the case of a C-section cantilevered column with torsional and flexural fixity at the base. If designed using the direct analysis method, the calculations of available strength [factored resistance] would be based on $K_x = K_y = 1.0$ when computing flexural buckling stresses as prescribed by Chapter E. However, a $K_t = 2.0$ would be used in computing the flexural-torsional buckling stress.

Any type of second-order elastic analysis capable of establishing static equilibrium on the displaced structure is permitted. Two examples of such analyses are the stability functions approach and the geometric stiffness approach. The latter is typically implemented in commercially available software. It is required to carry out a second-order analysis that considers both the effect of loads acting on the deflected shape of a member between joints or nodes ($P-\delta$ effects) and the effect of loads acting on the displaced location of joints or nodes in a structure ($P-\Delta$ effects). On a member level, $P-\delta$ effects need to be modeled explicitly. One possible method of accomplishing this is to employ an elastic analysis capable of capturing only $P-\Delta$ effects whereby $P-\delta$ effects are accounted for by modeling individual columns as a series of short column segments separated by intermediate nodes. These intermediate nodes do not need to account for the initial out-of-straightness for the member. This is because for members, the design equations based on flexural buckling column curves include the presence of the initial imperfections along the member length.

As an alternative to an elastic method of analysis capable of capturing both $P-\Delta$ and $P-\delta$ effects, users are permitted to employ a mixed approach, whereby $P-\Delta$ effects are captured explicitly in the analysis with the results of such an analysis subsequently amplified by the coefficient $B_1$, as defined in Specification Section C1.2. This method of analysis is typical of commercially available analysis software commonly used in practice. Relatively small conservativism occurs in moment frame systems due to the application of $B_1$ to both sway and non-sway components of the calculated moment.

Second-order frame analysis within the direct analysis method of design is permitted either on the out-of-plumb geometry without notional loads or on the plumb geometry by applying notional loads or minimum lateral loads as defined in Specification Section C1.1.1.2. Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The imperfections required to be considered in this section are imperfections in the locations of points of intersection of members. In typical building structures, the important imperfection of this type is the out-of-plumbness of columns. Initial out-of-straightness of individual members is not addressed in this section; it is accounted for in the compression member design provisions of Chapter E and need not be considered explicitly in the analysis as long as it is within the limits prescribed by the standard practice governing the member fabrication. The magnitude of the initial displacements should be based on permissible construction
tolerances, such as those specified in the AISI S202, Code of Standard Practice for Cold-Formed Steel Structural Framing (AISI, 2011), AISC 303, Code of Standard Practice (AISC, 2010c), other governing requirements, as applicable, or on actual imperfections, if known. The notional loads can lead to additional (generally small) fictitious base shears in the structure. The correct horizontal reactions at the foundation may be obtained by applying an additional horizontal force at the base of the structure, equal and opposite in direction to the sum of all notional loads, distributed among vertical load-carrying elements in the same proportion as the gravity load supported by those elements. The notional loads can also lead to additional overturning effects, which are not fictitious. An out-of-plumbness of 1/240, based on the Rack Manufacturer’s Institute Specification, RMI MH16.1:2008 (RMI, 2008), is selected as appropriate or conservative for use in a wide variety of cold-formed steel structures. An out-of-plumbness of 1/500, representing the maximum tolerance on column plumbness, is specified in the AISC 303. The usage of values smaller than 1/240 is permitted provided such values are substantiated by the applicable quality assurance standard or project-specific requirements. Various codes, such as EN 1993-1 (ECS, 2005), provide criteria and methods of computing the initial imperfection as a function of the number of stories and the number of participating columns in the resistance plane. For most building structures, the requirement regarding notional load direction may be satisfied as follows: For load combinations that do not include lateral loading, consider two alternative orthogonal directions of notional load application, in a positive and a negative sense in each of the two directions, in the same direction at all levels; for load combinations that include lateral loading, apply all notional loads in the direction of the resultant of all lateral loads in the combination. The notional load concept is applicable to all types of structures, but the specific requirements (1) through (3) given in Specification C1.1.1.2(b) are applicable only for the particular class of structure identified therein.

If second-order elastic analysis is used, whereby the effects of inelasticity and uncertainty are not explicitly included in the analysis, all stiffnesses maintaining the stability of the system are to be reduced as specified in Specification Section C1.1.1.3. In the application of the direct analysis method in ANSI/AISC 360-10 (AISC, 2010a), this results in a stiffness reduction factor of 0.8 for slender columns, capable of resisting factored axial loads of up to 0.5P_y. The factor of 0.8 is equivalent to the margin of safety implied by a strength reduction factor 0.9, prescribed in the ANSI/AISC 360-10 Chapter E, multiplied by the elastic flexural buckling column curve adjustment coefficient of 0.877. In the development of the direct analysis method, as implemented in ANSI/AISC 360-10, a distributed plasticity analysis was used. It can be shown, using distributed plasticity analysis, that using a factored elastic modulus, E, and yield stress, F_y, in the analysis will yield the same P-M interaction curve as if nominal values of E and F_y were used in the analysis and subsequently the abscissa and the ordinate of the P-M interaction curve are factored (White et al., 2006). However, a ten percent (10%) reduction in member stiffnesses, EI, namely multiplying EI by 0.9, is substantiated by Sarawit and Peköz for the cold-formed steel members whose required axial force does not exceed 0.5P_y. Specifically, Sarawit and Peköz (2006) showed that for typical industrial storage rack frames with a wide variety of section properties, configurations, and behavior modes, a reduction of 10 percent in member stiffnesses results in an increased conservatism of 10 percent in the calculated load-carrying capacity. A 20 percent (20%) reduction in member stiffnesses would lead to an increased conservatism of 20 percent (20%) in the calculated load-carrying capacity. However, a parametric study of individual columns in Sarawit and Peköz (2006) shows that
some unconservative results can be obtained in a few instances if the stiffness of members is not reduced in the analysis. Reducing the stiffness by 10 percent gives satisfactory results for these cases.

The study by Sarawit and Peköz did not incorporate the members with sections subjected to the required axial force in excess of 0.5\(P_y\)). It is furthermore not expected that such sections would be commonly found in cold-formed steel framing applications. However, to address the full scope of the Specification, the committee’s position is that such occurrences could be adequately addressed by applying the ANSI/AISC 360-10 (2010a) stiffness modifier, \(\tau_b\), which in addition to 0.9, has the role of capturing additional stiffness softness characteristic for stockier columns with the axial force approaching \(P_y\). This conclusion is further supported by the study by Ziemian and Kissell (2010) on aluminum members which noted a limited impact of \(\tau_b\) even for fairly stocky columns. In addition, the study findings suggest the ability to use a higher value of the reduced stiffness given the usage of a linear interaction diagram compared to a multi-linear bulged-forward interaction diagram in ANSI/AISC 360-10 (2010a).

Initial imperfections, as considered in this design method when performing an analysis, refer to the imperfections at the points of member intersections (i.e., column out-of-plumbness). Column out-of-straightness, referring to the initial imperfection occurring between the points of member intersections, is in turn not considered in the analysis, but its influence is considered in the column strength curves when computing the available strength [factored resistance] per Chapter E. In certain cases, the user may experience difficulty in determining the effective member length (i.e., \(KL_x=L_x\) and \(KL_y=L_y\)) for use in computations of available strength per Chapter E. An example of such a difficulty would be the exercise of determining the effective length of a gable long-span portal frame rafter. In such a case, the user may directly model initial imperfections along the length of the member and in exchange determine the available strength [factored resistance] based on the strength of the member section (i.e, \(L_x\) and \(L_y\) of zero). The initial displacements should be considered in the direction in which the effective member length \(KL=L\) is taken as zero. Similarly, even when member length is less ambiguous, such as in the cases of typical floor columns with clearly defined points of member intersection, it is permitted to explicitly include the column out-of-straightness in the analysis, and in exchange determining the available axial strength [factored resistance] considering local and distortional buckling only. Consideration of torsional and flexural-torsional buckling would be unaffected by this option.

It should be noted that the nominal axial and flexural strengths [resistances] computed per Chapters D, E, F, G, H, I, K and M are not intended to be calculated using the reduced value of stiffness.

**C.1.2 Direct Analysis Method Using Amplified First-Order Elastic Analysis**

The design method presented in this section is identical to that offered in Specification Section C1.1, except that it is permitted to perform the design using an amplified first-order elastic analysis. With this approach, the non-sway and sway components of the member moments resulting from a first-order elastic analysis are amplified by factors \(B_1\) and \(B_2\), respectively. Additionally, sway moment amplification will translate into additional axial forces in the system columns and thereby amplification of the column forces resulting from sway effects by the factor \(B_2\) is required as well. The amplified first-order elastic analysis
method, as configured here, was first introduced in the 1986 AISC LRFD Specification (AISC, 1986), and has also been used historically in some form in other major design specifications, such as ACI 318-14 (ACI, 2014). Both $B_1$ and $B_2$ represent nonlinear algebraic convergence functions relating the member forces from the undeformed structure equilibrium to the forces in a displaced member and structure, respectively. Consequently, diverging values of these functions will indicate instability. Similarly, a large value of $B_2$ is associated with a stability critical system. The reader will recognize the mathematical similarity of the amplifier included in the P-M interaction equation in AISI S100-12 (AISI, 2012a) with $B_1$ and $B_2$. The advantage of $B_1$ and $B_2$ over the amplifier previously used in the Specification is their ability to distinguish between the non-sway (member) and sway (story) effects and consequently capture the appropriate level of amplification associated with each effect. Consequently, it avoids potentially grossly conservative or unconservative results resulting from the application of a single amplifier previously contained in the Specification interaction equations. As $B_1$ and $B_2$ are stiffness-based terms representing an integral part of the analysis process, their application within the framework of Direct Analysis Method of design must be associated with the same stiffness reductions as mandated by the method for use in the first-order elastic analysis, with the exception that $B_1$ must be computed with a reduced stiffness even for members not contributing to the system stability. The evaluation of $B_2$ as configured in these provisions is based upon the story drift approach, rather than column buckling analogy. As a result, this analysis can be employed with any method of design, without the need to determine the effective length factor associated with story sway. $F$ and $\Delta F$ in Specification Equation C1.2.1.1-7 may be based on any lateral loading that provides a representative value of story lateral stiffness, $F/\Delta F$. The derivation of $B_1$ and $B_2$ was presented in many references, including Chen and Lui (1991).

The user should maintain awareness of the fact that the result of a first-order member force amplification represents a real system effect in the form of additional member, connection, bracing, foundation and anchorage forces. As a result of logistical convenience within the commercially available design-analysis software packages, when amplified first-order analysis is used, $B_1$ and $B_2$ are typically applied at the member proportioning stage, thus excluding such forces from the result of the analysis. Consequently, care should be taken to incorporate the effect of the amplifiers in the proportioning of other elements of the system, such as those listed above.

Further background on this method of analysis is presented in the Commentary to the ANSI/AISC 360-10 (2010b).

**C1.3 Effective Length Method**

The design method presented in this Specification section represents the traditional method of design, first introduced in the 1961 AISC Specification (AISC, 1961). Recognizing its traditional association with the amplified first-order elastic analysis in the 1986, 1993 and 1999 AISC Specifications, it is presented in such a form herein, though it is not the intention of the committee to limit the usages of other methods of analysis compatible with the effective length method framework as long as the items (a) through (f) listed in Specification Section C1 are considered. Notwithstanding the different formulation of second-order effect amplifiers, the effective length method historically constituted the primary approach of design for stability up until and including the 2012 edition of the Specification (AISI, 2012a), and the sole such
approach before the 2007 edition of the Specification (AISI 2007a) which introduced the notional load approach (direct analysis method) in its Appendix 2.

Unlike the methods of design set forth in Specification Sections C1.1 and C1.2, the effective length method relies on the calculations of available strength [factored resistance] through the application of effective length (typically larger than the actual unbraced member length) and the empirical column curves, incorporating a modified elastic and inelastic buckling range, to capture the effects of geometric imperfections and loss of stiffness due to residual stresses, local yielding as the capacity is approached, as well as other effects. As a result of this, the analysis, performed using nominal stiffnesses, need only capture the P-∆ and P-δ effects. Also, given the application of the effective length factor in member proportioning, notional forces are not required to safely configure a column solely on the basis of the axial forces and in-plane bending. Unfortunately, the application of the effective length factor for flexural buckling does not impart the forces resulting from initial imperfections into beams, framing connections, stability braces, foundations and base anchorage, which is particularly critical in designs controlled by gravity load combinations. For this reason, the Specification stipulates the application of notional forces, as described in Specification Section C1.1 in conjunction with all gravity load combinations.

In the design, many systems can be classified as sway and non-sway. For the former, effective length factor, Kx or Ky, as applicable, will be larger than 1.0; and for the latter, Kx or Ky, as applicable, can typically be taken as 1.0 or less, depending upon specific boundary conditions.

The calculation of the effective length factor, K, for flexural buckling depends upon the axis of bending, frame configuration, boundary conditions, and the stiffness properties of the column and the members attached thereto. For further information on various methods of computing K, the user is referred to Chen and Lui (1991), AISC Specification (2010a) and ASCE Task Committee on Effective Length Method (1997).

When B2 exceeds 1.5, this method of design is not permitted. Specifically, research found that the method considerably underestimates the internal system forces when B2 exceeds 1.5, where B2 is evaluated on the basis of unreduced (nominal) stiffness (White et al., 2006).

**C2 Member Bracing**

The provisions of this section cover the design of torsion (also known as primary, first-order, or load-resisting) bracing in Section C2.2 and stability (also known as secondary, second-order, or deformation-resisting) bracing in Sections C2.1 and C2.3.

Torsion bracing develops forces even when equilibrium in the undeformed shape is considered. For example, bracing designed to resist twist in a C-section loaded in the plane of the web develops forces due to the location of the shear center not coinciding with the web, and is considered torsion, or first-order, bracing. Also, bracing designed to resist twist in a Z-section will develop forces when it is desired to have loading and response occur in a geometric axis that does not coincide with a cross-section principal axis, and is also considered torsion, or first-order, bracing. First-order or torsion braces, traditionally, are designed with strength criteria alone. The forces that develop to directly resist the first-order demands in torsion braces scale directly with the applied loads and can be significant. The relatively large magnitude of the brace forces and the fact that they may be predicted independently of brace stiffness makes their design criteria slightly simpler than stability bracing, as explained below. A First-
Analysis that includes cross-section torsion can provide a means to predict bracing forces for torsion braces.

Stability bracing, on the other hand, is used to prevent a member from buckling. Stability bracing receives forces only if equilibrium in the deformed (buckled) shape requires forces in the braces. For example, bracing designed to resist minor-axis flexural buckling in a C-section is considered a stability, or second-order, bracing. If stability braces are stiff enough, they only develop very small forces. As a result, stability braces are typically designed with both stiffness and strength criteria. A second-order analysis that includes the potential buckling deformations the brace is intended to restrict along with appropriate imperfections can provide a means to predict bracing forces for stability braces as detailed further in Section C2.3.

Cases where a brace may need to act as both a torsion and a stability brace are possible. In such cases, the strength demands for torsion braces generally exceed those for stability braces and traditionally have been employed without additional consideration for behavior as a stability brace. For other cases beyond the scope of the Specification, brace forces predicted from a proper second-order analysis that can capture both torsion and stability brace demands are recommended.

C2.1 Symmetrical Beams and Columns

There are no simple, generally accepted techniques for determining the required strength [effect due to factored loads] 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.

C2.2 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 C2.2 of the Specification includes the requirements for spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material. The bracing requirements for members having one flange connected to deck or sheathing materials are provided in Specification Section I6.4.1.

C2.2.1 Neither Flange Connected to Sheathing That Contributes to the Strength and Stability of the Section

(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-C2.2.1-1. Figure C-C2.2.1-2, 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-I1.1-2, 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 on the order of one to two degrees.

In order to obtain the information for developing bracing provisions, different C-section shapes were 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 work was given by Winter, Lansing and McCalley (1949b). It is indicated in the 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 should not be greater than one-quarter of the span and defined the conditions under which an additional brace should be placed at a load concentration.

For such braces to be effective, it is necessary that their spacing be appropriately
limited and their strength should suffice to provide the force required to prevent the C-section from rotating. It is also necessary to determine the forces that will act in braces, such as those forces shown in Figure C-C2.2.1-3. These forces are found if one considers that the action of a load applied in the plane of the web (which causes a torque Qm) is equivalent to that same load when applied at the shear center (where it causes no torque) plus two forces $P = Qm/d$ which, together, produce the same torque Qm. As is sketched in Figure C-C2.2.1-4 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 C2.2.1 provide expressions for determining bracing forces $P_{L1}$ and $P_{L2}$, which the braces are required to resist at each flange.

![Figure C-C2.2.1-3 Lateral Forces Applied to C-Section](image)

![Figure C-C2.2.1-4 Half of C-Section Treated as a Continuous Beam Loaded by Horizontal Forces](image)

(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 that a similar load would produce on a C-section. However, in Z-
sections the principal axes are oblique to the web (Figure C-C2.2.1-5). A load applied in the plane of the web, resolved in the direction of the two axes, produces deflections along 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 the 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 work 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 braced C-beams. It is merely necessary, at the point of each actual vertical load, to apply a fictitious horizontal load, or \( Q \frac{I_{xy}}{I_x} \) or \( Q \frac{I_{xy}}{2I_x} \), 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-C2.2.1-5), except that certain modified section properties have to be used. To control the lateral deflection, brace forces, \( P \), must statically balance the fictitious force.

![Figure C-C2.2.1-5 Principal Axis of Z-Section](image)

In this manner it has been shown that as to location of braces, the same provisions that 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 Figure C-C2.2.1-5, in order to constrain bending of the section about the axis x-x. The directions of the bracing forces in the C-beam flanges are in the opposite direction, as shown in Figure C-C2.2.1-3, in order to 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) Slope Effect and Eccentricity

For a C- or Z-section member subjected to an arbitrary load, bracing forces, \( P_{L1} \) and \( P_{L2} \), on flanges need to resist: (1) force component \( P_x \) that is perpendicular to the web, (2) the torsion caused by eccentricity about the shear center, and (3) for the Z-section member, the lateral movement caused by component \( P_y \) that is parallel to the web.

To develop a set of equations applicable to any loading conditions, the x and y axes are oriented such that one of the flanges is located in the quadrant with both x and y axes positive. Since the torsion should be calculated about the shear center, coordinates \( x_s \) and \( y_s \), that go through the shear center and parallel to x and y axes, are established. Load eccentricities \( e_x \) and \( e_y \) should be measured based on \( x_s \) and \( y_s \) coordinate system.

\[
\begin{align*}
\text{Figure C-C2.2.1-6 C-Section Member Subjected to a Concentrated Load}
\end{align*}
\]

For the C-section member as shown in Figure C-C2.2.1-6, the bracing forces on both flanges are given in Equations C-C2.2.1-1 and C-C2.2.1-2.

\[
\begin{align*}
P_{L1} &= -\frac{P_x + M_z}{2d} \quad \text{(C-C2.2.1-1)} \\
P_{L2} &= -\frac{P_x - M_z}{2d} \quad \text{(C-C2.2.1-2)} \\
M_z &= -P_x e_{sy} + P_y e_{sx} \quad \text{(C-C2.2.1-3)}
\end{align*}
\]

where \( d \) = Overall depth of the web; \( e_{sx} \), \( e_{sy} \) = Eccentricities of load about the shear center in \( x_s \)- and \( y_s \)-direction, respectively; \( P_x \), \( P_y \) = Components of load in x- and y-direction, respectively; \( M_z \) = Torsional moment about the shear center; and \( P_{L1} \) = Bracing force applied to the flange located in the quadrant with both positive x and y axes, and \( P_{L2} \) = Bracing force applied on the other flange. Positive \( P_{L1} \) and \( P_{L2} \) indicate that a restraint is required to prevent the movement of the corresponding flange in the negative x-direction.
For a special case where load, Q, is through the web, as shown in Figure C-C2.2.1-3, \( P_y = Q, \ P_x = 0; \ e_{sx} = m, \ e_{sy} = d/2 \), and from Equation C-C2.2.1-3, \( M_z = -Qm \). Therefore:

\[
P_{L1} = -\frac{Qm}{d} \quad (C-C2.2.1-4)
\]

\[
P_{L2} = \frac{Qm}{d} \quad (C-C2.2.1-5)
\]

In which, \( m \) = Distance from centerline of web to the shear center.

\[
\frac{d^3}{cosPb} \sin P)I^2 = \theta - \theta + \theta - =
\]

\[
\frac{d^3}{cosPb} \sin P)I^2 = \theta + \theta - =
\]

In considering the distribution of loads and the braces along the member length, it is required that the resistance at each brace location along the member length be greater than or equal to the design load within a distance of 0.5\( a \) on each side of the brace for distributed loads. For concentrated loads, the resistance at each brace location should be greater than or equal to the concentrated load within a distance 0.3\( a \) on each side of the brace, plus 1.4\((1-l/a)\) times each load located farther than 0.3\( a \) but not farther than 1.0\( a \) from the brace. In the above, “\( a \)” is the distance between centerline of braces along the member length and “\( l \)” is the distance from concentrated load to the brace to be...
considered.

In Specification Section C2.2.1, a top-bar is added to the variables designed as the design load, which are calculated in accordance with ASD, LRFD, or LSD load combinations depending on the design method used.

(d) 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 Sections F2 and F3 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 Sections F2 and F3.

Research (Ellifritt, Sputo and Haynes, 1992) has also shown that the lateral-torsional buckling equations of Specification Sections F2 and F3 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 C2.2.1 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 of an open cross-section member should be determined by Specification Sections F2 and F3 using the distance between centerlines 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 K2.1, as an alternative, or use a rigorous analysis, which accounts for biaxial bending and torsion.

Section C2.2.1 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.

In 2007, the title of this section was changed to clarify that it is and was formerly anticipated that the C- and Z-sections covered by these provisions would be supporting sheathing and be loaded as a result of providing this support function. The revised title reflects that the supported sheathing is not contributing to the strength and stiffness of these members by virtue of the nature of its connection to the C- and Z-sections.
C2.2.2 Flange Connected to Sheathing That Contributes to the Strength and Stability of the C- or Z-Section

This section of the Specification reminds users that stability should be considered in accordance with provisions of Section I6.4.1 or I6.4.2 for members with sheathing attached. See commentary for those sections for detail.

C2.3 Bracing of Axially Loaded Compression Members

In 2012, second-order analysis was introduced as a method for establishing the required strength [brace force due to factored loads] and stiffness for column bracing. The analysis includes consideration of the initial out-of-straightness of the compression member as well as the bracing member properties, connections, and anchoring details. Specific requirements are provided in Section C2.2.3.

Alternatively, the bracing can be designed using the provisions provided in Specification Section C2.3. The requirements for bracing a single compression member were developed from a study by Green, et al. (2004) and adaptation of requirements in the AISC Specification (AISC, 2010a). These bracing provisions ensure that an individual concentrically loaded compression member can develop the required compressive axial strength [compressive axial force due to factored loads]; however, they do not necessarily allow individual concentrically loaded compression members to develop their fully braced capacity at an effective length equal to the length between braces. The required bracing stiffness ensures that the translation at the brace point is limited until the axial loads equal the required strength [compressive axial force due to factored loads], \( P_{ra} \), which is determined in accordance with the applied load combinations for the corresponding design method of ASD, LRFD, or LSD. The engineer should recognize that a column braced to these provisions has an available strength [factored resistance] equal to the required strength [compressive axial force due to factored loads], but not in excess of the required strength [compressive axial force due to factored loads]. If the engineer desires the available column strength [factored resistance] to exceed \( P_{ra} \), then the required brace strength [brace force due to factored loads] designed for \( P_{ra} \) should be increased. If the engineer desires the available column strength [factored resistance] to equal the fully braced column strength, the required axial compressive strength [compressive axial force due to factored loads], \( P_{ra} \), in Specification Equations C2.3-1, C2.3-2a and C2.3-2b should be replaced by the fully braced column available strength [factored resistance], \( P_n/\Omega \) for ASD or \( \phi \) for LRFD or LSD.

The requirements for brace stiffness for a single compression member are similar to the AISC provisions, with the exception that the number of braces is accounted for by including the term \( 2(4-(2/n)) \). As a simplification, AISC assumes \( n = \infty \), but this simplification is considered too conservative for cold-formed steel structures. Analytical modeling by Sputo and Beery (2006) has shown that these provisions may be applied to members of varied cross-sections. The safety factor \( (\Omega = 2.0) \) and resistance factor \( (\phi = 0.75) \) for calculating required brace stiffness in Specification Equations C2.3-2a and C2.3-2b are the same as those used in the AISC provisions (AISC, 2010a).

The brace provisions for lateral translation assume that the braces are perpendicular to the compression member being braced and located in the plane of buckling. For inclined brace members, the required brace strength [brace force due to factored loads] and stiffness should be increased as follows:
\[ P_{rb}' = \frac{P_{rb}}{\cos \theta} \]  
(C-C2.3-1)

where
\[ P_{rb}' = \text{Required strength [brace force due to factored loads]} \] of the inclined brace
\[ \theta = \text{Angle of brace from perpendicular} \]

The required stiffness is
\[ \beta_{rb} = \frac{P_{rb}}{\Delta} \]  
(C-C2.3-2)

And the required stiffness of the inclined brace, \( \beta_{rb}' \), is
\[ \beta_{rb}' = \frac{P_{rb}'}{\Delta'} \]  
(C-C2.3-3)

\[ \Delta' = \Delta \cos \theta \]  
(C-C2.3-4)

where
\[ \Delta = \text{Lateral movement of brace point} \]
\[ \Delta' = \text{Deformation of inclined brace} \]

Substituting Equations C-C2.3-1, C-C2.3-2, and C-C2.3-4 into Equation C-C2.3-3,
\[ \beta_{rb}' = \frac{\beta_{rb}}{\cos^2 \theta} \]  
(C-C2.3-5)

The stiffness requirements include the contributions of the bracing members, connections, and anchorage details.

Additional bracing or additional brace strength and stiffness may be required to brace members that may also be subject to bending, torsion, or torsional-flexural stresses. Bracing for these effects are not accounted for in Section C2.3 and should be determined through rational analysis or other methods.

Once the required brace strength [brace force due to factored loads] and required stiffness are determined in accordance with Specification Equations C2.3-1 and C2.3-2, the brace member should then be designed in accordance with Specification Section B3.2.1, B3.2.2, or B3.2.3, as appropriate, and with the safety and resistance factors determined in accordance with the applicable Specification section.
D. MEMBERS IN TENSION

In 2010, the provisions for tension members were consolidated and moved from the country-specific appendices to the main Specification. The available tensile strength [factored resistance] of axially loaded cold-formed steel tension members is determined either by yielding of the gross area of the cross-section or by rupture of the net area of the cross-section. At locations of connections, the nominal tensile strength [resistance] is also limited by the available strengths [factored resistances] specified in Specification Chapter J for tension in connected parts.

D2 Yielding of Gross Section

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 is well established in hot-rolled steel construction.

The resistance factor $\phi_t = 0.90$ and safety factor $\Omega_t = 1.67$ used for yielding of the gross section are consistent with the factors used in ANSI/AISC 360 Specification (AISC, 2010a) and CSA S16 Specification (CSA, 2009).

D3 Rupture of Net Section

The resistance factor of $\phi_t = 0.75$ and safety factor of $\Omega_t = 2.00$ used for rupture of the net section are consistent with the factors used in the ANSI/AISC 360 Specification (AISC, 2010a) and CSA S16 Specification (CSA, 2009).
E. MEMBERS IN COMPRESSION

E1 General Requirements

Cold-formed steel column members should be designed considering yielding and global (flexural, flexural-torsional and torsional) buckling in accordance with Specification Section E2; local buckling with yielding and global buckling in accordance with Specification Section E3; and distortional buckling in accordance with Specification Section E4; as applicable. Design tables and example problems may be found in Parts I and III of the AISI Cold-Formed Steel Design Manual (AISI, 2013).

Two approaches can be used in column design: the Effective Width Method (EWM) and the Direct Strength Method (DSM). The EWM traditionally addressed local and global buckling. In 2004, the distortional buckling strength prediction using DSM was adopted as an alternative method.

The calibration of the EWM has been reported in the Commentary on the 1991 edition of the AISI Specification. The brief discussion of the DSM is provided herein. In considering column yielding and global buckling, the DSM is essentially the same as the EWM. However, the approach of the two methods in predicting the strength due to local buckling is different. The DSM strength curves for local and distortional buckling of a fully braced column are presented in Figure C-E1-1. The curves are presented as a function of slenderness, which in this case refers to slenderness in the local or distortional mode, as opposed to traditional long column slenderness. Inelastic and post-buckling regimes are observed for both local and distortional buckling modes. The magnitude of the post-buckling reserve for the distortional buckling mode is less than the local buckling mode, as may be observed by the location of the strength curves in relation to the critical elastic buckling curve.

The development and calibration of the DSM provisions for columns are reported in Schafer...
The reliability of the column provisions was determined using the test data of Section B4.2 and the provisions of Section K2 of the Specification. Based on a target reliability, $\beta$, of 2.5, a resistance factor, $\phi$, of 0.84 was calculated for all the investigated columns. Based on this information, the safety and resistance factors of Chapter E were determined for the prequalified members. For the United States and Mexico, $\phi = 0.85$ was selected; while for Canada, $\phi = 0.80$, since a slightly higher reliability, $\beta$, of 3.0 is employed. The safety factor, $\Omega$, was back-calculated from $\phi$ at an assumed dead-to-live load ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the DSM to geometries outside the prequalified set is allowed. Given the uncertain nature of this extension, increased safety factors and reduced resistance factors are applied in that case, per the rational engineering analysis provisions of Section A1.2(c) of the Specification.

The provisions of Chapter E are summarized in Figure C-E1-2. The controlling strength is either by Specification Section E3, which considers local buckling interaction with long column buckling, or by Section E4, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-E1-2. Scatter exists throughout the data set, but the trends in strength are clearly shown, and further, the scatter (variance) is similar to that of the EWM.

The development and calibration of the DSM provisions for columns with holes was performed with experimental and simulation databases as reported in Moen and Schafer (2009a) and summarized in Moen and Schafer (2011). Note that both databases contain only lipped Cee cross-sections with discrete web holes because this is what was available in the research literature at the time. However, the philosophy of employing elastic buckling parameters ($P_{cr}^l$, $P_{crd}$, $P_{cre}$) to predict the ultimate strength of cold-formed steel columns with holes was thoroughly validated in Moen and Schafer (2009a), and is assumed to hold true for other cross-section shapes.
The generality of the DSM approach for holes was demonstrated across experiments and nonlinear finite element analysis collapse simulations across a wide range of spacing, shape, and size of holes for both cold-formed steel columns and beams. Based on a target reliability, $\beta$, of 2.5, the resistance factor, $\phi$, was calculated as 0.94 (experiments) and 0.89 (simulations) for columns with holes predicted to fail from local-global buckling interaction. For columns with holes predicted to experience a distortional buckling failure mode, $\phi$ was calculated as 0.96 (experiments) and 0.91 (simulations). The prediction accuracy for DSM for members with holes is greater than that for members without holes (Ganesan and Moen, 2012).

### E2 Yielding and Global (Flexural, Flexural-Torsional and Torsional) Buckling

In this section, the limit states of yielding and overall column buckling are discussed.

#### 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-E2-1:

$$P_y = A_g F_y$$  \hspace{1cm} (C-E2-1)

where $A_g$ is the gross area of the column and $F_y$ is the yield stress 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:

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2}$$  \hspace{1cm} (C-E2-2)

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

$$F_{cr} = \frac{(P_{cr})_e}{A_g} = \frac{\pi^2 E}{(KL/r)^2}$$  \hspace{1cm} (C-E2-3)

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-E2-3 exceeds the proportional limit, $F_{p_{cr}}$, the column will buckle in the inelastic range. Prior to 1996, the following equation was used in the Specification for computing the inelastic column buckling stress:

$$F_{cr} = F_y \left(1 - \frac{F_y}{4(P_{cr})_e} \right)$$  \hspace{1cm} (C-E2-4)
It should be noted that because Equation C-E2-4 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-E2-4 can be rewritten as follows:

$$(F_{cr})_e = \left(1 - \frac{\lambda_c^2}{4}\right)F_y$$

(C-E2-5)

where

$$\lambda_c = \frac{KL}{r\pi} \sqrt{\frac{F_y}{(F_{cr})_e}}$$

(C-E2-6)

Accordingly, Equation C-E2-5 is applicable only for $\lambda_c \leq \sqrt{2}$.

(c) Nominal Axial Strength [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 stress of steel. Therefore, the nominal axial strength [resistance] can be determined by the following equation:

$$P_n = A_g F_{cr}$$

(C-E2-7)

where

$P_n$ = Nominal axial strength [resistance]

$A_g$ = Gross area of cross-section

$F_{cr}$ = Column buckling stress

In the 1986 edition of the Specification, the nominal axial strength [resistance] for C- and Z-sections and single angle sections was limited by Equation C-E2-8, which is determined by the local buckling stress of the unstiffened element and the area of the full cross-section:

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

(C-E2-8)

This equation was deleted in the 1996 edition of the Specification based on a study conducted by Rasmussen at the University of Sydney (Rasmussen, 1994) and validated by Rasmussen and Hancock (1992).

In the 1996 Specification, the design equations for calculating the inelastic and elastic flexural buckling stresses were changed to those used in the AISC LRFD Specification (AISC, 1993). As given in Specification Section E2, these design equations are as follows:

For $\lambda_c \leq 1.5$:  $F_n = (0.658 \lambda_c^2) F_y$

(C-E2-9)

For $\lambda_c > 1.5$:  $F_n = \left(\frac{0.877}{\lambda_c^2}\right) F_y$

(C-E2-10)

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-E2-3. Consequently, the equation for determining the nominal axial strength [resistance] can be written as:
This is Equation E2-1 of the Specification.

The reasons for changing the design equations from Equation C-E2-5 to Equation C-E2-9 for inelastic buckling stress and from Equation C-E2-4 to Equation C-E2-10 for elastic buckling stress are:

\[
P_n = A_g F_n \quad \text{(C-E2-11)}
\]
The revised column design equations (Equations C-E2-9 and C-E2-10) are based on a different basic strength model and were shown to be more accurate by Peköz 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 flexural-torsional buckling.

Because the revised column design equations represent the maximum strength with due consideration given to initial crookedness and can provide a better fit to test results, the required safety factor can be reduced. In addition, the revised equations enable the use of a single safety factor for all $\lambda_c$ values even though the nominal axial strength [resistance] of columns decreases as the slenderness increases because of initial out-of-straightness. By using the selected safety factor 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.


Figure C-E2-1 shows a comparison of the critical flexural buckling stresses used in the 1986, 1991, 1996 and 2001 Specifications. No changes were made on critical flexural buckling stresses between the 2001 edition and the 2016 edition. Because of the use of a relatively smaller safety factor in the 1996 Specification (as well as in the Specifications to the 2016 edition), it can be seen from Figure C-E2-2 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 percent. For the LRFD method, the differences between the nominal axial strengths [resistances] used for the 1991,
1996, and the 2001 LRFD design provisions are shown in Figure C-E2-3. The curve for the LSD provisions would be the same as the curve for LRFD.

(d) 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-E2-4); 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-E2-1 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 stress, 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, research (Harper, LaBoube and Yu, 1995) has shown that the K values may be taken as 0.75 (AISI, 1995).

<table>
<thead>
<tr>
<th>Table C-E2-1</th>
<th>Effective Length Factors K for Concentrically Loaded Compression Members</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(a)</td>
</tr>
<tr>
<td>Theoretical K value</td>
<td>0.5</td>
</tr>
<tr>
<td>Recommended K value when lateral conditions are approximated</td>
<td>0.65</td>
</tr>
<tr>
<td>End condition code</td>
<td>Rotation fixed, Translation fixed</td>
</tr>
</tbody>
</table>

On the other hand, when no lateral bracing against sidesway is present, such as in the portal frame of Figure C-E2-5, 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 were 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-E2-6 (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-E2-6 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, 2005). 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):
The above equation is the same as Specification Equation E2.2-5, 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 $K_t L_t$ is the effective length for twisting.

For inelastic buckling, the critical torsional buckling stress can also be calculated according to Equation C-E2-9 by using $\sigma_t$ as $F_e$ in the calculation of $\lambda_c$.

D. Flexural-Torsional 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 flexural-torsional 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, flexural-torsional buckling is one of the possible buckling modes as shown in Figure C-E2-7. Non-symmetric sections will always buckle in the flexural-torsional mode.

It should be emphasized that one needs to design for flexural-torsional 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, flexural-torsional 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 flexural-torsional buckling so that this mode needs to be checked only for the individual component channels between batten connections (Winter, 1970).

The governing elastic flexural-torsional buckling load of a column can be found from the following equation (Chajes and Winter, 1965; Chajes, Fang and Winter, 1966; Yu and LaBoube, 2010):

\[
P_n = \frac{1}{2\beta} \left( (P_X + P_Z) - \sqrt{(P_X + P_Z)^2 - 4\beta P_X P_Z} \right)
\]  

(C-E2-13)

If both sides of this equation are divided by the cross-sectional area \(A\), one obtains the equation for the elastic flexural-torsional buckling stress \(F_{cre}\) as follows:

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

(C-E2-14)

For this equation, as in all provisions which deal with flexural-torsional 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-E2-12); and \(\beta = 1-(x_o/r_o)^2\). It is worth noting that the flexural-torsional 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 flexural-torsional buckling stress can also be calculated by using Equation C-E2-9.

An inspection of Equation C-E2-14 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 flexural-torsional buckling stress cannot be made as simple as that for flexural buckling. Formulas for typical C-sections, Z-sections, angle and hat sections are provided in Part I of the AISI Design Manual (AISI, 2013).

E. Additional Design Consideration for Angles

During the development of a unified approach to the design of cold-formed steel members, Peköz 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 Peköz 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 E3 and H1.2 of the Specification. A study conducted at the University of Sydney (Popovic, Hancock, and Rasmussen, 1999) indicated that for the design of singly-symmetric unstiffened angle 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 subjected to local buckling at stress \(F_y\). Consequently, clarification was made in the 2001 edition of the AISI Specification, and is retained in Section H1.2 of this
Equations E2-1 to E2-3 have been shown to be conservative in predicting the experimental failure loads obtained from tests of concentrically loaded pin-ended and fixed-ended angle columns. Tested columns exhibit end supports fixed with respect to warping and major-axis flexure, but pinned or fixed with respect to minor-axis flexure. Tests were performed by Popovic, et al. (1999) and Chodraui, et al. (2006) for columns with minor-axis pin-ends, and by Popovic, et al. (1999) and Young (2004, 2005) for columns with fixed-ends. The above underestimation is essentially due to the fact that Equations E2-1 to E2-3: (1) account twice for the local/flexural-torsional effects (Rasmussen, 2005), and (2) disregard the beneficial effect of the warping fixity (Shifferaw and Schafer, 2014). Dinis et al. (2012) and Mesacasa, et al. (2014) investigated the mechanics of these phenomena and showed that the collapse of intermediate plain angle columns is governed by the interaction between major-axis flexural-torsional buckling and minor-axis flexural buckling. Due to effective centroid shift effects (Young and Rasmussen, 1999), this interaction is much stronger in pin-ended columns. Several design methods/approaches have been proposed to estimate more accurately the angle column failure loads, thus accounting for the increased strength due to the warping fixity (e.g., Young, 2004; Rasmussen, 2005; Silvestre, et al., 2013; Shifferaw and Schafer, 2014; and Dinis and Camotim, 2015). Using flexural-torsional strength curves (instead of the local buckling strength curves), the research finding of angle end-fixity is valid for columns with pin-ends and fixed-ends, and provides reliable prediction of column failure loads.

F. Slenderness Ratios

The slenderness ratio, KL/r, of all compression members should preferably 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 concerns. The AISI Specification provisions adequately predict the capacities of slender columns and beam-columns, but the resulting strengths 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.

E2.1 Sections Not Subject to Torsional or Flexural-Torsional 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 without considering local buckling should be determined by using Equation E2-1 of the Specification. The elastic flexural buckling stress is given in Equation E2.1-1 of the
Specification, which is the same as Equation C-E2-3 of the Commentary. This provision is applicable to doubly-symmetric sections, closed cross-sections and any other sections not subject to torsional or flexural-torsional buckling.

**E2.1.1 Closed-Box Section**

For the determination of the compression strength of members of Grade 80 (550) Class 3 steels produced to ASTM A653/A653M and A792/792M, compression tests of steel produced to Australian Standard AS1397 G550 (which is similar to ASTM A792 Grade 80 (550) Class 3) were performed at the University of Sydney by Yang and Hancock (2004a, 2004b), and Yang, Hancock and Rasmussen (2004). For short-box sections where $F_n = F_Y$, the study (Yang and Hancock, 2004a) shows that the limit of the yield stress used in design can be 90 percent of the specified minimum yield stress, $F_{Sy}$, for low-ductility steels. For edge-stiffened elements with intermediate stiffener(s), stub compression testing on channel sections (Yang and Hancock, 2004b) confirms the provisions given in Specification Section 1.4.2. For long column tests of channel sections (Yang and Hancock, 2004b), distortional buckling as well as the interaction of local and distortional buckling controls the design. The use of $0.9 F_{Sy}$ in the distortional buckling equations produces reliable results.

Further, for calculating the nominal strength [resistance] of concentrically loaded compression members with a closed-box section, Specification Equations E2.1.1-1 and E2.1.1-2, based on the University of Sydney research findings (Yang, Hancock and Rasmussen, 2002), were added in the Specification Section E2.1.1 when determining the nominal axial strength [resistance] according to Sections E2 and E3. The reduction factor $R_r$ specified in Equation E2.1.1-2 is to be applied to the radius of gyration $r$ and allows for the interaction of local and flexural (Euler) buckling of thin high-strength low-ductility steel sections. The reduction factor is a function of the length varying from 0.65 at $KL = 0$ to 1.0 at $KL = 1.1L_0$, where $L_0$ is the length at which the local buckling stress equals the flexural buckling stress.

**E2.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling**

As discussed previously in Section E2, torsional buckling is one of the possible buckling modes for doubly- and point-symmetric sections. For singly-symmetric sections, flexural-torsional 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-E2-12. For flexural-torsional buckling, Equation C-E2-14 can be used to compute the elastic buckling stress. The following simplified equation for elastic flexural-torsional buckling stress is an alternative permitted by the AISI Specification:

$$ F_{cre} = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} $$  \hspace{1cm} (C-E2.2-1)

The above equation is based on the following interaction relationship given by Peköz and Winter (1969a):

$$ \frac{1}{P_n} = \frac{1}{P_x} + \frac{1}{P_z} $$  \hspace{1cm} (C-E2.2-2)

or
E2.3 Point-Symmetric Sections

This section of the Specification is for the design of discretely braced point-symmetric sections 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 E2.2-5 or the elastic flexural buckling stress about its minor principal axis, as defined in Specification Equation E2.1-1. Figure C-C2.2.1-5 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.

E2.4 Non-Symmetric Sections

For non-symmetric open shapes, the analysis for flexural-torsional 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 V of the AISI Design Manual (AISI, 2013) and the book by Yu and LaBoube (2010). Section E2.4 of the Specification states that calculations according to Appendix 2 should be used or tests according to Section K2 should be made when dealing with non-symmetric open shapes.

E2.5 Sections With Holes

The global buckling load, \( P_{cre} \), for columns decreases when holes are present (Sarawit, 2003; Moen and Schafer, 2009a). A “weighted average” approach is provided in Appendix 2, which modifies the section properties due to the existence of holes. Appendix 2 also provides rational elastic buckling analysis methods that can be used to determine the elastic buckling loads with the influence of the holes.

Within the limitations of the hole size given in Appendix 1.1.1, the hole influence on the global buckling stress is negligible when using the Effective Width Method; therefore, an exception is provided to exclude these cases from the additional requirements of Appendix 2.

E3 Local Buckling Interacting With Yielding and Global Buckling

The discussion in Section E2 refers to members subject to global (flexural, flexural-torsional and torsional) 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 global buckling. For this case, the effect of local buckling on the global buckling strength can be handled by using the Effective Width Method, which applies the effective area, \( A_e \), determined at the stress \( F_n \); or the Direct Strength Method, which takes into consideration the local and global buckling interaction in the strength predication equations.

The Effective Width Method’s approach to local buckling is to conceptualize the member as a collection of “elements” and investigate local buckling of each element separately.

The Direct Strength Method provides a means to incorporate all relevant global buckling modes into the design process. Further, all buckling modes are determined for the member as a
whole rather than element by element. This ensures that compatibility and equilibrium are maintained at element junctures. Consider, as an example, the lipped C-section shown in pure compression in Figure C-2.2.2-2(a). The member’s local elastic buckling load from the analysis is:

\[ P_{cr} = 0.12 \times 48.42 \text{kips} = 5.81 \text{kips} (25.84 \text{kN}) \]

The column has a gross area \((A_g)\) of 0.881 in\(^2\) (568.4 mm\(^2\)); therefore,

\[ f_{crit} = P_{cr}/A_g = 6.59 \text{ksi (45.44 MPa)} \]

The Effective Width Method determines a plate buckling coefficient, \(k\), for each element, then \(f_{crit}\), and finally the effective width. The centerline dimensions (ignoring corner radii) are \(h = 8.94\) in. (227.1 mm), \(b = 2.44\) in. (62.00 mm), \(d = 0.744\) in. (18.88 mm), and \(t = 0.059\) in. (1.499 mm), the critical buckling stress, \(f_{crit}\), of each element as determined from the Appendix 1 of the Specification:

- **lip**: \(k = 0.43, \quad f_{crit-lip} = 0.43[\pi^2E/(12(1-\mu^2))](t/d)^2 = 72.1 \text{ksi (497 MPa)}\)
- **flange**: \(k = 4, \quad f_{crit-flange} = 4.0[\pi^2E/(12(1-\mu^2))](t/b)^2 = 62.4 \text{ksi (430 MPa)}\)
- **web**: \(k = 4, \quad f_{crit-web} = 4.0[\pi^2E/(12(1-\mu^2))](t/h)^2 = 4.6 \text{ksi (32.0 MPa)}\)

Each element predicts a different buckling stress, even though the member is a connected group. These differences in the buckling stress are ignored in the Effective Width Method. The high flange and lip buckling stresses have little relevance given the low web buckling stress. The finite strip analysis, which includes the interaction amongst the elements, shows that the flange aids the web significantly in local buckling, increasing the web buckling stress from 4.6 ksi (32.0 MPa) to 6.59 ksi (45.4 MPa), but the buckling stress in the flange and lip are much reduced due to the same interaction.

The Direct Strength Method is a robust method, but the Effective Width Method, which has been used by design engineers for over two decades, also provides a comprehensive and reliable design solution.

### E3.1 Effective Width Method

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 [resistance] determined by Equation C-E2-7. The interaction effect of the local and overall column buckling may result in a reduction of the overall column strength. 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 and LaBoube, 2010). 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 has shown that this method can be improved. On the basis of the test results and analytical studies of DeWolf, Peköz, Winter, and Mulligan (DeWolf, Peköz and Winter, 1974; Mulligan and Peköz, 1984) and Peköz’s development of a unified approach for the design of cold-formed steel members (Peköz, 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 [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 AISI S902, *Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns* (AISI, 2013c). For a more in-depth discussion of the background for these provisions, see Peköz (1986b). Therefore, the nominal axial strength [resistance] of cold-formed steel compression members can be determined by the following equation:

\[ P_n = A_e F_{cr} \]  

(C-E3-1)

where \( F_{cr} \) is either elastic buckling stress or inelastic buckling stress, whichever is applicable, and \( A_e \) is the effective area at \( F_{cr} \).

In the *Effective Width Method*, column nominal strength [resistance] is calculated by multiplying the nominal column buckling stress, \( F_n \), by the effective area, \( A_e \), calculated at \( F_n \). This accounts for *local buckling* reductions in the actual column strength (i.e., local-global interaction).

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 stress, do not fail in a flexural-torsional mode and can be designed based on flexural buckling alone as specified in Specification Section E2.1. There is also no need to include a load eccentricity for these sections when using Specification Section H1.2 as explained in Item E of Section E2.

**E3.1.1 Members Without Holes**

**E3.1.1.1 Closed Cylindrical Tubular Sections**

Closed thin-walled cylindrical tubular members are economical 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 classic theory of *local buckling* of longitudinally compressed cylinders overestimates the actual *buckling* strength, and that inevitable imperfections and residual stresses reduce the actual strength 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 post-buckling 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 \( F_{ult} \) is the ultimate stress or collapse stress. As shown in Figure C-E3.1.1.1-1, 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 stress, and Line 3 represents the collapse stress occurring at yield stress. In the range of Line 3, *local buckling* will not occur before yielding. In Ranges 1 and 2, *local buckling* occurs before the yield stress is reached. The cylindrical tubes should be designed to safeguard against *local
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 A_1, 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,

\[
\left( \frac{E}{F_y} \right) \left( \frac{t}{D} \right) = 2.27, \quad \frac{F_{ult}}{F_y} = 0.75
\]  

(C-E3.1.1.1-1)

Using line A_1B_1, the maximum stress of cylindrical tubes can be represented by

\[
\frac{F_{ult}}{F_y} = 0.037 \left( \frac{E}{F_y} \right) \left( \frac{t}{D} \right) + 0.667
\]  

(C-E3.1.1.1-2)

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-E3.1.1.1-3)

The correlations between the available test data and Equations C-E3.1.1.1-2 and C-E3.1.1.1-3 are shown in Figure C-E3.1.1.1-2. 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.
As indicated in *Commentary* Section F2.3, *Specification* Section E3.1.1.1 is only applicable to members having a ratio of outside diameter-to-wall thickness, D/t, not greater than 0.441E/Fy 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.

![Figure C-E3.1.1.1-2](image)

**Figure C-E3.1.1.1-2 Correlation Between Test Data and AISI Criteria for Local Buckling of Cylindrical Tubes Under Axial Compression**

When closed cylindrical tubes are used as concentrically loaded compression members, the nominal axial strength [resistance] is determined by the same equation as given in *Specification* Section E2, except that: (1) the nominal buckling stress, Fe, is determined only for flexural buckling, and (2) the effective area, Ae, is calculated by Equation C-E3.1.1.1-4:

\[
A_e = [1 - (1 - R^2)(1 - A_o / A)]A
\]

where

\[
R = \sqrt{\frac{F_y}{2F_e}}
\]

\[
A_o = \left(\frac{0.037}{DF_y / tE} + 0.667\right)A \leq A
\]

A = area of the unreduced cross-section.

Equation C-E3.1.1.1-6 is used for computing the reduced area due to local buckling. It is derived from Equation C-E3.1.1.1-2 for inelastic local buckling stress (Yu and LaBoube, 2010).

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, \( R = F_y/(2F_e) \) is used rather than \( R = \sqrt{F_y/(2F_e)} \) as in the previous edition of
the AISI Specification. The equation for the effective area is simplified to \( A_e = A_o + R(A - A_o) \) as given in Equation E3.1.1.1-1 of the Specification.

**E3.1.2 Members With Circular Holes**

For members with circular holes, the provisions in Appendix 1.1.1 (a) should be used in determining the effective area, \( A_e \). The Specification permits ignoring the hole effect if the effective length region times the hole diameter divided by the effective length does not exceed 0.015.

**E3.2 Direct Strength Method**

In the Direct Strength Method, the local buckling is considered in two parts: the long column strength without any reduction for local buckling \( (P_{n,le}) \), and the long column strength considering local-global interaction \( (P_{nl}) \). The calibration of the Direct Strength Method has been provided in Section E1.

**E3.2.1 Members Without Holes**

The nominal strength [resistance] of compression members without holes is provided in Specification Section E3.2.1 with the buckling load, \( P_{cre} \), determined in accordance with Section E2.

**E3.2.2 Members With Holes**

The Direct Strength Method (DSM) approach to columns with holes utilizes the elastic buckling properties of a cold-formed steel column \( (P_{ct}, P_{crd}, \text{and } P_{cre}) \), including the influence of holes (e.g., flat punched holes in studs, patterned holes in rack sections, holes with edge stiffeners) to predict ultimate strength. In most cases, holes decrease the elastic buckling properties, \( P_{ct}, P_{crd}, \text{and } P_{cre} \) which increases a column’s local \( (\lambda_l) \), distortional \( (\lambda_d) \) and global \( (\lambda_c) \) slenderness and lowers the predicted strength. Simplified methods for predicting \( P_{ct}, P_{crd}, \text{and } P_{cre} \) including holes are presented in Appendix 2. Alternatively, full finite element elastic Eigen-buckling analysis can be performed.

The DSM strength prediction expressions have been modified to limit the maximum strength of a column with holes to the capacity of the net cross-section, \( P_{y,net} \) (Moen and Schafer, 2011). A transition from \( P_{y,net} \) through the inelastic regime, to the elastic buckling portion of the distortional buckling strength curve has also been included in the design provisions. The transition slope is dictated by the ratio of the net section capacity to gross section capacity, \( P_{y,net}/P_y \), which was derived based on observed trends in column simulations to collapse, reported in Moen and Schafer (2009a). If a member contains mostly holes, then the critical elastic buckling loads and the net section capacity approach zero. The DSM strength equations are written such that when the net section goes to zero, predicted capacity also degrades to zero.

The development and calibration of the Direct Strength Method provisions for columns with holes was performed with experimental and simulation databases as reported in Moen and Schafer (2009a) and summarized in Moen and Schafer (2011). Note that both databases contain only lipped Cee cross-sections with discrete web holes because this is what was available in the research literature at the time. However, the philosophy of
employing elastic buckling parameters ($P_{crb}$, $P_{crd}$, $P_{cre}$) to predict the ultimate strength of cold-formed steel columns with holes was thoroughly validated in Moen and Schafer (2009a), and is assumed to hold true for other cross-section shapes and for members with edge-stiffened holes. See Grey and Moen (2011), and Moen and Yu (2010).

Holes are common in cold-formed steel members, and their presence reduces structural member strength as defined by Direct Strength Method equations in Specification Section E3.2.2 for compression members and Section F3.2.2 for flexural members. Hole influence on strength can be counterintuitive and difficult to predict just with engineering judgment alone. Therefore, the strength reduction should be calculated, even if the holes are small. Rules of thumb on the influence of holes in both compression and flexural members are: (1) rectangular or elongated holes typically reduce local buckling strength more than square and circular holes; (2) web holes always decrease distortional buckling strength; (3) holes always reduce global (Euler) buckling strength; (4) the more holes along a member, the more the strength decreases; (5) hole patterns, such as those typically present in storage rack columns, can reduce strength as much as discrete holes; and (6) adding edge stiffeners to holes increases local buckling strength more than distortional buckling and global buckling strength.

In an approximate strength check, the influence of holes on unlapped compression or flexural members can be ignored when the sum of the length of holes along the member is less than or equal to 10 percent of the member length ($\Sigma (L_h/L) \leq 0.10$); the maximum hole depth (width) is greater than or equal to 25 percent of the hole length ($d_h/L_h \geq 0.25$); and the net cross-sectional area is greater than or equal to 95 percent of the gross cross-sectional area ($A_{net}/A_g \geq 0.95$). Members meeting these limits are expected to have a capacity reduction of 5 percent or less caused by the presence of holes.

**E4 Distortional Buckling**

The expression selected for distortional buckling of columns is shown in Figure C-E1-1 and Figure C-E1-2 and is discussed in Section E1. Based on experimental test data and on the success of the Australian/New Zealand code (see Hancock et al., 2001 for discussion and Hancock et al., 1994 for further details), the distortional buckling strength is limited to $P_y$ instead of $P_{ne}$. This presumes that distortional buckling failures are independent of long-column behavior, i.e., little if any distortional-global interaction exists. See Appendix 2 for information on rational analysis methods for calculation of $P_{crd}$.

**E4.1 Members Without Holes**

Distortional buckling is an instability that may occur in members with edge-stiffened flanges, such as lipped C- and Z-sections. As shown in Figure C-E4.1-1, this buckling mode is characterized by instability of the entire flange, as the flange along with the edge stiffener rotates about the junction of the flange and the web. The length of the buckling wave in distortional buckling is considerably longer than local buckling, and noticeably shorter than flexural or flexural-torsional buckling. The Specification provisions of Section 1.3 partially account for distortional buckling, but research has shown that a separate limit state check is required (Schafer, 2002). Thus, in 2007, treating distortional buckling as a separate limit state, Specification Section E4.1 was added to address distortional buckling in columns and Specification Section F4.1 was added to address distortional buckling in beams.
Determination of the nominal strength [resistance] in distortional buckling (Specification Equation E4.1-2) was validated by testing. Specification Equation E4.1-2 was originally developed for the Direct Strength Method. Calibration of the safety and resistance factors for Specification Equation E4.1-2 is provided in Commentary Section E1. In addition, the Australian/New Zealand Specification (AS/NZS 4600) has used an expression of similar form to Specification Equation E4.1-2, but yielding slightly less conservative strength predictions than Equation E4.1-2.

Distortional buckling is unlikely to control the strength of a column if: (a) the web is slender and triggers local buckling far in advance of distortional buckling, as is the case for many common C-sections, (b) edge stiffeners are sufficiently stiff and thus stabilize the flange (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lip stiffeners), (c) unbraced lengths are long and flexural or flexural-torsional buckling strength limits the capacity, or (d) adequate rotational restraint is provided to the flanges from attachments (panels, sheathing, etc.).

The primary difficulty in calculating the strength in distortional buckling is to efficiently estimate the elastic distortional buckling load, \( P_{crd} \). Recognizing the complexity of this calculation, Appendix 2 provides two alternatives: (a) numerical solutions, or (b) analytical formulas for C- and Z-section members and any open section with a single web and flanges of the same dimension. See the Appendix 2 commentary for further discussion. The Appendix 2 commentary also provides a simplified analytical formula method that may be useful in preliminary design, and was specifically derived as a conservative simplification to Specification 2.3.1.3.

**E4.2 Members With Holes**
Figure C-E4.2-1 compares the *distortional buckling* strength prediction curve for a column without holes to the prediction curve for the same column with holes, where \( P_{\text{ynet}} = 0.80P_y \). For the column with holes, \( P_{\text{nd}} \) is limited to a maximum strength of \( P_{\text{ynet}} \). As distortional slenderness increases, the prediction transitions from \( P_{\text{ynet}} \) to the same strength curve used for columns without holes. The transition is implemented to reflect the change in failure mode as slenderness increases, from yielding at the net section to elastic *distortional buckling* along the column.

The extension of the DSM approach to columns with holes utilizes the elastic buckling properties of a cold-formed steel column \( (P_{\text{cr}}, P_{\text{crd}}, \text{and } P_{\text{cre}}) \), including the influence of holes to predict ultimate strength. In most cases, holes decrease the elastic buckling properties, \( P_{\text{cr}}, P_{\text{crd}}, \text{and } P_{\text{cre}} \) which increases a column’s local (\( \lambda_l \)), distortional (\( \lambda_d \)) and global (\( \lambda_c \)) slenderness and lowers the predicted strength. Simplified methods for predicting \( P_{\text{cr}}, P_{\text{crd}}, \) and \( P_{\text{cre}} \) including holes are presented in Appendix 2. Alternatively, full finite element elastic Eigen-buckling analysis can be performed.

The DSM strength prediction expressions have been modified to limit the maximum strength of a column with holes to the capacity of the net cross-section, \( P_{\text{ynet}} \) (Moen and Schafer, 2011). A transition from \( P_{\text{ynet}} \) through the inelastic regime, to the elastic buckling portion of the *distortional buckling* strength curve has also been included in the design provisions. The transition slope is dictated by the ratio of the net section capacity to gross section capacity, \( P_{\text{ynet}}/P_y \), which was derived based on observed trends in column simulations to collapse, reported in Moen and Schafer (2009a). If a member contains mostly holes, then the critical elastic buckling loads and the net section capacity approach zero. The DSM strength equations are written such that when the net section goes to zero, predicted capacity also degrades to zero.
F. MEMBERS IN FLEXURE

This chapter provides the design requirements for flexural members.

In 2007, the design provisions related to metal roof and wall systems were moved to Section I6:

1. Flexural Members Having One Flange Through-Fastened to Deck or Sheathing,
2. Flexural Members Having One Flange Fastened to a Standing Seam Roof System,
3. Compression Members Having One Flange Through-Fastened to Deck or Sheathing, and

In 2016, the Specification was reorganized, moving the tension design provisions to Chapter D, compression design provisions to Chapter E, shear design provisions to Chapter G, and the combined load checks to Chapter H. Chapter F contains the flexural member design provisions only. In addition, the Direct Strength Method and the Effective Width Method are combined.

F1 General Requirements

In general, a common nominal strength [resistance] equation is provided in the Specification for a given limit state with a required safety factor (Ω) for Allowable Strength Design (ASD) and a resistance factor (φ) for Load and Resistance Factor Design (LRFD) or Limit States Design (LSD). Design provisions that are applicable to a specific country are provided in the corresponding lettered appendix.

The thin-walled nature of cold-formed beams complicates behavior and design. Elastic buckling analysis reveals at least three buckling modes: local, distortional, and lateral-torsional buckling (for members in strong-axis bending) that must be considered in design. Bending strengths of flexural members are determined by considering yielding, global (lateral torsional) buckling in Specification Section F2, local buckling interaction with global buckling in Specification Section F3, and distortional buckling in Specification Section F4. The member flexural strength is the least of the strengths after considering the above buckling modes.

Like column design, two approaches can be used in beam design: Effective Width Method (EWM) and Direct Strength Method (DSM). The EWM traditionally addressed local and global buckling. In 2004, the distortional buckling strength prediction using DSM was adopted.

In considering flexural member yielding and global buckling, the DSM follows the same practice as the EWM. The Effective Width Method provides the lateral-torsional buckling strength in terms of a stress, $F_n$ (Specification Equation F2.1-1). In the DSM, this is converted from a stress to a moment by multiplying by the gross section modulus, $S_f$, resulting in Specification Equation F2.1-1 for $M_{ne}$. The DSM emerged through the combination of more refined methods for local and distortional buckling prediction, improved understanding of the post-buckling strength and imperfection sensitivity in distortional buckling, and the relatively large amount of available experimental data.

In the Effective Width Method, for beams that are not fully braced and locally unstable, beam strength is calculated by multiplying the predicted stress for failure in lateral-torsional buckling, $F_{nt}$, by the effective section modulus, $S_e$, determined at stress $F_n$. This accounts for local buckling reductions in the lateral-torsional buckling strength (i.e., local-global interaction). In the DSM, this calculation is broken into two parts: the lateral-torsional buckling strength without any reduction for local buckling ($M_{ne}$), and the strength considering local-global interaction ($M_{nt}$).
The strength curves for local and distortional buckling of a beam fully braced against lateral-torsional buckling are presented in Figure C-F1-1 and compared to the critical elastic buckling curve. The post-buckling reserve for the local mode is predicted to be greater than that of the distortional mode. As depicted in Figure C-F1-1, provisions were added in 2012 for inelastic reserve capacity in bending, i.e., where \( M_n > M_y \).
If members are laterally supported, then they are proportioned according to the nominal section strength (Specification Section F3.1). Since distortional buckling has an intermediate buckling half wavelength, distortional buckling still needs to be considered even for braced members. See the Direct Strength Method Design Guide (AISI, 2006) for detailed discussion and design examples. If they are laterally unbraced, then the limit state is lateral-torsional buckling and possible interaction with local buckling (Specification Sections F2 and F3).

The extension of the DSM approach to beams with holes utilizes the elastic buckling properties of a cold-formed steel beam ($M_{tr}$, $M_{rd}$, and $M_{re}$) including the influence of holes to predict ultimate strength. In most cases, holes decrease $M_{tr}$, $M_{rd}$, and $M_{re}$; this increases the beam’s local ($\lambda_c$), distortional ($\lambda_d$) and global ($\lambda_c$) slenderness and lowers the predicted strength. Simplified methods for predicting $M_{tr}$, $M_{rd}$, and $M_{re}$ including holes are presented in Appendix 2. Alternatively, full finite element elastic Eigen-buckling analysis can be performed.

The calibration of the Effective Width Method was reviewed in the Commentary of the 1991 edition of the Specification. A brief discussion of the DSM is provided herein. The reliability of the DSM beam provisions was determined using test data defined by the limits of Section B4.1 and the provisions of Section K2 of the Specification. Based on a target reliability, $\beta$, of 2.5, a resistance factor, $\phi$, of 0.90 was calculated for all of the investigated beams. Based on this information, the safety and resistance factors of Specification Chapter F were determined for the prequalified members. The safety factor, $\Omega$, is back-calculated from $\phi$ at an assumed dead-to-live load ratio of 1 to 5. Since the range of prequalified members is relatively large, extensions of the DSM to geometries outside the prequalified set are allowed. However, given the uncertain nature of this extension, increased safety factors and reduced resistance factors are applied in that case, per the rational engineering analysis provisions of Section A1.2(c) of the Specification.

The provisions of Specification Chapter F, applied to the beams of Specification Section B4.1, are summarized in Figure C-F1-2. The controlling strength is determined either by Specification Section F3, which considers local buckling interaction with lateral-torsional buckling, or by...
Specification Section F4, which considers the distortional mode alone. The controlling strength (minimum predicted of the two modes) is highlighted for the examined members by the choice of marker. Overall performance of the method can be judged by examination of Figure C-F1-2. The scatter shown in the data is similar to that of the Effective Width Method.

The development and calibration of the DSM provisions for beams with holes were performed with a simulation database as reported in Moen and Schafer (2009a) and a set of 12 beam experiments summarized in Moen, et al. (2012). Note that the simulations and experiments only considered lipped Cee cross-sections with discrete web holes. However, the philosophy of employing elastic buckling parameters \(M_{cr}, M_{crd}, M_{cre}\) to predict the ultimate strength of cold-formed steel beams with holes, validated in Moen and Schafer (2009a), is assumed to hold true for other cross-section shapes.

Resistance factors for beams with holes were calculated by limit state with Section K2 of the main Specification. Based on a target reliability, \(\beta\), of 2.5, the resistance factor, \(\phi\), was calculated with the simulation database as 0.95 for laterally braced beams predicted to fail from local buckling. For beams predicted to experience a distortional buckling failure mode, \(\phi\) was calculated with the simulation database as 0.91 and with the Moen, et al. (2012) experiments as 0.94.

F2 Yielding and Global (Lateral-Torsional) Buckling

The bending capacity of flexural members can be limited by yielding or the lateral-torsional buckling strength of the member depending on the member’s lateral unbraced length. The design provisions for determining the nominal lateral-torsional buckling strength [resistance] are given in Specification Section F2.

F2.1 Initiation of Yielding Strength

In this section, the limit states of yielding and global (lateral-torsional) buckling are discussed.

A. Initiation of Yielding

For compact beams with short unbraced lengths, the member may fail by yielding. The yield moment is determined by Equation C-F2.1-1:

\[
M_y = S_f F_y
\]

where \(S_f\) is the unreduced elastic section modulus, and \(F_y\) is the yield stress.

B. Lateral-Torsional Buckling

(a) Elastic Buckling Stress for Doubly- or Singly-Symmetric Open Cross-Section

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-F2.1-2.

\[
\sigma_{cr} = \frac{\pi}{L S_f} \sqrt{E I_y G_j \left(1 + \frac{\pi^2 E C_w}{G J L^2}\right)}
\]

For other than simply supported end conditions, Equation C-F2.1-2 can be generalized as given in Equation C-F2.1-2a (Galambos, 1998):
In the above equation, \( K_y \) and \( K_t \) are effective length factors; \( L_y \) 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-flange I-members with simply supported end conditions both laterally and torsionally, Equation C-F2.1-3 can be used to calculate the elastic critical buckling stress (Winter, 1947a; Yu and LaBoube, 2010):

\[
\sigma_{cr} = \frac{\pi^2 E d}{2(1 + \mu)(I_{yc} + I_{yt}) S_f} \left( I_{yc} - I_{yt} + I_y \sqrt{1 + \frac{4GJL^2}{\pi^2 I_y Ed^2}} \right)^2
\]  

(C-F2.1-3)

In Equation C-F2.1-3, 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):

\[
\sigma_{cr} = \frac{\pi^2 E d}{2L^2 S_f} \left( I_{yc} - I_{yt} + I_y \sqrt{1 + \frac{4GJL^2}{\pi^2 I_y Ed^2}} \right)^2
\]  

(C-F2.1-4)

where \( I_{yc} \) and \( I_{yt} \) 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-F2.1-3 and C-F2.1-4 are identical.

For other than simply supported end conditions, Equation C-F2.1-4 can be generalized as given in Equation C-F2.1-4a:

\[
\sigma_{cr} = \frac{\pi^2 E d}{2(K_y L_y)^2 S_f} \left( I_{yc} - I_{yt} + I_y \sqrt{1 + \frac{4GJ(K_t L_t)^2}{\pi^2 I_y Ed^2}} \right)^2
\]  

(C-F2.1-4a)

In Equation C-F2.1-4a, 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-F2.1-4a can be simplified as shown in Equation C-F2.1-5 by considering \( I_y = I_{yc} + I_{yt} \) and neglecting the term \( 4GJ(K_t L_t)^2/(\pi^2 I_y Ed^2) \):

\[
\sigma_{cr} = \frac{\pi^2 E d I_{yc}}{(K_y L_y)^2 S_f}
\]  

(C-F2.1-5)

Equation C-F2.1-5 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, \( C_{br} \), to account for nonuniform bending and the symbol \( F_e \) is used for \( \sigma_{cr} \), i.e.,
\[
F_e = \frac{C_b \pi^2 E I_{yc}}{(K_y L_y)^2 S_f} \tag{C-F2.1-6}
\]

where \( C_b \) is the bending coefficient, which can conservatively be taken as unity, or calculated in accordance with Equation C-F2.1-11.

In the 1986 edition of the Specification, in addition to the use of Equation C-F2.1-6 for determining the critical stresses, more design equations (Specification Equations F2.1.1-1, F2.1.2-1, and F2.1.3-1) for elastic critical stress were added as alternative methods. These additional equations were developed from the previous studies conducted by Peköz, Winter and Celebi on flexural-torsional buckling of thin-walled sections under eccentric loads (Peköz and Winter, 1969a; Peköz 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:

\[
F_e = \frac{C_b A_0 \sigma_{ey}}{S_f} \sqrt{\sigma_{ey} \sigma_t} \tag{C-F2.1-7}
\]

where \( \sigma_{ey} \) and \( \sigma_t \) are the elastic buckling stresses as defined in Specification Equations F2.1.1-4 and F2.1.1-5, respectively.

In the 1996 edition of the Specification, this general form was adopted as the primary design equation and Equation C-F2.1-6 was retained as an alternative. In the 2016 edition of the Specification, the alternative equation was restricted to doubly-symmetric sections because it was derived for I-members and was determined to be unconservative for most singly-symmetric sections.

(b) Elastic Buckling Stress for Point-Symmetric Open Cross-Section

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 is used in the Specification, in which the elastic critical buckling stress is taken to be one-half of that for I-members.

(c) Elastic Buckling Stress for Closed Tubular Cross-Section

In computing the lateral-torsional buckling stress of closed-box sections, the warping constant, \( C_w \), may be neglected since the effect of nonuniform warping of box sections is small. The critical buckling stress is

\[
\sigma_{cr} = \frac{\pi}{(K_y L_y) S_f \sqrt{E I_y G J}} \tag{C-F2.1-8}
\]

The Saint-Venant torsional constant, \( J \), of a box section, neglecting the corner radii, may be conservatively determined as follows:

\[
J = \frac{2(ab)^2}{(a/t_1) + (b/t_2)} \tag{C-F2.1-9}
\]

where

- \( a \) = Distance between web centerlines
- \( b \) = Distance between flange centerlines
- \( t_1 \) = Thickness of flanges
- \( t_2 \) = Thickness of webs

(d) Bending Coefficient, \( C_b \)

Bending coefficient, \( C_b \), is applied to the critical buckling stress, \( \sigma_{cr} \), to account for
nonuniform bending. \( C_b \) can be determined as follows:

\[
C_b = 1.75 + 1.05 \left( \frac{M_1}{M_2} \right) + 0.3 \left( \frac{M_1}{M_2} \right)^2 \leq 2.3 \tag{C-F2.1-10}
\]

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 Specification. Because it is valid only for straight-line moment diagrams, Equation C-F2.1-10 was replaced by the following equation for \( C_b \) in the 1996 edition of the Specification and is retained in this edition of the Specification:

\[
C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 4M_B + 3M_C} \tag{C-F2.1-11}
\]

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

Equation C-F2.1-11, 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 ANSI/AISC S360 (AISC, 2010a).

Figure C-F2.1-1 shows the differences between Equations C-F2.1-10 and C-F2.1-11 for a straight line moment diagram.

\( (e) \) Inelastic Buckling Stress

It should be noted that Equations C-F2.1-2a and C-F2.1-6 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 following equation was used for calculating the critical moment in the 1986 edition of the AISI Specification:

\[(M_{cr})_e = M_y \left(1 - \frac{M_y}{4(M_{cr})_e}\right)\]  

(C-F2.1-12)

in which \((M_{cr})_e\) is the elastic critical buckling moment.

The following equation for determining the inelastic buckling stress, \(F_n\), was adopted in the 1996 edition of the Specification:

\[F_n = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e}\right)\]  

(C-F2.1-13)

where \(F_e\) is the elastic critical lateral-torsional buckling stress.

The general shape of the curve as represented by Equation C-F2.1-13 is consistent with the preceding edition of the Specification (AISI, 1986).

As specified in Specification Section F2.1, 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 \(F_n\) in Equation C-F2.1-13.

This liberalization of the inelastic lateral-torsional buckling curve for singly-, doubly-, and point-symmetric sections has been confirmed by research in beam-columns (Peköz and Sumer, 1992) and wall studs (Niu and Peköz, 1994).

(f) Limit of Unbraced Length

The elastic and inelastic critical stresses for the lateral-torsional buckling strength are shown in Figure C-F2.1-2. For any unbraced length, \(L\), less than \(L_u\), lateral-torsional buckling does not need to be considered. \(L_u\) is determined by setting \(F_{cre} = 2.78F_y\) and \(L_u = L_y = L_t\). \(L_u\) may then be calculated using the expression given below (AISI, 1996):

(1) For Singly-, Doubly- and Point-Symmetric Sections:

\[L_u = \left\{ \frac{GJ}{2C_1} + \left[ \frac{C_2}{C_1} + \left( \frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \]  

(C-F2.1-14)

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-F2.1-15)

\[C_1 = \frac{30.9}{AE} \left( \frac{K_y F_y S_f}{C_b \pi r_y} \right)^2 \]  

for point-symmetric sections  

(C-F2.1-16)

\[C_2 = \frac{\pi^2 EC_w}{(K_t)^2} \]  

(C-F2.1-17)
(2) For I-Sections, Singly-Symmetric C-Sections, or Z-Sections Bent About the Centroidal Axis Perpendicular to the Web
The following equations may be used in lieu of (1) (AISI, 1996):
For doubly-symmetric I-sections and singly-symmetric C-sections:

\[
L_u = \frac{1}{K_y} \left( \frac{0.36C_b \pi^2 EdI_{yc}}{F_y S_f} \right)^{0.5}
\]

(C-F2.1-18)

For point-symmetric Z-sections:

\[
L_u = \frac{1}{K_y} \left( \frac{0.18C_b \pi^2 EdI_{yc}}{F_y S_f} \right)^{0.5}
\]

(C-F2.1-19)

For members with unbraced length, \( L \leq L_u \), or elastic lateral-torsional buckling stress, \( F_{cre} \geq 2.78F_y \), the flexural strength (without considering local buckling) is determined by Specification Equation F2.1-1 with \( F_n = F_y \).

Due to the high torsional stiffness of closed-box sections, lateral-torsional 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 F2.1-4 equal to \( F_y \), with \( F_{cre} \) set equal to Specification Equation F2.1.4-2.

![Figure C-F2.1-2 Lateral-Torsional Buckling Stress](image)

**F2.2 Beams With Holes**

The hole effect was considered in determining the effective section modulus, \( S_e \), but was not previously considered in global buckling analysis. The research work (Moen and Schafer, 2009a, 2009c and 2010b) indicated that the existence of holes will reduce the member global buckling stress and lower the predicted strength. It was, therefore, decided that hole effect should be considered in determining the global buckling stress. Specification Appendix 2 provides both analytical and numerical analysis methods to consider the hole effect.
Within the limitations of the hole size given in Appendix 1.1.3, the hole influence on the lateral-torsional buckling stress is negligible when using the Effective Width Method; therefore, an exception is provided to exclude these cases from the additional requirements of Appendix 2.

**F2.3 Initiation of Yielding Strength [Resistance] for Closed Cylindrical Tubular Sections**

The discussion on cylindrical tubular member behavior and buckling modes is provided in Commentary Section E3.1.1.1. It should be noted that the design provisions of Specification Sections F2.3 and E3.1.1.1 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.

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 F2.3-2, F2.3-3 and F2.3-4 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 0.75F_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 0.441E/F_y is the same as for axial compression.
All three equations for determining the nominal flexural strength [resistance] of closed cylindrical tubular members are shown in Figure C-F2.3-1. These equations have been used in the Specification since 1986 and are retained in this edition. In 1999, the limiting D/t ratios for Specification Equations F2.3-2 and F2.3-3 were revised to provide an appropriate continuity. The safety factor, \( \Omega \), and the resistance factor, \( \phi \), are the same as used in Specification Section F2 for sectional bending strength.

**F2.4 Inelastic Reserve Strength**

**F2.4.1 Element-Based Method**

Prior to 1980, the inelastic reserve capacity of beams was not included in the 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, Peköz, Winter, and Yener at Cornell University (Reck, Peköz and Winter, 1975; Yener and Peköz, 1985a, 1985b). These studies showed that the inelastic reserve strength of cold-formed steel beams due to partial plastification of the cross-section and the moment redistribution of statically indeterminate beams can be significant for certain practical shapes. With proper care, this reserve strength can be utilized to achieve more economical design of such members.

In order to utilize the available inelastic reserve strength [factored 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 Specification. The same provisions are retained in this edition of the Specification. According to Section F2.4.1 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 \( 1.25M_y \), where \( M_y \) is the effective yield moment. The ratio of \( M_n/M_y \) represents the inelastic reserve strength of a beam cross-section.

The nominal moment [resistance], \( M_n \), is the maximum bending capacity of the beam by considering the inelastic reserve strength 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, Peköz 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 F2.4.1 (a) as shown in Figure C-F2.4.1-1.

On the basis of the maximum compression strain, \( \varepsilon_{cu} \), allowed in the Specification, the neutral axis can be located by using Equation C-F2.4.1-1 and the nominal moment [resistance] \( M_n \) can be determined by using Equation C-F2.4.1-2:

\[
\int \sigma dA = 0 \quad \text{(C-F2.4.1-1)}
\]

\[
\int \sigma y dA = M_n \quad \text{(C-F2.4.1-2)}
\]

where \( \sigma \) is the stress in the cross-section, and \( y \) is the distance measured from the neutral axis to the yield stress.
The calculation of $M_n$ based on inelastic reserve capacity is illustrated in Part I of the *AISI Cold-Formed Steel Design Manual* (AISI, 2013) and the textbook by Yu and LaBoube (2010).

In 2001, the shear force upper limit was clarified. The stress upper limit is $0.35F_y$ for ASD and $0.6F_y$ for LRFD and LSD in the Specification.

Additional equations were provided in Specification Section F2.4.1(b) since 2004 for determining the nominal moment strength [resistance], $M_n$, based on inelastic reserve capacity, for sections containing unstiffened compression elements under stress gradient. Based on research by Bambach and Rasmussen (2002b, 2002c) on I- and plain channel sections in minor axis bending, a compression strain factor, $C_y$, determines the maximum compressive strain on the unstiffened element of the section. The $C_y$ values are dependent on the stress ratio, $\psi$, and slenderness ratio, $\lambda$, of the unstiffened element, determined in accordance with Section 1.2.2(a) of the Specification.

### F2.4.2 Direct Strength Method

In 2012, provisions were added (Specification Sections F2.4.2, F3.2.3, and F4.3) to take advantage of the inelastic reserve strength for members that are stable enough to allow partial plastification of the cross-section. Such sections have capacities in excess of $M_y$ and potentially as high as $M_p$ (though practically, this upper limit is rarely achievable). As Figure C-F1-1 shows, the inelastic reserve capacity is assumed to linearly increase with decreasing slenderness.

### F3 Local Buckling Interacting With Yielding and Global Buckling

#### F3.1 Effective Width Method

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 of the member. The effect of local buckling on the critical
moment is considered by Equation F3.1-1 of the Specification by using the elastic section modulus, $S_e$, based on an effective section.

Using the nominal lateral-torsional buckling strength [resistance] determined in accordance with Specification Equation F3.1-1 with a resistance factor of $\phi_b = 0.90$, the reliability indexes of $\beta$ vary from 2.4 to 3.8 for the LRFD method.

For locally stable beams, the nominal moment, $M_n$, 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 Appendix 1 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 stress of the steel. This is the maximum bending capacity to be used in elastic design. Figure C-F3.1-1 shows several types of stress distributions for yield moment based on different locations of the neutral axis. For balanced sections (Figure C-F3.1-1(a)), the outer fibers in the compression and tension flanges reach the yield stress at the same time. However, if the neutral axis is eccentrically located, as shown in Figures C-F3.1-1(b) and (c), the initial yielding takes place in the tension flange for case (b) and in the compression flange for case (c).

Figure C-F3.1-1 Stress Distribution for Yield Moment:
(a) Balanced Sections, (b) Neutral Axis Close to Compression Flange, and (c) Neutral Axis Close to Tension Flange
Accordingly, the nominal section strength [resistance] for initiation of yielding is calculated by using Equation C-F3.1-1:

\[ M_n = S_e F_y \]  \hspace{1cm} (C-F3.1-1)

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.

Prior to the 2008 edition of the AISI Specification, the design flexural strength [factored resistance], \( \phi_b M_n \), employed different \( \phi_b \) factors depending on the compression flange. Based on the 1991 edition of the Specification and the work of Hsiao, Yu and Galambos (1988a), unstiffened flanges were specified at \( \phi_b = 0.90 \) and edge-stiffened or stiffened flanges at \( \phi_b = 0.95 \) (ASD used one \( \Omega \) factor for all cases). Examination of more recently available test data (Schafer and Trestain, 2002; Yu and Schafer, 2003) and consideration of the fact that the higher \( \phi_b \) existed in part due to inelastic reserve strength, which is already addressed in Specification Section F2.4.1, a uniform \( \phi_b = 0.90 \) was adopted for all members. This change also removed a conflict with the \( \phi_b \) factors adopted in 2007 for the Specification, when the member is fully effective.

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**Figure C-F3.1-2 Combined Sheet-Stiffener Sections**
The research work (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. See Section F4 for commentary on distortional buckling strength.

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-F3.1-2. 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. However, when the distortional buckling may occur, 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-F3.1-3. This distortional buckling strength can be determined using the design provisions provided in Specification Section F4. It should, however, be noted that for laterally unstable U-shaped beams, lateral-torsional buckling may still occur. Therefore, lateral-torsional buckling should still be considered for U-shaped members.

F3.1.1 Members Without Holes

The nominal strength [resistance] due to local buckling interacting with yielding or global buckling is determined by Specification Equation F3.1-1, in which the effective section modulus, $S_w$, is calculated at the extreme $F_n$.

F3.1.2 Members With Holes

For members with holes, the elements beside the holes are considered as unstiffened elements. The effective widths are then determined in accordance with Appendix 1 of the Specification. The nominal stress, $F_n$, should consider the effects of the holes in accordance with Specification Section F2.2.

F3.1.3 Members Considering Inelastic Reserve Strength

Specification Section F2.4.1 should be used for determining the inelastic reserve strength, as applicable. The discussion of inelastic reserve strength has been provided in Commentary Section F2.4.1.

F3.2 Direct Strength Method

In the Direct Strength Method (DSM), local buckling is considered through beam lateral-torsional buckling without any reduction for local buckling ($M_{ne}$), and beam strength is considered in local-global interaction ($M_{nl}$). The calibration of the DSM for beams was discussed in Commentary Section F1.
**F3.2.1 Members Without Holes**

The expression selected for local buckling of beams is shown in Figures C-F1-1 and C-F1-2 and is discussed in Section F1. The use of the DSM for local buckling and the development of the empirical strength expression are given in Schafer and Peköz (1998). The potential for local-global interaction is presumed; thus, the beam strength in local buckling is limited to a maximum of the nominal lateral-torsional buckling strength [resistance], $M_{ne}$. For fully braced beams, the maximum $M_{ne}$ value is the yield moment, $M_y$.

**F3.2.2 Members With Holes**

For beams with holes (e.g., flat-punched holes in studs, patterned holes in rack sections, holes with edge stiffeners), $M_{n/h}$ is limited to $M_{y/net}$ to reflect yielding and collapse of the net section when both local and global slenderness are low.

More discussions are provided in Commentary Section E3.2.2 regarding hole influences on member strength, including the treatment of stiffened holes (Grey and Moen, 2011; Moen and Yu, 2010).

**F3.2.3 Members Considering Local Inelastic Reserve Strength**

Unique expressions were derived for inelastic bending reserve in local buckling. This reserve is only allowed in cross-sections that are predicted to have inelastic bending reserve in lateral-torsional buckling (i.e., $M_{ne} > M_y$). The compressive strain which the cross-section may sustain in local buckling, $C_y\varepsilon_y$, is shown to increase as specified in Specification Equation F3.2.3-4 in both back-calculated strains from tested sections and average membrane strains from finite element models (Shifferaw and Schafer, 2010). Local strains in the corners and at the surface of the plates (comprising the cross-section) as they undergo bending may be significantly in excess of $C_y\varepsilon_y$ (Shifferaw and Schafer, 2010). As a result, and consistent with the main Specification, $C_y\varepsilon_y$ is limited to 3.

For sections with first yield in tension, the potential for inelastic reserve capacity is great, but the design calculations are more complicated. Specification Equation F2.4.2-1 only applies after the cross-section begins to yield in compression, i.e., when the moment reaches $M_{yc}$. Calculation of $M_{yc}$ requires the use of basic mechanics to determine the moment strength in the partially plastified cross-section. $M_y$ may be used in place of $M_{yc}$, but this is conservative (excessively so if the tensile strain demands are much higher than the compressive strain demands). Based on experience and past practice, it has also been determined that the tensile strain should not exceed three times the yield strain; thus the moment is also limited by this value, i.e., $M_{y3}$.

Note: The slenderness $\lambda_{sl}$ utilizes $M_y$, instead of $M_{ne}$, for simplicity in the inelastic reserve regime and provides continuity with the expressions of Specification Section F3.2.1. Further, the elastic buckling moment, $M_{cr,el}$, is determined based on the elastic bending stress distribution, not the plastic stress distribution. These simplifications were shown to be sufficiently accurate when compared with existing tests and a parametric study using rigorous nonlinear finite element analysis (Shifferaw and Schafer, 2010).
F4 Distortional Buckling

F4.1 Members Without Holes

Distortional buckling is an instability that may occur in members with edge-stiffened flanges, such as lipped C- and Z-sections. As shown in Figure C-F4.1-1, this buckling mode is characterized by instability of the entire flange, as the flange along with the edge stiffener rotates about the junction of the compression flange and the web. The length of the buckling wave in distortional buckling is considerably longer than local buckling, and noticeably shorter than lateral-torsional buckling. The Specification provisions of Section 1.3 partially account for distortional buckling, but research has shown that a separate limit state check is required (Ellifritt, Sputo, and Haynes, 1992; Hancock, Rogers, and Schuster, 1996; Kavanagh and Ellifritt, 1994; Schafer and Peköz, 1999; Hancock, 1997; Yu and Schafer, 2003 and 2006). Thus, in 2007, provisions were added to address distortional buckling as a separate limit state.

\[ \text{Distortional } \frac{M_{cr}}{M_y} = 0.77 \]

\[ \text{Local } \frac{M_{cr}}{M_y} = 0.85 \]

\[ M_y = 107.53 \text{kip-in.} \]

\[ \frac{M_{cr}}{M_y} \text{ vs. half-wavelength (in.)} \]

Figure C-F4.1-1 Rational Elastic Buckling Analysis of a Z-Section Under Restrained Bending

Showing Local, Distortional, and Lateral-Torsional Buckling Modes

Determination of the nominal strength [resistance] in distortional buckling (Specification Equation F4.1-2) was validated by testing. Results of one such study (Yu and Schafer, 2006) are shown in Figure C-F4.1-2. In addition, the Australian/New Zealand Specification (AS/NZS 4600) has used Specification Equation F4.1-2 since 1996. Calibration of the safety and resistance factors for Specification Equation F4.1-2 is provided in Commentary Section F1.

Distortional buckling is unlikely to control the strength if: (a) edge stiffeners are sufficiently stiff and thus stabilize the flange (as is often the case for C-sections, but typically not for Z-sections due to the use of sloping lips), (b) unbraced lengths are long and lateral-torsional buckling strength limits the capacity, or (c) adequate rotational restraint is provided to the compression flange from attachments (panels, sheathing, etc.).
The primary difficulty in calculating the strength in distortional buckling is to efficiently estimate the elastic distortional buckling moment, $M_{crd}$. Recognizing the complexity of this calculation, Appendix 2 provides two alternatives: (a) numerical solutions, or (b) analytical formula for C- and Z-section members and any open section with a single web and single edge-stiffened compression flange. See the Appendix 2 commentary for further discussion. The Appendix 2 commentary also provides a simplified analytical formula that may be useful in preliminary design, and was specifically derived as a conservative simplification to Specification Section 2.3.3.3.

**F4.2 Members With Holes**

The DSM strength prediction expressions have been modified to limit the maximum strength of a beam with holes to the capacity of the net cross-section, $M_{ynet}$ (Moen and Schafer, 2009b). A transition from $M_{ynet}$ through the inelastic regime, to the elastic buckling portion of the distortional buckling strength curve is also included in the design provisions as shown in Figure C-F4.2-1. The transition slope is dictated by the ratio of net section capacity to gross section capacity, $M_{ynet}/M_y$, which was derived based on observed trends in beam simulations to collapse reported in Moen and Schafer (2009b) and experiments (Moen et al., 2012).
F4.3 Members Considering Distortional Inelastic Reserve Strength

The inelastic reserve strength provisions were added in 2012 based on the research finding by Shifferaw and Schafer (2010). See commentary in Section F3.2.3 for detailed discussion. The provisions take advantage of the inelastic reserve strength for members that are stable enough to allow partial plastification of the cross-section. Such sections have capacities in excess of $M_y$ and potentially as high as $M_p$ (though practically, this upper limit is rarely achievable). As Figure C-F1-1 shows, the inelastic reserve capacity is assumed to linearly increase with decreasing slenderness.

F5 Stiffeners

F5.1 Bearing Stiffeners

Design requirements for attached bearing stiffeners (previously called transverse stiffeners) were added in the 1980 Specification and the same design equations are retained in Section F5 of the current Specification. The term “transverse stiffener” was changed to “bearing stiffeners” in 2004. The nominal strength [resistance] equation given in Item (a) of Specification Section F5.1 serves to prevent end crushing of the bearing 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 bearing 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 \( \frac{w}{t_s} \) ratio for the unstiffened elements of cold-formed steel bearing stiffeners was revised from \( 0.37 \sqrt{\frac{E}{F_{ys}}} \) to \( 0.42 \sqrt{\frac{E}{F_{ys}}} \) 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 bearing stiffeners.

**F5.2 Bearing Stiffeners in C-Section Flexural Members**

The provisions of this section are based on the research by Fox and Schuster (2001), which investigated the behavior of stud and track type bearing stiffeners in cold-formed steel C-section flexural members. These stiffeners fall outside of the scope of *Specification* Section F5.1. The research program investigated bearing stiffeners subjected to two-flange loading at both interior and end locations, and with the stiffener positioned between the member flanges and on the back of the member. A total of 263 tests were carried out on different stiffened C-section assemblies. The design expression in *Specification* Section F5.2 is a simplified method applicable with the limits of the test program. A more detailed beam-column design method is described in Fox (2002).

**F5.3 Nonconforming Stiffeners**

Tests on rolled-in stiffeners covered in *Specification* Section F5.3 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the *available strength [factored resistance]* of stiffeners should be determined by special tests.
**G. MEMBERS IN SHEAR AND WEB CRIPPLING**

**G1 General Requirements**

Chapter G defines the shear strength of flexural members with or without web holes. The design of transverse web stiffeners and the determination of web crippling strength are also treated.

**G2 Shear Strength [Resistance] of Webs Without Holes**

Previous editions of the AISI ASD Specification (AISI, 1986) used three different safety factors when evaluating the allowable shear strength 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 safety factor for ASD and one resistance factor for LRFD, 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 G2.1-5 was reduced from 0.64 to 0.60. In addition, the ASD safety factor 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.

**G2.1 Flexural Members Without Transverse Web Stiffeners**

The shear strength of flexural member webs is governed by either yielding or buckling, depending on the h/t ratio and the mechanical properties of steel. For flexural member webs having small h/t ratios, the nominal shear strength [resistance] is governed by shear yielding, i.e.,

\[
V_n = A_w \tau_y = A_w F_y / \sqrt{3} \approx 0.60 F_y h t
\]

in which \( A_w \) is the area of the flexural member web computed as \((ht)\), and \( \tau_y \) is the yield stress of steel in shear, computed as \( F_y / \sqrt{3} \).

For flexural member webs having large h/t ratios, the nominal shear strength [resistance] is governed by elastic shear buckling (Yu and LaBoube, 2010), 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 nominal shear strength [resistance], \( V_n \), can be determined as follows:

\[
V_n = 0.904 E k_v t^3 / h
\]

For flexural member webs having moderate h/t ratios, the nominal shear strength [resistance] is based on inelastic shear buckling (Yu and LaBoube, 2010), i.e.,

\[
V_n = 0.64 t^2 / \sqrt{k_v F_y E}
\]

The Specification provisions are applicable for the design of webs of flexural members and decks either with or without transverse web stiffeners.

The nominal strength [resistance] equations given in C-G2.1-1 to C-G2.1-4 above are similar to the nominal shear strength [resistance] equations given in the AISI LRFD Specification (AISI,
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).

In 2016, the Direct Strength Method (DSM) equations for determining the nominal shear strength [resistance] were adopted for sections prequalified to Specification Table B4.1-1, including flat webs and webs with small intermediate longitudinal stiffeners. The DSM equations provided in Specification Section G2.1 for shear are based on the nominal strength [resistance] Equations C-G2.1-1 to C-G2.1-4. Validation for the local buckling equations in DSM format has been confirmed (Pham and Hancock, 2012a) by tests on high-strength steel C-sections in shear, and combined bending and shear, and the tests of LaBoube and Yu (1978a).

**G2.2 Flexural Members With Transverse Web Stiffeners**

The Pham and Hancock tests show that considerable tension field action is available for local buckling if the web is fully restrained at the loading and support points over its full depth by bolted connections. This post-local buckling has been included in the Specification Equations G2.2-1 and G2.2-2 for aspect ratios up to 2:1 based on testing and FEM analyses (Pham and Hancock, 2012b). These equations allow elastic local critical shear buckling force, $V_{cr}$, to be determined by an elastic buckling analysis of the whole section or web in pure shear including longitudinal intermediate stiffeners. Experimental justification for inclusion of small longitudinal intermediate stiffeners in the value of $V_{cr}$ in Specification Equations G2.2-1 and G2.2-2 is given in Pham and Hancock (2012a). Distortional buckling in shear has been ignored at this stage.

Prior to 2016, the shear strength with transverse web stiffeners was predicted using Equations C-G2.1-1 to C-G2.1-4 while the transverse stiffener effect was considered in shear buckling coefficient $k_v$. Tension field action was, however, not considered.

**G2.3 Web Elastic Critical Shear Buckling Force, $V_{cr}$**

Specification Section G2.3 provides a simple analytical solution for shear buckling force, $V_{cr}$, of an unreinforced web. However, for prequalified webs according to Specification Table B4.1-1, a numerical analysis approach should be considered in accordance with Appendix 2, which provides for the contribution of the transverse stiffeners in buckling analysis.

**G3 Shear Strength 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 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 equation for reduction factor, $q_{sr}$, 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, $q_{sr}$, 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 $h/t \leq 0.96\sqrt{\frac{E}{F_y}}$, an anomaly exists; the calculated available shear strength [factored resistance] is independent of $t$ when $h$ is constant. In this region, the calculated available shear...
strength [factored 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-1.1.3-1 illustrates the \( L_h \) and \( d_h \) that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-1.1.3-2 illustrates the \( d_h \) 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 to the actual hole or holes. The reduction factor, \( q_s \), only applies when the shear buckling stress, \( V_{cr} \), is computed according to Section G2.3.

**G4 Transverse Web Stiffeners**

**G4.1 Conforming Transverse Web Stiffeners**

The requirements for transverse web stiffeners included in Specification Section G4.1 were primarily adopted from the AISC Specification (1978). The equations for determining the minimum required moment of inertia (Specification Equation G4.1-1) and the minimum required gross area (Specification Equation G4.1-2) of attached transverse web stiffeners are based on the studies summarized by Nguyen and Yu (1978a). In Specification Equation G4.1-1, 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 of beam webs with transverse web 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 \( \phi = 0.90 \).

**G4.2 Nonconforming Transverse Web Stiffeners**

Tests on rolled-in transverse web stiffeners covered in Specification Section G4.2 were not conducted in the experimental program reported by Nguyen and Yu (1978). Lacking reliable information, the available strength [factored resistance] of stiffeners should be determined by special tests, or rational engineering analysis.

**G5 Web Crippling Strength 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-G5-1 shows typical web crippling failure modes of unreinforced single hat sections (Figure C-G5-1(a)) and of I-sections (Figure C-G5-1(b)) unfastened to the support.

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 and LaBoube, 2010). 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 Hettrakul 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

Figure C-G5-1 Web Crippling of Cold-Formed Steel Sections

Figure C-G5-2 Loading Conditions for Web Crippling Tests:
(a) EOF Loading, (b) IOF Loading, (c) ETF Loading, (d) ITF Loading
All loading conditions are illustrated in Figure C-G5-2. 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-G5-3 and the assumed reaction or load distributions are illustrated in Figure C-G5-4.

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; Hetrakul and Yu, 1978) and the assumed distributions of loads or reactions acting on the web as shown in Figure C-G5-4. 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.

Figure C-G5-3 Application of Loading Case
Figure C-G5-4 Assumed Distribution of Reaction or Load
Prabakaran (1993) and Prabakaran and Schuster (1998) developed one consistent unified web crippling equation with variable coefficients (Specification Equation G5-1). 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 G5-1 to G5-5 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 G5-1, it can be seen that the nominal web crippling strength [resistance] 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 nondimensional, 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 in the Specification based on section type, as follows:
1) Built-up sections (Table G5-1),
2) Single web channel and C-sections (Table G5-2),
3) Single web Z-sections (Table G5-3),
4) Single hat sections (Table G5-4), and
5) Multi-web deck sections (Table G5-5).

Calibrations were carried out by Beshara and Schuster (2000) in accordance with Supornsilaphachai, Galambos and Yu (1979) to establish the safety factors, \( \Omega \), and the resistance factors, \( \phi \), for each web crippling case. Based on these calibrations, different safety factors and corresponding resistance factors are presented in the web crippling coefficient tables for the particular load case and section type. In 2005, the safety and the resistance factors for built-up and single hat sections with interior one-flange loading case were revised based on a more consistent calibration. For the fastened built-up sections, the factors were revised from 1.65 to 1.75 (for ASD), 0.90 to 0.85 (for LRFD) and 0.80 to 0.75 (for LSD). For the fastened single hat section, the factors were revised from 1.90 to 1.80 (for ASD) and 0.80 to 0.85 (for LRFD). For the unfastened
single hat sections, the factors were revised from 1.70 to 1.80 (for ASD), 0.90 to 0.80 (for LRFD) and 0.75 to 0.70 (for LSD). Also in 2005, the coefficients for built-up sections were revised to remove inconsistencies between unfastened and fastened conditions and to better reflect the calibration for the safety factor and the resistance factors. 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 (Figure C-G5-2(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-G5-2(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-G5-4, 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 nonuniform bearing load distribution on the web. The value of \( P_n \) will vary because of a transition from the interior one-flange loading (Figure C-G5-4(b)) to the end one-flange loading (Figure C-G5-4(a)) condition. These discrete conditions represent the experimental basis on which the design provisions were founded (Winter, 1970; Hetrakul and Yu, 1978). Based on additional updated calibrations, the resistance factor for Canada LSD for the unfastened interior one-flange loading (IOF) case in Specification Table G5-4 was changed from 0.75 to 0.70 in 2004.

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-G5-5(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-G5-5(b) (Bhakta, LaBoube and Yu, 1992).

![Figure C-G5-5 Typical Bolt Pattern for I-Section Test Specimens](image-url)

In Specification Table G5-1, the heading was changed in 2012 to indicate that the resulting nominal web crippling strength [resistance] is per web.

The 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 G5 of the 1996 *Specification*. This is now incorporated under “Fastened to Support” condition.

In 2005, the R/t limit in *Specification* Table G5-3 regarding interior one-flange loading for fastened Z-sections was changed from 5 to 5.5 to achieve consistency with *Specification* Equation H3-3, which stipulates a limit of R/t = 5.5.

For two nested Z-sections, the 1996 *Specification* permitted the use of a slightly different safety factor 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 *Specification* Table G5-3 of the *Specification* under the category of “Fastened to Support” for the interior one-flange loading case.

The coefficients in *Specification* Table G5-4 for one-flange loading or reaction with fastened to support condition are based on those with unfastened to support condition. For consistency, the R/t ratios for unfastened to support condition were revised in 2009 to be the same as the values of fastened to support condition. The table heading was changed to indicate that the resulting nominal web crippling strength [resistance] is per web.

The previous web crippling coefficients in Table G5-5 for end one-flange loading (EOF) of multi-web deck sections in the design provisions (AISI 2001) were based on limited data. This data was based on specimens that were not fastened to the support during testing; hence, the previous coefficients for this case were also being used conservatively for the case of fastened to the support. Based on extensive testing, web crippling coefficients were developed by James A. Wallace (2003) for both the unfastened and fastened cases of EOF loading. Calibrations were also carried out to establish the respective safety factors and resistance factors. The R/t ratio for interior one-flange loading with fastened to support condition was revised in 2012 to be consistent with the corresponding interior one-flange loading value of the unfastened condition. The heading of Table G5-5 was changed to indicate that the resulting nominal web crippling strength [resistance] is per web. A note was also added to the table to indicate that multi-web deck sections are considered unfastened for any support fastener spacing greater than 18 in. (460 mm) (Wallace, 2004).

In 2004, the definitions of “one-flange loading” and “two-flange loading” were revised according to the test setup, specimen lengths, development of web crippling coefficients, and calibration of safety factors and resistance factors. In Figures C-G5-3 and C-G5-4 of the *Commentary*, the distances from the edge of bearing to the end of the member were revised to be consistent with the *Specification*.

*Specification* Equation G5-2 for single web C- and Z-sections with an overhang or overhangs is based on a study of the behavior and resultant failure loads from an end one-flange loading investigation performed at the University of Missouri-Rolla (Holesapple and LaBoube, 2002). This equation is applicable within the limits of the investigation. The UMR test results indicated that in some situations with overhangs, the interior one-flange loading capacity may not be achieved, and the interior one-flange loading condition was therefore removed from Figures C-G5-3 and C-G5-4.

Tests were conducted on fastened to support, stiffened flange, single web 3-1/2 in. (88.9 mm) C-sections subjected to interior two-flange loading or reactions (ITF) that indicate the web crippling equation is unconservative by about 25 percent. Therefore, in 2012, the application of the web crippling equation was limited to a web depth greater than or equal to 4-1/2 in. (110 mm) or more to be consistent with the tests conducted by Schuster and Bashera in 1999. This revision
was based on the web crippling test observations (Yu, 2009 and 2009a).

G6 Web Crippling Strength 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 d_h/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 d_h/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-1.1.3-1 illustrates the L_h and d_h that may be used for a multiple hole pattern that fits within a non-circular virtual hole. Figure C-1.1.3-2 illustrates the d_h 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.
H. MEMBERS UNDER COMBINED FORCES

H1 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 subjected to combined tensile axial load and bending. Since the 2001 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.

H1.1 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 Chapters E, F, Sections I4, I6.1, and I6.2. Care must be taken not to overestimate the tensile load, as this could be unconservative.

Specification Equation H1.1-1 provides a design criterion to prevent yielding of the tension flange of a member under combined tensile axial load and bending. Therefore, the available flexural strengths [factored resistances], $M_{\text{ax,t}}$ and $M_{\text{ay,t}}$, are calculated based on the section modulus of full unreduced section relative to the extreme tension fiber. Specification Equation H1.1-2 provides a design criterion to prevent failure of the compression flange.

H1.2 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 (Peköz, 1986a; Peköz 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 2007, the Specification introduced the second-order analysis, which contained the direct analysis method approach as an optional method for structural stability analysis. In 2016, the Specification was reorganized and it provides three methods of design for system stability: the direct analysis method using rigorous second-order elastic analysis (Section C1.1), the direct analysis method using amplified first-order elastic analysis (Section C1.2) and the effective length method (Section C1.3). Since moment magnifications are considered in the system analysis in accordance with Specification Section C1, Section H1.2 was revised accordingly by deleting the terms relating to moment amplification ($1/\alpha$) and moment gradient ($C_m$) as these effects are now handled in Chapter C.

When a beam-column is subjected to an axial load $P$ and end moments $M$ as shown in Figure C-H1.2-1(a), the combined axial and bending stress in compression is given in Equation C-H1.2-1 as long as the member remains straight:
\[
\bar{f} = \frac{P}{A} + \frac{M}{S} \\
= \bar{f}_a + \bar{f}_b
\]

where
\[
\bar{f} = \text{Combined stress in compression} \\
P = \text{Required axial load determined in accordance with ASD, LRFD or LSD load combinations} \\
A = \text{Cross-sectional area} \\
M = \text{Required bending moment determined in accordance with ASD, LRFD or LSD load combinations} \\
S = \text{Section modulus} \\
\bar{f}_a = \text{Axial compressive stress} \\
\bar{f}_b = \text{Bending stress in compression}
\]

![Figure C-H1.2-1 Beam-Column Subjected to Axial Loads and End Moments](image)

In the design of a beam-column by using the ASD, LRFD or LSD method, the combined stress should be limited by certain available stress \( F_a \); that is,
\[
\bar{f}_a + \bar{f}_b \leq F_a
\]

or
\[
\frac{\bar{f}_a}{F_a} + \frac{\bar{f}_b}{F_a} \leq 1.0
\]  

(C-H1.2-2)

As specified in Sections F2 and F3, I6.1, I6.2 and Chapter E of the Specification, the safety factor or resistance factor for the design of compression members is different from the safety factor or resistance factor for beam design. Therefore, Equation C-H1.2-2 may be modified as follows:
\[
\frac{\bar{f}_a}{F_{a_{axial}}} + \frac{\bar{f}_b}{F_{a_{bending}}} \leq 1.0
\]

(C-H1.2-3)

where
\[
F_{a_{axial}} = \text{Available stress for the design of compression members} \\
F_{b_{bending}} = \text{Available stress for the design of beams}
\]
If the strength ratio is used instead of the stress ratio, Equation C-H1.2-3 can be rewritten as follows:

\[
\frac{\bar{P}}{P_a} + \frac{\bar{M}}{M_a} \leq 1.0
\]  

(C-H1.2-4)

where

\(P_a = \text{Available compressive strength [factored resistance]}\) determined in accordance with Chapter E

\(M_a = \text{Available flexural strength [factored resistance]}\) determined in accordance with Chapter F and Sections I6.1 and I6.2, as applicable.

Equation C-H1.2-4 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-H1.2-1(b) due to the applied moment, \(\bar{M}\), and the secondary moment resulting from the applied axial load, \(\bar{P}\), and the deflection of the member. This is why the increase of moment in the member due to member deformation (P-\(\delta\) effect), and story sway (P-\(\Delta\) effect), as well as initial imperfections, need to be considered in determining member forces. See Section C1 Commentary for further information.

In 2016, the Specification relaxed the requirement that the bending moment (\(\bar{M}\)) should be defined with respect to the centroidal axis of the effective section. The increased eccentricity due to local buckling may exist in an ideally simply-supported member; it becomes minor in continuous members or members with ends restrained so as to reduce such eccentricity. Further, the Direct Strength Method utilized in Chapter F for the available flexural strength [factored resistance], \(M_a\), has shown that accurate bending strength may be determined without consideration of neutral axis shift. In such an approach, the designer does not calculate effective properties or effective axes, and thus it is consistent to remove such a requirement from the beam-column interaction check. Research indicates that use of the gross centroidal axes is adequate for cold-formed steel beam-columns (Torabian, et al. 2013, 2014), and additional work is ongoing.

For the design of angle sections using the ASD, LRFD or LSD method, the required additional bending moment of \(PL/1000\) about the minor principal axis is discussed in Item E of Chapter E of the Commentary.

**H2 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), which is part of a unit circle:

\[
\left(\frac{f_b}{f_{cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1.0
\]  

(C-H2-1)

or
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 shear stiffeners, for which a diagonal tension field action may be developed. Based on the studies made by LaBoube and Yu (1978b), Equation C-H2-3 was developed for beam webs with shear stiffeners satisfying the requirements of Specification Section G4.

\[
0.6 \frac{f_b}{f_{b_{\text{max}}}} + \frac{\tau}{\tau_{\text{max}}} = 1.3
\]  

(C-H2-3)

Equation C-H2-3 was added to the AISI Specification in 1980. The correlations between Equation C-H2-3 and the test results of beam webs having a diagonal tension field action are shown in Figure C-H2-1.

Since 1986, the AISI Specification has used strength ratios (i.e., moment ratio for bending and force ratio for shear) instead of stress ratios for the interaction equations. Specification Equations H2-1 and H2-2 are based on Equations C-H2-2 and C-H2-3, respectively, by using the available flexural strength [factored resistance], \( M_{a/\sigma} \), and the available shear strength [factored resistance], \( V_a \).

The available flexural strength [factored resistance], \( M_{a/\sigma} \), for local buckling from Specification Section F3.1 or F3.2 has been used in the interaction equations since combined bending and shear occur in regions of high moment gradient where distortional buckling is unlikely to play a significant role. Distortional buckling is checked independently in Specification Section F4.
Validation of this approach has been confirmed from tests of lapped *purlins* (Pham and Hancock, 2009b) and tests on high-strength steel C-sections in combined bending and shear (Pham and Hancock, 2012a). However, where tension field action given by *Specification* Equations G2.2-1 and G2.2-2 is used to compute \( V_{ar} \), then *flange* distortion of unrestrained *flanges* requires that *distortional buckling* be considered when computing \( M_{a/o} \) (Pham and Hancock, 2012a).

**H3 Combined Bending and Web Crippling**

This *Specification* contains interaction equations for the combination of bending and *web crippling*. *Specification* Equations H3-1 and H3-2 are based on an evaluation of available experimental data using the *web crippling* equation included in the 2001 edition of the *Specification* (LaBoube, Schuster, and Wallace, 2002). The experimental data is based on research studies conducted at the University of Missouri-Rolla (Hettrakul and Yu, 1978 and 1980; Yu, 1981 and 2000), Cornell University (Winter and Pian, 1946), and the University of Sydney (Young and Hancock, 2000). For embossed *webs*, *crippling* strength should be determined by tests according to *Specification* Section K2.

The exception clause included in *Specification* Section H3 for single unreinforced *webs* applies to the interior supports of continuous spans using decks and beams, as shown in Figure C-H3-1. Results of continuous beam tests of steel decks (Yu, 1981) and several independent studies by manufacturers indicate that, for these types of members, the post-*buckling* behavior of *webs* at interior supports differs from the type of failure mode occurring under concentrated *loads* on single-span beams. This post-*buckling* strength enables the member to redistribute the moments in continuous spans. For this reason, *Specification* Equation H3-1 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-H3-1 and similar situations explicitly described in *Specification* Section H3.

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 ensure 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 behavior described above, the designer is urged to use Specification Equation H3-1.

In 1996, additional design information was added to Specification Section H3(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 2003, based on the test data of LaBoube, Nunnery, and Hodges (1994), the interaction equation for the combined effects of bending and web crippling was reevaluated because a new web crippling equation was adopted for Section G5 of the Specification.

In the development of the original LRFD equations, a total of 551 tests were calibrated for combined bending and web crippling strength. Based on $\phi_w = 0.75$ for single unreinforced webs and $\phi_w = 0.80$ for I-sections, the values of the reliability index vary from 2.5 to 3.3 as summarized in the AISI Commentary (AISI, 1991).

### H4 Combined Bending and Torsional Loading

When the transverse loads applied to a bending member do not pass through the shear center of the cross-section of the member, twisting and torsional stresses can develop. The torsional stresses consist of pure torsional shear stresses, shear stresses due to warping, and normal stresses due to warping. References, such as the AISC Steel Design Guide “Torsional Analysis of Structural Steel Members” (AISC, 1997a), describe the effect of torsion and how these stresses may be calculated.

Open cold-formed steel sections have little resistance to torsion, thus severe twisting and large stresses can develop. In many situations, however, the connection between a beam and the member delivering the load to the beam is such that it constrains twisting and in effect causes the resultant load to act as though it is delivered through the shear center. In such cases the torsional effects do not occur. Positive connections between the load-bearing flange and supported elements, in general, prevent torsional effects. An example of this is a purlin supporting a through-fastened roof panel that will prevent movement in the plane of the roof panel. It is important that the designer ensure that torsion is adequately constrained when evaluating a specific situation.

In situations where torsional loading cannot be avoided, discrete bracing will reduce the torsional effects. For most situations, the maximum torsional warping stresses will occur at discrete brace locations. Torsional bracing at the third points of the span would be adequate for most light construction applications. The bracing should be designed to prevent torsional twisting at the braced points.

Specification Section H4 provides design criteria for a singly- or doubly-symmetric member that is subjected to torsional loading. The provision uses a reduction factor, $R$, to reduce the
nominal moment strength [resistance] as determined by Specification Section F3 with \( F_n = F_y \) or \( M_{ne} = M_y \). This factor accounts for the normal stresses due to combined torsional warping and flexure. In 2012, the R factor was revised to accommodate situations where the maximum stress due to combined bending and torsional warping occurs at the tip of the flange stiffener. This R factor requires calculation of both the bending only stresses and the torsional warping stresses at critical points on the cross-section. The largest combination of these is the denominator of the reduction factor while the bending stress alone at the extreme fiber is the numerator. The member is then selected based on bending alone with the effect of torsion accounted for by the reduction in the nominal moment strength [resistance].

The largest combined stresses on the cross-section may occur at the junction of the web and flange, at the junction of the edge of flange and flange stiffener, or at the tip of the flange stiffener. The second and third conditions have a more severe effect on reducing the moment capacity of the member. These conditions can occur when the applied load is positioned off the member away from both the web and the shear center. This is shown from the test results reported in the referenced paper by Put, et al. (1999). This is not a practical situation for structural assemblies; however, this location of the critical stresses would occur at the position of a torsional brace located at midspan of a member. To allow for the more favorable situation, the provisions of Specification Section H4 permit the nominal moment strength [resistance] to be increased by 15 percent when the critical combination of stresses occurs at the junction of the flange and web. This is supported by tests on channels conducted by Winter, et al. (1950), which indicated that an overstress of 15 percent did not significantly affect the load-carrying capacity.

Rational engineering analysis should be used for sections, such as point-symmetric and non-symmetric sections, that are not covered by Specification Section H4. For these members, combined flexural with torsional warping stresses should be checked at both maximum tensile and compressive stress locations. A reasonable method would be to limit the combined bending stress and torsional warping stress to an allowable stress or factored stress using safety factors or resistance factors, respectively, provided in Specification F3, where \( F_n = F_y \) or \( M_{ne} = M_y \). Any location on the cross-section that may control design should be considered.

The provisions of this section are intended as a separate limit state for available flexural strength [factored resistance]. It is still necessary to check the other limit states listed in Specification Sections F2 through F4, but those limit states are calculated without the torsional R factor. In addition, the R factor is excluded from all interaction checks involving flexure such as combined bending and shear (Specification Section H2), combined bending and web crippling (Specification Section H3), and combined axial load and bending (Specification Section H1).
I. ASSEMBLIES AND SYSTEMS

I1  Built-Up Sections

I-sections made by connecting two C-sections back-to-back are one type of built-up section that is often used as either flexural or compression members. Cases (2) and (8) of Figure C-A1.3-2 and Cases (3) and (7) of Figure C-A1.3-3 show several built-up I-sections. For built-up flexural members, the Specification is limited to two back-to-back C-sections. For built-up compression members, other sections can be used.

I1.1  Flexural Members Composed of Two Back-to-Back C-Sections

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Equation I1.1-1 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 requirement is based on the strength and arrangement of connectors and the intensity of the load acting on the beam (Yu and LaBoube, 2010).

The second requirement for maximum spacing of connectors required by Specification Equation I1.1-1 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-I1.1-1. The tensile force of the top connector, Ts, can then be computed from the equality of the twisting moment, Qm, and the resisting moment, Tsg, that is:

\[ Qm = Tsg \]  \hspace{1cm} (C-I1.1-1)

\[ Ts = \frac{Qm}{g} \]  \hspace{1cm} (C-I1.1-2)

Figure C-I1.1-1 Tensile Force Developed in the Connector for C-Section

Considering that q is the intensity of the load and that s is the spacing of connectors as shown in Figure C-II.1-2, the applied load is \( Q = qs/2 \). The maximum spacing, \( s_{\text{max}} \), used in the Specification can easily be obtained by substituting the above value of Q into Equation C-II.1-2 of this Commentary. The determination of the load intensity, q, is based upon the type of loading applied to the beam. The requirement of three times the uniformly distributed load is applied to reflect that the assumed uniform load will not really be uniform. The Specification prescribes a conservative estimate of the applied loading to account for the likely concentration of loads near the welds or other connectors that join the two C-sections.
For simple C-sections without stiffening lips at the outer edges,

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

(C-I1.1-3)

For C-sections with stiffening lips at the outer edges,

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

(C-I1.1-4)

where

- \( w_f \) = Projection of flanges from the inside face of the web (for C-sections with flanges of unequal width, \( w_f \) should 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 [force due to 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 \( s_{\text{max}} = L/6 \) is the first requirement in Specification Equation I1.1-1.

1.2 Compression Members Composed of Two Sections in Contact

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:

\[
\left( \frac{KL}{r} \right)_m = \sqrt{\left( \frac{KL}{r} \right)_o^2 + \left( \frac{a}{r_1} \right)^2}
\]

(C-I1.2-1)

Note that in this expression, the overall slenderness ratio, \( (KL/r)_o \), is computed about the
same axis as the modified slenderness ratio, \((KL/r)_{m}\). Further, the modified slenderness ratio, \((KL/r)_{m}\), replaces \(KL/r\) in Specification Chapter E for both flexural and flexural-torsional buckling. This modified slenderness approach is used in other steel standards, including the AISC (AISC, 1999, 2005 and 2010a), 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 \(a/r_i \leq 0.5(KL/r)_o\). 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 II.2(b), adopted from AISC (AISC, 1999) and CAN/CSA S16.1 (CAN/CSA S16.1-94, 1994). These provisions were added to the North American Specification in 2001.

The intermediate fastener(s) or weld(s) at any longitudinal member tie location is required, as a group, to transmit a force equal to 2.5 percent of the nominal axial strength [resistance] of the built-up member. A longitudinal member tie is defined as a location of interconnection of the two members in contact. In the 2001 edition of the Specification, a 2.5 percent total force determined in accordance with appropriate load combinations was used for design of the intermediate fastener(s) or weld(s). This requirement was adopted from CSA S136-94. In 2004, the requirement was changed to be a function of the nominal axial strength [resistance]. This change ensures that the nominal axial strength [resistance] of the built-up member is valid and is not compromised by the strength of the member interconnections. To avoid confusion for different design methods, the minimum required strength [force due to factored loads] of the interconnection changed to 2.5 percent of the available strength [factored resistance] of the built-up member.

Note that the provision in Specification Section II.2 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 percent requirement for shear strength and the conservative spacing requirement \(a/r_i \leq 0.5(KL/r)_o\).

### I1.3 Spacing of Connections in Cover-Plated Sections

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 of the connected element. Figure C-I1.3-1 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 II.3 of the Specification.

Section I1.3(a) of the Specification requires that the necessary shear strength 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 I1.3(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-I1.3-1) at a stress less than $1.67f_c$ for ASD and $f_c$ for LRFD and LSD, where $f_c$ is the compressive stress in the connected compression element (Winter, 1970; Yu and LaBoube, 2010). The AISI requirement is based on the following Euler equation for column buckling:

$$\sigma_{cr} = \frac{\pi^2E}{(KL/r)^2}$$

by substituting $\sigma_{cr} = \alpha f_c$, where $\alpha = 1.67$ for ASD and $\alpha = 1.0$ for LRFD or LSD, $K = 0.6$, $L = s$, 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 the theoretical value for a column with fixed end supports.

![Figure C-I1.3-1 Spacing of Connectors in Composite Section](image)

Section I1.3(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 and LaBoube, 2010).

Specification Section 1.1.4 extends the limits of this section and uses the post-buckling strength of the edge-stiffened compression plate. Specification Section 1.1.4 specifies the parameter ranges that are validated by the research (Luttrell and Balaji, 1992; Snow and Easterling, 2008).

I2 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 bending strength 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 not only provide enclosure surface and support normal loads, but they can also provide diaphragm action in their own planes.

With the publication of AISI S310, North American Standard for the Design of Profiled Steel Diaphragm Panels, the provisions in Specification Section I2 have moved to AISI S310. See AISI S310-C for background information on floor, roof and wall steel diaphragm construction. See AISI S240 and AISI S400 for information on the design and construction of cold-formed steel framing with diagonal bracing or covered with sheathings other than fluted panels or cellular deck.
I3 Mixed Systems

When cold-formed steel members are used in conjunction with other construction materials, the design requirements of the other material specifications must also be satisfied.

I4 Cold-Formed Steel Light-Frame Construction

In 2007, the scope of Section I4 on “Wall Studs and Wall Stud Assemblies” of the 2001 edition of the Specification with 2004 Supplement was broadened to include light-frame construction. This was done in order to recognize the growing use of cold-formed steel framing in a broader range of residential and light commercial framing applications and to provide a means for either requiring or accepting use of the various ANSI-approved standards that have been developed by the AISI Committee on Framing Standards.

In 2012, the reference to nonstructural members was removed from Section I4 because the provisions for nonstructural members were moved from AISI S200, North American Standard for Cold-Formed Steel Framing - General Provisions, to the newly developed AISI S220, North American Standard for Cold-Formed Steel Framing – Nonstructural Members.

In 2016, the provisions for the design and installation of structural members and connections utilized in cold-formed steel light-frame construction applications were consolidated in AISI S240, North American Standard for Cold-Formed Steel Structural Framing, from the following previously referenced standards:

(a) AISI S200, North American Standard for Cold-Formed Steel Framing – General Provisions
(b) AISI S210, North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design
(c) AISI S211, North American Standard for Cold-Formed Steel Framing – Wall Stud Design
(d) AISI S212, North American Standard for Cold-Formed Steel Framing – Header Design
(e) AISI S213, North American Standard for Cold-Formed Steel Framing – Lateral Design
(f) AISI S214, North American Standard for Cold-Formed Steel Framing – Truss Design

In 2016, AISI S400 was developed to address the design and construction of cold-formed steel structural members and connections in seismic force-resisting systems and diaphragms in buildings and other structures. AISI S400 is applicable in the United States and Mexico in Seismic Design Categories (SDC) D, E, or F, or in SDC B or C with seismic response modification coefficient, $R$, used to determine the seismic design forces is taken as other than 3; and in Canada where the design spectral response acceleration $S(0.2)$ as specified in the NBCC is greater than 0.12 and the seismic force modification factors, $R_dR_o$, used to determine the seismic design forces, are taken as greater than or equal to 1.56.

AISI S220, AISI S240 and AISI S400 are available for adoption and use in the United States, Canada and Mexico, and provide an integrated treatment of Allowable Strength Design (ASD), Load and Resistance Factor Design (LRFD), and Limit States Design (LSD). These framing standards do not preclude the use of other materials, assemblies, structures or designs not meeting the criteria herein when the other materials, assemblies, structures or designs demonstrate equivalent performance for the intended use to those specified in the standards.

I4.1 All-Steel Design of Wall Stud Assemblies

It is well known that column strength 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 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 Peköz (1976), which are summarized by Yu (2000), contain procedures for computing the strength 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 Peköz (1976) is quite general and includes the case of studs braced on one as well as on both flanges. However, the provisions of Section 14 of the 1980 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 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 and integrity for the expected service life of the wall. Of particular concern is the use of gypsum sheathing in a moist environment.

In 2004, the sheathing-braced design provisions were removed from the Specification and a requirement added that sheathing-braced design be based on appropriate theory, tests, or rational engineering analysis that can be found in AISI (2004a); Green, Winter, and Cuykendall (1947); Simaan (1973); and Simaan and Peköz (1976).

In 2007, in addition to the revisions of Specification Section 14 as discussed in this Commentary, the provisions for noncircular holes were moved from Specification Section 14.1 to Section 1.1.1 on “Uniformly Compressed Stiffened Elements With Circular or Noncircular Holes”. Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs.

### I5 Special Bolted Moment Frame Systems

In 2015, AISI S110, Standard for Seismic Design of Cold-Formed Steel Structural Systems - Special Bolted Moment Frames, was incorporated into AISI S400.
I6 Metal Roof and Wall Systems

For members connected to deck or metal sheathing, the member flexural and compression strengths as well as bracing requirements are provided in Specification Section I6. Two strength prediction methods are provided—one for general cross-sections and system connectivity (Section I6.1), and one for specific cross-sections and system connectivity (Section I6.2). The provisions in Specification Section I6.1 directly calculate member capacity, including stiffness from connected roof or wall panels, bridging and bracing, span continuity, and torsion from loading eccentric to the shear center and from roof slope. The provisions in Specification Section I6.2 define wall and roof system capacity based on past experiments for variables within defined limits.

I6.1 Member Strength: General Cross-Sections and System Connectivity

This method provides a means for directly calculating the axial and flexural capacity of members (such as purlins and girts) connected to deck, sheathing, or through-fastened or standing seam panels. The approach employs the Direct Strength Method and available computational tools; for example, the finite strip elastic buckling program CUFSM (Li and Schafer, 2010).

An elastic buckling analysis is performed that includes the test-derived rotational, translational, and composite stiffness provided by the panel or sheathing connection to the members (Schafer, 2013; Gao and Moen, 2013a). The member critical elastic local, distortional, and global buckling loads or moments are calculated considering wall or roof connection stiffness, end support conditions, span continuity, and bridging and bracing. Member slenderness, including the wall or roof system influence, is determined within the Direct Strength Method to predict axial or flexural capacity.

Panel, deck, and sheathing rotational and translational stiffnesses are available for bare deck through-fastened to members (Gao and Moen, 2012; Pham et al., 2016), deck with rigid board insulation (Gao, 2012), and for through-fastened and standing seam insulated metal panels (IMPs) (Wu and Moen, 2015). Composite stiffness developed by the connection between the panel and a member can also be approximated (Vieira, 2011).

In many cases, the applied load on a member is eccentric to its shear center from forces applied through the flange connection or because of a sloped roof. The warping torsion stresses are directly calculated in these cases and a reduction factor, R, determined in accordance with Specification Equation H4-1, is used to reduce the capacity for combined flexure and torsion. The reduction factor should be applied to nominal strength [resistance], \( M_{n} \), considering local, distortional, and global buckling limit states.

The method described above can be applied to members with, generally, any cross-section and system connectivity. Supporting documentation for this method applied to metal building wall and roof systems comes from experimental, computational, and analytical studies conducted between 2009 and 2015, including Gao and Moen (2013a and 2013b). Example calculations are available for many of these systems (Moen, 2015), including standing seam roofs (Moen, et al., 2012).

The design methodology for general cross-sections and system connectivity has been thoroughly validated. The strength predictions were compared to a database of 62 through-fastened roof and wall tests containing the same experiments that form the basis for the provisions of Section I6.2. The test-to-predicted mean and coefficient of variance (COV) for
this database comparison is 1.05 and 0.18, respectively, corresponding to an LRFD resistance factor of 0.90. Extensive validation also exists for sheathed cold-formed steel framing (Vieira, 2011).

I6.2 Member Strength: Specific Cross-Sections and System Connectivity

I6.2.1 Flexural Members 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 (Pekőz 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 stress limit 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 I6.2.1 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, \( M = 0.08 \, W L^2 \).

The provisions of Specification Section I6.2.1 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 (Pekőz 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 I6.2.1-1) 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).

In 2007, the panel depth was reduced from 1-1/4 inch (32 mm) to 1-1/8 inch (29 mm). This reduction in depth was justified because the behavior during full-scale tests indicated that the panel deformation was restricted to a relatively small area around the screw attachment of the panel to the purlin. Also, tests by LaBoube (1986) demonstrated that the panel depth did not influence the rotational stiffness of the panel-to-purlin attachment.
Prior to the 2001 edition, the Specification specifically limited the applicability of these provisions to continuous purlin and girt systems in which any given span length did not vary from any other span length by more than 20 percent. This limitation was included in recognition of the fact that the research was based on systems with equal bay spacing. In 2007, the Specification was revised to permit purlin and girt systems with adjacent span lengths varying more than 20 percent to use the reduction factor, R, for the simply supported condition. The revision allows a row of continuous purlins or girts to be treated with a continuous beam condition R-factor in some bays and a simple span beam condition R-factor in others. The 20 percent span variation rule is a local effect and as such, only variation in adjacent spans is relevant.

In 2012, based on tests reported by Wibbenmeyer (2009), the limitation on the member depth was increased to 12 in. (305 mm), the ratio of depth-to-flange width was increased to 5.5, and a minimum flange width of 2.125 in. (54.0 mm) was added. The ratio of tensile strength to yield stress of 1.08 was added based on research at the University of Sydney (Pham and Hancock, 2009), which is also consistent with the applicable steels listed in Specification Section A2. The average depth-to-flange width ratio based on measured properties in the research by Wibbenmeyer (2009) was 5.3. However, the limit was increased to 5.5 in the Specification. This increased value was justified because the smallest measured purlin flange width for any of the members tested by Wibbenmeyer (2009) was 2.1875 in. (71.56 mm), which resulted in a ratio of depth-to-flange width of 5.5. Also, the reported value of R for the 12-in. (305-mm) deep purlins significantly exceeded those previously stipulated for 11.5-in. (292-mm) deep members.

The provisions of Specification Section H4, Combined Bending and Torsion, should not be used in combination with the bending provisions in Specification Section I6.2.1 since these provisions are based on tests in which torsional effects are present.

I6.2.2 Flexural Members 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 Appendix A.

I6.2.3 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 I6.2.3-1 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 stress of the C- or Z-section is not given in the Specification since Specification Equation I6.2.3-1 is based on elastic buckling criteria. A limitation on minimum length is not contained in the Specification because Equation I6.2.3-1 is conservative for spans less than 15 feet. The gross area, A, has been used rather than the effective area, A_eff, because the ultimate axial stress is generally not large enough to result in a significant reduction in the effective area for common cross-section geometries.
As indicated in the Specification, the strong axis axial load capacity is determined by 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 H1 of the Specification (Hatch, Easterling, and Murray, 1990).

Note: As stated in the Specification, when a member is designed in accordance with Section I6.2.3, Compression Members Having One Flange Through-Fastened to Deck or Sheathing, the provisions of Section E4.1, Distortional Buckling Strength [Resistance], need not be applied since distortional buckling is inherently included as a limit state in Section I6.2.3 on strength prediction equations.

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

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 Appendix A.

I6.3 Standing Seam Roof Panel Systems

I6.3.1 Strength [Resistance] of Standing Seam Roof Panel Systems

Under gravity loading, the nominal strength [resistance] of many panels can be calculated accurately. Under uplift loading, nominal strength [resistance] of standing seam roof panels and their attachments or anchors cannot be calculated with accuracy. Therefore, it is necessary to determine the nominal strength [resistance] by testing. Three test protocols have been used in this effort: FM 4471 developed by Factory Mutual, CEGS 07416 by the U.S. Army Corps of Engineers and ASTM E1592. In Supplement No. 1 to the 1996 edition of the Specification, (AISI, 1999), only the ASTM E1592-95 procedure was approved. In 2004, the Factory Mutual and Corps of Engineers protocols were also approved, provided that testing was in accordance with the AISI test procedure defined in S906 (AISI, 2002). While these test procedures have a common base, none define a design strength [factored resistance]. Specification Section I6.3.1 and AISI S906, Standard Procedures for Panel and Anchor Structural Tests, adopted in 1999, added closure to the question by defining appropriate resistance and safety factors. The safety factors determined in Section I6.3.1 will vary depending on the characteristics of the test data. In 2006, limits were placed on the safety factor and resistance factor determined in this section to require a minimum safety factor of 1.67 and a maximum resistance factor of 0.9.

The Specification permits end conditions other than those prescribed by ASTM E1592-01. Areas of the roof plane that are sufficiently far enough away from crosswise restraint can be simulated by testing the open/open condition that was permitted in the 1995 edition of ASTM E1592. In addition, eave and ridge configurations that do not provide crosswise restraint can be evaluated.

The relationship of strength to serviceability limits may be taken as strength limit/serviceability limit = 1.25, or

\[ \Omega_{\text{serviceability}} = \frac{\Omega_{\text{strength}}}{1.25} \]  

(C-I6.3.1-1)

It should be noted that the purpose of the test procedure specified in Specification Section I6.3.1 is not to set up guidelines to establish the serviceability limit. The purpose is to define the method of determining the available strength [factored resistance] whether based
on the serviceability limit or on the nominal strength [resistance]. The Corps of Engineers Procedure CEGS 07416 (1991) requires a safety factor of 1.65 on strength 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, $\beta_o$, 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 I6.3.1, a target reliability index of 2.5 is used for connection limits. It is used because the consequences of a panel fastener failure ($\beta_o = 2.5$) are not nearly as severe as the consequences of a primary frame connection failure ($\beta_o = 3.5$). The intermittent nature of wind load as compared to the relatively long duration of snow load further justifies the use of $\beta_o = 2.5$ for panel anchors. In Specification Section I6.3.1, the coefficient of variation of the material factor, $V_M$, 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 I6.3.1 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 percent value set in Specification Section K2.1. 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.

Commentary for load combinations including wind uplift is provided in Appendix A.

I6.4 Roof System Bracing and Anchorage

I6.4.1 Anchorage of Bracing for Purlin Roof Systems Under Gravity Load With Top Flange Connected to Metal Sheathing

In metal roof systems utilizing C- or Z-purlins, the application of gravity loads will cause torsion in the purlin and lateral displacements of the roof system. These effects are due to the slope of the roof, the loading of the member eccentric to its shear center, and for Z-purlins, the inclination of the principal axes. The torsional effects are not accounted for in the design provisions of Chapter F, Sections I6.1 and I6.2; and lateral displacements may create instability in the system. Lateral restraint is typically provided by the roof sheathing and lateral anchorage devices to minimize the lateral movement and the torsional effects. The anchorage devices are designed to resist the lateral anchorage force and provide the appropriate level of stiffness to ensure the overall stability of the purlins.

The calculation procedure in Specification Equations I6.4.1-1 through I6.4.1-6 determines the anchorage force by first calculating an upper bound force for each purlin, $P_i$, at the line of anchorage. This upper bound force is then distributed to anchorage devices and reduced due to the system stiffness based on the relative effective stiffness of each component. For the calculation procedure, the anchorage devices are modeled as linear springs located at the top of the purlin web. The stiffness of anchorage devices that do not attach at this location must be adjusted, through analysis or testing, to an equivalent lateral stiffness at...
the top of the web. This adjustment must include the influence of the attached purlin but not include any reduction due to the flexibility of the sheathing to purlin connection. Specification Equation I6.4.1-4 establishes an effective lateral stiffness for each anchorage device, relative to each purlin, that has been adjusted for the flexibility of the roof system between the purlin location and the anchorage location. It is important to note that the units of $A_p$ are area per unit width. Therefore the bay length, $L$, in this equation must have units consistent with the unit width used for establishing $A_p$. The resulting product, $L A_p$, has units of area. The total effective stiffness for a given purlin is then calculated with Specification Equation I6.4.1-5 by summarizing the effective stiffness relative to each anchorage device and the system stiffness from Specification Equation I6.4.1-6. The force generated by an individual purlin is calculated by Equation I6.4.1-2, and then distributed to an anchorage device based on the relative stiffness ratio in Specification Equation I6.4.1-1.

Lateral bracing forces will accumulate within the roof sheathing and must be transferred into the anchorage devices. The strength of the elements in this load path must be verified. AISI S912, Test Procedures for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection, provides a means to determine a lower bound strength for the complete load path. For through-fastened roof systems, this strength value can be reasonably estimated by rational analysis by assuming that the roof fasteners within 12 inches (305 mm) of the anchorage device participate in the force transfer.

The 1986 through 2001 Specifications included brace force equations that were based on the work by Murray and Elhouar (1985) with various extensions from subsequent work. The original work assumed the applied loading was parallel to the purlin webs. The later addition of the “cos$\theta$” and “sin$\theta$” terms attempted to account for the roof slope, but it failed to correctly model the system effect for higher-sloped roofs. Tests by Lee and Murray (2001) and Seek and Murray (2004) showed generally that the brace force equations conservatively predicted the lateral anchorage forces at slopes less than 1:12, but predicted unconservative lateral anchorage forces at steeper slopes. The new procedure outlined in Specification Section I6.4.1 was formulated to correlate better with test results. Also, the original work was based on the application of one anchorage device to a group of purlins. Until the work of Sears and Murray (2007), a generally accepted manual technique to extend this procedure to roofs with multiple anchors was not available.

Prior to the work by Seek and Murray (2006, 2007) and Sears and Murray (2007), the anchorage devices were assumed to have a constant and relatively high lateral stiffness. The current provisions recognize the finite stiffness of the anchorage device, and the corresponding decrease in anchorage forces for more flexible anchorage devices. Specification Equation I6.4.1-7 establishes a minimum effective stiffness that must be provided to limit the lateral displacement at the anchorage device to $d/20$. This required stiffness does not represent the required stiffness of each anchorage device, but instead the total stiffness provided by the stiffness of the purlin system ($K_{sys}$) and the anchorage devices relative to the most remote purlin.

Several alternative rational analysis methods have been developed to predict lateral anchorage forces for Z-section roof systems. A method for calculating lateral anchorage forces is presented by Seek and Murray (2006, 2007). The method is similar to the procedure outlined in Specification Section I6.4.1 but uses a more complex method derived from mechanics to determine the lateral force introduced into the system at each Z-section, $P_i$, and distributes the force to the components of the system according to the relative
lateral stiffness of each of the components. The method is more computationally intensive, but allows for analysis of more complex bracing configurations such as supports plus third points lateral anchorage and supports plus third points torsional braces.

A method to predict lateral anchorage forces using the finite element method is presented in Seek and Murray (2004). The model uses shell finite elements to model the Z-sections and sheathing in the roof system. The model accurately represents Z-section behavior and is capable of handling configurations other than lateral anchorage applied at the top flange. However, the computational complexity limits the size of the roof system that can be modeled by this method.

Rational analysis may also be performed using the elastic stiffness model developed by Sears and Murray (2007) upon which the provisions of Specification Section I6.4.1 are based. The model uses frame finite elements to represent the Z-sections and a truss system to represent the diaphragm. The model is computationally efficient, allowing for analysis of large systems.

Anchorage is most commonly applied along the frame lines due to the effectiveness and ease in which the forces are transferred out of the system. In the absence of substantial diaphragm stiffness, anchorage may be required along the interior of the span to prevent large lateral displacements. Torsional braces applied along the span of a Z- or C-section provide an alternative to interior anchorage.

### I6.4.2 Alternative Lateral and Stability Bracing for Purlin Roof Systems

Tests (Shadravan and Ramseyer, 2007) have shown that C- and Z-sections can reach the capacity determined by Specification Chapter F through the application of torsional braces along the span of the member. Torsional braces applied between pairs of purlins prevent twist of the section at a discrete location. The moments developed due to the torsional brace can be resolved by forces in the plane of the web of each section and do not require external anchorage at the location of the brace. The vertical forces should, however, be accounted for when determining the applied load on the section.

Torsional braces should be applied at or near each flange of the Z- or C-section to prevent deformation of the web of the section and ensure the effectiveness of the brace. When twist of the section is thus prevented, a section may deflect laterally and retain its strength. Second-order moments can be resisted by the rotational restraints. Therefore, a more liberal lateral deflection of L/180 between the supports is permitted for a C- or Z-section with torsional braces. Anchorage is required at the frame line to prevent excessive deformation at the support location that undermines the strength of the section. A lateral displacement limit, therefore, is imposed along the frame lines to ensure that adequate restraint is provided.

### 17 Rack Systems

Steel rack systems are designed and constructed in accordance with ANSI MH16.1. See the commentary on MH16.1 for information.
J. CONNECTIONS AND JOINTS

J1 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 2016 edition of the Specification contains provisions in Chapter J for welded connections, bolted connections, screw connections, and power–actuated fastener connections. Among these commonly used types of connections, the design provisions for using screws were developed in 1993 and were included in the 1996 Specification for the first time, and the design provisions for power–actuated fasteners were added in the 2012 Specification. The following brief discussions deal with the application 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 of rivets may be quite different from that of bolts. Additional design information on the strength 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 of connections must be obtained from manufacturers or from tests carried out by or for the user. Guidelines provided in Specification Section K2 are to be used in these tests.

The plans or specifications are to contain information and design requirement data for the adequate detailing of each connection if the connection is not detailed on the engineering design drawings.

In the 2001 edition of the Specification, the ASD, LRFD and LSD design provisions for welded and bolted connections were based on the 1996 edition of the Specification, with some revisions and additions which will be discussed in subsequent sections. Most of those design provisions were kept in this edition of the Specification. Some content reorganization was made in 2010, where shear rupture check for welds and fasteners was moved to Section J6.

J2 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 Peköz and McGuire (1979) and summarized by Yu and LaBoube (2010). 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. All possible failure modes are covered in the *Specification* since 1996, whereas the earlier *Specifications* mainly dealt with shear failure.

For most of the *connection* tests reported by Peköz 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 J2.1 through J2.7, were conducted on sections with single and double sheets (see *Specification* Figures J2.2-1 and J2.2-2). 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 3/16 inch (4.76 mm) or less. For arc spot welds, the maximum *thickness* of a single sheet (*Specification* Figure J2.2.2.1-1) and the combined *thickness* of double sheets (*Specification* Figure J2.2.2.1-2) are set at 0.15 inch (3.81 mm).

The upper limit of the *Specification* applicability was revised in 2004 from 0.18 in. (4.57 mm) to 3/16 in. (4.76 mm). This change was made to be consistent with the limit given in AWS D1.3 (1998).

In 2001, the *safety factors* 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 *Cold-Formed Steel Design Manual* (AISI, 2013).

### J2.1 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.

In 2010, *Specification* Section J2.1(a) was revised to delete the case for tension or compression parallel to the axis of the weld, so that *Specification* Equation J2.1-1 is applicable only to tension or compression normal to the *effective area* of the weld. For tension or compression parallel to the weld axis, the computation of the weld strength is not required (AISC, 2005 and 2010a).

### J2.2 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 pre-punched holes, but no pre-punched holes are required for the latter. 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 J2.2 apply to plug welds as well as spot welds.
J2.2.1 Minimum Edge and End Distance

In the 2001 and 2007 editions of the Specification, the distance measured in the line of force from the centerline of weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed was required to not be less than \( e_{\text{min}} \) which is equal to required strength \([\text{forces due to factored loads]}\) divided by \((tF_u)\). In 2010, an equivalent resistance was determined by the use of Section J6.1.

J2.2.2 Shear

J2.2.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member

The Cornell tests (Peköz 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 the 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-J2.2.2.1-1. 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 J2.2.2.1-2, consideration should also be given to possible shear failure between thin sheets.

Figure C-J2.2.2.1-1 Out-of-Plane Distortion of Welded Connection

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.

Specification Equation J2.2.2.1-1 shows that the nominal shear strength \([\text{resistance}]\) of arc spot welds is proportional to the square of effective diameter, \(d_e\), of fused area at plane of maximum shear transfer. Since \(d_e = 0.7d-1.5t \leq 0.55d\) in accordance with Specification Equation J2.2.2.1-5, a larger visible diameter, \(d\), may be needed if the welded sheet thickness, \(t\), is increased.
J2.2.2.2 Shear Strength for Sheet-to-Sheet Connections

The Steel Deck Institute (SDI) Diaphragm Design Manual (SDI, 1987 and 2004) stipulates that the shear strength for a sheet-to-sheet arc spot weld connection be taken as 75 percent of the strength of a sheet-to-structural connection. SDI further stipulates that the sheet-to-structural connection strength be defined by Specification Equation J2.2.2.1-2. This design provision was adopted by the Specification in 2004. Prior to accepting the SDI design recommendation, a review of the pertinent research by Luttrell (SDI, 1987) was performed by LaBoube (2001). The tested sheet thickness range that is reflected in the Specification documents is based on the scope of Luttrell’s test program. SDI suggests that sheet-to-sheet welds are problematic for thicknesses of less than 0.0295 in. (0.75 mm). Such welds result in “blow holes,” but the perimeter must be fused to be effective.

Quality control for sheet-to-sheet connections is not within the purview of AWS D1.3. However, using AWS D1.3 as a guide, the following quality control/assurance guidelines are suggested:

1. Measure the visible diameter of the weld face,
2. Ensure no cracks in the welds,
3. Maximum undercut = 1/8 of the weld circumference, and
4. Sheets are to be in contact with each other.

J2.2.3 Tension

For tensile capacity of arc spot welds, the design provisions in the 1989 Specification 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 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 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 (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 safety factors and resistance factors recognize the behavior of a panel system with many connections versus the behavior of a member connection and the potential for a catastrophic failure in each application. In Specification Section J2.2.3, 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 I6.3.1 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 nominal strength [resistance] given by Specification Equation J2.2.3-2 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 as depicted by Figure C-J2.2.3-1.

At a lap *connection* between two deck sections as shown in Figure C-J2.2.3-1, 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 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]*.

**J2.2.4 Combined Shear and Tension on an Arc Spot Weld**

The Steel Deck Institute *Diaphragm Design Manual* (2004) provides a design equation for evaluating the strength of an arc spot weld *connection* subject to combined shear and tension forces. An experimental investigation was conducted at the University of Missouri–Rolla to study the behavior and to develop design recommendations for the relationship (interaction) of the tension and shear forces on an arc spot weld *connection* (Stirnemann and LaBoube, 2007).

The experimental study focused on six variables that were deemed to be the key parameters that could influence the strength of the arc spot weld *connection*. These variables were the sheet *thickness*; sheet material properties including *yield stress*, *tensile strength* and ductility of the sheet; visible diameter of the arc spot weld; and the relationship between the magnitude of the shear force and tension force. Based on an analysis of the test results, the Steel Deck Institute’s interaction equation was found to provide an acceptable estimate of the strength of the arc spot weld *connection*.

**J2.3 Arc Seam Welds**

The general behavior of arc seam welds is similar to that of arc spot welds. In 2010, Section J2.3 was reorganized to be consistent with provisions provided for arc spot welds.

**J2.3.2 Shear**

**J2.3.2.1 Shear Strength for Sheet(s) Welded to a Thicker Supporting Member**

No simple shear failures of arc seam welds were observed in the Cornell tests (Peköz
and McGuire, 1979). Therefore, Specification Equation J2.3.2.1-1, which accounts for shear failure of welds, is adopted from the AWS welding provisions for sheet steel (AWS, 1998).

Specification Equation J2.3.2.1-2 is intended to prevent failure through a combination of tensile tearing plus shearing of the cover plates.

**J2.3.2.2 Shear Strength for Sheet-to-Sheet Connections**

In 2010, the provisions for determining the shear strength of sheet-to-sheet arc spot weld connections were adopted for arc seam weld connections. This is conservative because the length of the seam weld is not considered.

**J2.4 Top Arc Seam Sidelap Welds**

*Top arc seam sidelap welds* (often referred to as TSWs) have commonly been used to attach the edges of standing seam steel roof and floor deck panels, particularly those used for diaphragms. The *top arc seam sidelap weld connection* is formed by a vertical sheet leg (edge stiffener of deck) inside an overlapping sheet hem, or by two vertical sheet legs back-to-back. *Top arc seam welds* have been referenced in some historical diaphragm design standards as part of a system without defining the strength of individual connections. Similarly, AWS D1.3 has shown the weld as a possible variation of an arc seam weld, without clear provisions to determine weld strength. The research to develop the design provisions for the *top arc seam welds* is presented in the S. B. Barnes Associates (Nunna and Pinkham, 2012; Nunna, et al., 2012) report.

**J2.4.1 Shear Strength of Top Arc Seam Sidelap Welds**

The design limitations are due to the scope of the test program that served as the basis for these provisions. The tests included typical weld spacing of approximately 12 in. (305 mm) o.c. and this established the strength of the welds with the stated limits. All testing was performed on joints with a vertical sheet leg inside an overlapping sheet hem configuration, but the behavior of connections with back-to-back vertical sheet legs is assumed to be similar.

Testing was performed in general accordance with AISI S905 (AISI, 2008), with the specimen dimensions in S905 Table 2 modified as required to address the described deck edge configuration. The ductility of the tested steels ranged from $F_u/F_{sy} = 1.01$ to $F_u/F_{sy} = 1.52$. The limits were extended to permit the use of the full range of recognized steels. Application should be based on the specified $F_u/F_{sy}$ for steels recognized in Section A3 of the Specification. The exclusion of the connection design restrictions for *top arc seam welds* used in diaphragms considers that the shear in the side lap welds is flowing from the sheet into each weld such that each weld is loaded as if it were a singular weld by its tributary length. This mitigates the concern over load sharing in brittle connections, and the strength reduction of lower ductility steels is based on the tests and built into Specification Equation J2.4.1-1.

The impact of shear rupture in the sheet can be calculated based on Specification Section J6 and this can be used to determine minimum acceptable weld spacing. The distance from the centerline of any weld and the centerline of adjacent weld can be checked by using Equation C-J2.4.1-1. Equation C-J2.4.1-1 is derived by equating the nominal shear strength
[resistance] expression from Specification Section J6 (Eq. J6.1-1 with $A_{nv} = st$) to the nominal shear strength [resistance] expression from Specification Section J2.4.1.

$$s = [6.67(F_u/F_{sy})-2.53]L_w(t/L_w)^{0.33}$$  \hspace{1cm} (C-J2.4.1-1)

where

- $s$ = Minimum distance from centerline of any weld to centerline of adjacent weld
- $s/2$ = Minimum distance from centerline of weld to end of connected member
- $L_w$ = Specified weld length
- $t$ = Base steel thickness (exclusive of coatings) of the thinner connected sheet
- $F_u$ = Minimum tensile strength of connected sheets as determined in accordance with Specification Section A3.1.1, A3.1.2 or A3.1.3
- $F_{sy}$ = Minimum specified yield stress of connected sheets as determined in accordance with Specification Section A3.1.1, A3.1.2 or A3.1.3

The steel deck sheets at the sidelap need to be tightly interlocked by crimping or pinching the sidelap prior to welding. When using the joint variation shown in Specification Figure J2.4.1-1(b), contact must be maintained between the two vertical legs while welding. For sidelaps with overlapping hem, Specification Figure J2.4.1-1(a) illustrates a crimped area nominally longer than the length of fusion, and the top of the overlapping hem sidelap must be burned through to allow fusion with the top of the inner vertical leg. Holes are commonly present at either or both ends of the completed welds. The holes do not necessarily indicate deficient welds or poor workmanship provided the specified length of fusion is obtained. Holes may aid in determining proper fusion with the inner vertical leg.

**J2.5 Fillet Welds**

For fillet welds on the lap joint specimens tested in the Cornell research (Peköz 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 J2.5-1). In connections of this type, the fillet weld throat is commonly larger than the throat of 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-J2.5-1).

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 (Peköz and McGuire, 1979), the last provision in this section covers 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. Additional research at the University of Sydney (Zhao and Hancock, 1995) has further indicated that weld throat failure may even occur between the thicknesses 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 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, 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 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.
In 2012, the design provisions were modified to take into consideration that the connected parts may have different tensile strengths.

J2.6 Flare Groove Welds

The primary mode of failure in cold-formed steel sections welded by flare groove welds, loaded transversely or longitudinally, was found to be sheet tearing along the contour of the weld (see Figure C-J2.6-1).

Figure C-J2.6-1 Flare Groove Weld Failure Modes

Except for Specification Equation J2.6-4, the provisions of this Specification section are intended to prevent shear tear failure. Specification Equation J2.6-4 covers the possibility that thicker sections may have effective throats less than the thickness of the channel and weld failure may become critical.

In 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).

In 2010, two figures were added showing reference dimensions for flare-bevel groove welds and flare V-groove welds, respectively, which replaced the figures for these welds in the previous editions of the Specification. Specification Equations J2.6-5 and J2.6-7 were added to more accurately define the effective throat of these welds. Filled flush throat depths were modified to match those specified in AWS D1.1-2006 Section 2.3.1.4 and Table 2.1.
process designations in Specification Tables J2.6-1 and J2.6-2 were based on AWS D1.1 Annex K, where SMAW stands for “shielded metal arc welding,” FCAW-S stands for “flux cored arc welding-self shielded,” GMAW stands for “gas metal arc welding,” FCAW-G stands for “flux cored arc welding-gas shielded,” and SAW stands for “submerged arc welding.” No change was needed in the Specification requirements from previous editions except in the definitions of the effective throat for use in Specification Equation J2.6-4.

**J2.7 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). These 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 these values are based; however, they may require special welding conditions. In view of the fact that AWS C1.1-66 and AWS C1.3-70 Standards were incorporated in AWS C1.1-2000, resistance welds should be performed in accordance with AWS C1.1-2000 (AWS, 2000).

In the 2001 edition of the Specification, a design equation is used to determine the nominal shear strength [resistance] that replaces the tabulated values given in previous editions of the Specification. The upper limit of Specification Equations J2.7-1, J2.7-3 and J2.7-5 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 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 J2.7-2, J2.7-4 and J2.7-6 due to the thickness limit set forth in Specification Section J2.

**J3 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 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 better coordination with the specifications of the Research Council on Structural Connections (RCSC, 1980) and AISC (1978). In 1986, design provisions for the maximum size of bolt holes and the allowable tension stress for bolts were added to the Specification (AISI, 1986). In the 1996 edition of the Specification, minor changes to the safety factors were made for computing the allowable and design tensile and shear strengths [factored resistances] of bolts. The allowable tensile stress for the bolts subject to the combination of shear and tension was determined by the equations provided in Specification Table J3.4-2 with the applicable safety factor.

(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 *Specification* criteria are applicable only to cold-formed steel members or elements 3/16 inch (4.76 mm) or less in thickness. For materials greater than 3/16 inch (4.76 mm), ANSI/AISC 360 (AISC, 2015) should be used for the United States and Mexico and CSA S16 (CSA, 2014) should be used for Canada.

Because of the 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 steel members where the thickness of the thinnest connected part is 3/16 inch (4.76 mm) or less, 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 sufficient safety against initial slip at least equal to that implied by the provisions of ANSI/AISC 360 and CSA S16. 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 than if no gap existed. Structural performance of connections containing unavoidable gaps between plies would require tests in accordance with *Specification* Section K2.1.

(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 ASTM F3125 Grades A325/A325M and A490/A490M bolts are available only for diameters of 1/2 inch (12 mm) and larger, A449 and A354 Grade BD bolts should be used as an equivalent of ASTM F3125 Grades A325/A325M and A490/A490M bolts, respectively, whenever smaller bolts (less than 1/2 inch (12 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 Section K2 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 pre-tensioning 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 are not subject to vibration or fatigue.

Bolts in slip-critical connections, however, must be tightened in a manner which ensures 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 those encountered in usual cold-formed steel 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
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(ASTM F959), whose published clamping forces are independent of grip, can be used for tightening slip-critical connections.

(d) Hole Sizes

For bolts having diameters less than 1/2 inch (12 mm), the diameter of a standard hole is the diameter of bolt plus 1/32 inch (1 mm). In 2014, metric hole sizes were adjusted to whole millimeters. Hole sizes for 1 inch (24 mm) and larger bolts were increased in line with AISC practices (AISC, 2015).

An alternative short-slotted hole size was added to Table J3 as a result of a research project undertaken by Yu and Xu (2010), who investigated bolted connections having various hole dimensions.

When using oversized holes or short-slotted 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 back-up plates should be used over oversized or short-slotted holes in an outer ply when the bolt hole deformation is considered in design. For connections using long-slotted holes, Specification Section J3 requires that the washers or back-up plates be used and that the shear capacity of bolts be determined by tests because a reduction in strength may be encountered.

Design information for oversized and slotted holes is included in Section J3.3.1 because such holes are often used in practice to meet dimensional tolerances during erection.

When the bolt hole deformation is considered in design, standard holes should be used in bolted connections. Oversized holes and slotted holes are only permitted as approved by the designer. An exception to the provisions for slotted holes is made in the case of slotted holes in lapped and nested zees. Resistance is provided in this situation partially by the nested components, rather than direct bolt shear and bearing. An oversized or slotted hole is required for proper fit-up due to offsets inherent in nested parts. Research (Bryant and Murray, 2001) has shown that lapped and nested zee members with 1/2-in. (12-mm) diameter bolts without washers and 9/16 in. × 7/8 in. (15 mm x 23 mm) slotted holes can develop the full moment in the lap.

J3.3 Bearing

Previous bolted connection tests have shown that bearing strength 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 Specification for determining the bearing strength 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 of bolted connections.
J3.3.1 Bearing Strength Without Consideration of Bolt Hole Deformation

Rogers and Hancock (1998) developed the design equation for bearing of bolted connections with washers (Specification Table J3.3.1-1). 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 J3.3.1-2). In Specification Table J3.3.1-1, the bearing factor, C, depends on the ratio of bolt diameter to member thickness, d/t. The design equations in Specification Section J3.3.1 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 safety factor and resistance factors are based on calibration of available test data (Wallace, Schuster, and LaBoube, 2001b).

Yu and Xu (2010) conducted testing of bolted connections without washers on oversized and short-slotted holes. Based on the test data, Yu and Xu developed new equations for bearing factor, C, and new values for modification factor, m_f. The hole dimensions investigated in Yu and Xu (2010) are consistent with those in Table J3. The added provisions for oversized and short-slotted holes do not apply to the slotted holes in lapped and nested zees. The safety factor and resistance factors are verified by Yu and Xu (2010) to be applicable for bolted connections using oversized and short-slotted holes.

J3.3.2 Bearing Strength 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 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.

Since the nominal strength [resistance] value with consideration of bolt hole deformation should not exceed the nominal strength [resistance] without consideration of the hole deformation, this limit was added in 2004.

J3.4 Shear and Tension in Bolts

The design provisions of this section are given in Section J3.4 of Appendix A or B. In Appendix A, the commentary is provided for Section J3.4.

J4 Screw Connections

The results of over 3500 tests worldwide were analyzed to formulate screw connection provisions (Peköz, 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 are not available for the particular application. A higher degree of accuracy can be obtained by testing any particular connection geometry (AISI, 1992).

Over 450 elemental connection tests and eight diaphragm tests were conducted in which compressible fiberglass insulation, typical of that used in metal building roof systems (MBMA, 2002), was placed between steel sheet samples in the elemental connection tests and between the
deck and *purlin* in the *diaphragm* tests (Lease and Easterling, 2006a, 2006b). The results indicate that the equations in Section J4 of the *Specification* are valid for applications that incorporate 6-3/8 in. (162 mm) or less of compressible fiberglass insulation.

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-J4-1 gives the correlation between the common number designation and the nominal diameter for screws. See Figure C-J4-1 for the measurement of nominal diameters.

**Table C-J4-1 Nominal Diameter for Screws**

<table>
<thead>
<tr>
<th>Number Designation</th>
<th>Nominal Diameter, d</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.</td>
</tr>
<tr>
<td>0</td>
<td>0.060</td>
</tr>
<tr>
<td>1</td>
<td>0.073</td>
</tr>
<tr>
<td>2</td>
<td>0.086</td>
</tr>
<tr>
<td>3</td>
<td>0.099</td>
</tr>
<tr>
<td>4</td>
<td>0.112</td>
</tr>
<tr>
<td>5</td>
<td>0.125</td>
</tr>
<tr>
<td>6</td>
<td>0.138</td>
</tr>
<tr>
<td>7</td>
<td>0.151</td>
</tr>
<tr>
<td>8</td>
<td>0.164</td>
</tr>
<tr>
<td>10</td>
<td>0.190</td>
</tr>
<tr>
<td>12</td>
<td>0.216</td>
</tr>
<tr>
<td>1/4</td>
<td>0.250</td>
</tr>
</tbody>
</table>

**Figure C-J4-1 Nominal Diameter for Screws**

**J4.1 Minimum Spacing**

Minimum spacing is the same as specified for bolts.

**J4.2 Minimum Edge and End Distances**

In 2001, the minimum edge distance was decreased from 3d to 1.5d.
J4.3 Shear

J4.3.1 Shear Strength [Resistance] 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-J4.3.1-1).

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-J4.3.1-2. 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-J4.3.1-3.

It is necessary to determine the lower bearing capacity of the two members based on the product of their respective thicknesses and tensile strengths.

Figure C-J4.3.1-2 Design Equations for $t_2/t_1 \geq 2.5$

\[
\begin{align*}
\text{tellowing} & \quad \text{N/A} \\
\text{bearing} & \quad P_{ns} = 2.7 t_2 d F_{u1} \\
\end{align*}
\]

Figure C-J4.3.1-3 Design Equations for $t_2/t_1 \leq 1.0$

\[
\begin{align*}
\text{tilting} & \quad P_{ns} = 4.2 (t_2 d)^{1/2} F_{u2} \\
\text{bearing} & \quad P_{ns} = 2.7 t_2 d F_{u2}
\end{align*}
\]

J4.3.2 Shear in Screws

Shear strength of the screw fastener itself should be known and documented from testing. Screw strength should be established and published by the manufacturer. In order
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To prevent the brittle and sudden shear fracture of the screw, the Specification applies a 25 percent adjustment to the safety factor or the resistance factor where determined in accordance with Specification Section K2.1.

J4.4 Tension

Screw connections loaded in tension can fail either by the screw pulled out from the plate (pull-out); material pulled 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 J4.4.

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 sufficiently thick to withstand bending forces with little or no deformation. In 2010, the minimum washer thickness requirement of 0.050 in. (1.27 mm) was relaxed for the washers in connections where t1 does not exceed 0.027 in. (0.686 mm), with the evidence that the washer thickness of as low as 0.024 in. (0.610 mm) does not adversely impact the pull-over strength of the connection for such top substrate thicknesses (Mujagic, 2008). In 2012, the washer dimension requirements were modified to harmonize the limitations of Specification Sections J4.5 with J4.4, given similar pull-over models in the two sections. Based on the findings of Zwick and LaBoube (2002), washers with outside diameter of 5/8 to 3/4 in. (15.9 mm to 19.1 mm) and a minimum thickness of 0.063 in. (1.60 mm) were included in the scope of Specification Section J4.4. Designers should include minimum required washer thickness in project documents.

J4.4.1 Pull-Out Strength

For the limit state of pull-out, Specification Equation J4.4.1-1 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 Peköz (1990).

J4.4.2 Pull-Over Strength

For the limit state of pull-over, Specification Equation J4.4.2-1 was derived on the basis of the modified British Standard and the results of a series of tests as reported by Peköz (1990). In 2007, a rational allowance was included to cover the contribution of steel washers beneath screw heads. For the special case of screws with domed washers (washers that are not solid or do not seat flatly against the sheet metal in contact with the washer), the calculated nominal pull-over strength [resistance] should not exceed 1.5t1d′wFu1 with d′w = 5/8 in. (15.9 mm). The 5/8 in. (15.9 mm) limit does not apply to solid steel washers in full contact with the sheet metal. In accordance with Specification Section J4, testing is allowed as an alternative method to determine fastener capacity. To use test data in design, the tested material should be consistent with the design. When a polygon-shaped washer is used and capacity is determined using Specification Equation J4.4.2-1, the washer should have rounded corners to prevent premature tearing.

In 2010, the pancake head washer screws and domed washers integral with the screw head were added and defined to assist the designer in proper determination of computational variables.
J4.4.3 Tension in Screws

Tensile strength 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 prevent the brittle and sudden tensile fracture of the screw, the Specification applies a 25 percent adjustment to the safety factor or the resistance factor where determined in accordance with Section K2.1.

J4.5 Combined Shear and Tension

Section J4.5 checks three failure modes where shear and tension are present at a connection: connection failures due to combined shear and pull-over, and combined shear and pull-out, as well as screw failure in the shank due to combined shear and tension.

J4.5.1 Combined Shear and Pull-Over

Research pertaining to the behavior of a screw connection has been conducted at West Virginia University (Luttrell, 1999). Based on the review and analysis of West Virginia University’s data for the behavior of a screw connection subject to combined shear and tension (Zwick and LaBoube, 2002), equations were derived that enable the evaluation of the strength of a screw connection when subjected to combined shear and tension. The tests indicated that at failure, the sheet beneath the screw head pulled over the head of the screw or the washer. Therefore, the nominal tensile strength [resistance] is based solely on $P_{nov}$. Although both nonlinear and linear equations were developed for ease of computation and because the linear equation provides regions of $\frac{V}{P_{nv}}$ and $\frac{T}{P_{nov}}$ equal to unity, the linear equation was adopted for the Specification. The proposed equation is based on the following test program limits:

$0.0285 \text{ in. (0.724 mm)} \leq t_1 \leq 0.0445 \text{ in. (1.13 mm)}$

No. 12 and No. 14 self-drilling screws with or without washers
d_w \leq 0.75 \text{ in. (19.1 mm)}

$62 \text{ ksi (427 MPa or 4360 kg/cm}^2\text{)} \leq F_{u1} \leq 70.7 \text{ ksi (487 MPa or 4970 kg/cm}^2\text{)}$

$t_2 / t_1 \geq 2.5$

The limit $t_2 / t_1 \geq 2.5$ reflects the fact that the test program (Luttrell, 1999) focused on connections having sheet thicknesses that precluded the tilting limit state from occurring. Thus, this limit ensures that the design equations will only be used when tilting limit state is not the controlling limit state.

The standard washer with outside diameter of 3/4 in. (19.1 mm) has a minimum thickness of 0.063 in. (1.60 mm). In 2011, the washer dimension limitations of Specification Sections J4.4 and J4.5 were harmonized, given similar pull-over models in the two sections.

The linear form of the equation as adopted by the Specification is similar to the following more conservative linear design equation that has been used by engineers:

$\frac{V}{P_{nv}} + \frac{T}{P_{nov}} \leq 1.0$  \hspace{1cm} C-J4.5.1-1

See Specification Section J4.5.1 for the definitions of the variables.

An eccentric load on a clip connection may create a nonuniform stress distribution around the fastener. For example, tension tests on roof panel welded connections have shown that under an eccentrically applied tension force, the resulting connection capacity is
50 percent of the tension capacity under a uniformly applied tension force. Thus, the Specification stipulates that the nominal pull-over strength [resistance] shall be taken as 50 percent of $P_{nov}$. If the eccentric load is applied by a rigid member such as a clip, the resulting tension force on the screw may be uniform; thus the force in the screw can be determined by mechanics, and the capacity of the fastener should be reliably estimated by $P_{nov}$. Based on the field performance of screw-attached panels, the 30 percent reduction associated with welds at sidelaps need not be applied when evaluating the strength of sidelap screw connections at supports or for sheet-to-sheet. The reduction is due to transverse prying or peeling. It is acceptable to apply the 50 percent reduction at panel ends due to longitudinal prying.

### J4.5.2 Combined Shear and Pull-Out

Research pertaining to the behavior of a screw connection has been conducted at the Missouri University of Science and Technology (Francka and LaBoube, 2010). Based on the findings of this research, equations were derived that enable the evaluation of the strength of a screw connection when subjected to combined shear and tension. The tests indicated that at failure, the screw pulled out of the bottom sheet of the connection. Therefore, the nominal tensile strength [resistance] is based solely on the tilting and tearing failure mode, Specification Equation J4.5.2-2. Although both nonlinear and linear equations were developed, the reliability of the nonlinear and linear equations was comparable. Therefore, for ease of computation, the linear equation was adopted for the Specification. The proposed equation is based on the test program limits as defined in the Specification. Evaluation of the connection for the combined shear and pull-out does not negate the need to evaluate the shear alone and pull-out alone limit states.

### J4.5.3 Combined Shear and Tension in Screws

In 2012, new provisions were added to account for shear and tension interaction in screws. Based on the rational engineering analysis, the same strength interaction as that used for bolts, Specification Equations J3.4-2 (ASD) and J3.4-3 (LRFD and LSD) (but in a different form) are used for screws.

### J5 Power-Actuated Fastener (PAF) Connections

In 2012, Section J5 was added to address connections with power-actuated fasteners (PAFs) connecting steel elements in non-diaphragm applications. These provisions do not preclude evaluation of any limit state on any power-actuated fastener through manufacturer or independent laboratory testing. The safety and resistance factors for any nominal strength [resistance] established through testing should be determined using provisions of Section K2 of the Specification.

In Specification Section J5, the provisions for determining the available strengths [factored resistances] were developed based on the study by Mujagic et al. (2010). Applicability constraints of these provisions correspond to the limitations of data available in the study (Mujagic et al., 2010).

In the provisions, the term “near side of the embedment material” refers to the surface of the embedment material from which the PAF is driven. The term “far side of the embedment material” refers to the embedment material surface from which the driven fastener exits.
J5.1 Minimum Spacing, Edge and End Distances

The minimum center-to-center spacing of the PAFs and the edge distances in the Specification are those stipulated by Table 2 of ASTM E1190 (ASTM, 2008). While larger spacing and edge distances are frequently found in test reports, the minimum distances given in ASTM E1190 (ASTM, 2008) are deemed sufficient in eliminating the detrimental effects of inadequate edge distance or fastener grouping.

J5.2 Power-Actuated Fasteners (PAFs) in Tension

Applicable limit states in tension include tension fracture, pull-out, and pull-over. The determination of available strength [factored resistance] due to any particular limit state for the fasteners depicted in Specification Figure J5 should be accomplished through appropriate testing. Alternatively, the available strength [factored resistance] should be determined using Sections J5.2.1 through J5.2.3 of the Specification.

J5.2.1 Tension Strength of Power-Actuated Fasteners (PAFs)

Power-actuated fasteners (PAFs) typically possess the Rockwell hardness (HRC) values of 49 to 58. Adequate HRC values represent one of the most critical design, installation and behavioral features of PAFs. The HRC values can be properly related to tensile strength in most ranges of HRC. The study by Mujagic et al. (2010) showed that the nominal tensile fracture strength [resistance] can be determined using the value of 260,000 psi (1790 MPa) for the HRC range in excess of 52. The user is cautioned to distinguish between the strength properties and HRC of pre-hardened steel from which a fastener is made and those of the hardened steel representing the final fastener product.

Specification Equation J5.2.1-1 was developed with the PAF driven such that no part of the length, \( l_{dp} \), as illustrated in Specification Figure J5, is located above the near side of the embedment material.

J5.2.2 Pull-Out Strength

The nominal pull-out strength [resistance] of power-actuated fasteners (PAFs) greatly depends on minute metallurgical, geometric, installation, and other design (often proprietary) features. PAFs develop their pull-out strength through partial fusion to the embedment material and friction resulting from the confinement stresses imposed by the displaced embedment material. Mechanical interlock or keying with PAF shank knurling and brazing effects due to zinc plating of the PAF also contribute to strength. While various behavioral trends can be established, it is not possible to develop a generic prediction model for PAFs which captures the above-mentioned, often proprietary, specific design features. Consequently, it was decided to stipulate testing as the only viable method of determining the pull-out strength. This approach is similar to how the pull-out strength is addressed in the EN 1993-1-3 (CEN 2006). The currently available testing protocols for determining the pull-out strength are given in AISI S905 (AISI, 2013) and ASTM E1190 (ASTM, 2011).

The tabulated nominal pull-out strengths [resistances] in Table C-J5.2.2-1 are provided for informational purposes. The table is extracted from the study by Mujagic et al. (2010), and it represents lower bound values from a limited selection of industry fastener and embedment plate combinations available to the study. Table C-J5.2.2-1 is only applicable to
fasteners embedded in steel plate for which manufacturer applicability guidelines stipulate embedment condition whereby no part of the length, $l_{dp}$, of PAF point, as illustrated in Specification Figure J5, is located above the near side of the embedment material. The values in Table C-J5.2.2-1 were scaled such that a safety factor of 3.0 computed in accordance with Section K2 of the Specification can be justified for the nominal strength [resistance] value of each of the considered fasteners. Since these are lower bound solutions, the actual safety factor for some of the fasteners would be higher than 3.0. The table is only applicable to fastener types and geometries depicted in Specification Figure J5. The current design practice generally involves reliance on tested capacities established per International Code Council Evaluation Service (ICC-ES) Acceptance Criteria 70 (AC70) (ICC-ES, 2010). The AC70 stipulates a minimum safety factor of 5.0, thus in many cases resulting in lower allowable strength values than those implied by Table C-J5.2.2-1. The approaches for establishing the safety factor stipulated by Section K2 of the Specification and by ICC-ES AC70 are not consistent. However, the values in Table C-J5.2.2-1 can be conservatively related to the current practice by reducing the nominal strength [resistance] values given therein by a factor of 0.6 (i.e., 3/5).

![Table C-J5.2.2-1](image)

Table C-J5.2.2-1
Nominal Tensile Pull-Out Strength of PAFs in Steel, $P_{not}$, lbs (N)

<table>
<thead>
<tr>
<th>Embedment Thickness, in. (mm)</th>
<th>1/8 (3.18)</th>
<th>3/16 (4.76)</th>
<th>1/4 (6.35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PAF Shank Diameter, $d_{so}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>in. (mm)</td>
<td>0.106 (2.69) ≤ $d_{so}$ &lt; 0.146 (3.71)</td>
<td>450 (2000)</td>
<td>915 (4070)</td>
</tr>
<tr>
<td>0.177 (4.50) ≤ $d_{so}$ &lt; 0.206 (5.23)</td>
<td>-</td>
<td>-</td>
<td>1970 (8760)</td>
</tr>
</tbody>
</table>

Where statistical indices required to compute the safety and resistance factors in accordance with Specification Section K2 are not given for a pull-out strength provided by a manufacturer, a safety factor of 4.0 and a resistance factor of 0.40 (0.35 for LSD) can be applied to the nominal strengths [resistance] provided in Table C-J5.2.2-1. This option was provided based on the study by Mujagic et al. (2010) which shows that 4.0 represents a conservative lower bound value of safety factor for a variety of fastener types and models when the computed safety factor or data required for its computation is not available to the user.

J5.2.3 Pull-Over Strength

The pull-over limit state in PAF connections is fundamentally the same as that in screw connections. The Specification addresses the screw-like PAFs in an identical manner that screw connections are dealt with in Specification Section J4. The two notable exceptions represent connections with tapered-head fasteners that consistently yield about 20 percent lower pull-over strength than screw-like PAF connections, and connections with collapsible spring washers that consistently yield about 30 percent higher strength than screw-like PAF connections. The Specification addresses the two special cases by varying the constant multiplier of the pull-over equation.

J5.3 Power-Actuated Fasteners (PAFs) in Shear

Applicable limit states in shear are shear fracture, bearing and tilting, pull-out, net section checks, and nominal shear strength [resistance] limited by edge distance.
J5.3.1 Shear Strength of Power-Actuated Fasteners (PAFs)

Nominal shear strength [resistance] of PAFs is determined by relating the ultimate tensile strength in tension to that in shear by a factor of 0.6.

J5.3.2 Bearing and Tilting Strength

The nominal bearing strength [resistance] is based on the equation proposed in the study by Mujagic et al. (2010) based on the data for which $t_2/t_1 \geq 2.0$ and $t_2 \geq 1/8$ in. (3.2 mm). While some decrease in calculated strength was observed with decreasing $t_2/t_1$ ratio, thus suggesting the presence of tilting at lower ratios of $t_2/t_1$, it was noted that the bearing and tilting strength can be predicted by setting the constant multiplier in the bearing equation to 3.7. Since the study by Mujagic et al. (2010) was based only on the types of fasteners shown in Specification Figures J5(c) and J5(d), the ENV 1993-1-3 (ECS, 2006) equation constant of 3.2 is conservatively adopted for other types of PAFs.

J5.3.3 Pull-Out Strength in Shear

Pull-out in shear is essentially a derivative of fastener tilting in steel. The pull-out failures were reported at wide range of $t_2/t_1$ ratios. The bearing strength equation of Specification Section J5.3.2 considers the effect of tilting deformation on bearing failures at low ratios of $t_2/t_1$. However, as expected, it does not accurately predict the connection strength where tilting is the predicted failure mode. The Specification, therefore, stipulates a separate pull-out check over the entire range of $t_2/t_1$ ratios and thicknesses covered by the Specification.

J5.3.4 Net Section Rupture Strength

Based on the recommendations of Beck and Engelhardt (2002), the PAF hole is required to be calculated based on a width of 1.10 times the PAF diameter. The effect of partially driven PAFs (i.e., where the PAF point length, $\ell_{dp}$, is fully or partially located in the embedment material) on net section properties of a connection are not presently known. The Specification, therefore, stipulates that the PAF shank diameter, $d_s$, be used in determination of net section properties.

J5.3.5 Shear Strength Limited by Edge Distance

The Specification presently stipulates the application of the same criteria given for screws in Specification Section J6.1, recognizing fundamental similarities in behavior and application of screw and PAF connections. Favorable local effects of sheath folding and local hardening of the sheathing near the PAF hole may render the screw connection criteria slightly conservative when applied to PAF connections. The effect of partially driven PAFs (i.e., where the PAF point length, $\ell_{dp}$, is fully or partially located in the embedment material) on edge distance properties of a connection are not presently known. The Specification, therefore, stipulates that the PAF shank diameter, $d_s$, be used in edge distance checks.
J5.4 Combined Shear and Tension

Combined shear and tension in the PAF connection should include the interaction of combined shear and pull-over, combined shear and pull-out, and fracture due to combined shear and tension on the PAF fastener itself. Currently available research does not address PAF connections subject to combined tension and shear. Consequently, the Specification does not at present provide equations for consideration of such connections. The ICC-ES AC 70 (ICC-ES, 2010) criteria can be used to consider combined tension and shear through testing. Alternatively, such a condition can be evaluated in accordance with Specification Section A1.2. Based upon fundamental principles of fastener mechanics, Equation C-J5.4-1 represents an exact interaction between tension and shear when fastener fracture governs. Since the actual interaction curve is not presently known for other combinations of tension and shear limit states, the power coefficient of one, rendering the Equation C-J5.4-1 a linear interaction, can be used as a conservative check when both shear and tension are not limited by fracture.

\[
\left( \frac{T}{P_{at}} \right)^n + \left( \frac{V}{P_{av}} \right)^n \leq 1.0
\]  
\( (C-J5.4-1) \)

where
- \( T \) = Required tension strength [force due to factored loads]
- \( P_{at} \) = Available tension strength [factored resistance] determined in accordance with Specification Section J5.2
- \( V \) = Required shear strength [shear force due to factored loads]
- \( P_{av} \) = Available shear strength [factored resistance] determined in accordance with Specification Section J5.3
- \( n \) = 2 when both tension and shear are governed by the fracture limit state
  - 1 in all other cases

J6 Rupture

The provisions contained in Specification Section J6 and its subsections are applicable only when the thinnest connected part is 3/16 inch (4.76 mm) or less in thickness. For materials thicker than 3/16 inch (4.76 mm), the design should follow ANSI/AISC 360 for the United States and Mexico and CSA S16 for Canada.

Significant changes were made to the format of Specification Section J6 in 2010. Connections may be subject to shear rupture, tension rupture, block failure in tension, block failure, or any combinations of these failures in shear depending upon the relationship of the connectors to the connection geometry and loading direction. Specification Table J6.2-1 provides adjustment factors consistent with prior editions of the Specification to cover shear lag factors. Other adjustment factors provide allowances for staggered connector patterns and nonuniform stress distribution on the tensile plane. In 2012, the Committee added a reference to PAFs in Table J6-1, permitting the use of the same safety and resistance factors as for screws. This step was taken recognizing inherent similarities in configurations and behavior of screw and PAF connections as they relate to net fracture of connected elements. Furthermore, partial fusion occurring between the embedment steel and PAF should result in a conservative design with respect to application of resistance and safety factors for screw connections.

(a) Shear Lag for Flat Sheet Connections
Earlier 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. Consequently, specific shear lag factors were developed. However, it was found by Teh and Gilbert (2014) that, for the limit state of net section tension rupture, there is no noticeable difference in the shear lag factors between different types of bolted connections. The apparent differences in the shear lag factors “due to joint rotation and out-of-plane deformation of flat sheets” cited in the earlier Specification edition were actually due to a different failure mode, namely tilt bearing failure, which is considered separately. For flat sheet connections using multiple connectors in the line of force and having less out-of-plane deformations, the strength reduction was not required in the 2012 edition of the Specification (Rogers and Hancock, 1998). A single shear lag reduction factor given by Specification Equation J6.2-4 (Teh and Gilbert, 2014) now applies to all cases (both single and multiple bolts in the line of the force, and single and double shear connections) in the 2016 edition of the Specification.

(b) Staggered Holes

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 rupture. It was first analytically studied by Cochrane (1922), who derived the adjustment term \( s^2/(4g + 2d_h) \) shown in Specification Equation J6.2-3. 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 rupture plane could be adjusted by use of \( s^2/4g \) with an additional 10 percent reduction factor. More recent testing on the critical tensile path involving stagger has been carried out by Fox and Schuster (2010), the results of which indicate that the 10 percent reduction is not required. However, the neglect of the variable involving the bolt hole diameter, \( d_h \), in the earlier Specification edition was not required, as it did not lead to meaningful simplification while potentially leading to 10 percent overestimation (Teh & Gilbert, 2014). Consequently, \( d_h \) was included in Equation J6.2-3 of the 2016 edition of the Specification.

(c) Shear Lag for Other Than Flat Sheet Connections

Shear lag has a debilitating effect on the tensile capacity of a cross-section. Based on the University of Missouri-Rolla 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 connection (See Figures C-J6-1 and C-J6-2). The research by Teh and Gilbert (2014) has shown that the shear lag factors for bolted
connections in angle and channel members should take into account the width ratio between the connected and the unconnected parts, in addition to the traditional ratio between the connection eccentricity, $x$, and the connection length, $L$. Specification Equations J6.2-6 and J6.2-8 developed by Teh and Gilbert (2014) lead to accurate results for bolted connections in angle and channel members of various configurations and material properties. Additionally, there are no artificial lower or upper bound values for the computed shear lag factors.

Research has also shown that for cold-formed steel sections using single-bolt connections, bearing usually controlled the nominal strength [resistance], not rupture in the net section.

(d) Block Shear

Block shear is a limit state in which the resistance is determined by the sum of the shear strength on a failure path(s) parallel to the force and the tensile strength on the segment(s) perpendicular to the force. 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 equations may be applied to cold-formed steel members.

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane. Specification Equations J6.3-1 and J6.3-2 check both conditions.

Connection tests conducted by Birkemoe and Gilmor (1978) have shown that on coped beams, a tearing failure mode as shown in Figure C-J6-5 can occur along the perimeter of the holes. Hardash and Bjorhovde (1985) have demonstrated these effects for tension members as illustrated in Figure C-J6-4. The research paper “AISC LRFD Rules for Block Shear in Bolted Connections – A Review” (Kulak and Grondin, 2001) provides a summary of test data for block shear rupture strength.

The distribution of tensile stresses is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001). For shear forces on coped beams, an additional multiplier, $U_{bsr}$, of 0.5 is used when more than one row of bolts is present. This approach is consistent with the provisions of ANSI/AISC 360 (AISC, 2005 and 2010a).

Tests performed at the University at Missouri-Rolla have indicated that the current design equations for shear and tilting provide a reasonably good estimate of the connection performance for multiple screws in a pattern (LaBoube and Sokol, 2002).
Examples of failure paths can be found in Figures C-J6-3 through C-J6-7.

 семейной

(Tension Rupture)

Failure Path 1, 2, 3, 4

*Specification* Section J6.2 applies:

\[ A_{\text{e}} = U_{\text{sl}} A_{\text{nt}} \]

\[ U_{\text{sl}} \text{ in accordance with } \text{Specification Equation J6.2-4} \]

\[ A_{\text{nt}} = (w_g - d_h) t \]

(Shear Rupture)

Failure Path 5, 2, 3, 6

*Specification* Section J6.1 applies:

\[ A_{\text{nv}} = 2n(e - d_h/2) t \]

\[ n = 1 \text{ as there is only a single fastener} \]

(Tension Rupture)

Failure Path 9, 4, 6, 10

*Specification* J6.2 applies:

\[ A_{\text{nt}} = A_g - 2d_h t \]

Failure Path 11, 2, 4, 5, 6, 7, 12

\[ A_{\text{nt}} = A_g - 5d_h t + t(4s'^2)/(4g + 2d_h) \]

(Block Shear Rupture)

*Specification* Section J6.3 applies:

\[ A_{\text{gv}} = 2et \]

\[ A_{\text{nv}} = 2(e - d_h/2)t \]
$U_{bs} = 1.0$

**Failure Path 3, 2, 5, 7, 8**

$A_{nt} = (4g - 2dh)t$

**Failure Path 3, 2, 4, 5, 6, 7, 8**

$A_{nt} = 4[g - dh + s^2/(4g + 2dh)]t$

---

**Figure C-J6-5 Potential Failure Path of Coped Stiffened Channel (Block Shear Rupture)**

(Block Shear Rupture)

Failure Path 1, 2, 3, 4, 5, 6

*Specification* Section J6.3 applies:

$A_{g_v} = (2g + e_1)t$

$A_{nv} = A_{g_v} - 2.5d_h t$

$A_{nt} = [(s + e_2) - 1.5d_h]t$

$U_{bs} = 0.5$

---

**Figure C-J6-6 Potential Failure Path of Multiple-Fastener Lap Joint (Tension Rupture)**

(Tension Rupture)

Failure Path 1, 2, 3, 4, 5, 6

*Specification* Section J6.2 applies:

$A_e = U_{sl}A_{nt}$

$U_{sl}$ in accordance with *Specification* Eq. J6.2-4

$A_{nt} = (w - 4d_h)t$
J7 Connections to Other Materials

When a cold-formed steel structural member is connected to other materials, such as hot-rolled steel, aluminum, concrete, masonry or wood, the connection strength should be the smallest of the strength of the fastener, the strength of the fastener attachment to the cold-formed steel structural member, or the strength of the fastener attachment to the other material.

In 2016, provisions were added to Specification Section J7.2 for power-actuated fasteners (PAFs) connecting cold-formed steel framing track-to-concrete base materials. These provisions were based on an experimental study where cold-formed steel wall tracks were attached to concrete base materials and subjected to monotonic and cyclic/seismic test loads (AISI, 2013h). In 2018, these provisions were removed to avoid unconservative designs of track and other cold-formed steel structural member attachments to concrete and to avoid unintended interpretation of the validity of these provisions in different applications.

J7.1 Connection Strength to Other Materials

The design of connections to other materials should be in accordance with the applicable building code, including those referenced standards, as applicable. When the applicable building code provides no requirement with respect to consideration of specific limit states, other codes and standards and manufacturers’ technical reports and catalogues acceptable to the authority having jurisdiction may be utilized. The following is a list of suggested references:

(a) Cold-Formed Steel Attached to Steels Over 3/16-Inch (4.76-mm) Thick

(1) For Welded Connections:
   In the U.S. and Mexico:
   AWS D1.1/D1.1M, Structural Welding Code – Steel
   AWS D1.3/D1.3M, Structural Welding Code – Sheet Steel
   In Canada:
   CSA W47.1, Certification of Companies for Fusion Welding of Steel
Commentary on the North American Cold-Formed Steel Specification, 2016 Edition With Supplement 1

CSA W55.3, Certification of Companies for Resistance Welding of Steel and Aluminum
CSA W59, Welded Steel Construction (Metal Arc Welding)

(2) For Bolted Connections:
   - In the U.S. and Mexico: ANSI/AISC 360, Specification for Structural Steel Buildings
   - In Canada: CSA S16, Design of Steel Structures

(3) For Screw Connections:
   - Published manufacturers’ technical reports and catalogs

(4) For Power-Actuated Fastener Connections:
   - Published manufacturers’ technical reports and catalogs

(b) Cold-Formed Steel Attached to Aluminium

(1) For Bolted or Screw Connections:
   - In the U.S. and Mexico: ADM1, Aluminum Design Manual: Part 1—Specification for Aluminum Structures
   - In Canada: CSA S157, Strength Design in Aluminum

(c) Cold-Formed Steel Attached to Concrete

(1) For Post-Installed Anchors and Cast-in-Place Anchors:
   - In the U.S. and Mexico: ACI 318, Building Code Requirements for Structural Concrete
   - In Canada: CSA A23.3, Design of Concrete Structures
   - Published manufacturers’ technical reports and catalogs

(2) For Power-Actuated Fasteners:
   - Published manufacturers’ technical reports and catalogs

(d) Cold-Formed Steel Attached to Masonry

(1) For Cast-in-Place Bolts:
   - In the U.S. and Mexico: TMS 402/ACI 530/ASCE 5, Building Code Requirements for Masonry Structures
   - In Canada: CSA S370, Connectors for Masonry

(2) For Power-Actuated Fasteners and Other Post-Installed Anchors:
   - Published manufacturers’ technical reports and catalogs

(e) Cold-Formed Steel Attached to Wood

(1) For Bolt or Screw Connections:
   - In the U.S. and Mexico: ANSI/AWC NDS, National Design Specification (NDS) for Wood Construction
   - In Canada: CSA O86, Engineering Design in Wood
   - Published manufacturers’ technical reports and catalogs

(f) Cold-Formed Steel Attached to Plywood

(1) For Screw Connections:
   - APA Technical Note E830E, Fastener Loads for Plywood-Screws

J7.1.1 Bearing

The design provisions for the nominal bearing strength [resistance] on the other materials should be derived from appropriate material specifications.
**J7.1.2 Tension**

This section is included in the *Specification* to raise the awareness of the design engineer regarding tension on fasteners and the connected parts.

**J7.1.3 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.
K. RATIONAL ENGINEERING ANALYSIS AND TESTING

K1 Test Standards

Specification Section K1 lists standards developed for testing cold-formed steel elements, connections, or assemblies. Commentaries are provided along with the test standards as needed.

K2 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. The design and testing of cold-formed steel diaphragms should be in accordance with the standards specified in Specification Section I2. Accordingly, the statement that the provisions in Specification Section K2 do not apply to cold-formed steel diaphragms was deleted in 2016.

K2.1 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 A1.2 of the Specification or specifically permitted elsewhere in the Specification.

K2.1.1 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 φ factor can be computed from Specification Equation K2.1.1-2 as developed in the Commentary as Equation C-B3.2.2-15.

The calibration coefficient, \( C_\phi \), and coefficient of variation of the load effect, \( V_Q \), are dependent on the selected load combination and load ratio (e.g., dead-to-live load ratio). Justification for the selected choices is provided in Commentary Sections B3.2.2 and B3.2.3. If the special case being considered deviates significantly from the assumed governing load combination (1.2D + 1.6L in the United States and 1.25D + 1.5L in Canada) or dead-to-live load ratio (1:5 in the United States and 1:3 in Canada), then updated values such as those provided in Meimand and Schafer (2014) for \( C_\phi \) and \( V_Q \) may be considered. With the exception of earthquake load combinations, the constant values for \( C_\phi \) and \( V_Q \) that the Specification provides were shown to result in φ factors within 15 percent of more exact approximations (Meimand and Schafer, 2014).

The correction factor, \( C_P \), is used in Specification Equation K2.1.1-2 for determining the φ factor to account for the influence due to a small number of tests (Peköz 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 Specification, Equation K2.1.1-4 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 ±10 percent deviation limit was relaxed to ±15 percent. The use of \( C_P \) with a minimum \( V_P \) reduces the need for this restriction. In Specification Equation K2.1.1-4, 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 K2.1.1-1, assuming that $V_P$ will be no larger than about 0.20. The requirements of Specification Section K2.1.1(a) for $n = 3$ help to ensure this outcome.

The 0.065 minimum value of $V_P$, when used in Specification Equation K2.1.1-2 for the case of three tests, produces safety factors 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:

$$V_P = \frac{\sqrt{s^2}}{R_n}$$

where

$\sqrt{s^2} = \text{Sample variance of all test results}$

$$= \frac{1}{n-1} \sum_{i=1}^{n} (R_i - R_n)^2$$

$R_n = \text{Mean of all test results}$

$R_i = \text{Test result i of n total results}$

Alternatively, $V_P$ can be calculated as the sample standard deviation of $n$ ratios $R_i/R_n$.

If the nominal strength [resistance] is determined in accordance with a rational engineering analysis while the safety and resistance factors are calculated based on tests, the coefficient of variation, $V_P$, is determined in accordance with Specification Equation K2.1.1-6 with $P_m$ determined in accordance with Specification Equation K2.1.1-3.

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 I6.2.1 of this Commentary for detailed discussion).

The additional statistical data needed for the determination of the resistance factor are listed in Specification Table K2.1.1-1. Table K2.1.1-1 was simplified and updated in 2016 to reflect current limit states, to provide clarity in its use for rational engineering analysis and test-based methods, and to reflect the actual accuracy of the selected $M_m$, $V_M$, $F_m$, and $V_F$ statistics. The original basis for the Table K2.1.1-1 member statistics is the LRFD calibration of Hsiao, Yu and Galambos (1988a). Connection statistics are based on Rang, Galambos, and Yu (1979b) and Peköz (1990). Values for power-actuated fasteners are based on Mujagic et al. (2010). The statistics data for connections to structural concrete and wood are based on those employed in AISI S310. In 2007, the Specification more clearly defined the appropriate material properties that are to be used when evaluating test results by specifying that supplier-provided properties are not to be used.

In 2012, statistical data of $M_m$, $V_M$, $F_m$, and $V_F$ were added for power-actuated fasteners to accompany the newly created Specification Section J5, based on the study by Mujagic et al. (2010).

In 2012, Section K2.1.1(c) was revised to permit the use of mill certificates to establish
the mechanical properties of small connectors and devices. As a general practice, the yield stress, $F_y$, is determined by testing a tensile specimen that is either cut from the test specimen, or the steel coil or sheet used to produce the test specimen. However, for some cold-formed steel components such as small hurricane ties and clips, it is often impossible to cut a standard size or sub-size tensile specimen that would meet the requirements of ASTM A370 (ASTM, 2015). Since mill certificate tensile specimens are taken from the lead or tail of the master coil which may not be representative of the entire coil, and because coiling and uncoiling operations can alter mechanical properties, it is necessary to reduce $M_m$. When using mill certificates instead of tensile specimens for a range of 21 coils (Stauffer and McEntee, 2012), it has been shown that using $M_m = 0.85$ will provide corresponding $\phi$ and $\Omega$ values that are on average 15 percent more conservative. In order to use mill certificates to establish material properties, it is important to maintain proper records and procedures that can trace the connector or device to the master coil. The use of mill certificates is not permitted for members. In addition, although mill certificates are routinely used to establish the raw material properties for fasteners such as screws or power-actuated fasteners, they should not be used to establish the final material properties. This is because the raw steel undergoes secondary operations such as heat treating that alters its final properties.

In 2012, Section A1.2(b) and Section K2.1.1(b) were added as an optional method to calibrate safety and resistance factors for a proposed strength theory using test data. In order to use this optional method, sufficient correlation must exist between the proposed strength theory and the test data. The correlation coefficient, $C_c$, used in this section is a statistical measure of the agreement between the strength predictions ($R_{n,i}$) and test results ($R_{t,i}$):

$$C_c = \frac{n \sum R_{t,i} R_{n,i} - (\sum R_{t,i})(\sum R_{n,i})}{\sqrt{n(\sum R_{t,i}^2) - (\sum R_{t,i})^2} \sqrt{n(\sum R_{n,i}^2) - (\sum R_{n,i})^2}}$$

where

- $R_{t,i}$ = Tested strength [resistance], corresponding to test $i$
- $R_{n,i}$ = Predicted nominal strength [resistance], corresponding to test $i$.

The value of the correlation coefficient reveals information about the potential quality of the proposed strength theory, namely:

1. High or moderately high positive correlation indicates that the theory and tests either agree substantially as they are, or can be brought into good agreement by using a constant factor. This means that bias factor, $P_m$, will compensate for the bias, as intended, in the calibration procedure to determine the resistance factor.

2. Low or nearly zero correlation is an indicator of independence; in other words, no relationship between the tests and theory can be discerned. Using the theory will produce bad results and it should be rejected.

3. Negative correlation indicates that the theory and test data not only disagree but actually have opposite relationships. For example, when the theory says the strength increases, it actually decreases. Using the theory will produce bad results and it should be rejected.

The square of the correlation coefficient is referred to as the coefficient of
determination. It gives the proportion of the variance (fluctuation) of one variable (tested strength [resistance]) that is predicted by the other variable (strength theory). For example, for \( C_c^2 = (0.8)^2 \), 64 percent of the variance is accounted for by the theory. Alternative values for the minimum correlation coefficient could be used, but values above \( C_c = 0.707 \) have the desirable characteristic that \( C_c^2 \geq 0.5 \); that is, more than 50 percent of the variance is explained by the theory.

In general, higher values of the correlation coefficient are desirable, and indicate a better agreement with the theory, lower \( V_P \), and a better result for the product of the resistance factor times the nominal strength [resistance] given by the theory.

Another advantage of a correlation coefficient criterion is that it is less restrictive and easier to satisfy than alternative criteria based on individual deviations, such as a 15 percent deviation restriction. \( C_c \) is obtained from the full data set and does not apply to individual values. Also, there are multiple ways to obtain a good correlation coefficient. For example, if the test data and strength theory differ by a constant factor; i.e., they are proportional, one will still get a correlation coefficient of 1.0, as if they had agreed directly. This advantage also holds for moderately high correlation coefficients. As mentioned above, this will improve the effectiveness of bias factor, \( P_m \), and the resistance factor.

It is important that users not only test at the upper and lower bounds of the desired parameter range, but that even coverage of tests is provided throughout the range. This is emphasized in the Specification in order to ensure that potential minima or maxima within the test range are detected and that the resistance factor and safety factor calibrated using the test data properly reflect any variation from the minima/maxima.

The Specification provides methods for determining the deflection of some members for serviceability consideration, but the Specification does not provide serviceability limits. Justification is discussed in Section B3.7 of the Commentary.

**K2.1.2 Allowable Strength Design**

The equation for the safety factor, \( \Omega \) (Specification Equation K2.1.2-2), converts the resistance factor, \( \phi \), from LRFD test procedures in Specification Section K2.1.1 to an equivalent safety factor for the Allowable Strength Design. The average of the test results, \( R_n \), is then divided by the safety factor to determine an allowable strength. It should be noted that Specification Equation K2.1.2-2 is identical with Equation C-B3.2.2-16 for D/L = 0.

**K2.2 Tests for Confirming Structural Performance**

Members, connections and assemblies that can be designed according to the provisions of Chapters A through J, L, and M of the Specification need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm the results of calculations by test. 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 for the tested specimen or assembly to demonstrate that the strength is not less than the applicable nominal resistance, \( R_n \).
K2.3 Tests for Determining Mechanical Properties

K2.3.1 Full Section

Explicit methods for utilizing the effects of cold work are incorporated in Section A3.3.2 of the Specification. In that section, it is specified that as-formed mechanical properties, in particular the yield stress, 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 cross-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 A3.3.2. For details of testing procedures which have been used for such purposes, but which in no way should be regarded as mandatory, see Specification (1968), Chajes, Britvec and Winter (1963), and Karren (1967). AISI S902, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns, provides testing procedures (AISI, 2013c).

K2.3.2 Flat Elements of Formed Sections

Specification Section K2.3.2 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 A3.3.2 for calculating the average yield stress of the formed section by considering the strength increase from cold work of forming.

K2.3.3 Virgin Steel

For steels other than the ASTM Specifications listed in Specification Section A3.1, the tensile properties of the virgin steel used for calculating the increased yield stress of the formed section should also be determined in accordance with the Standard Methods of ASTM A370 (2015).
L. DESIGN FOR SERVICEABILITY ($l_{eff}$)

Reduced stiffness values used in the direct analysis method, described in Chapter C, are not intended for use with the provisions of this chapter.

L1 Serviceability Determination for Effective Width Method

The effective moment of inertia is calculated based on the reduced cross-section at the service load level. Examples are provided in the Cold-Formed Steel Design Manual (AISI, 2013).

L2 Serviceability Determination for Direct Strength Method

The provisions of this section use a simplified approach to deflection calculations that assume the moment of inertia of the section for deflection calculations is linearly proportional to the strength of the section, determined at the allowable stress of interest. This approximation avoids lengthy effective section calculations for deflection determination.

L3 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 L3-1 of the Specification permits one to compute the maximum permissible flange width, $w_f$, for a given amount of flange curling, $c_f$. The equation has been shown to be conservative when compared with more recent experimental data and more exact analytical expressions for predicting flange curling are now available (Lecce and Rasmussen 2008, 2009).

It should be noted that Section L3 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. An example in the AISI Cold-Formed Steel Design Manual (AISI, 2013) illustrates the design consideration for flange curling.
M. DESIGN FOR 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-M-1(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-M-1(b) also need not be considered.

Research by Barsom et al. (1980) and Klippstein (1980, 1981, 1985, 1988) 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

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**Figure C-M-1 Welded Angle Members**
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, Equation C-M-1.

\[
\log N = C_f - m \log F_{SR} \\
C_f = B - (n s) 
\]

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-M-1
- \( n \) = number of standard deviations to obtain a desired confidence level
- \( s \) = approximate standard deviation of the fatigue data
- \( = 0.25 \) (Klippstein, 1988)

The database for these design provisions is based upon cyclic testing of real joints; therefore, stress concentrations have been accounted for by the categories in Table M1-1 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 M3-1 of the Specification is to be used to calculate the design stress range for the chosen design life, \( N \). Table M1 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 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 1, EFFECTIVE WIDTH OF ELEMENTS

In cold-formed steel construction, individual elements of steel structural members are thin and the width-to-thickness ratios are large as compared to hot-rolled steel shapes. These thin elements may buckle locally at a stress level lower than the yield stress of steel when they are subjected to compression in flexural bending, axial compression, shear, or bearing. Figure C-1-1 illustrates some local buckling patterns of certain beams and columns (Yu and LaBoube, 2010).

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 post-buckling strength of structural components. Section B4.1 and Appendix 1 of the Specification contain 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 F5 for flexural members.

It is well known that the structural behavior and the load-carrying capacity of a stiffened compression element such as the compression flange of a 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 stress of steel and the strength of the compression element is governed by yielding. For the compression flange with large w/t ratios, local buckling (Figure C-1-2) will occur at the following elastic critical buckling stress:

\[ f_{cr} = \frac{k\pi^2E}{12(1-\mu^2)(w/t)^2} \]  

(C-1-1)
where
\( k = \) Plate buckling coefficient (Table C-1-1)
\( = 4 \) for stiffened compression elements supported by a web on each longitudinal edge
\( E = \) Modulus of elasticity of steel
\( \mu = \) Poisson’s ratio = 0.3 for steel in the elastic range
\( w = \) Flat width of the compression element
\( t = \) Thickness of the compression element

When the elastic critical buckling stress computed according to Equation C-1-1 exceeds the proportional limit of the steel, the compression element will buckle in the inelastic range (Yu and LaBoube, 2010).

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 post-buckling strength of the compression elements and is most pronounced for stiffened compression elements with large \( w/t \) ratios. The mechanism of the post-buckling action of compression elements was discussed by Winter in previous editions of the 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 in Figure C-1-3. 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 stresses, 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, and those nearest the center being able to deflect most.

In consequence of (a), the model will not collapse and fail when its buckling stress (Equation C-1-1) 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,

<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></td>
<td>Compression</td>
<td>4.0</td>
</tr>
<tr>
<td>(b)</td>
<td>Fixed</td>
<td>Compression</td>
<td>6.97</td>
</tr>
<tr>
<td>(c)</td>
<td>Fixed</td>
<td>Compression</td>
<td>0.425</td>
</tr>
<tr>
<td>(d)</td>
<td>Fixed</td>
<td>Compression</td>
<td>1.277</td>
</tr>
<tr>
<td>(e)</td>
<td>Fixed</td>
<td>Compression</td>
<td>6.42</td>
</tr>
<tr>
<td>(f)</td>
<td></td>
<td>Shear</td>
<td>5.34</td>
</tr>
<tr>
<td>(g)</td>
<td>Fixed</td>
<td>Shear</td>
<td>8.98</td>
</tr>
<tr>
<td>(h)</td>
<td></td>
<td>Bending</td>
<td>23.9</td>
</tr>
<tr>
<td>(i)</td>
<td>Fixed</td>
<td>Bending</td>
<td>41.8</td>
</tr>
</tbody>
</table>

Table C-1-1
Values of Plate Buckling Coefficients
continue to resist increasing load with hardly any increasing deflection. For the plate, this means that the hitherto uniformly distributed compression stress redistributes itself in a manner shown in Figure C-1-4, with the stresses being largest at the edges and smallest in the center. With further increase in load, this nonuniformity increases, as also shown in Figure C-1-4. 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 stress \( F_y \).

This post-buckling strength 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 post-buckling strength can be found in the series of photographs in Figure 7 of Winter (1959b).

The model of Figure C-1-3 is representative of the behavior of a compression element supported along both longitudinal edges, as the flange in Figure C-1-2. In fact, such elements buckle into approximately square waves.

In order to utilize the post-buckling strength of the stiffened compression element for design purposes, the Specification has used the effective design width approach to determine the sectional properties since 1946. In Appendix 1 of the Specification, design equations for computing the effective widths are provided for the following cases: (1) uniformly compressed stiffened elements, (2) uniformly compressed stiffened elements with circular or noncircular holes, (3) webs and other stiffened elements with stress gradient, (4) unstiffened elements with uniform or gradient stress, and (5) C-section webs with holes under stress gradient. The background information on various design requirements is discussed in subsequent sections.
1.1 Effective Width of Uniformly Compressed Stiffened Elements

(a) Strength 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-1-4. 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:

\[
b = 1.9t \left[ \frac{E}{f_{\text{max}}} \right] \sqrt{1 - 0.475 \left( \frac{t}{w} \right) \frac{E}{f_{\text{max}}}}
\]  

(C-1.1-1)

The above equation can be written in terms of the ratio of \( F_{\text{cr}}/f_{\text{max}} \) as follows:

\[
\frac{b}{w} = \frac{F_{\text{cr}}}{f_{\text{max}}} \left[ 1 - 0.25 \left( \frac{F_{\text{cr}}}{f_{\text{max}}} \right) \right]
\]  

(C-1.1-2)

where \( F_{\text{cr}} \) is the critical elastic buckling stress of a plate, and is expressed in Equation C-1-1.

Thus, the effective width expression (e.g., Equation C-1.1-1) 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 Specification design provision for the determination of the effective design width was based on Equation C-1.1-1. Accumulated experience has demonstrated that a more realistic equation as shown below may be used for the determination of the effective width b (Winter, 1970):

\[
b = 1.9t \left[ \frac{E}{f_{\text{max}}} \right] \sqrt{1 - 0.415 \left( \frac{t}{w} \right) \frac{E}{f_{\text{max}}}}
\]  

(C-1.1-3)
The correlation between the test data on stiffened compression elements and Equation C-1.1-3 is illustrated by Yu and LaBoube (2010).

It should be noted that Equation C-1.1-3 may also be rewritten in terms of the $F_{cr}/f_{max}$ ratio as follows:

\[
\frac{b}{w} = \sqrt{\frac{F_{cr}}{f_{max}} (1 - 0.22 \sqrt{\frac{F_{cr}}{f_{max}}})}
\]  

(C-1.1-4)

Therefore, the effective width, $b$, can be determined as

\[
b = \rho w
\]  

(C-1.1-5)

where $\rho$ = reduction factor

\[
\rho = (1 - 0.22 / \sqrt{\frac{f_{max}}{F_{cr}}}) / \sqrt{\frac{f_{max}}{F_{cr}}} = (1 - 0.22 / \lambda) / \lambda \leq 1
\]  

(C-1.1-6)

In Equation C-1.1-6, $\lambda$ is a slenderness factor determined below.

\[
\lambda = \sqrt{\frac{f_{max}}{F_{cr}}}
\]  

(C-1.1-7)

Figure C-1.1-1 shows the relationship between $\rho$ and $\lambda$. It can be seen that when $\lambda \leq 0.673$, $\rho = 1.0$.

![Figure C-1.1-1 Reduction Factor, $\rho$, vs. Slenderness Factor, $\lambda$](image)

Based on Equations C-1.1-5 through C-1.1-7 and the unified approach proposed by Peköz (1986b and 1986c), the 1986 edition of the Specification adopted the non-dimensional format in Section 1.1 for determining the effective design width, $b$, for uniformly compressed stiffened elements. The same design equations were used in the 1996 edition of the Specification and were retained in this edition of the Specification. For design examples, see Part I of the AISI Cold-Formed Steel Design Manual (AISI, 2013).

(b) Serviceability Determination

The effective design width equations discussed above for strength determination can also be used to obtain a conservative effective width, $b_d$, for serviceability determination. It is included in Section 1.1(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 Peköz (1986) indicated that Equations 1.1-6 through 1.1-8 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 either of the two procedures for determining the effective width to be used for serviceability determination.

### 1.1.1 Uniformly Compressed Stiffened Elements With Circular or Noncircular 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 of individual component elements and the overall strength 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 1.1.1 of the Specification for uniformly compressed stiffened elements with circular holes are based on a study conducted by Ortiz-Colberg and Peköz at Cornell University (Ortiz-Colberg and Peköz, 1981). For additional information on the structural behavior of perforated elements, see Yu and Davis (1973a) and Yu and LaBoube (2010).

In 2004, Specification Equation 1.1.1-2 was revised to provide continuity at \( \lambda = 0.673 \).

Within the limitations stated for the size and spacing of perforations and section depth, the provisions were deemed appropriate for members with uniformly compressed stiffened elements, not just wall studs. The validity of this approach for C-section wall studs was verified in a Cornell University project on wall studs reported by Miller and Peköz (1989 and 1994). The limitations included in Specification Section 1.1.1 for the size and spacing of perforations and the depth of studs are based on the parameters used in the test program. Although Figure 1.1.1-1 in the Specification shows a hole centered within the flat width, \( w \), holes not centered within \( w \) are allowed. For such a case, the unstiffened strip, \( c \), and resulting effective width, \( b \), must be calculated separately for the strips on each side of the hole. For sections with perforations which do not meet these limits, the effective area, \( A_e \), can be determined by stub column tests.

The geometric limitations (\( w/t \), etc.) and hole size for the circular and noncircular hole provisions in Specification Section 1.1.1 are not consistent with one another. This anomaly in the limitations is due to the differing scopes of the test programs that serve as the basis for these effective width equations. The provisions for noncircular holes generally give a more conservative prediction of the effective width than the provisions for circular holes, as long as \( d_h/w < 0.4 \). Provisions for designing perforated members using the Direct Strength Method (DSM) can be found in Specification Sections E and F, and Appendix 2.

### 1.1.2 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-1-1, 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-1-1.
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 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 Specification on the basis of the studies made by Peköz (1986b), Cohen and Peköz (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 post-buckling strength (LaBoube and Yu, 1982b; Yu, 1985).

Prior to 2001, the b1 and b2 expressions used in the Specification for the effective width of webs (Equations 1.1.2-3 through 1.1.2-5) implicitly assumed that the flange provided beneficial restraint to the web. Collected data (Cohen and Peköz (1987), Elhouar and Murray (1985), Ellisfritt 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- and Z-sections indicate that Specification Equations 1.1.2-3 through 1.1.2-5 can be unconservative if the overall web width (ho) to overall flange width (bo) ratio exceeds 4. Consequently, in 2001, in the absence of a comprehensive method for handling local web and flange interaction, the Specification adopted a two-part approach for the effective width of webs: an additional set of alternative expressions (Equations 1.1.2-6 and 1.1.2-7), originally developed by Cohen and Peköz (1987), were adopted for ho/bo > 4; while the expressions adopted in the 1986 edition of the Specification (Equations 1.1.2-3 through 1.1.2-5) remain for ho/bo ≤ 4. For flexural members with local buckling in the web, the effect of these changes is that the strengths will be somewhat lower when ho/bo > 4 compared with the 1996 Specification (AISI, 1996). When compared with the CSA S136 (CSA, 1994) Standard, there are only minor changes for members with ho/bo > 4, but an increase in strength will be experienced when ho/bo ≤ 4.

It should be noted that in the Specification, the stress ratio, ψ, is defined as an absolute value. As a result, some signs for ψ have been changed in Specification Equations 1.1.2-2, 1.1.2-3, 1.1.2-6 and 1.1.2-7 as compared with the 1996 edition of the Specification (AISI, 1996).

1.1.3 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/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 noncircular 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 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 $d_h/h < 0.38$. For situations where the $d_h/h \geq 0.38$, 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 must be determined by test in accordance with Section K2.1.

The provisions for circular and noncircular holes also apply to any hole pattern that fits within an equivalent virtual hole. For example, Figure C-1.1.3-1 illustrates the $L_h$ and $d_h$ that may be used for a multiple-hole pattern that fits within a noncircular virtual hole. Figure C-1.1.3-2 illustrates the $d_h$ 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 and web crippling strength of C-section webs are discussed in Sections G3 and G6 of the Commentary, respectively.

### 1.1.4 Uniformly Compressed Elements Restrained by Intermittent Connections

Section I1.3 limits the spacing of connections in compression elements so that the strength of the section is fully developed before buckling occurs between connections. In practice this limitation is often exceeded. Luttrell and Balaji (1992) and Snow and Easterling (2008) developed a method to determine the effective width of compression elements when greater connection spacing exists. The design provisions in Specification Section 1.1.4 were based on
the research work by Snow and Easterling (2008) with 82 standard roof deck tests. All test specimens had multiple flutes and the depth range was between 1-½ in. (38.1 mm) and 7-½ in. (191 mm). As shown in Figures C-1.1.4-1 and C-1.1.4-2, all test compression plates had edge stiffeners.

Figure C-1.1.4-1 Built-Up Deck

Figure C-1.1.4-2 Built-Up Deck in Bending

The full stress potential of the “built-up” section is determined by recognizing the post-buckling strength of the compression plate after local waves form between connections. The method models an equivalent composite transformed section and maintains the classical assumption of linear strain distribution. The critical compression stress, \( F_c \), is based on “column-like” buckling in the plate. The connections provide fixed-end column restraint and \( K = 0.5 \). Before such buckling occurs (\( f < F_c \)), the effective width of the section is calculated using Section 1.1 with the connection lines treated as webs. When the critical stress is reached and exceeded (\( f \geq F_c \)), the compression plate might not resist the same stress, \( f_c \), as the adjacent element that theoretically has slightly less strain. An equivalent width is determined to provide the approximate true force contribution of the buckled plate in resisting bending. This equivalent width is assumed to have an artificially high stress, \( f \), which is compatible with both a constant “E” and linear strain distribution across the “built-up” section; however, the actual stress might be between \( F_c \) and \( f \). \( \rho_t \) provides the effective width at \( F_c \), and \( \rho_m \) allows further effective width reduction to provide the equivalent force. The equivalent transformed section properties cannot be greater than the section calculated using Specification Section 1.1 at the stress level, \( f \). The moment of inertia for deflection is determined by substituting the maximum stress at service load for \( F_y \) and the compression stress at service load, \( f_d \), for \( f \) in Specification Section 1.1.4.

Figure C-1.1.4-2 shows the built-up deck section in bending. In Figures C-1.1.4-1 and C-
1.1.4-2, s is the center-to-center connection spacing along the plate, w is the center-to-center connection spacing across the plate, t is the thickness of cover plate, t₂ is the thickness of the member connected to the cover plate, f is the compression stress in the cover plate, f_c is the compression stress in the element connected to the cover plate, f_t is the maximum tension stress in the member connected to the cover plate, and d is the overall depth of the built-up member.

In 2012, provisions for determining the effective width between the first line of fasteners and the edge stiffener and the effective length of the stiffener were added. The post-buckling stress at the first interior line of connections is applied across the first interior width, w₁ or w₃, as illustrated in Figure C-1.1.4-1, and at the edge stiffener. Specification Equation 1.1.4-7 is based on the approximate shape of the half sine wave restrained by the connectors in the compression element and by the edge stiffener. w’ given in Specification Equation 1.1.4-7 is twice the distance from the stiffener to the apex of the wave and models w in Specification Section 1.3 for the same wave length. Equation 1.1.4-6 sets w as e before “column-like” buckling occurs. Specification Equations 1.3-7 to 1.3-10 are then applied based on w and f. When f reaches or exceeds F_c, Specification Equations 1.3-7 to 1.3-10 are applied based on w’ and f’ to evaluate the stiffener. ρmf approximates the post-buckling stress that cannot be less than F_c since the stiffener must resist F_c as buckling begins.

Jones (Jones et al., 1997) validated Luttrell’s method (1992), but the researchers cautioned its use for single-fluted members having compression plates with edge stiffeners. Luttrell and Balaji (1992) tested built-up deck with compression plate thickness between 0.045 in. (1.14 mm) and 0.06 in. (1.52 mm). Jones (1997) investigated unstiffened cover plates to 0.017 in. (0.432 mm). The research work at the University of Missouri-Rolla (UMR) indicated that the method worked reasonably well for single-fluted members having unstiffened compression plates when the plate thickness exceeded 0.045 in. (1.14 mm). See the illustrative example in the AISI Cold-Formed Steel Design Manual (AISI, 2013).

1.2 Effective Widths of Unstiffened Elements

Similar to stiffened compression elements, the stress in the unstiffened compression elements can reach to the yield stress 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.

When the w/t ratio of the unstiffened element is large, local buckling (Figure C-1.2-1) will occur at the elastic critical stress determined by Equation C-1-1 with a value of k = 0.43. This buckling coefficient is listed in Table C-1-1 for case (c). For the intermediate range of w/t ratios, the unstiffened element will buckle in the inelastic range. Figure C-1.2-2 shows the relationship between the maximum stress for unstiffened compression elements and the w/t ratio, in which Line A is the yield stress of steel, Line B represents the inelastic buckling stress, and 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 stresses in the Specification up to 1986 (Winter, 1970; Yu and LaBoube, 2010). Also shown in Figure C-1.2-2 is Curve E, which represents the maximum stress on the basis of the post-buckling strength of the unstiffened element. The correlation between some test data on unstiffened elements and the predicted maximum stresses is shown in Figure C-1.2-3 (Yu and LaBoube, 2010).
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 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, Peköz, and Winter at Cornell University (Kalyanaraman, Peköz, and Winter, 1977; Kalyanaraman and Peköz, 1978). The evaluation of the test data using $k = 0.43$ was presented and summarized by Peköz in the AISI report (Peköz, 1986b), which indicates that Equation C-1.1-6 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 Peköz in the same report. It was found that the maximum amplitude of the out-of-plane distor
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 the 1986 Specification for the design of cold-formed steel members having unstiffened compression elements.

1.2.1 Uniformly Compressed Unstiffened Elements

In the Specification, it is specified that the effective widths, b, of uniformly compressed unstiffened elements can be determined in accordance with Section 1.1(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-1-1. For serviceability determination, the effective widths of uniformly compressed unstiffened elements can only be determined according to Procedure I of Section 1.1(b) of the Specification, because Procedure II was developed only for stiffened compression elements. See Part I of the AISI Cold-Formed Steel Design Manual for design examples (AISI, 2013).

1.2.2 Unstiffened Elements and Edge Stiffeners With 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 compression element are present, the compressive stress in the edge stiffener is not uniform but varies in proportion to the distance from the neutral axis in flexural members. The unstiffened element (the edge stiffener) in this case has compressive stress applied at both longitudinal edges. The unstiffened element of a section may also be subjected to stress gradients causing tension at one longitudinal edge and compression at the other longitudinal edge. This can occur in I-sections, plain channel sections and angle sections in minor axis bending.
Prior to the 2001 edition of the Specification, unstiffened elements with stress gradient were designed using the Winter effective width equation (Equation C-1.1-4) and \( k = 0.43 \). In 2004, Section 1.2.2 of the Specification adopted the effective width method for unstiffened elements with stress gradient proposed by Bambach and Rasmussen (2002a, 2002b and 2002c), based on an extensive experimental investigation of unstiffened plates tested as isolated elements in combined compression and bending. The effective width, \( b \), (measured from the supported edge) of unstiffened elements with stress gradient causing compression at both longitudinal edges, is calculated using the Winter equation. For unstiffened elements with stress gradients causing tension at one longitudinal edge and compression at the other longitudinal edge, modified Winter equations are specified when tension exists at either the supported or the unsupported edges. The effective width equations apply to any unstiffened element under stress gradient, and are not restricted to particular cross-sections. Figure C-1.2.2-1 demonstrates how the effective width of an unstiffened element increases as the stress at the supported edge changes from compression to tension. As shown in the figure, the effective width curve is independent of the stress ratio, \( \psi \), when both edges are in compression. In this case, the effect of stress ratio is accounted for by the plate buckling coefficient, \( k \), which varies with stress ratio and affects the slenderness, \( \lambda \). When the supported edge is in tension and the unsupported edge is in compression, both the effective width curve and the plate buckling coefficient depend on the stress ratio, as per Equations 1.2.2-4 and 1.2.2-5 of the Specification.

Equations are provided for \( k \), determined from the stress ratio, \( \psi \), applied to the full element width, and \( k \) will usually be higher than 0.43. The equations for \( k \) are theoretical solutions for long plates assuming simple support along the longitudinal edge. A more accurate determination of \( k \) by accounting for interaction between adjoining elements is permitted for plain channels in minor axis bending (causing compression at the unsupported edge of the unstiffened element), based on research of plain channels in compression and bending by Yiu and Peköz (2001).

In the 2016 edition of the Specification, the definition of the stresses \( f_1 \) and \( f_2 \) was revised to
reflect that the effective width calculations for unstiffened elements should be determined iteratively due to a shift of the neutral axis location, as with other elements in the cross-section. However, if all other elements are fully effective, the stresses $f_1$ and $f_2$ may be based on the gross cross-section such that iteration is not required.

The effective width is located adjacent to the supported edge for all stress ratios, including those producing tension at the unsupported edge. Research has found (Bambach and Rasmussen, 2002a) that for the unsupported edge to be effective, tension must be applied over at least half of the width of the element starting at the unsupported edge. For less tension, the unsupported edge will buckle and the effective part of the element is located adjacent to the supported edge. Further, when tension is applied over half of the element or more starting at the unsupported edge, the compressed part of the element will remain effective for elements with w/t ratios less than the limits set out in Section B4.1 of the Specification.

The method for serviceability determination is based on the method used for stiffened elements with stress gradient in Section 1.1.2(b) of the Specification.

1.3 Effective Width of Uniformly Compressed Elements With a Simple Lip Edge Stiffener

An edge stiffener is used to provide continuous support along a longitudinal edge of the compression flange to improve the buckling stress. In most cases, the edge stiffener takes the form of a simple lip. Other types of edge stiffeners can be beneficial and are also used for cold-formed steel members, but are not covered in Specification Section 1.3.

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. 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 has to be determined either on the basis of a flat element disregarding the stiffener or through tests.

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 the 1986 Specification were based on the investigations of adequately stiffened and partially stiffened elements conducted by Desmond, Peköz and Winter (1981a), with additional research work by Peköz and Cohen (Peköz, 1986b). These design provisions were developed on the basis of the critical buckling criterion and the post-buckling strength criterion.

Specification Section 1.3 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 $A_s$ (Peköz, 1986b).

In the 1996 edition of the Specification (AISI, 1996), the design equations for buckling coefficient were changed for further clarity. The requirement of $140^\circ \geq \theta \geq 40^\circ$ for the applicability of these provisions was decided on an intuitive basis. For design examples, see Part I of the AISI Cold-Formed Steel Manual (AISI, 2013).

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 Specification Section 1.3.
In 2001, Dinovitzer’s expressions (Dinovitzer, et al., 1992) for n (Specification Equation 1.3-11) 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.

In 2007, the expressions were limited to cover only simple lip edge stiffeners, as the previously employed expressions for complex lip stiffeners were found to be unconservative in comparison with rigorous nonlinear finite element analysis (Schafer, et al., 2006). Design of members with complex lips may be handled via the Direct Strength Method provided in Chapters E and F, as applicable. In addition, the design provisions for the uniformly compressed elements with one intermediate stiffener were deleted in the 2007 edition of the Specification due to the fact that the effective width of such members can be determined in accordance with Specification Section 1.4.1.

1.4 Effective Widths of Stiffened Elements With Single or Multiple Intermediate Stiffeners or Edge-Stiffened Elements With Intermediate Stiffener(s)

1.4.1 Effective Width of Uniformly Compressed Stiffened Elements With Single or Multiple Intermediate Stiffeners

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.

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 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. In view of the fact that for some cases the design requirements for intermediate stiffeners included in the 1980 Specification could be unduly conservative (Peköz, 1986b), the design provisions were revised in 1986 according to Peköz’s research findings (Peköz, 1986b and 1986c). In 2007, the design of uniformly compressed elements with multiple or single intermediate stiffeners was merged. The multiple intermediate stiffener provisions were developed based on Peköz’s continuing research on intermediate stiffeners (Schafer and Peköz, 1998) and the finding that the method developed in Section 1.4.1 of the Specification for multiple intermediate stiffeners could provide the same reliability as the method for single intermediate stiffeners (Yang and Schafer, 2006) in the previous edition of the Specification (AISI, 2001).

Prior to 2001, the AISI Specification and the Canadian Standard provided different 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 Specification, the design requirements of Section 1.4 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 1.4.1 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-1.4.1-1. Experimental research shows that the distortional mode is prevalent for members with multiple intermediate stiffeners.

![Diagram of local and distortional buckling](image)

**Figure C-1.4.1-1 Local and Distortional Buckling of a Uniformly Compressed Element 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 ensures 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.

In 2010, Specification Equation 1.4.1.1-1 was replaced by

\[
\kappa_{loc} = 4\left(\frac{b_o}{b_p}\right)^2 \tag{C-1.4.1-1}
\]

where

- \( \kappa_{loc} \) = Plate buckling coefficient of element
- \( b_o \) = Total flat width of stiffened element
- \( b_p \) = Sub-element flat width for flange with equally spaced stiffeners

This replacement ensures that Specification Sections 1.4.1.1 and 1.4.1.2 provide the same answer for sub-element local buckling, and replaces the overly conservative estimate of the 2007 edition of the Specification Equation 1.4.1.1-1, which ignored the stiffener width (Schafer,}
1.4.2 Edge-Stiffened Elements With Intermediate Stiffener(s)

The buckling modes for edge-stiffened elements with one or more intermediate stiffeners include local sub-element buckling, distortional buckling of the intermediate stiffener, and distortional buckling of the edge stiffener, as shown in Figure C-1.4.2-1. 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 1.3), 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 1.4.1. However, an edge-stiffened element does not have the same web rotational restraint as a stiffened element; therefore, the constant R of Specification Section 1.4.1 is conservatively limited to be less than or equal to 1.0.

If the edge-stiffened element is partially effective (\(b_o/t > 0.328S\) and \(I_s < I_a\) and thus \(k < 4\), per the rules of Specification Section 1.3), then the intermediate stiffener(s) should be ignored and the provisions of Specification Section 1.3 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 \(\pm 10\) percent for practical intermediate and edge stiffener sizes.

When applying Specification Section 1.4.2 for effective width determination of edge-stiffened elements with intermediate stiffeners, the effective width of the intermediately stiffened flange, \(b_o\), is replaced by an equivalent flat section (as shown in Specification Figure 1.4.1-2). The edge stiffener should not be used in determining the centroid location of the equivalent flat effective width, \(b_o\), for the intermediately stiffened flange.

Stub compression testing performed in 2003 demonstrates the adequacy of this approach (Yang and Hancock, 2003).
APPENDIX 2, ELASTIC BUCKLING ANALYSIS OF MEMBERS

Elastic buckling stress, or stress resultants (axial force, shear force, bending moment, etc.) are used extensively in the Specification for the determination of strength. The buckling of cold-formed steel members includes traditional global buckling modes such as flexural buckling and lateral-torsional buckling, as well as buckling modes that include cross-sectional deformation such as local buckling and distortional buckling.

It is important to realize that elastic buckling itself is not a limit state. Elastic buckling stress or stress resultants are instead used as inputs in various strength equations throughout the Specification. For example, in determining the nominal strength [resistance] of a column, Section E2 requires the global buckling stress, and Section E3 requires the local buckling stress either implicitly in determining the effective width in Section E3.1 or explicitly after conversion to a local buckling force in the Direct Strength Method of Section E3.2. Section E4 requires the input of distortional buckling force. In each case, the elastic buckling stress (or its resultant) is employed in strength expressions that provide varying degrees of post-buckling reserve and interaction with yielding and other buckling modes in determining the nominal strength [resistance] in a given limit state.

2.1 General Provisions

The Specification does not place a preference for what methods are used to determine elastic buckling stress or stress resultants. Conversion between stress and stress resultants is provided.

2.2 Numerical Solutions

2.2.1 Elastic Buckling of Cold-Formed Steel Members

The fundamental buckling modes in a cold-formed steel member include: local buckling, distortional buckling, and global buckling modes: flexural buckling, torsional buckling, and flexural-torsional buckling for compression members, and lateral-torsional buckling for bending members. The fundamental buckling modes are illustrated in Figure C-2.2.1-1.

Figure C-2.2.1-1 Illustration of Fundamental Elastic Buckling Modes for a Lipped Channel in Compression
The elastic buckling load (force) is the load in which the equilibrium of the member is neutral between two alternative states: buckled and straight. Thin-walled cold-formed steel members have at least three relevant elastic buckling modes: local, distortional, and global (Figure C-2.2.2-2). The global buckling mode includes flexural, torsional, or flexural-torsional buckling for columns, and lateral-torsional buckling for beams.

The Effective Width Method traditionally addressed local and global buckling. The distortional buckling consideration was added in 2004. Further, the Effective Width approach to local buckling is to conceptualize the member as a collection of “elements” and investigate local buckling of each element separately.

The Direct Strength Method, introduced in 2004, provides a means to incorporate all three relevant buckling modes into the design process. Further, all buckling modes are determined for the member as a whole rather than element by element. This ensures that compatibility and equilibrium are maintained at element junctures.

Local Buckling. Limit state of buckling of a compression element where the line junctions between elements remain straight and angles between elements do not change.

Local buckling involves significant distortion of the cross-section, but this distortion involves only rotation, not translation, at the fold lines of the member, as shown in Figure C-2.2.1-1. The buckling half-wavelength ($L_{cr}$) for local buckling is less than the largest characteristic dimension of the member under compressive stress (this length is demarcated with a short vertical dashed line in the examples of Figure C-2.2.2-2). Since the local buckling half-wavelength is short, local buckling is difficult to retard, and in general must always be considered. Changes to the geometry of the member (stiffeners, change of thickness, etc.) are the most effective means for changing local buckling loads or moments.

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

Distortional buckling involves both translation and potentially rotation at the fold line of a member. Distortional buckling involves distortion of one portion of the cross-section and predominantly rigid response of a second portion. For instance, the edge-stiffened flanges of the lipped C-section in Figure C-2.2.1-1 are primarily responding as a rigid cross-section while the web is distorting. Distortional buckling occurs at a buckling half-wavelength ($L_{cr,d}$) intermediate to local and global buckling modes. The half-wavelength typically several times larger than the largest characteristic dimension of the member; however, $L_{cr,d}$ is highly dependent on both the loading and the geometry. For some members, distortional buckling may not occur. Bracing can be effective in retarding distortional buckling and boosting the strength of a member.

Global Buckling. A mode of buckling that does not involve distortion of the cross-section. The global buckling includes the following buckling modes:

- **Flexural Buckling.** Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.
- **Torsional Buckling.** Buckling mode in which a compression member twists about its shear center axis.
- **Flexural-Torsional Buckling.** Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.
**Lateral-Torsional Buckling** Buckling mode of a flexural member involving deflection out of the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

For columns, global buckling modes include flexural, torsional and flexural-torsional buckling. For beams bent about their strong-axis, lateral-torsional buckling is the global buckling mode of interest. Figure C-2.2.1-1 illustrates the uncoupled global buckling modes; but for the singly-symmetric section illustrated, the strong-axis flexure (x) and torsion (t) are coupled as two flexural-torsional buckling modes. Global buckling modes involve translation (flexure) and/or rotation (torsion) of the entire cross-section. No distortion exists in any of the elements in the cross-section. The global buckling half-wavelength is equal to the unbraced length ($L_x, L_y$ or $L_t$). Bracing can be effective in retarding global buckling and boosting the member strength.

### 2.2.2 Summary of Available Numerical Solution Methods

**Finite Strip Analysis**

The semi-analytical Finite Strip Method is a numerical solution utilizing plate bending strips to discretize a cold-formed steel cross-section. For a model with simply supported end boundary conditions, a finite strip buckling analysis leads to the member’s signature curve which provides the local, distortional, and global elastic buckling loads or moments as needed in the Specification. Each buckling mode is associated with a particular cross-section shape and a buckling half-wavelength that together provide a complete description of the buckling mode. An example signature curve for a lipped channel in pure compression is provided in Figure C-2.2.2-1, and additional examples are provided in Figure C-2.2.2-2.

Finite strip analysis is a specialized variant of the Finite Element Method. For elastic stability of cold-formed steel structures, it is one of the most efficient and popular methods. Cheung and Tham (1998) explain the basic theory while Hancock et al. (2001) and Ádány and Schafer (2006) provide specific details for stability analysis with this method. Hancock and his researchers pioneered the use of finite strip analysis for stability of cold-formed steel members and convincingly demonstrated the important potential of finite strip analysis in both cold-formed steel design and behavior.

AISI has sponsored research that, in part, has led to the development of the freely available program, CUFSM, which employs the Finite Strip Method for elastic buckling determination of any cold-formed steel cross-section. The program is available at [www.ce.jhu.edu/bschafer/cufsm](http://www.ce.jhu.edu/bschafer/cufsm) and runs on both Windows and Mac platforms. Tutorials and examples are available online at the same address. Other programs that provide similar solutions include THIN-WALL (Hancock, 1995), and CFS. Steel Smart System uses an embedded version of CUFSM.
Figure C-2.2.2-1 Semi-Analytical Finite Strip Analysis Signature Curve Results for Lipped Channel in Compression

As detailed in Commentary Sections 2.2.3 to 2.2.10, specialized variants of the Finite Strip Method exist for shear, general end boundary conditions, members with holes, members with attachments, and for numerically (and automatically) identifying the local, distortional, and global buckling modes, and other special cases.
(a) 9CS2.5x059 of AISI Cold-Formed Steel Design Manual (2002), Example I-8

Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method
(b) 8ZS2.25x059 of AISI Cold-Formed Steel Design Manual (2002), Example I-10

Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis
With Finite Strip Method (cont.)
Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)
(d) 3HU4.5x135 of AISI Cold-Formed Steel Design Manual (2002), Example I-13

Figure C-2.2.2-2 Examples of Bending and Compression Elastic Buckling Analysis With Finite Strip Method (cont.)
Shell Finite Element Methods

Finite element models of cold-formed steel members developed from plate or shell finite elements are capable of providing appropriate buckling solutions for local, distortional, and global buckling. The buckling modes illustrated in Figure C-2.2.1-1 were generated from an eigen-buckling analysis using shell finite elements. Incorporation of specialized details of the section, including holes (as illustrated in Figure C-2.2.2-3(b)), or any other variation along the length, as well as unique end boundary conditions, attachments, etc. are all possible using shell finite element models. In general, the more complicated the situation, the greater the preference for the use of shell finite element-based models.

However, categorization of the numerically determined buckling solutions into local, distortional, and global buckling for use in the Specification often requires significant engineering judgment. A typical shell finite element model may require visual evaluation of as many as 100 modes to find the fundamental buckling modes. Buckling modes often appear as coupled, such as in Figure C2.2.2-3(a), further complicating the identification effort. No direct equivalent to the finite strip analysis signature curve exists for shell finite element models. Additional discussion of identification is provided in Commentary Section 2.2.3.

Figure C-2.2.2-3 Shell Finite Element Elastic Buckling Results for a Lipped Channel in Compression

Most basic finite element texts for solid mechanics include the full details of thin-plate and thin-shell finite elements appropriate for modeling thin-walled cold-formed steel members (e.g., see Cook et al. (1989), or Zienkiewicz and Taylor (1989, 1991)). Due to the common practice of using linear or polynomial shape functions in the Finite Element Method, the number of elements required for reasonable accuracy can be significant and mesh convergence studies may need to be performed to ensure adequate accuracy, particularly for local buckling modes.

A large variety of commercial software provides plate or shell finite elements capable of accurately predicting the elastic buckling modes of cold-formed steel members, including (but not limited to): ABAQUS, ANSYS, MARC, and MSC NASTRAN.

Generalized Beam Theory

Generalized Beam Theory enriches a standard beam finite element with additional cross-section deformation modes consistent with local and distortional buckling and can provide elastic buckling solutions appropriate for use in the Specification. Generalized Beam Theory is capable of generating a member's signature curve for stability as in Figure C-2.2.2-1. In common implementations, the method is directly comparable to the Finite Strip Method, though generally utilizing less degrees of freedom. Specialized variants of Generalized Beam Theory exist for a variety of member conditions, loading conditions, shear deformations, etc.

Generalized Beam Theory originally was developed by Schardt (1989), disseminated by Davies et al. (1994), implemented by Davies and Jiang (1996, 1998), and further expanded by
Silvestre and Camotim (2002a, 2002b), Bebiano et al. (2007, 2015), Camotim et al. (2008) and Basaglia and Camotim (2013). Research on Generalized Beam Theory remains active. The method provides an explicit ability to separate the different buckling modes, making the approach especially amenable in design. Professor Camotim’s group at University of Lisbon developed the program GBTUL and made it available for Generalized Beam Theory based buckling analysis. Version 2 can be used to analyze members with arbitrary flat-walled cross-sections and handles general loading and support conditions (Bebiano et al., 2014).

**Other Solutions**

Any numerical method that incorporates plate theory has the potential to provide an accurate elastic buckling solution for cold-formed steel members. For example, beyond finite strip analysis, finite element analysis, and Generalized Beam Theory, finite differences and boundary elements have both been successfully used in related stability problems (e.g., Harik et al. (1991), Elzein, 1991). In addition, many of the analytical solutions provided in Specification Section 2.3 can be generalized and applied as numerical solutions.

Beam elements used in typical structural analysis software are not capable of including cross-sectional distortion and thus do not include local buckling or distortional buckling. Beam elements used in typical structural analysis software do not explicitly include warping torsion and thus do not accurately model torsional, flexural-torsional, or lateral-torsional buckling. Beam elements used in typical structural analysis software do not account for torsion demands inherent in sections where the shear center and centroid do not coincide, and thus should be used with care for singly- and un-symmetric sections.

**2.2.3 Numerical Solutions – Identifying Buckling Modes**

Once a model is constructed in any of the available methods, the appropriate local, distortional, and global buckling modes must be identified. In some cases this can be a challenge; however, it is often easy to identify that a particular buckling mode is higher than a certain value due to the nature of most analyses which report results from the smallest buckling load (moment) to the largest. For all buckling modes—local, distortional, and global—if the elastic buckling value is large enough, then the cross-section will develop its full capacity (e.g., the yield moment in bending, \( M_y \), or the squash load in compression, \( P_y \)). Using the strength prediction equations of the *Specification*, the following limits can be generated:

*Flexural Members (not considering inelastic reserve)*

- if \( M_{crf} > 1.66M_y \), then no reduction will occur due to local buckling
- if \( M_{crd} > 2.21M_y \), then no reduction will occur due to distortional buckling
- if \( M_{cre} > 2.78M_y \), then no reduction will occur due to global buckling

*Compression Members*

- if \( P_{crf} > 1.66P_y \), then no reduction will occur due to local buckling
- if \( P_{crd} > 3.18P_y \), then no reduction will occur due to distortional buckling
- if \( P_{cre} \geq 3.97P_y \), a 10% or less reduction will occur due to global buckling
- if \( P_{cre} \geq 8.16P_y \), a 5% or less reduction will occur due to global buckling
- if \( P_{cre} \geq 41.64P_y \), a 1% or less reduction will occur due to global buckling
When considering the limits for local buckling, the given values are conservative, since local buckling interacts with global buckling. \( M_y \) and \( P_y \) can be replaced by \( M_{ne} \) and \( P_{ne} \) for the local buckling upper-bounds, where \( M_{ne} \) and \( P_{ne} \) are the nominal strengths determined in the Specifcation for global buckling limit states.

Identification in Finite Strip Analysis

Finite strip analysis is generally the preferred tool for predicting elastic buckling, and in some cases identification of the buckling modes is readily apparent. For example, in Figure C-2.2.2-1, local buckling is the first minimum in the signature curve, distortional buckling is the second minimum in the signature curve, and global buckling is the final descending branch of the signature curve and can be read directly at the global buckling effective length, \( KL \). This is the ideal scenario. Study of the examples of Figure C-2.2.2-2 indicates that immediate identification from the signature curve is often, but not always, possible. If any buckling mode can be identified to be at a buckling value greater than the preceding limits (e.g., \( P_{crd} > 3.18P_y \)), then further identification of that mode need not be pursued.

Finite strip analysis may have indistinct minima in the signature curve. For example, distortional buckling in the Z-section in compression of Figure C-2.2.2-2 is difficult to identify. The basic definitions in Commentary Section 2.2.1 may be used to identify appropriate half-wavelengths and cross-section deformations for manual identification of the modes; however, this can be fairly subjective. In some cases \( (K_xL_x \neq K_yL_y \neq K_tL_t) \) or \( KL < L_{crd} \), it may be easier to use finite strip analysis for local and distortional buckling determination, but use analytical solutions for global buckling. An extension of the Finite Strip Method has been developed that allows for automatic identification and full separation of each mode, termed the constrained Finite Strip Method (Ádány and Schafer, 2008). The method is applied to practical identification of cold-formed steel members in Li and Schafer (2010), is the basis for tabular solutions for lipped channels in CFSEI Tech Note G103-11 (Li and Schafer, 2011), and is provided within the freely available finite strip program CUFSM (Li and Schafer, 2010b). The method is not without its own limitations, and is under active development (Li et al., 2013).

Another study has shown that numerical evaluation of mode shape displacements can be used to identify buckling modes (Glauz, 2016). This study separates section and axial deformations, and quantifies mode shape deformation work to categorize the buckling mode.

Identification in Shell Finite Element Models

Shell finite element models provide the greatest power and flexibility with respect to construction of a model and calculation of the elastic buckling modes and associated loads (moments). However, shell finite element models provide no tools for identification of the modes, and the process can be subjective, time consuming, and difficult to automate. In general, the modes are ordered from smallest to largest and the analyst must visually investigate each mode. Visual identification proceeds using the basic definitions of Commentary Section 2.2.1, but the process can be somewhat subjective.

A conservative approach to identification in shell finite element models is to find the smallest buckling mode that has characteristics similar to a basic definition; for example, flange/lip translation associated with distortional buckling, and assign the buckling load (or moment) to that mode. In some cases, no deformations will be present in the initial results.
that match a given mode (e.g., local buckling in a thicker member). The limits of the preceding section are useful in this process; if any buckling mode can be identified to be at a buckling value greater than the preceding limits (e.g., $P_{cr} > 1.66P_y$), then further identification of that mode need not be pursued.

Numerical tools that augment shell finite element models and allow for automatic identification are under development (Li et al., 2013). The deformation work method (Glauz, 2016) described in Identification in Finite Strip Analysis could be adapted to finite element models for selected cross-sections of the member.

Identification in Generalized Beam Theory Models

The identification of buckling modes in models using Generalized Beam Theory is relatively direct. The analyst determines which deformation modes are to be employed in the model and for any buckling mode can assess to what extent local, distortional, or global buckling modes are engaged based on what deformation modes were included. Models must use sharp corners (no corner radius).

2.2.4 Numerical Solutions - End Boundary Conditions

The semi-analytical Finite Strip Method, which is used to generate the signature curve of Figures C-2.2.2-1 and C-2.2.2-2, is based on ends that are simply supported and warping free. This is consistent with all of the plate buckling solutions traditionally used in the Specification and now provided in Appendix 1. In addition, this is consistent with the boundary conditions used for deriving global buckling modes in Chapter E, Chapter F, and the Analytical Solutions of Appendix 2 in the Specification. Global buckling modes can be modified to account for different end conditions using effective length, $KL$; a similar method is not available for local and distortional buckling. This is because even in a fixed end member, if the length is great enough, local and distortional buckling will be free to form in the interior of the specimen and will converge to the pinned end (warping free) solution. For local buckling, the length where a fixed-end solution converges to the simply supported value is only three to five times the largest characteristic dimension of the member; however, for distortional buckling the length is greater (see Li and Schafer, 2009). For distortional buckling, an approximate solution to correct the simply supported boundary conditions to account for fixed ends, developed by Moen (2008), is recommended:

$$P_{crd}^{\text{fixed}} = D_{\text{boost}}P_{crd}^{\text{pinned}}$$

$$D_{\text{boost}} = 1 + \frac{1}{2}\left(\frac{L_{crd}}{L}\right)^2$$

where

$L_{crd}$ = Buckling half-wavelength for distortional buckling with pinned ends

$L$ = Unbraced length of the member with respect to distortional buckling

Generally, most available methods can directly model a variety of end boundary conditions. However, if the end conditions are not simply supported, the signature curve cannot be constructed, and identification can be more complicated. For finite strip analysis, CUFSM provides a solution for general end boundary conditions (Li and Schafer, 2010b), GBTUL provides a similar solution for generalized beam theory, and, of course, arbitrary end boundary conditions may be included in shell finite element models.
2.2.5 Numerical Solutions – Shear Buckling

Elastic shear buckling is treated as a separate buckling mode (despite being inextricably tied to moment gradient) and the related shear flow is provided for a lipped channel in Figure C-2.2.5-1. Conventional finite strip analysis, Generalized Beam Theory, and even some plate finite element formulations only include the destabilizing effect of longitudinal stresses. Therefore, the Finite Strip Method utilized in CUFSM and the conventional Generalized Beam Theory of GBTUL cannot provide a prediction for shear buckling.

![Figure C-2.2.5-1 Shear Flow Distributions in a Lipped Channel](image)

Available numerical solutions include: (1) a generalized version of the semi-analytical Finite Strip Method (SAFSM) developed by Plank and Wittrick (1974) and implemented in Hancock and Pham (2011), (2) a new version of SAFSM which accounts for the restraint from simply supported ends in the shear mode developed by Hancock and Pham (2013), (3) the Spline Finite Strip Method (SFSM) as developed by Lau and Hancock (1986) and implemented in Pham and Hancock (2009a), or (4) shell finite element models as previously discussed.

Members in pure shear can also cause buckling of the whole section in the form of shear local buckling as shown in Figure C-2.2.5-2(a) or shear distortional buckling as shown in Figure C-2.2.5-2(b) depending on the geometry of the section, loading, and restraint. Shear buckling is different from that for compression or bending in that the nodal lines are not perpendicular to the axis of the section as shown for the shear local buckling mode in Figure C-2.2.5-2(a). The modes shown as Semi-Analytical Finite Strip Method (SAFSM) apply to a single half-wavelength of an infinitely long section, and those designated as Spline Finite Strip Method (SFSM) apply to a section of finite length with simply supported ends. SFSM results are directly comparable to shell finite element method results. Typically, the local mode dominates at short half-wavelengths, and shear distortional buckling is evident at longer half-wavelengths in some instances. The buckling stress versus half-wavelength curves from Hancock and Pham (2011) are shown in Figure C-2.2.5-2(c). The minimum on the SAFSM curve corresponds to the value on the SFSM curve at longer half-wavelengths where end conditions do not affect the buckling.
2.2.6 Numerical Solutions – Members With Holes

Members with holes may be directly modeled using shell finite elements. Identification can be challenging, but model creation and analysis is straightforward (See Figure C-2.2.2-3(b)). Generalized Beam Theory is not well suited for handling holes in members, nor is finite
strip analysis. Spline finite strip analysis has been extended to members with holes (Yao and Rasmussen, 2012), but is not generally available, nor markedly more efficient than shell finite element models.

Given the popularity of finite strip analysis, approximate numerical methods have been developed for finding the local, distortional, and global buckling modes of members with holes using finite strip analysis. The methods generally apply to isolated perforations/holes as found in cold-formed steel framing and related applications. Members with flanged or stiffened holes and members with patterned holes (storage racks) currently require a shell finite element model to establish the elastic buckling values. Work is ongoing to provide general simplified methods for these cases in the near future (Grey and Moen, 2011; Casafont et al., 2012; Smith and Moen, 2014). In general, the provided methods are complementary to the analytical methods for members with holes provided in Appendix 2 of the Specification.

Local Buckling of Members With Holes Using Finite Strip Analysis

Researchers have observed that holes can change the local buckling mode shapes of thin plates and cold-formed steel columns and beams (Kumai, 1952; Schlack, Jr., 1964; Kawai and Ohtsubo, 1968; Vann, 1971; Kesti, 2000; El-Sawy and Nazmy, 2001; Sarawit, 2003; and Moen and Schafer, 2009b). A finite strip approximate method for predicting $P_{cr}$ and $M_{cr}$ including the influence of holes is described in Moen and Schafer (2009c). The method assumes that local buckling occurs as either buckling of the unstiffened strip(s) adjacent to a hole at the net section or as local buckling of the gross section between holes. This approach is an improvement over element-based methods because the interaction between the unstiffened strip and the connected cross-section is explicitly considered. For a column with holes:

$$P_{cr} = \min(P_{cr/nh}, P_{cr/h})$$

where

$P_{cr/nh} = \text{Local buckling load of the gross section by a finite strip analysis}$

$P_{cr/h} = \text{Local buckling load of the net section by a finite strip analysis (e.g., in CUFSM),}$

but restraining the deformations to local buckling and examining only those buckling half-wavelengths shorter than the length of the hole.

To calculate $P_{cr/h}$, a finite strip analysis of the net section is performed as shown in Figure C-2.2.6-1. To ensure a consistent comparison of $P_{cr/h}$ and $P_{cr/nh}$, the reference stress used in the net section and gross section finite strip analyses should be calculated with the same reference load (e.g., 1 kip (4.45 kN) on the net section, 1 kip (4.45 kN) on the gross section).

![Figure C-2.2.6-1. Modeling a Column Net Cross-Section in the Finite Strip Method (e.g., CUFSM): (a) C-Section With a Web Hole, (b) C-Section With a Flange Hole, (c) Hat Section With Web Holes](image)

Eigen-buckling analysis of the restrained cross-section results in an elastic buckling curve.
similar to Figure C-2.2.6-2, where the buckled half-wavelength at the minimum buckling load is $L_{cr/h}$. When the hole length, $L_h$, is less than $L_{cr/h}$, as shown in Figure C-2.2.6-2(a), $P_{cr/h}$ is equal to the buckling load for a single half-wave forming over the length of the hole. (This case is common for circular and square holes, where $L_h$ is less than the width of the cross-sectional element containing the hole.) If $L_h \geq L_{cr/h}$ (Figure C-2.2.6-2(b)), $P_{cr/h}$ is the minimum on the buckling curve, corresponding to a single half-wave forming within the length of the hole. Note that use of the net cross-section for buckling half-wavelengths greater than $L_h$ is conservative by failing to reflect the stiffness contributions of the gross section. Knowledge of the specific buckling half-wavelength of interest allows the Finite Strip Method to be extended by utilizing the net section, but only for half-waves less than the length of the hole, $L_h$.

![Figure C-2.2.6-2 Local Elastic Buckling Curve of Net Cross-Section](image)

The same approach described previously for columns is also applicable to beams, i.e., $M_{cr} = \min(M_{cr/hv}, M_{cr/h})$. In this case, the applied reference stress in the finite strip analysis should be represented as a moment, i.e., 1 kip-in. (113 kN-mm) on the net section and 1 kip-in. (113 kN-mm) on the gross cross-section. See Moen and Schafer (2010b).

A similar approach is recommended for patterned type hole patterns. See Smith and Moen (2014) for additional examples and complete details.

**Distortional Buckling of Members With Holes Using Finite Strip Analysis**

The distortional buckling loads $P_{crd}$ and $M_{crd}$ are, at least in part, dictated by the bending stiffness provided by the web of an open cross-section as it restrains the attached flange from rotating (see Figures C-2.2.1-1, C-2.2.2-1 and C-2.2.2-2). If a hole with length $L_h$ is introduced into the web of an open cross-section, the rotational restraint provided by the web is decreased, resulting in a lower critical distortional buckling load (Kesti, 2000; Moen and Schafer, 2009a). An approximate method for calculating $P_{crd}$ and $M_{crd}$ including the influence of flat-punched unstiffened web holes has been developed by Moen and Schafer (2009c). To implement the method, a finite strip analysis is performed with the gross cross-section to identify the distortional buckling half-wavelength, $L_{crd}$. Then, the web thickness is
modified from \( t \) to \( t_r \) to simulate the reduction in bending stiffness caused by the presence of a web hole, where:

\[
\frac{1}{3}
\]

\[
= \left( 1 - \frac{L_h}{L_{crd}} \right)
\]

(C-2.2.6-2)

and \( L_h \) is the length of the hole. Note that the cross-sectional thickness is modified over the full depth of the web, not just at the location of the hole in the cross-section. The buckling load \( P_{crd} \) or \( M_{crd} \) (including the influence of holes) is obtained with another finite strip analysis of the modified cross-section performed just at \( L_{crd} \) of the gross cross-section with the reduced thickness. The second analysis is only conducted at \( L_{crd} \) as this is the only length for which the reduced thickness \( t_r \) has any relevance. This finite strip elastic buckling simplified method is only appropriate for the case of flat-punched discrete holes in the web or flange (or both).

For patterned holes, as detailed in Smith and Moen (2014), a different reduction is required, specifically:

\[
\frac{1}{3}
\]

\[
= \left( 1 - \frac{A_{web,net}}{A_{web,gross}} \right)
\]

(C-2.2.6-3)

where \( t \) is the thickness of the web, \( A_{web,net} \) is the net area of the web, and \( A_{web,gross} \) is the gross area of the web. Since the reduction is along the full length of the member, the model, with modified thickness, should be completed along the full length of the member. A new finite strip analysis is conducted to find the new \( L_{crd} \) and resulting buckling load, \( P_{crd} \). The model should be loaded with a reference force to account for the reduced area due to the holes. This method has been validated for compressive members and is recommended for use with flexural members as well.

**Global Buckling of Members With Holes Using Finite Strip Analysis**

A general approach to including the influence of holes for global buckling in a finite strip analysis is not available. Bending rigidities \( E I_x \) and \( E I_y \), torsion rigidity \( G J \), and warping rigidity \( E C_w \) each require different reductions in the section to provide the appropriate reduced properties to account for the holes. For example, the reduced thickness needed to provide \( I_{x,avg}, J_{avg}, \) and \( C_{w,net} \) as discussed in the Specification Sections 2.3.2.1 and 2.3.4.1 for analytical solutions are all different—since these rigidities are typically coupled, one finite strip model cannot have two different thickness reductions. As a result, the analytical solutions of Specification Sections 2.3.2.1 and 2.3.4.1, as developed by Moen and Schafer (2009c), are preferred, or shell finite element models may be used directly. Note that the section property calculator in CUFSM does provide a convenient means to calculate the necessary average and net properties.

**2.2.7 Numerical Solutions – Bracing and Attachments**

Bracing and other attachments to a member (sheathing, sheeting, etc.) can have a significant impact on the elastic buckling load (moment, etc.) of a member. Thus, it is often desirable to include such additional elements in the elastic buckling analysis. The most common method is the inclusion of a spring. However, it is possible in shell finite element models to make complete models of a member and its relevant bracing and attachments, and then perform the elastic buckling analysis.
Bracing, or attachments made with fasteners, typically occur with a discrete spacing. This spacing is highly relevant when considering the impact of the bracing on the various buckling modes. Every buckling mode has a characteristic buckling half-wavelength (Figure C-2.2.2.2-2). If bracing or attachment is introduced to the member at a shorter length than this $L_{cr}$, then it may be beneficial to include this support in the elastic buckling prediction. For local buckling, $L_{cr}$ is short, and it is uncommon to have tight enough fastener spacing to significantly impact the mode. For distortional and global buckling typical fastener spacing is relevant, and bracing and attachments should be included.

For distortional buckling, bracing or attachments that restrict rotation at the web/flange juncture are typically of greatest importance. Commentary Section 2.3.3.3 provides a complete discussion of methods for determination of the relevant rotational stiffness. Such stiffness may be modeled discretely in shell finite element models, or smeared into foundation stiffness (along the length) for use in a finite strip analysis. Depending on the implementation, Generalized Beam Theory may use either the discrete or smeared stiffness method.

Significant effort has been directed at determining the restraining effect of sheathing on wall studs (Vieira and Schafer, 2013; Peterman and Schafer, 2014; and Schafer, 2013). The work specifically details methods for conversion of attached sheathing into springs appropriate for use in shell finite element models and finite strip models for elastic buckling determination. The work has wide potential applicability. New guidelines and procedures are expected in the near future.

### 2.2.8 Numerical Solutions – Moment Gradient or Stress Gradient

Moment gradient influences the elastic buckling of a section. For shell finite element models, it is possible to explicitly model the loading conditions and include moment gradient. For Generalized Beam Theory, inclusion of moment gradient is also possible and is available in Version 2 of GBTUL.

Finite strip analysis typically does not include moment gradient (a constant moment is assumed). For local buckling, due to the short half-wavelength of the buckling mode, moment gradient only has a minor influence and no correction needs to be made. For distortional buckling, the moment gradient will increase the buckling moment, and $\beta$ of Specification Equation 2.3.3.3-3 may be applied to increase the result from a finite strip analysis. For global buckling, the moment gradient is also beneficial, and $C_b$ of Specification Equation F2.1.1-2 may be applied.

### 2.2.9 Numerical Solutions—Members With Variation Along Length

Shell finite element models are best suited for handling unusual members with significant variation along the length. In some cases, conservative simplifications using finite strip analysis or Generalized Beam Theory are possible.

### 2.2.10 Numerical Solutions – Built-Up Sections and Assemblages

Elastic buckling of built-up sections may be explicitly considered with shell finite element models. Care must be taken to ensure the end boundary conditions are realistic and that appropriate stiffness is selected for the attachments between members. Finite strip analysis may be used if it is appropriate to smear the attachments along the length of the member—
see Schafer (2013) for a related discussion. Research is underway to develop improved elastic buckling prediction methods for built-up sections.

In some cases, it is both possible and desirable to treat an assemblage as a member—such as trusses, wall panels, and floor systems—for elastic buckling determination. Common practice is to model such assemblages with traditional beam finite elements. Care must be taken with this approach, since local, distortional, and often flexural-torsional buckling are not present in typical beam element models. Secondary models will be required to capture these buckling modes. Shell finite element models do provide a means to include complete assemblage information, but with added complexity.

### 2.3 Analytical Solutions

The Specification provides analytical solutions for elastic buckling of typical cold-formed steel cross-sections. Additional analytical solutions may be found in the SSRC Guide (Ziemian, 2010), the Direct Strength Method Design Guide (AISI, 2006), as well as other reference texts (Allen and Bulson, 1980; Chajes, 1974; and Timoshenko and Gere, 1961). The use of alternative analytical formulae for elastic buckling determination falls under the rational engineering analysis clause of Chapter A.

Many of the analytical solutions provided are relatively complex due to the lack of symmetry and the thin-walled nature of typical cold-formed steel members. In general, numerical solutions, as detailed in Specification Section 2.2, can provide efficient predictions for arbitrary cross-sections, boundary conditions, and loading conditions, and thus are recommended whenever practical. Also, for common sections, elastic buckling solutions are tabulated in the AISI Cold-Formed Steel Design Manual (AISI, 2013) and in CFSEI Tech Note G103-11 (Li and Schafer, 2011).

#### 2.3.1 Members Subject to Compression

##### 2.3.1.1 Global Buckling ($F_{cre}$, $P_{cre}$)

Formulae for global flexural, torsional, and flexural-torsional buckling are provided in Specification Section E2. For a general non-symmetric section, analytical formulae were previously only available in the AISI Cold-Formed Steel Design Manual (AISI, 2013). In 2016, the general solution for global elastic buckling was provided in this section of the Specification. Derivation of the solution is provided in Timoshenko and Gere (1961), and other common reference texts (e.g., Yu and LaBoube, 2010).

The advantage of the provided formulae is that they are applicable to any cross-section including those covered in Specification Section E2. Therefore, if programmed, they provide a general solution. The disadvantage of the formulae is that they are complex. Roots of a cubic equation are required as are torsional cross-section properties that may not be commonly available. The AISI Cold-Formed Steel Design Manual (AISI, 2013) provides examples for calculation of these cross-section properties. In general, the torsion-related cross-section properties may be found from the following:

\[
J = \text{Saint Venant torsion constant of the cross-section, in}^4 \text{ (mm}^4\text{)}
\]

\[
J = \frac{1}{3}(\ell_1 t_1^3 + \ell_2 t_2^3 + ... + \ell_n t_n^3)
\]

\[
C_w = \text{Warping constant of torsion of the cross section, in}^6 \text{ (mm}^6\text{)}
\]
\[ x_0 = \text{Distance from centroid to shear center along the principal x-axis, in. (mm)} \]
\[ y_0 = \text{Distance from centroid to shear center along the principal y-axis, in. (mm)} \]
\[ w_c = \text{Sectorial area measured from centroid, in.}^2 \text{ (mm}^2) \]
\[ w_o = \text{Sectorial area measured from shear center, in.}^2 \text{ (mm}^2) \]

where

\[ \ell_i = \text{Length of cross-section middle line of segment i, in. (mm)} \]
\[ t_i = \text{Wall thickness of segment i, in. (mm)} \]
\[ \ell = \text{Total length of middle line of cross-section, in. (mm)} \]
\[ s = \text{Distance measured along middle line of cross-section from one end to Point P (See Figure C-2.3.1.1-1), in. (mm)} \]
\[ A = \text{Total area of cross-section, in.}^2 \text{ (mm}^2) \]
\[ x, y = \text{Coordinates of principal coordinate system, measured from centroid of any point P along middle line of cross-section, in. (mm)} \]

Figure C-2.3.1.1-1 Non-Symmetric Cross-Section
\[ I_x, I_y = \text{Centroidal moment of inertia of cross-section about principal x- and y-axes, in.}^4 (\text{mm}^4) \]

\[ R_c, R_o = \text{Perpendicular distances from centroid (C.G.) and shear center (S.C.), respectively, to middle line at Point P, in.} R_c \text{ or } R_o \text{ is positive if a vector tangent to the middle line at P in the direction of increasing } s \text{ has a counter-clockwise moment about C.G. or S.C. as shown in Figure C-2.3.1.1-1, in. (mm)} \]

**2.3.1.2 Local Buckling (\( F_{cr}\), \( P_{cr} \))**

*Local buckling* is synonymous with plate *buckling*, and the classic plate *buckling* expression is:

\[
F_{cr} = k \left( \frac{\pi^2 E}{12(1-\mu^2)} \right) \left( \frac{t}{w} \right)^2 \quad \text{(Eq. C-2.3.1.2-1)}
\]

This equation is used extensively in the *Specification*. For the *Effective Width Method* detailed in Appendix 1, Equation 1.1-4 uses \( F_{cr} \) directly to determine the slenderness of an element, which in turn is used to find the *effective width* of the element. For every type of element and for different *stress* gradients on the elements, different solutions are provided for the plate *buckling* coefficient, \( k \), in Appendix 1.

For example, consider a lipped channel in compression with *web* depth, \( h = 8.94 \text{ in.} \) (227.1 mm), *flange* width, \( b = 2.44 \text{ in.} \) (62.00 mm), lip length \( d = 0.744 \text{ in.} \) (18.88 mm), and \( t = 0.059 \text{ in.} \) (1.499 mm) (and ignoring corner radius for this example). In this case:

- **Lip:** \( k = 0.43, F_{cr\text{-lip}} = 0.43 \left[ \frac{\pi^2 E}{12(1-\mu^2)} \right] \left( \frac{t}{d} \right)^2 = 72.1 \text{ ksi (497 MPa)} \)
- **Flange:** \( k \approx 4, F_{cr\text{-flange}} = 4.0 \left[ \frac{\pi^2 E}{12(1-\mu^2)} \right] \left( \frac{t}{b} \right)^2 = 62.4 \text{ ksi (430 MPa)} \)
- **Web:** \( k = 4, F_{cr\text{-web}} = 4.0 \left[ \frac{\pi^2 E}{12(1-\mu^2)} \right] \left( \frac{t}{h} \right)^2 = 4.6 \text{ ksi (32.0 MPa)} \)

Each separate *local buckling stress* is used for determining the element effective width.

However, if the *Direct Strength Method* given in *Specification* Section E3.2 is used for finding the *local buckling strength*, the *local buckling load*, \( P_{cr} \), not *stress*, \( F_{cr} \) is required. Obviously, the three separate element (plate) solutions predict three separate \( P_{cr} \). The *Specification* requires using the minimum \( F_{cr} \), thus the *web local buckling stress* would be used in the preceding example.

In this example, the *web local buckling stress* is significantly lower than the other elements. The User Note in the *Specification* Section 2.3.1.2 warns that in this case, prediction of \( P_{cr} \) based on the minimum \( F_{cr} \) may be very conservative. In this example, the *flange* provides beneficial restraint to the *web* that can be accounted for. The *DSM Design Guide* (Schafer, 2006) provides additional discussion, and improved analytical formulas are available (Schafer, 2001 and 2002; and Schafer and Peköz, 1999). However, for direct numerical solutions or tabulated numerical solutions, the AISI *Cold-Formed Steel Design Manual* (AISI, 2013) and CFSEI Tech Note G103-11 (Li and Schafer, 2011) are preferred since they can readily account for the interaction of the elements.

**2.3.1.3 Distortional Buckling (\( F_{crd}, P_{crd} \))**

The expressions employed in *Specification* Section 2.3.1.3 are derived in Schafer (2002)
and verified for complex stiffeners in Schafer et al. (2006). The equations used for the 
distortional buckling stress in AS/NZS 4600 (1996) are similar, except that when the web is 
very slender and is restrained by the flange, AS/NZS 4600 formulae use a simpler, 
conservative treatment. Since the provided expressions can be complicated, solutions for 
the geometric properties of C- and Z-sections based on centerline dimensions are provided in 
Specification Table 2.3.1.3-1. More refined values including corner radius are possible and 
permitted.

In many cases, the flange will have full or partial rotational restraint due to attachment 
to a brace, panel, or sheeting. In this case the appropriate rotational stiffness, $k_{\phi}$ from the 
restraining elements may be added to the solution. The Commentary Section 2.3.3.3 
provides additional details on $k_{\phi}$ and its determination.

Application of the method is involved and examples are provided in the AISI Cold-
Formed Steel Design Manual (AISI, 2013). While numerical methods or tabulated solutions 
from numerical methods (CFSEI Tech Note G103-11 by Li and Schafer, 2011) are generally 
preferred, a simplified method was provided until 2010 in the Specification. In 2010, the 
simplified approach was moved to the Commentary (as shown below), reflecting the intent 
that the method be used in preliminary design only, as it intentionally provides a lower 
bound solution.

**Simplified Method for Unrestrained C- and Z-Sections With Simple Lip Stiffeners**

For C- and Z-sections that have no rotational restraint of the flange and that are within 
the dimensional limits provided in this section, Equation C-2.3.1.3-1 can be used to 
calculate a conservative prediction of distortional buckling stress, $F_{crd}$, provided the 
following dimensional limits are met:

1. $50 \leq h_o/t \leq 200,$
2. $25 \leq b_o/t \leq 100,$
3. $6.25 < D/t \leq 50,$
4. $45^\circ \leq \theta \leq 90^\circ,$
5. $2 \leq h_o/b_o \leq 8,$ and
6. $0.04 \leq D \sin \theta/b_o \leq 0.5$

where

- $h_o = \text{Out-to-out web depth as defined in Specification Figure 1.1.2-2}$
- $b_o = \text{Out-to-out flange width as defined in Specification Figure 1.1.2-2}$
- $D = \text{Out-to-out lip dimension as defined in Specification Figure 1.3-1}$
- $t = \text{Base steel thickness}$
- $\theta = \text{Lip angle as defined in Specification Figure 1.3-1}$

$$F_{crd} = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{b_o} \right)^2$$  \hspace{1cm} (C-2.3.1.3-1)

where

- $\alpha = \text{A value that accounts for the benefit of an unbraced length, } L_{mb}, \text{ shorter than } L_{cr}, \text{ but can be conservatively taken as } 1.0$
- $= 1.0$ \hspace{1cm} for $L_{m} \geq L_{cr}$
\[ \left( \frac{L_m}{L_{cr}} \right)^{n\left(1 - \frac{L_m}{L_{cr}}\right)} \quad \text{for } L_m < L_{cr} \]  

\[ L_m = \text{Distance between discrete restraints that restrict distortional buckling} \]

\[ L_{cr} = 1.2h_o\left( \frac{b_oD\sin\theta}{h_o t} \right)^{0.6} \leq 10h_o \]  

\[ k_d = 0.05 \leq 0.1\left( \frac{b_oD\sin\theta}{h_o t} \right)^{1.4} \leq 8.0 \]

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

\[ \mu = \text{Poisson’s ratio of steel} \]

### 2.3.2 Members With Holes Subject to Compression

#### 2.3.2.1 Global Buckling ($F_{cre}$, $P_{cre}$) for Members With Holes

2.3.2.1.1 Sections With Holes Not Subject to Torsional or Flexural-Torsional Buckling

The global flexural buckling load decreases when holes are present (Sarawit, 2003; Moen and Schafer, 2009a). This is due to a reduction in the bending rigidity, EI, due to the presence of the holes. The “weighted average” approach to determination of the moment of inertia as used in Specification Table 2.3.2-1 has been shown to provide sufficient accuracy when compared with numerical solutions (Moen and Schafer, 2009c).

If the holes are not spaced uniformly about the mid-height of the column, then a more precise approximation of $I_{avg}$ can be employed where:

![Figure C-2.3.2.1.1-1 A Column With j = 1, 2, ..., n Holes or Net Section Regions](image-url)
Appendix 2, Elastic Buckling Analysis of Members

\[ I_{\text{avg}} = \frac{I_g L_g + I_{\text{net}} L_{\text{net}} + T(I_g - I_{\text{net}})}{L} \]  

(C-2.3.2.1.1-1)

where

\[ T = \frac{L}{2\pi} \sum_{j=1}^{n} \cos \left( \frac{2\pi c_j}{L} \right) \sin \left( \frac{\pi L_{h,j}}{L} \right) \]  

(C-2.3.2.1.1-2)

\[ L_{h,j} = \text{Length of hole or net section region, } j \]

\[ c_j = \text{Distance from top of column to hole centerline or net section region; see Figure C-2.3.2.1.1-1} \]

\[ L_{\text{net}} = \sum_{j=1}^{n} L_{h,j} \]  

(C-2.3.2.1.1-3)

All other variables are defined in Specification Table 2.3.2-1. Note that Equation C-2.3.2.1.1-2 reduces to the expression for \( I_{\text{avg}} \) in Specification Table 2.3.2-1, when the holes are symmetric about the mid-height.

2.3.2.1.2 Doubly- or Singly-Symmetric Sections (With Holes) Subject to Torsional or Flexural-Torsional Buckling

The “weighted average” approach for flexural buckling can be extended to the general case of flexural-torsional buckling as described in Moen and Schafer (2009c). The key extensions are the determination of the influence of holes on torsion rigidities: \( GJ \) and \( EC_w \) and the distances between the shear center and centroid, \( x_o \) and \( y_o \) along the corresponding principal axes, and related polar radius of gyration \( r_o \). The form of the “weighted average” employed for flexural rigidity \( EI \) is found to also work for \( GJ \), and a \( J_{\text{avg}} \) approximation is provided in Specification Table 2.3.2-1 as well. Similar “weighted average” approximations are provided for \( x_o \), \( y_o \), and \( r_o \). The warping torsion rigidity, \( EC_w \), does not follow the “weighted average” approximation, as the presence of holes prevents warping resistance from developing (Moen and Schafer, 2009c). A viable approximation for warping stiffness at the net section is \( EC_{w,\text{net}} \).

Some care must be exercised in the use of average vs. gross area. The buckling load is derived based on the cross-section rigidities: \( EI \), \( GJ \), and \( EC_w \) and the buckling load is independent of the cross-sectional area. Therefore, conversion to buckling stress uses the gross cross-sectional area if the rigidities have been properly reduced to account for holes. Average cross-sectional area is only necessary for calculating the radius of gyration since this quantity is directly tied to the rigidities.

Note that all net section properties, i.e., \( I_{x,\text{net}} \), \( I_{y,\text{net}} \), \( A_{\text{net}} \), \( x_{o,\text{net}} \), \( y_{o,\text{net}} \), \( J_{\text{net}} \), and \( C_{w,\text{net}} \) can be readily calculated with the built-in section property calculator in the freely available open source program CUFSM (Schafer and Ádány, 2006) by setting the element thicknesses to zero at the holes. See Moen and Schafer (2010a).

Also note that based on the hole distribution, it may be feasible to take the conservative approach of using the net section properties instead of the average properties. Similarly, for certain hole distributions, it is reasonable to assume that holes are symmetric about the longitudinal mid-height and use the equations provided in the Specification.
2.3.2.1.3 Point Symmetric Sections With Holes

The provided method for point symmetric sections with holes is a direct extension of the method without holes.

2.3.2.1.4 Non-Symmetric Sections With Holes

Similar to Specification Section 2.3.1.1, Specification Section 2.3.2.1.4 provides a general analytical solution for flexural-torsional buckling. The method is a direct extension of Specification Section 2.3.2.1.2 and may be used for any cross-section.

2.3.2.2 Local Buckling ($F_{ct}$, $P_{ct}$) for Members With Holes

As an extension to the example of Commentary Section 2.3.1.2, consider the same lipped channel in compression with web depth, $h = 8.94$ in. (227.1 mm); flange width, $b = 2.44$ in. (62.0 mm); lip length, $d = 0.744$ in. (18.9 mm); and thickness, $t = 0.059$ in. (1.50 mm); and now a 4-in. (102-mm) deep hole is located at the mid-depth of the web (again, ignoring corner radius). The unstiffened elements at the hole net section have width $a = (h - 4$ in.)/2 = 2.47 in. (62.7 mm), and the $A_{net} = 0.66$ in$^2$ (430 mm$^2$) and $A_g = 0.90$ in$^2$ (583 mm$^2$). The $F_{ct}$ are:

- lip: $k = 0.43$, $f_{ct}$-lip = 0.43[$\pi^2E/(12(1-\mu^2))$](t/d)$^2$ = 72.1 ksi (497 MPa)
- flange: $k \approx 4$, $f_{ct}$-flange = 4.0[$\pi^2E/(12(1-\mu^2))$](t/b)$^2$ = 62.4 ksi (430 MPa)
- web: $k = 4$, $f_{ct}$-web = 4.0[$\pi^2E/(12(1-\mu^2))$](t/h)$^2$ = 4.6 ksi (32.0 MPa)
- web at hole: $k = 0.43$, $f_{ct}$-web = 0.43[$\pi^2E/(12(1-\mu^2))$](t/a)$^2$(A$A_{net}$/A$_g$) = 4.8 ksi (33.3 MPa)

In this case, the net section does not control at the hole and the web local buckling stress away from the hole would still be multiplied by $A_g$ to determine $P_{ct}$. In this case, a smaller hole would have actually reduced the buckling stress at the hole location, e.g., a 2-in. (50.8-mm) hole yields a net section local buckling stress lower than away from the hole. Mitigating this circumstance is the fact that the net section squash load changes as well, and for the net section squash load, smaller holes are always better. Numerical methods may provide superior solutions since they can account for the beneficial restraint provided by the attached elements and can account for details such as edge-stiffened holes, etc.

2.3.2.3 Distortional Buckling ($F_{crd}$, $P_{crd}$) for Members With Holes

The distortional buckling load $P_{crd}$ is, at least in part, dictated by the bending stiffness provided by the web of an open cross-section as it restrains the attached flange from rotating (see Figure C-2.2.6-1). If a hole with length $L_h$ is introduced into the web of an open cross-section, the rotational restraint provided by the web is decreased, resulting in a smaller critical distortional buckling load (Kesti, 2000; Moen and Schafer, 2009a).

An approximate method developed for calculating $P_{crd}$ including the influence of flat-punched unstiffened web holes for finite strip analysis has been developed by Moen and Schafer (2009c) and adapted here for use in Specification Section 2.3.2.3. The key to the method is the reduction of the bending stiffness of the web. This is completed by modifying the web thickness from $t$ to $t_r$. This modification is only required for the rotational stiffness terms; correction of the distortional buckling half-wavelength, $L_{crd}$, is not required.
A similar reduction may also be applied to members that have patterned perforations along the full length of the web (Smith and Moen, 2014). In this case, the reduced stiffness is not only at the hole location but throughout the length of the member.

2.3.3 Members Subject to Flexure

2.3.3.1 Global Buckling ($F_{cre}$, $M_{cre}$)

Global (lateral-torsional) buckling is discussed extensively in Commentary Section F2.1(B). It is worth noting that for lateral-torsional buckling of doubly- and singly-symmetric cross-sections, Specification Section F2.1.1 provides the most general solution available in the Specification; however, completely general solutions for unsymmetric sections, similar to Section 2.3.1.1 for columns, have been derived (Peköz and Winter, 1969a; Peköz and Celebi, 1969b, Yu and LaBoube, 2010).

2.3.3.2 Local Buckling ($F_{cr}$, $M_{cr}$)

The local buckling moment, $M_{cr}$, is determined using the same–minimum of the elements–approach as used for columns in Specification Section 2.3.1.2. $M_{cr}$ is required for the Direct Strength Method of Specification Section F3.2 and may be approximated from the element local buckling stress. Note that the Effective Width Method of Specification Section F3.1 and Appendix 1 utilizes the element local buckling stress directly and ignores interaction amongst the elements.

Since it is a common practice to determine element local buckling stress utilizing the flat portion of a cross-section, for a member under a stress gradient, elements do not have a common reference location. Consider major-axis bending of a braced lipped channel with a 7.8-in. (198-mm) deep web (or overall 8-in. (203-mm) deep), 2.3-in. (58.4-mm) wide flange, 0.068-in. (1.73-mm) thick, and an outer corner radius of 0.10 in. (2.54 mm). Consider only the flange and web for this example (ignore the lip). From Specification Appendix 1, the plate buckling coefficients, $k$, would be found and are approximated here to be 23.9 for the web and 4.0 for the flange.

Flange: $k = 4, \quad F_{cr-flange} = 4.0\left(\frac{\pi^2E}{(12(1-\mu^2))}(0.068/2.3)^2\right) = 93.2$ ksi (643 MPa)

$F_{cr-flange-ext} = \left(F_{cr-flange}\right)(\frac{4}{4 - 0.068/2}) = 94.0$ ksi (648 MPa)

Web: $k = 23.9, \quad F_{cr-web} = 23.9\left(\frac{\pi^2E}{(12(1-\mu^2))}(0.068/7.8)^2\right) = 48.4$ ksi (334 MPa)

$F_{cr-web-ext} = \left(F_{cr-web}\right)(\frac{4}{4 - 0.10}) = 49.7$ ksi (342 MPa)

where $F_{cr-flange}$ and $F_{cr-web}$ are the local buckling stresses of the flange and the web, respectively; and $F_{cr-flange-ext}$ and $F_{cr-web-ext}$ are the corresponding stresses referenced to the extreme compression fiber, respectively.

The results show that the web controls, as 49.7 ksi (342 MPa) is less than 94.0 ksi (648 ksi). Therefore, the web local buckling stress, referenced to the extreme compression fiber is the governing $F_{cr}$, and may be multiplied by the gross section modulus to estimate $M_{cr}$.

2.3.3.3 Distortional Buckling ($F_{crd}$, $M_{crd}$)

The expressions employed here are derived in Schafer (2002) and verified for complex stiffeners in Schafer et al. (2006). The equations used for the distortional buckling stress in
AS/NZS 4600 (1996) are similar, except that when the web is very slender and is restrained by the flange, AS/NZS 4600 uses a simpler, conservative treatment. Since the provided expressions can be complicated, solutions for the geometric properties of C- and Z-sections based on centerline dimensions are provided in Appendix 2 Table 2.3.1.3-1; more refined values including corner radius are possible and permitted.

Application of the method is involved and examples are provided in the AISI Cold-Formed Steel Design Manual (AISI, 2013). Numerical methods or tabulated solutions from numerical methods in CFSEI Tech Note G103-11 (Li and Schafer, 2011) is often preferred.

(a) $k_\phi$ Determination

In many cases, the flange will have full or partial rotational restraint due to attachment to a brace, panel, or sheathing. In this case the appropriate rotational stiffness, $k_\phi$, from the restraining element(s) may be added to the solution. While it is always conservative to ignore the rotational restraint, $k_\phi$, in most cases it is beneficial to include this effect. Due to the large variety of possible conditions, no specific method is provided for determining the rotational restraint.

For framing applications: studs, joists, girts, etc. sheathed with plywood, OSB, or gypsum board, AISI S240 provides provisions for determining $k_\phi$ developed based on mechanics and testing (Schafer, Sangree and Guan 2007 and 2008; Schafer et al. 2010). For metal building applications: purlins and girts with through-fastened sheathing (both with and without insulation), Gao and Moen (2012) provide a method for determining $k_\phi$ confirmed by testing. As reference, past testing on 8-in. and 9.5-in. (203-mm and 241-mm) deep Z-sections with a thickness between 0.069 in. (1.75 mm) and 0.118 in. (3.00 mm), through-fastened 12 in. (205 mm) o.c., to a 36-in. (914 mm) wide, 1-in. (25.4 mm) and 1.5-in. (38.1 mm) high steel panels, with up to 6 in. (152 mm) of blanket insulation between the panel and the Z-section, results in a $k_\phi$ between 0.15 to 0.44 kip-in./rad./in. (0.667 to 1.96 kN-mm/rad./mm) (MRI, 1981).

Additional testing on C- and Z-sections with pairs of through-fasteners provides considerably higher rotational stiffness: for 6-in. and 8-in. (152-mm and 203-mm) deep C-sections with a thickness between 0.054 and 0.097 in. (1.27 and 2.46 mm), fastened with pairs of fasteners on each side of a 1.25-in. (31.8-mm) high steel panel flute at 12 in. (305 mm) o.c., $k_\phi$ is 0.4 kip-in./rad./in. (1.78 kN-mm/rad./mm); and for 8.5-in. (216-mm) deep Z-sections with a thickness between 0.070 in. and 0.120 in. (1.78 mm to 3.05 mm), fastened with pairs of fasteners on each side of 1.25 in. (31.8 mm) high steel panel flute at 12 in. (305 mm) o.c., $k_\phi$ is 0.8 kip-in./rad./in. (3.56 kN-mm/rad./mm) (Yu and Schafer, 2003; Yu, 2005).

Test determination of $k_\phi$ may use AISI S901 (AISI, 2013g). K from this method is a lower bound estimate of $k_\phi$. The member lateral deformation may be removed from the measured lateral deformation to provide a more accurate estimate of $k_\phi$ as detailed in Schafer, Sangree and Guan, 2008; and Schafer et al., 2010.

(b) Moment Gradient

The presence of moment gradient can also increase the distortional buckling moment. However, this increase is lessened if the moment gradient occurs over a longer length. Thus, in determining the influence of moment gradient, $\beta$, the ratio of the end moments, $M_1/M_2$, and the ratio of the critical distortional buckling length to the unbraced length, $L/L_{m}$, should both be accounted for. In 2010, the sign convention on the ratio of moments
M1 and M2 was changed to be consistent with moment gradient expressions for CTF (Specification Equation F2.1.2-3) and Cm (Specification Equation in Section C1) used elsewhere in the Specification. Specification Equation 2.3.3.3-3 and Commentary Equation C-2.3.3.3-2 were revised accordingly. Yu (2005) performed elastic buckling analysis with shell finite element models of C- and Z-sections under different moment gradients to examine this problem. Significant scatter exists in the results; therefore, a lower bound prediction (Specification Equation 2.3.3.3-3) for the increase was selected.

(c) Simplified Method for Unrestrained C- and Z-Sections With Simple Lip Stiffeners

Due to the complexity of the expressions, a simplified method was provided until 2010 in the Specification. In 2010, the simplified approach was moved to the Commentary, reflecting the intent that the method be used in preliminary design only—as it intentionally provides a lower bound solution. For C- and Z-sections that have no rotational restraint of the compression flange and are within the dimensional limits provided in this section, Equation C-2.3.3.3-1 can be used to calculate a conservative prediction of the distortional buckling stress, \( F_{crd} \). See Specification Section 2.3.3.3 or 2.2 for alternative provisions and for members outside the dimensional limits.

The following dimensional limits apply:
(1) \( 50 \leq \frac{h_o}{t} \leq 200 \),
(2) \( 25 \leq \frac{b_o}{t} \leq 100 \),
(3) \( 6.25 < \frac{D}{t} \leq 50 \),
(4) \( 45^\circ \leq \theta < 90^\circ \),
(5) \( 2 \leq \frac{h_o}{b_o} \leq 8 \), and
(6) \( 0.04 \leq \frac{D \sin \theta}{b_o} \leq 0.5 \).

where

- \( h_o \) = Out-to-out web depth as defined in Specification Figure 1.1.2-2
- \( t \) = Base steel thickness
- \( b_o \) = Out-to-out flange width as defined in Specification Figure 1.1.2-2
- \( D \) = Out-to-out lip dimension as defined in Specification Figure 1.3-1
- \( \theta \) = Lip angle as defined in Specification Figure 1.3-1

The distortional buckling stress, \( F_{crd} \), can be calculated as follows:

\[
F_{crd} = \beta k_d \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{b_o} \right)^2
\]  

(C-2.3.3.3-1)

where

- \( \beta \) = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

\[
= 1.0 + 0.4 \left( \frac{L}{L_m} \right)^{0.7} \left( 1 + M_1/M_2 \right)^{0.7} \leq 1.3
\]  

(C-2.3.3.3-2)

where

- \( L \) = Minimum of \( L_{cr} \) and \( L_m \)

\[
L_{cr} = 1.2 h_o \left( \frac{b_o D \sin \theta}{h_o t} \right)^{0.6} \leq 10 h_o
\]  

(C-2.3.3.3-3)

\[
L_m = \text{Distance between discrete restraints that restrict distortional buckling}
\]
(for continuously restrained members \( L_{m} = L_{cr} \))

\( M_{1} \) and \( M_{2} \) = Smaller and larger end moment, respectively, in the unbraced segment \( (L_{m}) \) of the beam; \( M_{1}/M_{2} \) is positive when the moments cause reverse curvature and negative when bent in single curvature

\[
kd = 0.5 \leq 0.6 \left( \frac{b_{o}D\sin \theta}{h_{o}t} \right)^{0.7} \leq 8.0 \tag{C-2.3.3.3-4}
\]

\( E \) = Modulus of elasticity of steel

\( \mu \) = Poisson’s ratio of steel

2.3.4 Members With Holes Subject to Flexure

2.3.4.1 Global Buckling \((F_{cre}, M_{cre})\) for Members With Holes

The “weighted average” method is also applicable to cold-formed steel beams with holes. Bending rigidity \((EI)\), St. Venant torsion rigidity \((GJ)\), shear center location \((x_{o} \text{ and } y_{o})\), and polar radius of gyration \((r_{o})\), all are reduced based on average properties as detailed in Specification and Commentary Section 2.3.2.1. Only warping torsion rigidity \((E_{cW})\) directly employs the net section as detailed in Commentary Section 2.3.2.1.

2.3.4.2 Local Buckling \((F_{cr}, M_{cr})\) for Members With Holes

Local buckling for flexural members with hole(s) in the web follows the same approach as for compression members, as detailed in the Commentary Section 2.3.2.2. When the net section with the hole is checked for local buckling, the unstiffened element buckling stress should be multiplied by the net section modulus and divided by the gross section modulus to develop the approximate stress on the gross cross-section. This stress can then be referenced to the extreme compression fiber and compared with all other elements.

2.3.4.3 Distortional Buckling \((F_{crd}, M_{crd})\) for Members With Holes

Distortional buckling for flexural members with hole(s) in the web follows the same approach as for compressive members, as detailed in Commentary Section 2.3.2.3.

2.3.5 Shear Buckling \((V_{cr})\)

Traditionally, the shear buckling stress and its resultant (shear buckling force) are based on the web alone ignoring interaction from the flanges, and are consistent with Section G2.3 of the Specification. For C- and Z-sections, Specification Section 2.3.5 provides a more refined calculation based on the work of Aswegan and Moen (2012). Pham and Hancock (2011) also provide tabulated solutions for a range of lipped channel section geometries calculated using the Spline Finite Strip Method (SFSM).
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Appendix A

Commentary on Provisions

Applicable to the United States and Mexico

2016 EDITION WITH SUPPLEMENT 1
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APPENDIX A, COMMENTARY ON PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

This commentary on Appendix A provides a record of reasoning behind, and justification for, provisions that are applicable to the United States and Mexico. The format used herein is consistent with that used in Appendix A of the Specification.

I6.2.2 Flexural Members 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 I6.2.2 was added in the 1996 edition of the AISI Specification for determining the nominal flexural strength [resistance] of beams having one flange fastened to a standing seam roof system. In Specification Equation I6.2.2-1, the reduction factor, R, can be determined by AISI S908. Application of the base test method for uplift loading was subsequently validated after further analysis of the research results.

The provisions of Specification Section H4, Combined Bending and Torsion, should not be used in combination with the bending provisions in Specification Section I6.2.2 since these provisions are based on tests in which torsional effects are present.

I6.2.4 Z-Section Compression Members Having One Flange Fastened to a Standing Seam Roof

The strength of axially loaded Z-sections having one flange attached to a standing seam roof may be limited by either a combination of torsional buckling and lateral buckling in the plane of the roof, or by flexural buckling in a plane perpendicular to the roof. As in the case of Z-sections carrying gravity or wind loads as beams, the roof diaphragm and purlin clips provide a degree of torsional and lateral bracing restraint that is significant, but not necessarily sufficient, to develop the full strength of the cross-section.

Specification Equation I6.2.4-1 predicts the lateral buckling strength using an ultimate axial buckling stress ($k_{af}RF_y$) that is a percentage of the ultimate flexural stress ($RF_y$) determined from uplift tests performed using AISI S908, Base Test Method for Purlins Supporting a Standing Seam Roof System, as published by AISI (2013f). This equation, developed by Stolarczyk, et al. (2002), was derived empirically from elastic finite element buckling studies and calibrated to the results of a series of tests comparing flexural and axial strengths using the uplift “Base Test” setup. The full unreduced cross-sectional area, $A$, has been used rather than the effective area, $A_e$, because the ultimate axial stress is generally not large enough to result in a significant reduction in the effective area for common cross-section geometries.

Specification Equation I6.2.4-1 may be used with the results of uplift “Base Tests”
conducted with and without discrete point bracing. There is no limitation on the minimum length because Equation I6.2.4-1 is conservative for spans that are smaller than those tested under the “Base Test” provisions.

The strength of longer members may be governed by axial buckling perpendicular to the roof; consequently, the provisions of Specification Sections E2 and E2.1 should also be checked for buckling about the strong axis.

I6.3.1a Strength of Standing Seam Roof Panel Systems

The introduction of the wind uplift loading required strength factor of 0.67 was a result of research conducted to correlate the static uplift capacity represented by tests performed in accordance with AISI S906 and the dynamic behavior of real wind, by Surry, et al. (2007). This research utilized two separate methods of comparison. The first method utilized full-scale tests conducted at Mississippi State University (MSU) using simulated wind loads on a portion of a standing seam metal roof. The second method utilized model-scale wind tunnel tests carried out at the University of Western Ontario of an aeroelastic “failure” model of the same roof system. In spite of these significantly different approaches, the results obtained were very consistent. It was found that the ASTM E1592 uniform pressure test contains conservatism of about 50 percent for the roof system tested by both approaches, and up to about 80 percent for the other roof systems tested only at MSU. This conservatism arises if the roof system is required to withstand the code-recommended pressure applied as uniform pressure in the ASTM E1592 test, without accounting for the reality of the dynamic spatially-varying properties of the wind-induced pressures. The limits of applicability of this factor (panel thickness and width) are conservatively listed based on the scope of the research. The failure mode is restricted to those failures associated with the load in the clip because this was how the research measured and compared the static and dynamic capacities. Therefore, the 2012 Specification was clarified with respect to the strength factor of 0.67 applying to the clips and fasteners as well as the standing seam roof panels. The required strength factor of 0.67 is not permitted to be used with other observed failures. In addition, the research does not support or confirm whether interpolation would be appropriate between ASTM E1592 tests of the same roof system with different spans, where one test meets the requirements, such as a clip failure, and another test does not, such as a panel failure.

It was determined that the strength factor, 0.67, when applied to the corner and edge zones of steeper slope roofs (greater than 27-degree slope) could yield a nominal wind load less than that in the field of the roof, based on ASCE 7 (2010). So, the limiting value of the wind load in the field of the roof was introduced in the 2012 Specification.

An AISI interpretation was issued in 2012 that clarified the strength factor, 0.67, that was based on research that compared the static and dynamic capacities of these types of roof systems, is justified to be used with the loads or load combinations in the International Building Code (IBC), since this strength factor is based on structural behavior caused by rate or duration of load. Therefore, this 0.67 factor is not duplicative of the consideration given for multiple variable loads in both the strength design load combinations and the allowable stress load combinations used in IBC and ASCE 7 (ASCE, 2010). It would be appropriate to utilize the 0.67 factor on the nominal wind load for any load combination that includes wind uplift as long as all of the conditions stated in Specification Section I6.3.1a (Appendix A) are met.
It is recognized that there are other analytical tools available, especially advanced finite element analyses, that have made strides in replicating the behavior of standing seam roof systems and determining their dynamic uplift capacity. Therefore, alternative means of analysis may be available to compare the dynamic and static behavior that could be used to extend the applicability of this method, provided it was sufficiently calibrated to the existing test data. Any alternative method should also comply with the rational engineering analysis requirements of Section A1.2, including the appropriate safety factor and resistance factor for members and connections.

### J3.4 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 J3.4 for bolts subjected to tension until 1986. In Specification Table J3.4-1, the allowable stresses specified for ASTM A307 (d ≥ 1/2 inch (12.7 mm)), A325, and A490 bolts were based on Section 1.5.2.1 of the AISC Specification (AISC, 1978). It should be noted that the same values were also used in Table J3.2 of the AISC ASD Specification (AISC, 1989). For ASTM 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 (AISI, 1996), Table J3.4-1 provided nominal tensile strengths [resistance] for various types of bolts with applicable safety factors. The allowable tension stresses computed from $F_{nt}/\Omega$ were approximately the same as those permitted by the AISI 1986 ASD Specification. The same table also gave the resistance factor to be used for the LRFD method. In 2012, the table values were realigned with the AISC Specification (AISC, 2010).

The design provisions for bolts subjected to a combination of shear and tension were added in AISI Specification Section J3.4 in 1986. Those design equations were based on Section 1.6.3 of the AISC Specification (AISC, 1978) for the design of bolts used for bearing-type connections.

In 1996, tables which listed 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 were retained in the 2001 edition. In 2007, those tables were replaced by Specification Equations J3.4-2 and J3.4-3 to determine the reduced tension stress of bolts subjected to the combined tension and shear. Specification Equations J3.4-2 and J3.4-3 were adopted to be consistent with the AISC Specification (AISC, 2005).

In 2016, Table J3.4-1 was brought into agreement with AISC Table J3.2 (AISC, 2010) in all related respects, both with regard to safety factors and $F_n$ nominal strengths. As previously stated, the nominal tensile strength values have been reduced by 10 percent for all bolts and threaded fasteners less than 1/2-in. (12-mm) diameter. The nominal shear strength values have also been reduced by 10 percent when threads are not excluded from the shear planes for all bolts and threaded fasteners less than 1/2-in. (12-mm) diameter.
Note that when the required \textit{stress}, f, in either shear or tension, is less than or equal to 20 percent of the corresponding available \textit{stress}, the effects of combined \textit{stress} need not be investigated.

For bolted \textit{connection} design, the possibility of pull-over 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 \textit{non-symmetric sections}, such as C- and Z-sections used as \textit{purlins} or \textit{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 AISI \textit{Cold-Formed Steel Design Manual} (AISI, 2013).
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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. The format used herein is consistent with that used in Appendix B of the Specification.

C2a Lateral and Stability 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 Sections C2.1 and C2.2 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.

C2.1a Symmetrical Beams and Columns

C2.1.1 Discrete Bracing for Beams

This section was revised to retain the two percent requirement for the compressive force in the compressive flange of a flexural member at the braced location only. The discrete bracing provisions for columns are provided in Specification Section C2.3.

C2.2a 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.

C2.2.2 Discrete Bracing

This section provides for brace intervals to prevent the member from rotating about the shear centre 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 Section K2 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.

**C2.2.3 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.
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