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

2007 EDITION

Approved in Canada by the
Canadian Standards Association
CSA S136-07

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

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

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

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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 2001 edition 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, and the previous editions of CSA Standard S136, Cold Formed Steel Structural Members, published by the Canadian Standards Association.

The Specification was developed by a joint effort of the American Iron and Steel Institute’s Committee on Specifications, the Canadian Standards Association’s Technical Committee on Cold Formed Steel Structural Members (S136), and Camara Nacional de la Industria del Hierro y del Acero (CANACERO) in Mexico. This effort was coordinated through the North American Specification Committee, which was made up of members from the AISI Committee on Specifications and CSA’s 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 G and Appendix 1 and 2, that is intended for use in all three countries, and two country-specific appendices (A and B). In this edition of the Specification, what was previously Appendix C has been combined with Appendix A. The new Appendix A is for use in both the United States and Mexico, and Appendix B is for use in Canada. A symbol (\( \phi \)) is used in the main document to point out that additional provisions are provided in the corresponding appendices indicated by the letters.

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

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 developed by AISI and CSA.

The AISI and CSA 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, and CANACERO acknowledge the continuing dedication of the members of the specifications committees and their subcommittees. The membership of these committees follows this Preface.
In this edition of the *Specification*, the terminology jointly used by AISC and AISI is applied. Terms defined in Section A1.3 are italicized when they appear for the first time in each section. A new standard numbering system has been introduced for standards developed by AISI: for example, this *Specification* will be referred as AISI S100-07, where the last two digits represent the year that this standard is updated. All AISI test procedures are referenced by a number with the format “S9xx-yy”, where “xx” is the sequence number, starting from “01”, and “yy” is the year the test standard is developed or updated.

In addition, design provisions are reorganized according to their applicability to wall studs and wall stud assemblies (Section D4), floor, roof, or wall steel diaphragm construction (Section D5), and metal roof and wall systems (Section D6). Accordingly, provisions under Chapters C and D of previous editions are relocated.

The other major technical changes made in this edition of the *Specification*, compared to the previous edition are summarized below.

**Materials**
- Provisions for applications of other steels (Section A2.2) have been rewritten.

**Strength**
- Strength reduction provisions (Section A2.3.2) are introduced for high-strength and low-ductility closed-box section members.

**Elements**
- The effective width equation (Eq. B2.2-2) for uniformly compressed stiffened elements with circular holes has been revised;
- New provisions for unstiffened elements and edge stiffeners with stress gradient (Section B3.2) are introduced.
- The provisions for determining the effective width of uniformly compressed elements with one intermediate stiffener (previously in Section B4.1) have been replaced by the provisions of B5.1.

**Members**
- Provisions for distortional buckling for beams (Section C3.1.4) and columns (C4.2) are introduced.
- The design provisions for bearing stiffeners (previously termed “transverse stiffeners”) have been revised.
- Provisions for web crippling strength for C- or Z-members with an overhang are added in Section C3.4.1.
- The equations for members subjected to combined bending and web crippling have been recalibrated.
- Provisions for considering combined bending and torsional loading (Section C3.6) are added.

**Member Bracing**
- Explicit equations for determining the required bracing force for members having neither
flange connected to sheathing are provided.

- Provisions for determining the required bracing force and stiffness of a compression member are introduced.

Wall Stud and Wall Stud Assemblies

- The sheathing braced design provisions have been removed.
- New framing standards are referenced.

Floor, Roof, or Wall Steel Diaphragm Construction

- The safety factors and the resistance factors for diaphragms (Section D5) have been revised.

Metal Roof and Wall System

- New provisions for Z-section compression members having one flange fastened to a standing seam roof (Section D6.1.4) are added for the United States and Mexico.
- For standing seam roof panel systems, a load reduction is permitted in the United States and Mexico for load combinations that include wind uplift.
- The provisions for determining the anchorage forces and required stiffness for a purlin roof system under gravity load with the top flange connected to metal sheathing have been revised.

Connections

- Provisions for shear strength determination of welded sheet-to-sheet connections are added.
- An interaction check for screws subjected to combined shear and pull-over is added.
- The design provisions for block shear rupture (Section E5.3) have been revised.

Appendix B

- The section for delivered minimum thickness for Canada is deleted.
- The specified loads (Section A3.1) and the load factors and load combinations for LSD (Section A6.1.2) for Canada have been revised.

New Appendices

- Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method, is added. The Direct Strength Method provides alternative design provisions for several sections of Chapters C and D.
- Appendix 2, Second Order Analysis, is added. Appendix 2 provides alternative method for considering the second order effect in members subjected to compression and bending.

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

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July 2007
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S. Rajan Alpine Engineering Products, Inc.
N. A. Rahman The Steel Network, Inc.
C. Ramseyer University of Oklahoma
C. Rogers McGill University
V. E. Sagan Wiss, Janney, Elstner Associates, Inc.
H. Salim University of Missouri-Columbia
B. W. Schafer Johns Hopkins University
N. Schillaci Dofasco Inc.
W. E. Schultz Nucor Vulcraft
R. M. Schuster Consultant
P. A. Seaburg Consultant
R. Serrette Santa Clara University
D. R. Sherman Consultant
W. L. Shoemaker Metal Building Manufacturers Association
K. S. Sivakumaran McMaster University
M. Sommerstein M&H Engineering
T. Sputo Sputo and Lammert Engineering
K. Taing PauTech Corporation
C.R. Taraschuk National Research Council Canada
M. A. Thimons CENTRIA
S. J. Thomas Varco-Pruden Buildings
P. Tian Berridge Manufacturing Company
T. W. J. Trestain T. W. J. Trestain Structural Engineering
L. Vavak Aglo Services Inc.
P. Versavel Behlen Industries Ltd.
R. Vincent Canam Group Inc.
J. Walker Canadian Standards Association
J. Walsh American Buildings Company
D. P. Watson B C Steel
J. Wellinghoff Clark Steel Framing
K. L. Wood K. L. Wood Engineering
L. Xu University of Waterloo
C. Yu University of North Texas
W. W. Yu Consultant
R. Zadeh Marino/Ware
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NORTH AMERICAN SPECIFICATION
FOR THE DESIGN OF COLD-FORMED
STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

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 and Definitions, Chapters A through G, Appendices A and B, and Appendices 1 and 2 that shall apply as follows:

• Appendix A — the United States and Mexico,
• Appendix B — Canada,
• Appendix 1 — alternative design provisions for several sections of Chapter C, and
• Appendix 2 — second-order analysis.

Symbol is used to point out that additional provisions are provided in the appendices indicated by the letter(s).

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 — United States and Mexico, and
• LSD — Canada

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

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

Where the composition or configuration of such components is such that calculation of strength [resistance] and/or stiffness cannot be made in accordance with those provisions, structural performance shall be established from either of the following:

(a) Available strength [factored resistance] or stiffness by tests, undertaken and evaluated in accordance with Chapter F,
(b) Available strength [factored resistance] or stiffness by rational engineering analysis based on appropriate theory, related testing if data is available, and engineering judgment. Specifically, the available strength [factored resistance] is determined from the calculated nominal strength [nominal resistance] by applying the following safety factors or resistance factors:
For members
\[
\begin{align*}
\Omega &= 2.00 \quad \text{(ASD)} \\
\phi &= 0.80 \quad \text{(LRFD)} \\
\phi &= 0.75 \quad \text{(LSD)}
\end{align*}
\]

For connections
\[
\begin{align*}
\Omega &= 2.50 \quad \text{(ASD)} \\
\phi &= 0.65 \quad \text{(LRFD)} \\
\phi &= 0.60 \quad \text{(LSD)}
\end{align*}
\]

When rational engineering analysis is used to determine the nominal strength [nominal resistance] for a limit state already provided in this Specification, the safety factor shall not be less than the applicable safety factor (Ω) nor shall the resistance factor exceed the applicable resistance factor (φ) 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 “shall be permitted” is used to express an option or that which is permissible within the limits of the Specification. In Standards developed by the Canadian Standards Association, “shall be permitted” is expressed by “may”.

The following terms are italicized when they appear for the first time in a sub-section of the Specification. Terms 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 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 [nominal 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 the provisions of Chapters A through G, Appendices A and B, and Appendices 1 and 2 of this Specification or its specific references, 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, \( A_e \), calculated using the *effective widths* of component elements in accordance with Chapter B. If the effective widths of all component elements, determined in accordance with Chapter B, 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 Strength Method.** An alternative design method detailed in Appendix 1 that provides predictions of member strengths [resistances] 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.

**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.

**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. Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

Local Bending. Limit state of large deformation of a flange under a concentrated transverse force.

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.

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.

Performance Test. Test made on structural members, connections, and assemblies whose performance cannot be determined in accordance with Chapters A through G 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.

Published Specification. Requirements for a steel listed by a manufacturer, processor, producer, purchaser, or other body, which (1) are generally available in the public domain or are available to the public upon request, (2) are established before the steel is ordered, and (3) 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-Δ 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, φ**. Factor that accounts for unavoidable deviations of the nominal strength 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-δ and P-Δ, unless specified otherwise) are included.

**Second-Order Effect.** Effect of loads acting on the deformed configuration of a structure; includes P-δ effect and P-Δ effect.

**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.

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

**SS (Structural Steel).** ASTM designation for certain sheet steels intended for structural applications.

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

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

**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.

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

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

**Unsymmetric Section**. Section not symmetric either about an axis or a point.

**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.

**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.

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**ASD and LRFD Terms (USA 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 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.

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.

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

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

LSD Terms (Canada):

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

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

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

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 shall be 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 Newtons and length in millimeters), and MKS units (force in kilograms and length in centimeters).

A2 Material

A2.1 Applicable Steels

This Specification requires the use of steels intended for structural applications as defined in general by the specifications of the American Society for Testing and Materials listed in this Section. The term SS shall designate sheet material and the terms HSLAS and HSLAS-F shall designate high-strength low-alloy steels.

ASTM A36/A36M, Standard Specification for Carbon Structural Steel

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

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

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 A588/A588M, Standard Specification for High-Strength Low-Alloy Structural Steel
with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick

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

ASTM A653/A653M (SS Grades 33 (230), 37 (255), 40 (275), 50 (340) Class 1, Class 3 and Class 4, and 55 (380); 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

ASTM A792/A792M (Grades 33 (230), 37 (255), 40 (275), and 50 (340) Class 1 and Class 4), Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

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, 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)), Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process. Thicknesses of Grades 55 (380) and higher that do not meet the minimum 10% elongation requirement are limited per Section A2.3.2.

A2.2 Other Steels

See Section A2.2 of Appendix A or B.

A2.3 Ductility

Steels not listed in Section A2.1 and used for structural members and connections in accordance with Section A2.2 shall comply with ductility requirements in either Section A2.3.1 or Section A2.3.2:

A2.3.1 The ratio of tensile strength to yield stress shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch (50 mm) gage length or 7 percent for an eight-inch (200 mm) gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 in. (12.7 mm) gage length across the fracture shall
not be less than 20 percent, and (2) uniform elongation outside the fracture shall not be less than 3 percent. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material shall be restricted to the design of purlins, girts, and curtain wall studs in accordance with Sections C3.1.1(a), C3.1.2, D6.1.1, D6.1.2, D6.2.1, and country-specific requirements given in A2.3.1a of the Appendix A or B. For purlins, girts, and curtain wall studs subject to combined axial load and bending moment (Section C5), $\frac{\Omega P}{P_n}$ shall not exceed 0.15 for ASD, $\frac{P_u}{\phi_c P_n}$ shall not exceed 0.15 for LRFD, and $\frac{P_t}{\phi_c P_n}$ shall not exceed 0.15 for LSD.

A2.3.2 Steels conforming to ASTM A653/A653M SS Grade 80 (550), A1008/A1008M SS Grade 80 (550), A792/A792M Grade 80 (550), A875/A875M SS Grade 80 (550), thicknesses of ASTM A1039 Grades 55 (380), 60 (410), 70 (480), and 80 (550) that do not meet the minimum 10 percent elongation requirement in Section A2.3.1, and other steels that do not meet the provisions of Section A2.3.1 shall be permitted for concentrically loaded closed box section compression members as given in Exception 2 below and for multi-web configurations such as roofing, siding, and floor decking as given in Exception 1 provided that:

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

Alternatively, the suitability of such steels for any multi-web configuration shall be demonstrated by load tests in accordance with the provisions of Section F1. Available strengths [factored resistances] based on these tests shall not exceed the available strengths [factored resistances] calculated in accordance with Chapters B through G, Appendices A and B, and Appendices 1 and 2, using the specified minimum yield stress, $F_{sy}$, and the specified minimum tensile strength, $F_u$.

Exception 1: For multiple-web configurations, a reduced specified minimum yield stress, $R_b F_{sy}$, shall be permitted for determining the nominal flexural strength [moment resistance] in Section C3.1.1(a), for which the reduction factor, $R_b$, shall be determined in accordance with (a) or (b):

(a) For stiffened and partially stiffened compression flanges
   - For $w/t \leq 0.067E/F_{sy}$, $R_b = 1.0$
   - For $0.067E/F_{sy} < w/t < 0.974E/F_{sy}$, $R_b = 1-0.26[wF_{sy}/(tE) - 0.067]^{0.4}$ (Eq. A2.3.2-1)
   - For $0.974E/F_{sy} \leq w/t \leq 500$, $R_b = 0.75$

(b) For unstiffened compression flanges
   - For $w/t \leq 0.0173E/F_{sy}$, $R_b = 1.0$
   - For $w/t > 0.0173E/F_{sy}$, $R_b = 1.0$
For $0.0173E/F_{sy} < w/t \leq 60$

$$R_b = 1.079 - 0.6 \sqrt{wF_{sy} / (tE)} \quad (Eq. \ A2.3.2-2)$$

where

$w = \text{Flat width of compression flange}$
$t = \text{Thickness of section}$
$E = \text{Modulus of elasticity of steel}$
$F_{sy} = \text{Specified minimum yield stress determined in accordance with Section A7.1}$

$\leq 80 \text{ ksi (550 MPa, or 5620 kg/cm}^2)$

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

Exception 2: For concentrically loaded compression members with a closed box section, a reduced yield stress, $0.9F_{sy}$, shall be permitted to be used in place of $F_y$ in Eqs. C4.1-2, C4.1-3, and C4.1-4 for determining the axial strength in Section C4. A reduced radius of gyration $(R_r)(r)$ shall be used in Eq. C4.1.1-1 when the value of the effective length $KL$ is less than $1.1L_0$, where $L_0$ is given by Eq. A2.3.2-3, and $R_r$ is given by Eq. A2.3.2-4.

$$L_0 = \pi r \sqrt[4]{E / F_{cr}} \quad (Eq. \ A2.3.2-3)$$

$$R_r = 0.65 + 0.35(KL) / 1.1L_0 \quad (Eq. \ A2.3.2-4)$$

where

$L_0 = \text{Length at which local buckling stress equals flexural buckling stress}$
$r = \text{Radius of gyration of full unreduced cross section}$
$F_{cr} = \text{Minimum critical buckling stress for section calculated by Eq. B2.1-5}$
$R_r = \text{Reduction factor}$
$KL = \text{Effective length}$

A2.4 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 thicknesses shall be permitted at bends, such as corners, due to cold-forming effects.

A3 Loads

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

A4 Allowable Strength Design

A4.1 Design Basis

Design under this section of the Specification shall be based on Allowable Strength Design (ASD) principles. All provisions of this Specification shall apply, except for those in Sections A5 and A6 and in Chapters C and F designated for LRFD and LSD.
A4.1.1 ASD Requirements

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

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

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

where

\[ R \quad \text{Required strength} \]
\[ R_n = \text{Nominal strength specified in Chapters B through G and Appendix 1} \]
\[ \Omega = \text{Safety factor specified in Chapters B through G and Appendix 1} \]
\[ R_n/\Omega = \text{Allowable strength} \]

A4.1.2 Load Combinations for ASD

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

A5 Load and Resistance Factor Design

A5.1 Design Basis

Design under this section of the Specification shall be based on Load and Resistance Factor Design (LRFD) principles. All provisions of this Specification shall apply, except for those in Sections A4 and A6 and in Chapters C and F designated for ASD and LSD.

A5.1.1 LRFD Requirements

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

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

\[ R_u \leq \phi R_n \]  \hspace{1cm} (Eq. A5.1.1-1)

where

\[ R_u = \text{Required strength} \]
\[ \phi = \text{Resistance factor specified in Chapters B through G and Appendix 1} \]
\[ R_n = \text{Nominal strength specified in Chapters B through G and Appendix 1} \]
\[ \phi R_n = \text{Design strength} \]

A5.1.2 Load Factors and Load Combinations for LRFD

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

A6 Limit States Design

A6.1 Design Basis

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

**A6.1.1 LSD Requirements**

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

\[ \phi R_n \geq R_f \]  \hspace{1cm} (Eq. A6.1.1-1)

where

- \( \phi \) = Resistance factor specified in Chapters B through G and Appendix 1
- \( R_n \) = Nominal resistance specified in Chapters B through G and Appendix 1
- \( \phi R_n \) = Factored resistance
- \( R_f \) = Effect of factored loads

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

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

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

**A7.1 Yield Stress**

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

**A7.2 Strength Increase from Cold Work of Forming**

Strength increase from cold work of forming shall be permitted by substituting \( F_{ya} \) for \( F_y \), where \( F_{ya} \) is the average yield stress of the full section. Such increase shall be limited to Sections C2, C3.1 (excluding Section C3.1.1(b)), C4, C5, D4, and D6.1. The limits and methods for determining \( F_{ya} \) shall be in accordance with (a), (b) and (c).

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity \( \rho \) for strength determination is unity as determined in accordance with Section B2 for each of the component elements of the section, the design yield stress, \( F_{yar} \) of the steel shall be determined on the basis of one of the following methods:

1. full section tensile tests [see paragraph (a) of Section F3.1],
2. stub column tests [see paragraph (b) of Section F3.1],
3. computed in accordance with Eq. A7.2-1.

\[ F_{ya} = CF_{yc} + (1 - C) F_{yt} \leq F_{uv} \]  \hspace{1cm} (Eq. A7.2-1)

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

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

(Eq. A7.2-2)

Eq. A7.2-2 applies only when \( \frac{F_{uv}}{F_{yv}} \geq 1.2 \), \( R/t \leq 7 \), and the included angle \( \leq 120^o \).

where

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

(Eq. A7.2-3)

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

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

\[ t = \text{Thickness of section} \]

\[ m = 0.192 \left( \frac{F_{uv}}{F_{yv}} \right) - 0.068 \]

(Eq. A7.2-4)

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

\[ F_{yf} = \text{Weighted average tensile yield stress of flat portions established in accordance with Section F3.2 or virgin steel yield stress if tests are not made} \]

(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.

A8 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.

A9 Referenced Documents

The following documents or portions thereof are referenced in this Specification and shall be considered part of the requirements of this Specification. Refer to Section A9a of Appendix A or B for documents applicable to the corresponding country.

1. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:
   - AISI S200-07, North American Standard for Cold-Formed Steel Framing - General Provisions
   - AISI S210-07, North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design
   - AISI S211-07, North American Standard for Cold-Formed Steel Framing – Wall Stud Design
   - AISI S212-07, North American Standard for Cold-Formed Steel Framing – Header Design
   - AISI S214-07, North American Standard for Cold-Formed Steel Framing – Truss Design
   - AISI S901-02*, Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies
   - AISI S902-02, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns
   - AISI S906-04, Standard Procedures for Panel and Anchor Structural Tests
Note: * AISI test procedures previously designated as AISI TSn-xx are re-designated to AISI S9n-xx, where “n” is the test procedure sequence number and “xx” is the year the standard was developed or updated.

2. American Society of Mechanical Engineers (ASME), 1828 L Street, NW, Washington, DC 20036:
   ASME B46.1-2000, Surface Texture, Surface Roughness, Waviness, and Lay

3. American Society for Testing and Materials (ASTM), 100 Barr Harbor Drive, West Conshohocken, Pennsylvania 19428-2959:
   ASTM A36/A36M-05, Standard Specification for Carbon Structural Steel
   ASTM A194/A194M-06, Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service, or Both
   ASTM A242/A242M-04e1, Standard Specification for High-Strength Low-Alloy Structural Steel
   ASTM A283/A283M-03, Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
   ASTM A307-04, Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength
   ASTM A325-06, Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
   ASTM A325M-05, Standard Specification for Structural Bolts, Steel, Heat Treated, 830 MPa Minimum Tensile Strength [Metric]
   ASTM A354-04, Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
   ASTM A490-06, Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength
   ASTM A490M-04a, Standard Specification for High Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]
   ASTM A500-03a, Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
   ASTM A529/A529M-05, Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality
   ASTM A563-04, Standard Specification for Carbon and Alloy Steel Nuts
   ASTM A572/A572M-06, Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
   ASTM A588/A588M-05, Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4-in. [100 mm] Thick
ASTM A606-04, Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A653/A653M-06, Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

ASTM A792/A792M-05, Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process

ASTM A847/A847M-05, Standard Specification for Cold-Formed Welded and Seamless High Strength, Low Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance

ASTM A875/A875M-05, Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process

ASTM A1003/A1003M-05, Standard Specification for Steel Sheet, Carbon, Metallic- and Nonmetallic-Coated for Cold-Formed Framing Members

ASTM A1008/A1008M-05b, 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-05a, 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-04, Standard Specification for Steel, Sheet, Hot Rolled, Carbon, Commercial and Structural, Produced by the Twin-Roll Casting Process


ASTM F436-04, Standard Specification for Hardened Steel Washers

ASTM F436M-04, Standard Specification for Hardened Steel Washers [Metric]

ASTM F844-04, Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959-05a, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

ASTM F959M-04, Standard Specification for Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners [Metric]

4. U. S. Army Corps of Engineers:

5. Factory Mutual, Corporate Offices, 1301 Atwood Avenue, P.O. Box 7500, Johnston, RI 02919: FM 4471, Approval Standard for Class 1 Metal Roofs, 1995
B. Elements

B1 Dimensional Limits and Considerations

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

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

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

1. **Stiffened compression element** having one longitudinal edge connected to a web or flange element, the other stiffened by:
   - Simple lip, w/t ≤ 60
   - Any other kind of stiffener
     i) when I_s < I_a, w/t ≤ 60
     ii) when I_s ≥ I_a, w/t ≤ 90

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

2. **Stiffened compression element** with both longitudinal edges connected to other stiffened elements, w/t ≤ 500

3. **Unstiffened compression element**, w/t ≤ 60

It shall be noted that unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full available strength [factored resistance], without affecting the ability of the member to develop the required strength [effect of factored loads].

Stiffened elements having w/t ratios greater than 500 provide adequate available strength [factored resistance] to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design equations of this Specification.

(b) Flange Curling

Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, Eq. B1.1-1 shall be permitted to be applied to compression and tension flanges, either stiffened or unstiffened as follows:

\[ w_f = \sqrt{0.061tdE/f_{av} \frac{d}{100c_t/d}} \]  \hspace{1cm} (Eq. B1.1-1)

where
- \( w_f \) = Width of flange projecting beyond web; or half of distance between webs for box- or U-type beams
- \( t \) = Flange thickness
- \( d \) = Depth of beam
- \( f_{av} \) = 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.)

$$c_f = \text{Amount of curling displacement}$$

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

Where the beam has a span of less than 30\(w_f\) (\(w_f\) as defined below) and it carries one concentrated load, or several loads spaced farther apart than 2\(w_f\), the effective design width of any flange, whether in tension or compression, shall be limited by the values in Table B1.1(c).

<table>
<thead>
<tr>
<th>(L/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.

B1.2 Maximum Web Depth-to-Thickness Ratios

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

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

(b) For webs which are provided with bearing stiffeners satisfying the requirements of Section C3.7.1:

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

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

where

\(h\) = Depth of flat portion of web measured along plane of web

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

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) Strength Determination

The effective width, \(b\), shall be calculated from either Eq. B2.1-1 or Eq. B2.1-2 as follows:
Chapter B, Elements

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

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

where

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

\[ \rho = \text{Local reduction factor} \]

\[ = (1 - 0.22/\lambda)/\lambda \quad (\text{Eq. B2.1-3}) \]

\[ \lambda = \text{Slenderness factor} \]

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

where

\[ f = \text{Stress in compression element computed as follows:} \]

For flexural members:

1. If Procedure I of Section C3.1.1 is used:
   - When the initial yielding is in compression in the element considered, \( f = F_y \).
   - When the initial yielding is in tension, the compressive stress, \( f \), in the element considered is determined on the basis of the effective section at \( M_y \) (moment causing initial yield).

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

3. If Section C3.1.2.1 is used, \( f \) is the stress \( F_c \) as described in that Section in determining effective section modulus, \( S_c \).

For compression members, \( f \) is taken equal to \( F_n \) as determined in accordance with Section C4.

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

where

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

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

\[ \text{Values for different types of elements are given in the applicable sections.} \]

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

\[ t = \text{Thickness of uniformly compressed stiffened element} \]

\[ \mu = \text{Poisson’s ratio of steel} \]

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated from either Eq. B2.1-6 or Eq. B2.1-7 as follows:

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

\[ b_d = \rho w \quad \text{when} \quad \lambda > 0.673 \quad (\text{Eq. B2.1-7}) \]

where

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

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

1. Procedure I:
   - A conservative estimate of the effective width is obtained from Eqs. B2.1-3 and
B2.1-4 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:

$\rho = 1$ when $\lambda \leq 0.673$

$\rho = \frac{(1.358 - 0.461/\lambda)}{\lambda}$ when $0.673 < \lambda < \lambda_c$  \hspace{1cm} (Eq. B2.1-8)

$\rho = \frac{(0.41 + 0.59 \sqrt{F_y/f_d} - 0.22/\lambda)}{\lambda}$ when $\lambda \geq \lambda_c$  \hspace{1cm} (Eq. B2.1-9)

$\rho \leq 1$ for all cases.

where

$\lambda = $ A value as defined by Eq. B2.1-4, except that $f_d$ is substituted for $f$

$\lambda_c = 0.256 + 0.328 (w/t) \sqrt{F_y/E}$  \hspace{1cm} (Eq. B2.1-11)

---

**Figure B2.1-1 Stiffened Elements**

### B2.2 Uniformly Compressed Stiffened Elements with Circular or Non-Circular Holes

(a) **Strength Determination**

For circular holes:

The effective width, $b$, shall be calculated by either Eq. B2.2-1 or Eq. B2.2-2 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 $\geq 3d_h$

$\begin{align*}
  b &= w - d_h & \text{when } \lambda \leq 0.673 \\
  b &= \left[ 1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w} \right] \left[ \frac{w}{\lambda} - \frac{(0.085d_h)}{w} \right] & \text{when } \lambda > 0.673
\end{align*}$  \hspace{1cm} (Eq. B2.2-1)

In all cases, $b \leq w - d_h$

where

$w = $ Flat width

$t = $ Thickness of element

$d_h = $ Diameter of holes
\( \lambda \) = A value as defined in Section B2.1

For non-circular holes:

A uniformly compressed stiffened element with non-circular holes shall be assumed to consist of two unstiffened strips of flat width, c, adjacent to the holes (see Figure B2.2-1). The effective width, b, of each unstiffened strip adjacent to the hole shall be determined in accordance with B2.1(a), except that 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, \( s \geq 24 \text{ in.} \) (610 mm),
2. Clear distance from the hole at ends, \( s_{\text{end}} \geq 10 \text{ in.} \) (254 mm),
3. Depth of hole, \( d_h \leq 2.5 \text{ in.} \) (63.5 mm),
4. Length of hole, \( L_h \leq 4.5 \text{ in.} \) (114 mm), and
5. Ratio of the depth of hole, \( d_h \), to the out-to-out width, \( w_o \), \( d_h/w_o \leq 0.5 \).

Alternatively, the effective width, b, shall be 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 B2.1(b), except that \( f_d \) is substituted for \( f \), where \( f_d \) is the computed compressive stress in the element being considered.

\[ \psi = \left| \frac{f_2}{f_1} \right| \quad (\text{Eq. B2.3-1}) \]

---

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

The following notation shall apply in this section:

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

(i) For webs under stress gradient ($f_1$ in compression and $f_2$ in tension as shown in Figure B2.3-1(a)), the effective widths and plate buckling coefficient shall be calculated as follows:

\[ k = 4 + 2(1 + \psi)^3 + 2(1 + \psi) \]  \hspace{1cm} (Eq. B2.3-2)

For $h_0/b_0 \leq 4$

- $b_1 = b_e/(3 + \psi)$  \hspace{1cm} (Eq. B2.3-3)
- $b_2 = b_e/2$ when $\psi > 0.236$  \hspace{1cm} (Eq. B2.3-4)

In addition, $b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.
For $h_0/b_o > 4$
\[
b_1 = \frac{b_o}{3 + \psi} \quad \text{(Eq. B2.3-6)}
\]
\[
b_2 = \frac{b_o}{1 + \psi} - b_1 \quad \text{(Eq. B2.3-7)}
\]

(ii) For other stiffened elements under stress gradient ($f_1$ and $f_2$ in compression as shown in Figure B2.3-1(b))
\[
k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \quad \text{(Eq. B2.3-8)}
\]
\[
b_1 = \frac{b_o}{3 - \psi} \quad \text{(Eq. B2.3-9)}
\]
\[
b_2 = b_o - b_1 \quad \text{(Eq. B2.3-10)}
\]

(b) Serviceability Determination

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

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

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

The provisions of Section B2.4 shall apply within the following limits:
1. $d_h/h \leq 0.7$,
2. $h/t \leq 200$,
3. Holes centered at mid-depth of web,
4. Clear distance between holes $\geq 18$ in. (457 mm),
5. Non-circular holes, corner radii $\geq 2t$,
6. Non-circular holes, $d_h \leq 2.5$ in. (64 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 mm).

where
- $d_h$ = Depth of web hole
- $h$ = Depth of flat portion of web measured along plane of web
- $t$ = Thickness of web
- $L_h$ = Length of web hole
\[ b_1, b_2 = \text{Effective widths defined by Figure B2.3-1} \]

(a) Strength Determination

When \( \frac{d_h}{h} < 0.38 \), the effective widths, \( b_1 \) and \( b_2 \), shall be determined in accordance with Section B2.3(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 B3.1(a), assuming the compression portion of the web consists of an unstiffened element adjacent to the hole with \( f = f_1 \), as shown in Figure B2.3-1.

(b) Serviceability Determination

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

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) Strength Determination

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

(b) Serviceability Determination

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

\[ \text{Figure B3.1-1 Unstiffened Element with Uniform Compression} \]

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

The following notation shall apply in this section:

\[ b = \text{Effective width measured from the supported edge, determined in accordance with Section B2.1(a), with } f \text{ equal to } f_1 \text{ and with } k \text{ and } \rho \text{ being determined in accordance with this section} \]

\[ b_o = \text{Overall width of unstiffened element of unstiffened C-section member as defined in Fig. B3.2-3} \]

\[ f_1, f_2 = \text{Stresses shown in Figures B3.2-1, B3.2-2, and B3.2-3 calculated on the basis of the gross section. Where } f_1 \text{ and } f_2 \text{ are both compression, } f_1 \geq f_2. \]

\[ h_o = \text{Overall depth of unstiffened C-section member as defined in Fig. B3.2-3} \]

\[ k = \text{Plate buckling coefficient defined in this section or, otherwise, as defined in Section} \]
B2.1(a)

\[ t = \text{Thickness of element} \]

\[ w = \text{Flat width of unstiffened element, where } w/t \leq 60 \]

\[ \psi = \left| \frac{f_2}{f_1} \right| \text{ (absolute value)} \quad (\text{Eq. B3.2-1}) \]

\[ \lambda = \text{Slenderness factor defined in Section B2.1(a) with } f = f_1 \]

\[ \rho = \text{Reduction factor defined in this section or, otherwise, as defined in Section B2.1(a)} \]

(a) **Strength Determination**

The effective width, \( b \), of an unstiffened element under stress gradient shall be determined in accordance with Section B2.1(a) with \( f \) equal to \( f_1 \) 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 B2.1(a) shall be determined in accordance with this section.

(1) When both \( f_1 \) and \( f_2 \) are in compression (Fig. B3.2-1), the plate buckling coefficient shall be calculated in accordance with either Eq. B3.2-2 or Eq. B3.2-3 as follows:

If the stress decreases toward the unsupported edge (Figure B3.2-1(a)):

\[ k = \frac{0.578}{\psi + 0.34} \quad (\text{Eq. B3.2-2}) \]

If the stress increases toward the unsupported edge (Figure B3.2-1(b)):

\[ k = 0.57 - 0.21\psi + 0.07\psi^2 \quad (\text{Eq. B3.2-3}) \]
(2) When \( f_1 \) is in compression and \( f_2 \) in tension (Fig. B3.2-2), the reduction factor and plate buckling coefficient shall be calculated as follows:

(i) If the unsupported edge is in compression (Figure B3.2-2(a)):
\[
\rho = \frac{1}{\lambda} \left(1 - \frac{0.22(1 + \psi)}{\lambda}\right) \quad \text{when} \quad \lambda \leq 0.673(1 + \psi)
\]
\[
\rho = (1 + \psi) \left(1 - \frac{0.22}{\lambda}\right) \quad \text{when} \quad \lambda > 0.673(1 + \psi)
\]
\[
k = 0.57 + 0.21\psi + 0.07\psi^2
\]

(Eq. B3.2-4)

(ii) If the supported edge is in compression (Fig. B3.2-2(b)):
For \( \psi < 1 \)
\[
\rho = \frac{1}{\lambda} \quad \text{when} \quad \lambda \leq 0.673
\]
\[
\rho = (1 - \psi) \left(1 - \frac{0.22}{\lambda}\right) + \psi \quad \text{when} \quad \lambda > 0.673
\]
\[
k = 1.70 + 5\psi + 17.1\psi^2
\]

(Eq. B3.2-6)

For \( \psi \geq 1 \),
\[
\rho = 1
\]

The effective width, \( b \), of the unstiffened elements of an unstiffened C-section member shall be 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 B3.2-3 (a)):
\[
b = w \quad \text{when} \quad \lambda \leq 0.856
\]
\[
b = \rho w \quad \text{when} \quad \lambda > 0.856
\]

where
\[
\rho = \frac{0.925}{\sqrt{\lambda}}
\]
\[
k = 0.145(b_o/h_o) + 1.256
\]

(Eq. B3.2-10)

(Eq. B3.2-11)

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 in tension (Figure B3.2-3(b)), the effective width is determined in accordance with Section B2.3.

Figure B3.2-3 Unstiffened Elements of C-Section under Stress Gradient for Alternative Methods

In calculating the effective section modulus \( S_e \) in Section C3.1.1 or \( S_c \) in Section C3.1.2.1, the extreme compression fiber in Figures B3.2-1(b), B3.2-2(a), and B3.2-3(a) shall be taken as
the edge of the effective section closer to the unsupported edge. In calculating the effective section modulus \( S_e \) in Section C3.1.1, the extreme tension fiber in Figures B3.2-2(b) and B3.2-3(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 B3.2(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 B3.2-1, B3.2-2, and B3.2-3, respectively, based on the gross section at the load for which serviceability is determined.

B4 Effective Width of Uniformly Compressed Elements with a Simple Lip Edge Stiffener

The effective widths of uniformly compressed elements with a simple edge stiffener shall be calculated in accordance with (a) for strength determination and (b) for serviceability determination.

(a) Strength Determination

For \( \frac{w}{t} \leq 0.328S \):

\[
I_a = 0 \quad \text{(no edge stiffener needed)}
\]

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

\[
b_1 = b_2 = \frac{w}{2} \quad \text{(Eq. B4-2)}
\]

\[
d_s = d'_s \quad \text{(Eq. B4-3)}
\]

For \( \frac{w}{t} > 0.328S \)

\[
b_1 = \frac{b}{2} \text{ (} R_1 \text{)} \quad \text{(Eq. B4-4)}
\]

\[
b_2 = b - b_1 \quad \text{(Eq. B4-5)}
\]

\[
d_s = d'_s \text{ (} R_1 \text{)} \quad \text{(Eq. B4-6)}
\]

where

\[
S = 1.28\sqrt{E/f} \quad \text{(Eq. B4-7)}
\]

\[
w = \text{Flat dimension defined in Figure B4-1}
\]

\[
t = Thickness \text{ of section}
\]

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

\[
b = \text{Effective design width}
\]

\[
b_1, b_2 = \text{Portions of effective design width as defined in Figure B4-1}
\]

\[
d_s = \text{Reduced effective width of stiffener as defined in Figure B4-1, and used in computing overall effective section properties}
\]

\[
d'_s = \text{Effective width of stiffener calculated in accordance with Section B3.2 (see Figure B4-1)}
\]

\[
(R_1) = I_s/I_a \leq 1 \quad \text{(Eq. B4-9)}
\]

where

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

\[ = \frac{(d^3 t \sin^2 \theta)}{12} \tag{Eq. B4-10} \]

See Figure B4-1 for definitions of other dimensional variables.

The effective width, \( b \), in Eqs. B4-4 and B4-5 shall be calculated in accordance with Section B2.1 with the plate buckling coefficient, \( k \), as given in Table B4-1 below:

### Table B4-1

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

where

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

(b) Serviceability Determination

The effective width, \( b_d \), used in determining serviceability shall be calculated as in Section B4(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.

![Diagram of Simple Lip Edge Stiffener](image)

**Figure B4-1 Elements with Simple Lip Edge Stiffener**
B5 Effective Widths of Stiffened Elements with Single or Multiple Intermediate Stiffeners or Edge Stiffened Elements with Intermediate Stiffener(s)

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

The following notation shall apply as used in this section.

- $A_g =$ *Gross area* of element including stiffeners
- $A_s =$ Gross area of stiffener
- $b_e =$ *Effective width* of element, located at centroid of element including stiffeners; see Figure B5.1-2
- $b_o =$ Total flat width of stiffened element; see Figure B5.1-1
- $b_p =$ Largest sub-element *flat width*; see Figure B5.1-1
- $c_i =$ Horizontal distance from edge of element to centerline(s) of stiffener(s); see Figure B5.1-1
- $F_{cr} =$ 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 which restrict distortional buckling of element
- $R =$ Modification factor for distortional plate buckling coefficient
- $n =$ Number of stiffeners in element
- $t =$ Element *thickness*
- $i =$ Index for stiffener “$i$”
- $\lambda =$ Slenderness factor
- $\rho =$ Reduction factor

The effective width shall be calculated in accordance with Eq. B5.1-1 as follows:

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

where

$$\rho = \begin{cases} 1 & \text{when } \lambda \leq 0.673 \\ \frac{(1 - 0.22/\lambda)}{\lambda} & \text{when } \lambda > 0.673 \end{cases} \quad (Eq. \ B5.1-2)$$
The plate buckling coefficient, \( k \), shall be determined from the minimum of \( R_k d \) and \( k_{loc} \), as determined in accordance with Section B5.1.1 or B5.1.2, as applicable.

\[ k = \text{the minimum of } R_k d \text{ and } k_{loc} \]  
(Eq. B5.1-5)

\[ R = \begin{cases} 2 & \text{ when } b_o/h < 1 \\ \frac{11 - b_o/h}{5} \geq \frac{1}{2} & \text{ when } b_o/h \geq 1 \end{cases} \]  
(Eq. B5.1-6)

### B5.1.1 Specific Case: \( n \) Identical Stiffeners, Equally Spaced

For uniformly compressed elements with multiple identical and equally spaced stiffeners, the plate buckling coefficients and effective widths shall be calculated as follows:

**(a) Strength Determination**

\[
k_{loc} = 4(n + 1)^2 \]  
(Eq. B5.1.1-1)

\[
k_d = \frac{(1 + \beta^2)^2 + \gamma(1 + n)}{\beta^2(1 + \delta(n + 1))} \]  
(Eq. B5.1.1-2)

where

\[
\beta = (1 + \gamma(n + 1))^{1/4} \]  
(Eq. B5.1.1-3)

where

\[
\gamma = \frac{10.92I_{sp}}{b_o t^3} \]  
(Eq. B5.1.1-4)

\[
\delta = \frac{A_s}{b_o t} \]  
(Eq. B5.1.1-5)

If \( L_{br} < \beta b_o \), \( L_{br}/b_o \) shall be 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 B5.1.1(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.

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

For uniformly compressed stiffened elements with multiple stiffeners of arbitrary size,
location and number, the plate buckling coefficients and effective widths shall be calculated as follows:

(a) **Strength Determination**

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

where

\[ \beta = \left( \frac{2 \sum_{i=1}^{n} \gamma_i \omega_i + 1}{n} \right)^{1/4} \]

where

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

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

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

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

---

Figure B5.1-1 Plate Widths and Stiffener Locations

Figure B5.1-2 Effective Width Locations
(b) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be calculated as in Section B5.1.2(a), except that $f_d$ 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.

B5.2 Edge Stiffened Elements with Intermediate Stiffener(s)

(a) Strength Determination

For edge stiffened elements with intermediate stiffener(s), the effective width, $b_o$, 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 $b_o/t > 0.328S$, then the plate buckling coefficient, $k$, is determined in accordance with Section B4, but with $b_o$ replacing $w$ in all expressions:
  - If $k$ calculated from Section B4 is less than 4.0 ($k < 4$), the intermediate stiffener(s) is ignored and the provisions of Section B4 are followed for calculation of the effective width.
  - If $k$ calculated from Section B4 is equal to 4.0 ($k = 4$), the effective width of the edge stiffened element is calculated from the provisions of Section B5.1, with the following exception:
    - $R$ calculated in accordance with Section B5.1 is less than or equal to 1.

where

$\begin{align*}
    b_o &= \text{Total flat width of edge stiffened element} \\
    \text{See Sections B4 and B5.1 for definitions of other variables.}
\end{align*}$

(b) Serviceability Determination

The effective width, $b_d$, used in determining serviceability shall be calculated as in Section B5.2(a), except that $f_d$ 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.
C. MEMBERS

C1 Properties of Sections

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

C2 Tension Members

See Section C2 of Appendix A or B for the provisions of this section.

C3 Flexural Members

C3.1 Bending

The nominal flexural strength [moment resistance], $M_n$, shall be the smallest of the values calculated in accordance with sections C3.1.1, C3.1.2, C3.1.3, C3.1.4, D6.1.1, D6.1.2, and D6.2.1, where applicable.

See Section C3.6, as applicable, for laterally unrestrained flexural members subjected to both bending and torsional loading, such as loads that do not pass through the shear center of the cross-section, a condition which is not considered in the provision of this section.

C3.1.1 Nominal Section Strength [Resistance]

The nominal flexural strength [moment resistance], $M_n$, shall be calculated either on the basis of initiation of yielding of the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II), as applicable. The applicable safety factors and the resistance factors given in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5 or A6.

For sections with stiffened or partially stiffened compression flanges:

$$\Omega_b = 1.67 \quad (ASD)$$
$$\Phi_b = 0.95 \quad (LRFD)$$
$$= 0.90 \quad (LSD)$$

For sections with unstiffened compression flanges:

$$\Omega_b = 1.67 \quad (ASD)$$
$$\Phi_b = 0.90 \quad (LRFD)$$
$$= 0.90 \quad (LSD)$$

(a) Procedure I — Based on Initiation of Yielding

The nominal flexural strength [moment resistance], $M_n$, for the effective yield moment shall be calculated in accordance with Eq. C3.1.1-1 as follows:

$$M_n = S_e F_y \quad (Eq. \ C3.1.1-1)$$
where

\[ S_e = \text{Elastic section modulus of effective section calculated relative to extreme compression or tension fiber at } F_y \]

\[ F_y = \text{Design yield stress determined in accordance with Section A7.1} \]

(b) Procedure II – Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity shall be permitted to be used when 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 \( \lambda_1 \).
4. The shear force does not exceed 0.35\( F_y \) for ASD, and 0.6\( F_y \) for LRFD and LSD times the web area (ht for stiffened elements or wt for unstiffened elements).
5. The angle between any web and the vertical does not exceed 30°.

The nominal flexural strength [moment resistance], \( M_n \), shall not exceed either 1.25\( S_e F_y \), as determined in accordance with Procedure I of Section C3.1.1 (a) or that causing a maximum compression strain of \( C_y e_y \) (no limit is placed on the maximum tensile strain).

where

\[ h = \text{Flat depth of web} \]
\[ t = \text{Base steel thickness of element} \]
\[ e_y = \text{Yield strain} = \frac{F_y}{E} \]
\[ w = \text{Element flat width} \]
\[ E = \text{Modulus of elasticity} \]
\[ C_y = \text{Compression strain factor calculated as follows:} \]

(i) Stiffened compression elements without intermediate stiffeners

For compression elements without intermediate stiffeners, \( C_y \) shall be calculated as follows:

\[ C_y = 3 \text{ when } w/t \leq \lambda_1 \]
\[ C_y = 3 - 2 \left( \frac{w}{t} - \lambda_1 \right) \left( \frac{\lambda_2 - \lambda_1}{\lambda_2 - \lambda_1} \right) \text{ when } \lambda_1 < \frac{w}{t} < \lambda_2 \]  
(Eq. C3.1.1-2)

\[ C_y = 1 \text{ when } w/t \geq \lambda_2 \]

where

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

(Eq. C3.1.1-4)

(ii) Unstiffened compression elements

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

(ii-1) Unstiffened compression elements under stress gradient causing compression at one longitudinal edge and tension at the other longitudinal edge:

\[
C_y = \begin{cases} 
3.0 & \text{when } \lambda \leq \lambda_3 \\
3 - 2\left[\frac{(\lambda - \lambda_3)}{(\lambda_4 - \lambda_3)}\right] & \text{when } \lambda_3 < \lambda < \lambda_4 \\
1 & \text{when } \lambda \geq \lambda_4 
\end{cases}
\]

(Eq. C3.1.1-5)

where

\[ \lambda_3 = 0.43 \]
\[ \lambda_4 = 0.673(1 + \psi) \]  

(Eq. C3.1.1-6)

\[ \psi = \text{A value defined in Section B3.2} \]

(ii-2) Unstiffened compression elements under stress gradient causing compression at both longitudinal edges:

\[ C_y = 1 \]

(ii-3) Unstiffened compression elements under uniform compression:

\[ C_y = 1 \]

(iii) 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:

\[ C_y = 1 \]

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

C3.1.2 Lateral-Torsional Buckling Strength [Resistance]

The provisions of this section shall apply to members with either an open cross-section as specified in Section C3.1.2.1 or closed box sections as specified in Section C3.1.2.2.

Unless otherwise indicated, the following safety factor and resistance factors and the nominal strengths calculated in accordance with Sections C3.1.2.1 and C3.1.2.2 shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[ \Omega_b = 1.67 \quad (ASD) \]
\[ \phi_b = 0.90 \quad (LRFD) \]
\[ = 0.90 \quad (LSD) \]
C3.1.2.1 Lateral-Torsional Buckling Strength [Resistance] of Open Cross-Section Members

The provisions of this section shall apply to I-, Z-, C-, and other singly-symmetric section flexural members (not including multiple-web deck, U- and closed box-type members, and curved or arch members) subject to lateral-torsional buckling. The provisions of this section shall not apply to laterally unbraced compression flanges of otherwise laterally stable sections. See Section D6.1.1 for C- and Z-purlins in which the tension flange is attached to sheathing.

For laterally unbraced segments of singly-, doubly-, and point-symmetric sections subject to lateral-torsional buckling, the nominal flexural strength [moment resistance], $M_n$, shall be calculated in accordance with Eq. C3.1.2.1-1.

$$M_n = S_c F_c$$

(Eq. C3.1.2.1-1)

where

- $S_c$ = Elastic section modulus of effective section calculated relative to extreme compression fiber at $F_c$

$F_c$ shall be determined as follows:

For $F_e \geq 2.78 F_y$

The member segment is not subject to lateral-torsional buckling at bending moments less than or equal to $M_y$. The available flexural strength [moment resistance] shall be determined in accordance with Section C3.1.1(a).

For $2.78 F_y > F_e > 0.56 F_y$

$$F_c = 10 F_y \left(1 - \frac{10 F_y}{36 F_e}\right)$$

(Eq. C3.1.2.1-2)

For $F_e \leq 0.56 F_y$

$$F_c = F_e$$

(Eq. C3.1.2.1-3)

where

- $F_y$ = Design yield stress as determined in accordance with Section A7.1

- $F_e$ = Elastic critical lateral-torsional buckling stress calculated in accordance with (a) or (b)

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

(i) For bending about the symmetry axis:

$$F_e = \frac{C_b r_o A}{S_f} \sqrt{\sigma_{ey} \sigma_t}$$

for singly- and doubly-symmetric sections

(Eq. C3.1.2.1-4)

$$F_e = \frac{C_b r_o A}{2 S_f} \sqrt{\sigma_{ey} \sigma_t}$$

for point-symmetric sections

(Eq. C3.1.2.1-5)

where

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

(Eq. C3.1.2.1-6)
where

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

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

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

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

\( C_b \) shall be 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.

\( r_o \) = Polar radius of gyration of cross-section about shear center

\[
 r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} \quad (\text{Eq. C3.1.2.1-7})
\]

where

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

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

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

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

\[
 \sigma_{ey} = \frac{\pi^2 E}{(K_y L_y/r_y)^2} \quad (\text{Eq. C3.1.2.1-8})
\]

where

\( E \) = Modulus of elasticity of steel

\( K_y \) = Effective length factors for bending about y-axis

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

\[
 \sigma_t = \frac{1}{A r_o^2} \left[ \frac{\pi^2 E C_w}{K_t L_t} \right] \quad (\text{Eq. C3.1.2.1-9})
\]

where

\( G \) = Shear modulus

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

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

\( K_t \) = Effective length factors for twisting

\( L_t \) = Unbraced length of member for twisting

For singly-symmetric sections, x-axis shall be the axis of symmetry oriented such that the shear center has a negative x-coordinate.

For point-symmetric sections, such as Z-sections, x-axis shall be the centroidal axis perpendicular to the web.

Alternatively, \( F_e \) shall be permitted to be calculated using the equation given in (b) for doubly-symmetric I-sections, singly-symmetric C-sections, or point-symmetric Z-sections.
(ii) For singly-symmetric sections bending about the centroidal axis perpendicular to the axis of symmetry:

\[
F_e = \frac{C_s \sigma_{ex}}{C_{TF} S_f} \left[ j + C_s \sqrt{2 + \frac{\sigma_t}{\sigma_{ex}} (\sigma_t/\sigma_{ex})} \right]
\]

(Eq. C3.1.2.1-10)

where

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

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

(Eq. C3.1.2.1-11)

where

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

\( C_{TF} = 0.6 - 0.4 (M_1/M_2) \)

(Eq. C3.1.2.1-12)

where

- \( M_1 \) and \( M_2 \) = 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

\[
j = \frac{1}{2 l_y} \left[ \int_A x^3 dA + \int_A xy^2 dA \right] - x_o
\]

(Eq. C3.1.2.1-13)

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

\[
F_e = C_b \pi^2 E d I_{yc} / S_f (K_y L_y)^2
\]

for doubly-symmetric I-sections and singly-symmetric C-sections

(Eq. C3.1.2.1-14)

\[
F_e = C_b \pi^2 E d I_{yc} / 2 S_f (K_y L_y)^2
\]

for point-symmetric Z-sections

(Eq. C3.1.2.1-15)

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

See (a) for definition of other variables.

C3.1.2.2 Lateral-Torsional Buckling Strength [Resistance] of Closed Box Members

For closed box members, the nominal flexural strength [moment resistance], \( M_{nv} \), shall be determined in accordance with this section.
If the laterally unbraced length of the member is less than or equal to \( L_u \), the nominal flexural strength [moment resistance] shall be determined in accordance with Section C3.1.1. \( L_u \) shall be calculated as follows:

\[
L_u = \frac{0.36C_b \pi \sqrt{E G J I_y}}{F_y S_f} \tag{Eq. C3.1.2.2-1}
\]

See Section C3.1.2.1 for definition of variables.

If the laterally unbraced length of a member is larger than \( L_u \), as calculated in Eq. C3.1.2.2-1, the nominal flexural strength [moment resistance] shall be determined in accordance with Section C3.1.2.1, where the critical lateral-torsional buckling stress, \( F_e \), is calculated as follows:

\[
F_e = \frac{C_b \pi \sqrt{E G J I_y}}{K_y L_y S_f} \tag{Eq. C3.1.2.2-2}
\]

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

See Section C3.1.2.1 for definition of other variables.

### C3.1.3 Flexural Strength [Resistance] of Closed Cylindrical Tubular Members

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 [moment resistance], \( M_n \), shall be calculated in accordance with Eq. C3.1.3-1. The safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
M_n = F_c S_f \tag{Eq. C3.1.3-1}
\]

\[
\Omega_b = 1.67 \quad (ASD)
\]

\[
\phi_b = 0.95 \quad (LRFD)
= 0.90 \quad (LSD)
\]

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

\[
F_c = 1.25 \ F_y \tag{Eq. C3.1.3-2}
\]

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

\[
F_c = \left[ 0.970 + 0.020 \left( \frac{E/F_y}{D/t} \right) \right] F_y \tag{Eq. C3.1.3-3}
\]

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

\[
F_c = 0.328E/(D/t) \tag{Eq. C3.1.3-4}
\]

where
\[
D = \text{Outside diameter of cylindrical tube}
\]
\[
t = \text{ Thickness}
\]
\[
F_c = \text{Critical flexural buckling stress}
\]
\[
S_f = \text{Elastic section modulus of full unreduced cross section relative to extreme}
\]
compression fiber

See Section C3.1.2.1 for definitions of other variables.

C3.1.4 Distortional Buckling Strength [Resistance]

The provisions of this section shall apply to I-, Z-, C-, and other open cross-section members that employ compression flanges with edge stiffeners, with the exception of members that meet the criteria of Section D6.1.1, D6.1.2 when the R factor of Eq. D6.1.2-1 is employed, or D6.2.1. The nominal flexural strength [moment resistance] shall be calculated in accordance with Eq. C3.1.4-1 or Eq. C3.1.4-2. The safety factor and resistance factors given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
\Omega_b = 1.67 \quad \text{(ASD)}
\]

\[
\phi_b = 0.90 \quad \text{(LRFD)}
\]

\[
= 0.85 \quad \text{(LSD)}
\]

For \( \lambda_d \leq 0.673 \)

\[
M_n = M_y \quad \text{(Eq. C3.1.4-1)}
\]

For \( \lambda_d > 0.673 \)

\[
M_n = \left(1 - 0.22 \left( \frac{M_{crd}}{M_y} \right)^{0.5} \right) \left( \frac{M_{crd}}{M_y} \right)^{0.5} M_y \quad \text{(Eq. C3.1.4-2)}
\]

where

\[
\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \quad \text{(Eq. C3.1.4-3)}
\]

\[
M_y = S_{fy} F_y \quad \text{(Eq. C3.1.4-4)}
\]

where

\[
S_{fy} = \text{Elastic section modulus of full unreduced section relative to extreme fiber in first yield}
\]

\[
M_{crd} = S_f F_d \quad \text{(Eq. C3.1.4-5)}
\]

where

\[
S_f = \text{Elastic section modulus of full unreduced section relative to extreme compression fiber}
\]

\[
F_d = \text{Elastic distortional buckling stress calculated in accordance with either Section C3.1.4(a), (b), or (c)}
\]

(a) Simplified Provision for Unrestrained C- and Z-Sections with Simple Lip Stiffeners

For C- and Z-sections that have no rotational restraint of the compression flange and are within the dimensional limits provided in this section, Eq. C3.1.4-6 shall be permitted to be used to calculate a conservative prediction of the distortional buckling stress, \( F_d \). See Section C3.1.4(b) or C3.1.4(c) for alternative provisions and for members outside the dimensional limits of this section.

The following dimensional limits shall apply:

1. \( 50 \leq h_0 / t \leq 200 \),
(2) \(25 \leq \frac{b_0}{t} \leq 100\),
(3) \(6.25 < \frac{D}{t} \leq 50\),
(4) \(45^\circ \leq \theta < 90^\circ\),
(5) \(2 \leq \frac{h_o}{b_0} \leq 8\), and
(6) \(0.04 \leq \frac{D \sin \theta}{b_0} \leq 0.5\).

where

- \(h_o\) = Out-to-out web depth as defined in Figure B2.3-2
- \(t\) = Base steel thickness
- \(b_0\) = Out-to-out flange width as defined in Figure B2.3-2
- \(D\) = Out-to-out lip dimension as defined in Figure B4-1
- \(\theta\) = Lip angle as defined in Figure B4-1

The distortional buckling stress, \(F_d\), shall be calculated as follows:

\[
F_d = \beta k_d \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{b_o} \right)^2
\]

(Eq. C3.1.4-6)

where

\(\beta\) = A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

\[
\beta = 1.0 + 0.4(L/L_m)^{0.7} (1-M_1/M_2)^{0.7} \leq 1.3
\]

(Eq. C3.1.4-7)

where

- \(L\) = Minimum of \(L_{cr}\) and \(L_m\)

\[
L_{cr} = 1.2 h_o \left( \frac{b_o \left( \frac{D \sin \theta}{h_o t} \right)^{0.6}}{h_o t} \right) \leq 10 h_o
\]

(Eq. C3.1.4-8)

\(L_m\) = Distance between discrete restraints that restrict distortional buckling

(For continuously restrained members \(L_m = L_{cr}\))

\(M_1\) and \(M_2\) = The smaller and the larger end moment, respectively, in the unbraced segment (\(L_m\)) of the beam; \(M_1/M_2\) is negative when the moments cause reverse curvature and positive when bent in single curvature

\[
k_d = 0.5 \leq 0.6 \left( \frac{b_o \left( \frac{D \sin \theta}{h_o t} \right)^{0.7}}{h_o t} \right) \leq 8.0
\]

(Eq. C3.1.4-9)

\(E\) = Modulus of elasticity

\(\mu\) = Poisson’s ratio

(b) For C- and Z-Sections or any Open Section with a Stiffened Compression Flange Extending to One Side of the Web where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall be permitted to apply to any open section with a single web and single edge stiffened compression flange, including those meeting the geometric limits of Section C3.1.4 (a). The distortional buckling stress, \(F_d\), shall be calculated in accordance with Eq. C3.1.4-10 as follows:
\[
F_d = \beta \frac{k_{fe} + k_{we} + k_{\phi}}{k_{fg} + k_{wg}} 
\]

(Eq. C3.1.4-10)

where

\[
\beta = \text{A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0}
\]

\[
= 1.0 \leq 1 + 0.4(L/L_m)^{0.7} (1 - M_1/M_2)^{0.7} \leq 1.3 
\]

(Eq. C3.1.4-11)

where

\[
L = \text{Minimum of } L_{cr} \text{ and } L_m
\]

where

\[
L_{cr} = \left( \frac{4\pi^4 h_o(1 - \mu^2)}{t^3} \right) \left( \frac{I_{xf}(x_o - h_x)^2 + C_{wf}}{I_{yf}(x_o - h_x)^2} \right) + \frac{\pi^4 h_o^4}{720} \left( \frac{x_o - h_x}{I_{xf}} \right)
\]

(Eq. C3.1.4-12)

where

\[
h_o = \text{Out-to-out web depth as defined in Figure B2.3-2}
\]

\[
\mu = \text{Poisson’s ratio}
\]

\[
t = \text{Base steel thickness}
\]

\[
I_{xf} = \text{x-axis moment of inertia of the flange}
\]

\[
x_o = \text{x distance from the flange/web junction to the centroid of the flange}
\]

\[
h_x = \text{x distance from the centroid of the flange to the shear center of the flange}
\]

\[
C_{wf} = \text{Warping torsion constant of the flange}
\]

\[
I_{xyf} = \text{Product of the moment of inertia of the flange}
\]

\[
I_{yf} = \text{y-axis moment of inertia of the flange}
\]

In the above, \(I_{xf}, I_{yf}, I_{xyf}, C_{wf}, x_o, \) and \(h_x\) are properties of the compression flange plus edge stiffener about an x-y axis system located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid.

\[
L_m = \text{Distance between discrete restraints that restrict distortional buckling (for continuously restrained members } L_m = L_{cr})
\]

\[
M_1 \text{ and } M_2 = \text{The smaller and the larger end moments, respectively, in the unbraced segment } (L_m) \text{ of the beam; } M_1/M_2 \text{ is negative when the moments cause reverse curvature and positive when bent in single curvature}
\]

\[
k_{\phi fc} = \text{Elastic rotational stiffness provided by the flange to the flange/web juncture}
\]

\[
= \left( \frac{\pi}{L} \right)^4 \left( EI_{xf}(x_o - h_x)^2 + EC_{wf} - E \frac{I_{xyf}}{I_{yf}}(x_o - h_x)^2 \right) + \left( \frac{\pi}{L} \right)^2 GJ_f
\]

(Eq. C3.1.4-13)

where

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

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

\[
J_f = \text{St. Venant torsion constant of the compression flange, plus edge stiffener}
\]
about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid.

\[ k_{\phi_{\text{WE}}} = \text{Elastic rotational stiffness provided by the web to the flange/web juncture} \]

\[
\frac{E t^3}{12 (1 - \mu^2)} \left( \frac{3}{h_o} + \left( \frac{\pi}{L} \right)^2 \frac{19}{60} \frac{h_o}{L} + \left( \frac{\pi}{L} \right)^4 \frac{h_o^3}{240} \right)
\]

(Eq. C3.1.4-14)

\[ k_{\phi} = \text{Rotational stiffness provided by a restraining element (brace, panel, sheathing) to the flange/web juncture of a member (zero if the compression flange is unrestrained)} \]

\[ \tilde{k}_{\phi_{\text{FG}}} = \text{Geometric rotational stiffness (divided by the stress } F_d) \text{ demanded by the flange from the flange/web juncture} \]

\[
\left( \frac{\pi}{L} \right)^2 A_f \left( x_o - h_x \right)^2 \left( \frac{I_{\text{xyf}}}{I_{\text{yf}}} \right)^2 - 2y_o \left( x_o - h_x \right) \left( \frac{I_{\text{xyf}}}{I_{\text{yf}}} \right) + h_x^2 + y_o^2 + I_{xf} + I_{yf}
\]

(Eq. C3.1.4-15)

where

\[ A_f = \text{Cross-sectional area of the compression flange plus edge stiffener about an x-y axis located at the centroid of the flange, with the x-axis measured positive to the right from the centroid, and the y-axis positive down from the centroid} \]

\[ y_o = \text{y distance from the flange/web junction to the centroid of the flange} \]

\[ \tilde{k}_{\phi_{\text{WG}}} = \text{Geometric rotational stiffness (divided by the stress } F_d) \text{ demanded by the web from the flange/web juncture} \]

\[
\frac{h_o t_n^2}{13440} \left[ \frac{45360(1 - \xi_{\text{WEB}}) + 62160}{h_o} \right]^2 + 448 \pi^2 + \frac{h_o^2}{L} \left[ 53 + 3(1 - \xi_{\text{WEB}}) \right] \pi^4 \]

(Eq. C3.1.4-16)

where

\[ \xi_{\text{WEB}} = (f_1 - f_2)/f_1, \text{ stress gradient in the web, where } f_1 \text{ and } f_2 \text{ are the stresses at the opposite ends of the web, } f_1 > f_2, \text{ compression is positive, tension is negative, and the stresses are calculated on the basis of the gross section, (e.g., pure symmetrical bending, } f_1 = -f_2, \xi_{\text{WEB}} = 2) \]

(c) Rational Elastic Buckling Analysis

A rational elastic buckling analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section C3.1.4 (a) or (b). The safety and resistance factors in Section C3.1.4 shall apply.
C3.2 Shear

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

The nominal shear strength [resistance], $V_n$, shall be calculated in accordance with Eq. C3.2.1-1. The safety factor and resistance factors given in this section shall be used to determine the allowable shear strength or design shear strength [factored shear resistance] in accordance with the applicable design method in Section A4, A5, or A6.

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

$\Omega_v = 1.60$  \hspace{1cm} \text{(ASD)}

$\phi_v = 0.95$  \hspace{1cm} \text{(LRFD)}

$= 0.80$  \hspace{1cm} \text{(LSD)}

(a) For $h/t \leq \sqrt{\frac{E k_v}{F_y}}$

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

(b) For $\sqrt{\frac{E k_v}{F_y}} < h/t \leq 1.51 \sqrt{\frac{E k_v}{F_y}}$

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

(c) For $h/t > 1.51 \sqrt{\frac{E k_v}{F_y}}$

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

$$= 0.904 E k_v / (h/t)^2$$  \hspace{1cm} \text{(Eq. C3.2.1-4b)}

where

$V_n$ = Nominal shear strength [resistance]

$A_w$ = Area of web element

$= ht$  \hspace{1cm} \text{(Eq. C3.2.1-5)}

where

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

t = Web thickness

$F_v$ = Nominal shear stress

$E$ = Modulus of elasticity of steel

$k_v$ = Shear buckling coefficient calculated in accordance with (1) or (2) as follows:

(1) For unreinforced webs, $k_v = 5.34$

(2) For webs with transverse stiffeners satisfying the requirements of Section C3.7 when $a/h \leq 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2}$$  \hspace{1cm} \text{(Eq. C3.2.1-6)}

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2}$$  \hspace{1cm} \text{(Eq. C3.2.1-7)}
where
\[ a = \text{Shear panel length of unreinforced web element} \]
\[ = \text{Clear distance between transverse stiffeners of reinforced web elements} \]
\[ F_y = \text{Design yield stress as determined in accordance with Section A7.1} \]
\[ \mu = \text{Poisson’s ratio} \]
\[ = 0.3 \]

For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

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

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

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 C3.2.1, multiplied by the reduction factor, \( q_S \), as defined in this section.

When \( c/t \geq 54 \)
\[ q_S = 1.0 \]
When \( 5 \leq c/t < 54 \)
\[ q_S = \frac{c}{(54t)} \quad (Eq. C3.2.2-1) \]

where
\[ c = \frac{h}{2} - \frac{d_h}{2.83} \quad \text{for circular holes} \]
\[ = \frac{h}{2} - \frac{d_h}{2} \quad \text{for non-circular holes} \quad (Eq. C3.2.2-2) \]

### C3.3 Combined Bending and Shear

#### C3.3.1 ASD Method

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

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

\[
\left( \frac{\Omega_b M}{M_{nxo}} \right)^2 + \left( \frac{\Omega_v V}{V_n} \right)^2 \leq 1.0
\]

(Eq. C3.3.1-1)
For beams with transverse web stiffeners, when \( \Omega_b \frac{M}{M_{n xo}} > 0.5 \) and \( \Omega_v \frac{V}{V_n} > 0.7 \), \( M \) and \( V \) shall also satisfy the following interaction equation:

\[
0.6 \left( \frac{\Omega_b M}{M_{n xo}} \right) + \left( \frac{\Omega_v V}{V_n} \right) \leq 1.3
\]  
(Eq. C3.3.1-2)

where:

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

### C3.3.2 LRFD and LSD Methods

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

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

\[
\left( \frac{\bar{M}}{\phi_b M_{n xo}} \right)^2 + \left( \frac{\bar{V}}{\phi_v V_n} \right)^2 \leq 1.0
\]  
(Eq. C3.3.2-1)

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

\[
0.6 \left( \frac{\bar{M}}{\phi_b M_{n xo}} \right) + \left( \frac{\bar{V}}{\phi_v V_n} \right) \leq 1.3
\]  
(Eq. C3.3.2-2)

where:

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

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

The nominal web crippling strength [resistance], \( P_n \), shall be determined in accordance with Eq. C3.4.1-1 or Eq. C3.4.1-2, as applicable. The safety factors and resistance factors in Tables C3.4.1-1 to C3.4.1-5 shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
P_n = Ct^2F_y \sin \theta \left[ 1 - C_R \left( \frac{R}{t} \right) \right] \left[ 1 + C_N \left( \frac{N}{t} \right) \right] \left( \frac{h}{t} \right) \]  
\text{(Eq. C3.4.1-1)}

where:
- \( P_n \) = Nominal web crippling strength [resistance]
- \( C \) = Coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4, or C3.4.1-5
- \( t \) = Web thickness
- \( F_y \) = Design yield stress as determined in accordance with Section A7.1
- \( \theta \) = 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 C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4, or C3.4.1-5
- \( R \) = Inside bend radius
- \( C_N \) = Bearing length coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4, or C3.4.1-5
- \( N \) = Bearing length [3/4 in. (19 mm) minimum]
- \( C_h \) = Web slenderness coefficient from Table C3.4.1-1, C3.4.1-2, C3.4.1-3, C3.4.1-4, or C3.4.1-5
- \( h \) = 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, shall be permitted to be calculated as follows, except that \( P_{nc} \) shall not be larger than the interior-one-flange loading condition:

\[
P_{nc} = \alpha P_n \]  
\text{(Eq. C3.4.1-2)}

where:
- \( P_{nc} \) = Nominal web crippling strength [resistance] of C and Z-sections with overhang(s)
- \( \alpha \) = \[
0.13 \frac{(0.09 + 0.009(h/t) + 0.3)}{L_o/h^{0.26}} \geq 1.0
\text{(Eq. C3.4.1-3)}

where
- \( L_o \) = 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. C3.4.1-1 and Tables C3.4.1-2 and C3.4.1-3

Eq. C3.4.1-2 shall be limited to \( 0.5 \leq L_o/h \leq 1.5 \) and \( h/t \leq 154 \). For \( L_o/h \) 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 webs consisting of two or more such sheets, $P_{n}$ and $P_{nc}$ shall be calculated for each individual sheet and the results added to obtain the nominal strength for the full section.

One-flange loading or reaction 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 C3.4.1-1 shall apply to I-beams made from two channels connected back-to-back where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$ and $\theta = 90^\circ$. See Section C3.4.1 of Commentary for further explanation.

**TABLE C3.4.1-1**

**Safety Factors, Resistance Factors, and Coefficients for Built-Up Sections**

<table>
<thead>
<tr>
<th>Support and Flange Conditions</th>
<th>Load Cases</th>
<th>C</th>
<th>$C_R$</th>
<th>$C_N$</th>
<th>$C_h$</th>
<th>USA and Mexico</th>
<th>Canada LSD</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>One-Flange Loading or Reaction</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>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>One-Flange Loading or Reaction</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>
<td>15.5</td>
<td>0.09</td>
<td>0.08</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</td>
<td>Interior</td>
<td>36</td>
<td>0.14</td>
<td>0.08</td>
<td>0.04</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>10</td>
<td>0.14</td>
<td>0.28</td>
<td>0.001</td>
<td>2.00</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>One-Flange Loading or Reaction</td>
<td>Interior</td>
<td>20.5</td>
<td>0.17</td>
<td>0.11</td>
<td>0.001</td>
<td>1.75</td>
<td>0.85</td>
</tr>
</tbody>
</table>
Table C.3.4.1-2 shall apply to single web channel and C-Sections members where $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$, and $\theta = 90^\circ$. In Table C.3.4.1-2, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least $2.5h$. For unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least $1.5h$.

**TABLE C3.4.1-2**

<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 ASD $\Omega_w$</th>
<th>LRFD $\phi_w$</th>
<th>Canada LSD $\phi_w$</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td>Stiffened or Partially Stiffened Flanges</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>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>20</td>
<td>0.10</td>
<td>0.08</td>
<td>0.031</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.85</td>
<td>0.80</td>
<td>0.70</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>
<td>0.80</td>
</tr>
<tr>
<td>Unstiffened Flanges</td>
<td>Two-Flange Loading or Reaction</td>
<td>End</td>
<td>13</td>
<td>0.32</td>
<td>0.05</td>
<td>0.04</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<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 Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>2</td>
<td>0.11</td>
<td>0.37</td>
<td>0.01</td>
<td>2.00</td>
<td>0.75</td>
<td>0.65</td>
</tr>
<tr>
<td></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>
<td>0.65</td>
</tr>
</tbody>
</table>
Table C3.4.1-3 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 C3.4.1-3, for interior two-flange loading or reaction of members having flanges fastened to the support, the distance from the edge of bearing to the end of the member shall be extended at least 2.5h; for unfastened cases, the distance from the edge of bearing to the end of the member shall be extended at least 1.5h.

**TABLE C3.4.1-3**

**Safety Factors, Resistance Factors, and Coefficients for Single Web Z-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>ANSI ASD ( \Omega_w )</th>
<th>LRFD ( \phi_w )</th>
<th>USA and Mexico</th>
<th>Canada LSD ( \phi_w )</th>
<th>Limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastened to Support</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</td>
<td>End</td>
<td>4</td>
<td>0.14</td>
<td>0.35</td>
<td>0.02</td>
<td>1.75</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>13</td>
<td>0.23</td>
<td>0.14</td>
<td>0.01</td>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
<td>R/t ≤ 5.5</td>
</tr>
<tr>
<td></td>
<td>Two-Flange Loading or Reaction</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>
<td>0.75</td>
<td>R/t ≤ 12</td>
</tr>
<tr>
<td></td>
<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>
<td>R/t ≤ 12</td>
</tr>
<tr>
<td>Unfastened</td>
<td>Stiffened or Partially Stiffened Flanges</td>
<td>One-Flange Loading or Reaction</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></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>
<td>R/t ≤ 3</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>
<td>1.65</td>
<td>0.90</td>
<td>0.80</td>
<td>R/t ≤ 3</td>
</tr>
<tr>
<td></td>
<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>
<td>R/t ≤ 3</td>
</tr>
<tr>
<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>
<td>0.85</td>
<td>0.70</td>
<td>R/t ≤ 2</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>
<td>0.70</td>
<td>R/t ≤ 1</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>
<td>0.65</td>
<td>R/t ≤ 1</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>
<td>0.65</td>
<td>R/t ≤ 1</td>
</tr>
</tbody>
</table>
Table C3.4.1-4 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 C3.4.1-4**

<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>
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<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 C3.4.1-5 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 C3.4.1-5**

<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>
<tr>
<td></td>
<td></td>
<td>Interior</td>
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<td>0.11</td>
<td>0.21</td>
<td>0.020</td>
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<td>0.75</td>
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<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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<td></td>
<td></td>
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<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>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>
C3.4.2 Web Crippling Strength [Resistance] 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 C3.4.1, multiplied by the reduction factor, $R_c$, given in this section.

The provisions of this section shall apply within the following limits:
1. $d_h/h \leq 0.7$,
2. $h/t \leq 200$,
3. Hole centered at mid-depth of web,
4. Clear distance between holes $\geq 18$ in. (457 mm),
5. Distance between end of member and edge of hole $\geq d$,
6. Non-circular holes, corner radii $\geq 2t$,
7. Non-circular holes, $d_h \leq 2.5$ in. (64 mm) and $L_h \leq 4.5$ in. (114 mm),
8. Circular holes, diameters $\leq 6$ in. (152 mm), and
9. $d_0 > 9/16$ in. (14 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 C3.4.1-1 with Table C3.4.1-2) where a web hole is not within the bearing length, the reduction factor, $R_c$, shall be calculated as follows:

$$R_c = 1.01 - 0.325 \frac{d_h}{h} + 0.083 x / h \leq 1.0 \quad (Eq. \ C3.4.2-1)$$

where $x \geq 1$ in. (25 mm)

For interior-one flange reaction (Equation C3.4.1-1 with Table C3.4.1-2) where any portion of a web hole is not within the bearing length, the reduction factor, $R_c$, shall be calculated as follows:

$$R_c = 0.90 - 0.047 \frac{d_h}{h} + 0.053 x / h \leq 1.0 \quad (Eq. \ C3.4.2-2)$$

where $x \geq 3$ in. (76 mm)

C3.5 Combined Bending and Web Crippling

C3.5.1 ASD Method

Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed such that the moment, $M$, and the concentrated load or reaction, $P$, satisfy $M \leq M_{nxo}/\Omega_b$ and $P \leq P_n/\Omega_w$. In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced webs, Eq. C3.5.1-1 shall be satisfied as follows:

$$0.91 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{nxo}} \right) \leq \frac{1.33}{\Omega} \quad (Eq. \ C3.5.1-1)$$
Exception: At the interior supports of continuous spans, Eq. C3.5.1-1 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. C3.5.1-2 shall be satisfied as follows:

\[
0.88 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n_{x0}}} \right) \leq \frac{1.46}{\Omega}
\]

(Eq. C3.5.1-2)

(c) For the support point of two nested Z-shapes, Eq. C3.5.1-3 shall be satisfied as follows:

\[
0.86 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n_{x0}}} \right) \leq \frac{1.65}{\Omega}
\]

(Eq. C3.5.1-3)

Eq. C3.5.1-3 shall apply to shapes that meet the following limits:

- \( h/t \leq 150 \),
- \( N/t \leq 140 \),
- \( F_y \leq 70 \text{ ksi} \) (483 MPa or 4920 kg/cm²), and
- \( R/t \leq 5.5 \).

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.
4. The ratio of the thicker to the thinner part does not exceed 1.3.

The following notation shall apply to this section:

- \( M \) = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction, \( P \)
- \( P \) = Required strength for concentrated load or reaction in the presence of bending moment
- \( M_{n_{x0}} \) = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1.1
- \( \Omega_{p} \) = Safety factor for bending (See Section C3.1.1)
- \( P_n \) = Nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section C3.4
- \( \Omega_{w} \) = Safety factor for web crippling (See Section C3.4)
- \( \Omega \) = Safety factor for combined bending and web crippling
- \( \Omega = 1.70 \)

**C3.5.2 LRFD and LSD Methods**

Unreinforced flat webs of shapes subjected to a combination of bending and
concentrated load or reaction shall be designed such that the moment, $M$, and the concentrated load or reaction, $P$, satisfy $M \leq \phi_b M_{n\infty}$ and $P \leq \phi_w P_n$. In addition, the following requirements in (a), (b), and (c), as applicable, shall be satisfied.

(a) For shapes having single unreinforced webs, Eq. C3.5.2-1 shall be satisfied as follows:

$$0.91 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n\infty}} \right) \leq 1.33 \phi$$

(Eq. C3.5.2-1)

where

\[\phi = 0.90 \text{ (LRFD)}\]
\[= 0.75 \text{ (LSD)}\]

Exception: At the interior supports of continuous spans, Eq. C3.5.2-1 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. C3.5.2-2 shall be satisfied as follows:

$$0.88 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n\infty}} \right) \leq 1.46 \phi$$

(Eq. C3.5.2-2)

where

\[\phi = 0.90 \text{ (LRFD)}\]
\[= 0.75 \text{ (LSD)}\]

(c) For two nested Z-shapes, Eq. C3.5.2-3 shall be satisfied as follows:

$$0.86 \left( \frac{P}{P_n} \right) + \left( \frac{M}{M_{n\infty}} \right) \leq 1.65 \phi$$

(Eq. C3.5.2-3)

where

\[\phi = 0.90 \text{ (LRFD)}\]
\[= 0.80 \text{ (LSD)}\]

Eq. C3.5.2-3 shall apply to shapes that meet the following limits:

- $h/t \leq 150$,
- $N/t \leq 140$,
- $F_y \leq 70 \text{ ksi (483 MPa or 4920 kg/cm}^2)$, and
- $R/t \leq 5.5$.

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.
4. The ratio of the thicker to the thinner part does not exceed 1.3.
The following notation shall apply in this section:

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

\( = M_u \) (LRFD)
\( = M_f \) (LSD)

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

\( = P_u \) (LRFD)
\( = P_f \) (LSD)

\( \phi_b \) = Resistance factor for bending (See Section C3.1.1)

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

\( \phi_w \) = Resistance factor for web crippling (See Section C3.4)

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

### C3.6 Combined Bending and Torsional Loading

For laterally unrestrained flexural members subjected to both bending and torsional loading, the available flexural strength \([\text{factored moment resistance}]\) calculated in accordance with Section C3.1.1(a) shall be reduced by multiplying it by a reduction factor, \( R \).

As specified in Equation C3.6-1, the reduction factor, \( R \), shall be equal to the ratio of the 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.

\[
R = \frac{f_{\text{bending}}}{f_{\text{bending}} + f_{\text{torsion}}} \leq 1 \quad \text{(Eq. C3.6-1)}
\]

Stresses shall be calculated using full section properties for the torsional stresses and effective section properties for the bending stresses. For C-sections with edge stiffened flanges, if the maximum combined compressive stresses occur at the junction of the \(web\) and flange, the \(R\) factor shall be 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 be applied when the provisions of Sections D6.1.1 and D6.1.2 are used.

### C3.7 Stiffeners

#### C3.7.1 Bearing Stiffeners

Bearing stiffeners attached to beam \(\text{webs}\) at points of concentrated \(\text{loads}\) 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 E. For concentrated loads or reactions, the \textit{nominal strength} \([\text{resistance}]\), \(P_n\), shall be the smaller value calculated by (a) and (b) of this section. The \textit{safety factor} and \textit{resistance factors} provided in this section shall be used to determine the \textit{allowable strength}, or design
strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[ \Omega_c = 2.00 \text{ (ASD)} \]
\[ \phi_c = 0.85 \text{ (LRFD)} \]
\[ = 0.80 \text{ (LSD)} \]

(a) \[ P_n = F_{wy}A_c \quad (Eq. \text{C3.7.1-1}) \]

(b) \[ P_n = \text{Nominal axial strength [resistance] evaluated in accordance with Section C4.1(a), with } A_c \text{ replaced by } A_b \]

where

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

where

\[ t = \text{Base steel thickness of beam web} \]
\[ A_s = \text{Cross-sectional area of bearing stiffener} \]
\[ A_b = b_1t + A_s \text{ for bearing stiffener at interior support or under concentrated load} \]
\[ = b_2t + A_s \text{ for bearing stiffener at end support} \]

where

\[ b_1 = 25t \left[ 0.0024 \left( \frac{L_{st}}{t} \right) + 0.72 \right] \leq 25t \quad (Eq. \text{C3.7.1-6}) \]
\[ b_2 = 12t \left[ 0.0044 \left( \frac{L_{st}}{t} \right) + 0.83 \right] \leq 12t \quad (Eq. \text{C3.7.1-7}) \]

where

\[ L_{st} = \text{Length of bearing stiffener} \]

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, and \( t_s \) is the thickness of the stiffener steel.

**C3.7.2 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 C3.7.1, the nominal strength [resistance], \( P_{nv} \), shall be calculated in accordance with Eq. C3.7.2-1. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[ P_n = 0.7(P_{wc} + A_cF_y) \geq P_{wc} \quad (Eq. \text{C3.7.2-1}) \]

\[ \Omega = 1.70 \text{ (ASD)} \]
\[ \phi = 0.90 \text{ (LRFD)} \]
\[ = 0.80 \text{ (LSD)} \]

where

\[ P_{wc} = \text{Nominal web crippling strength [resistance] for C-section flexural member calculated in accordance with Eq. C3.4.1-1 for single web members, at end or interior locations} \]
A_e = Effective area of bearing stiffener subjected to uniform compressive stress, calculated at yield stress

F_y = Yield stress of bearing stiffener steel

Eq. C3.7.2-1 shall apply within the following limits:

1. 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.

2. Stiffeners are C-section stud or track members with a minimum web depth of 3-1/2 in. (89 mm) and a minimum base steel thickness of 0.0329 in. (0.84 mm).

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

4. 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.

5. The length of the stiffener is not less than the depth of the flexural member minus 3/8 in. (9 mm).

6. The bearing width is not less than 1-1/2 in. (38 mm).

C3.7.3 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be based on the nominal shear strength [resistance], V_n, permitted by Section C3.2, and the ratio a/h shall not exceed [260/(h/t)]^2 nor 3.0.

The actual moment of inertia, I_s, of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value calculated in accordance with Equation C3.7.3-1 as follows:

\[
I_{s_{\text{min}}} = 5ht^3\left[\frac{h}{a} - 0.7\left(\frac{a}{h}\right)\right] \geq \left(\frac{h}{50}\right)^4
\]

(Eq. C3.7.3-1)

where

h and t = Values as defined in Section B1.2

a = Distance between shear stiffeners

The gross area of shear stiffeners shall not be less than:

\[
A_{st} = \frac{1 - C_v}{2} \left[ a \left(\frac{a}{h}\right)^2 \right] \left(\frac{a}{h}\right) + \sqrt{1 + \left(\frac{a}{h}\right)^2} \right] YDht
\]

(Eq. C3.7.3-2)

where

\[
C_v = \begin{cases} 
\frac{1.53E_{kv}}{F_y(h/t)^2} & \text{when } C_v \leq 0.8 \\
\frac{1.11E_{kv}}{h/t} & \text{when } C_v > 0.8
\end{cases}
\]

(Eq. C3.7.3-3)

(Eq. C3.7.3-4)

where

\[
k_v = \begin{cases} 
4.00 + \frac{5.34}{(a/h)^2} & \text{when } a/h \leq 1.0 \\
5.34 + \frac{4.00}{(a/h)^2} & \text{when } a/h > 1.0
\end{cases}
\]

(Eq. C3.7.3-5)

(Eq. C3.7.3-6)
Y = Yield stress of web steel
   / Yield stress of stiffener steel

D = 1.0 for stiffeners furnished in pairs
   = 1.8 for single-angle stiffeners
   = 2.4 for single-plate stiffeners

C3.7.4 Non-Conforming Stiffeners

The available strength [factored resistance] of members with stiffeners that do not meet the requirements of Section C3.7.1, C3.7.2, or C3.7.3, such as stamped or rolled-in stiffeners, shall be determined by tests in accordance with Chapter F or rational engineering analysis in accordance with Section A1.2(b).

C4 Concentrically Loaded Compression Members

The available axial strength [factored compressive resistance] shall be the smaller of the values calculated in accordance with Sections C4.1, C4.2, D1.2, D6.1.3, and D6.1.4, where applicable.

C4.1 Nominal Strength for Yielding, Flexural, Flexural-Torsional and Torsional Buckling

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

(a) The nominal axial strength [compressive resistance], \( P_n \), shall be calculated in accordance with Eq. C4.1-1. The safety factor and resistance factors in this section shall be used to determine the allowable axial strength or design axial strength [factored compressive resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
P_n = A_e F_n
\]

\( \Omega_c = 1.80 \) (ASD)

\( \phi_c = 0.85 \) (LRFD)

\( \phi_c = 0.80 \) (LSD)

where

\( A_e \) = Effective area calculated at stress \( F_n \). For sections with circular holes, \( A_e \) is determined from the effective width in accordance with Section B2.2(a), subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, it is permitted to determine \( A_e \) by ignoring the holes. For closed cylindrical tubular members, \( A_e \) is provided in Section C4.1.5.

\( F_n \) shall be calculated as follows:

For \( \lambda_c \leq 1.5 \)

\[
F_n = \left( 0.658 \lambda_c^2 \right) F_y
\]

(Eq. C4.1-2)
For $\lambda_c > 1.5$

$$F_n = \left[ \frac{0.877}{\lambda_c^2} \right] F_y$$  \hspace{0.5cm} (Eq. C4.1-3)

where

$$\lambda_c = \frac{F_y}{\sqrt{F_e}}$$  \hspace{0.5cm} (Eq. C4.1-4)

$F_e$ = The least of the applicable elastic flexural, torsional and flexural-torsional buckling stress determined in accordance with Sections C4.1.1 through C4.1.5.

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

**C4.1.1 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, $F_e$, shall be calculated as follows:

$$F_e = \frac{\pi^2 E}{(KL/r)^2}$$  \hspace{0.5cm} (Eq. C4.1.1-1)

where

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

In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, $K$, for compression members that do not depend upon their own bending stiffness for lateral stability of the frame or truss shall be taken as unity, unless analysis shows that a smaller value is suitable. In a frame that depends upon its own bending stiffness for lateral stability, the effective length, $KL$, of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

**C4.1.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling**

For singly-symmetric sections subject to flexural-torsional buckling, $F_e$ shall be taken as the smaller of $F_e$ calculated in accordance with Section C4.1.1 and $F_e$ calculated as follows:

$$F_e = \frac{1}{2\beta} \left[ (\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta \sigma_{ex} \sigma_t} \right]$$  \hspace{0.5cm} (Eq. C4.1.2-1)

Alternatively, a conservative estimate of $F_e$ shall be permitted to be calculated as follows:
\[ F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \]  
\text{(Eq. C4.1.2-2)}

where
\[ \beta = 1 - \left(\frac{x_0}{r_0}\right)^2 \]  
\text{(Eq. C4.1.2-3)}

\( \sigma_t \) and \( \sigma_{ex} \) = Values as defined in Section C3.1.2.1

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

For \textit{doubly-symmetric sections} subject to \textit{torsional buckling}, \( F_e \) shall be taken as the smaller of \( F_e \) calculated in accordance with Section C4.1.1 and \( F_e = \sigma_t \), where \( \sigma_t \) is defined in Section C3.1.2.1.

For singly-symmetric unstiffened angle sections for which the effective area \( (A_e) \) at stress \( F_y \) is equal to the \textit{full unreduced cross-sectional area} \( (A) \), \( F_e \) shall be computed using Eq. C4.1.1-1 where \( r \) is the least radius of gyration.

\textbf{C4.1.3 Point-Symmetric Sections}

For \textit{point-symmetric sections}, \( F_e \) shall be taken as the lesser of \( \sigma_t \) as defined in Section C3.1.2.1 and \( F_e \) as calculated in Section C4.1.1 using the minor principal axis of the section.

\textbf{C4.1.4 Nonsymmetric Sections}

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

\textbf{C4.1.5 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 \( E/F_y \) and in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section, the elastic \textit{flexural buckling} stress, \( F_e \), shall be calculated in accordance with Section C4.1.1, and the effective area, \( A_e \), shall be calculated as follows:

\[ A_e = A_o + R(A - A_o) \]  
\text{(Eq. C4.1.5-1)}

where

\[ A_o = \left[ \frac{0.037}{(DF_y)/(tE)} + 0.667 \right] A \leq A \quad \text{for} \quad \frac{D}{t} \leq 0.441 \quad \frac{E}{F_y} \]  
\text{(Eq. C4.1.5-2)}

where

\( D = \text{Outside diameter of cylindrical tube} \)
\( F_y = \text{Yield stress} \)
\( t = \text{Thickness} \)
\( E = \text{Modulus of elasticity of steel} \)
\( A = \text{Area of full unreduced cross-section} \)
\( R = F_y/(2F_e) \leq 1.0 \)  
\text{(Eq. C4.1.5-3)}
C4.2 Distortional Buckling Strength [Resistance]

The provisions of this section shall apply to I-, Z-, C-, HAT, and other open cross-section members that employ flanges with edge stiffeners, with the exception of members that are designed in accordance with Section D6.1.2. The nominal axial strength [compressive resistance] shall be calculated in accordance with Eqs. C4.2-1 and C4.2-2. The safety factor and resistance factors in this section shall be used to determine the allowable compressive strength or design compressive strength [resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[ \Omega_b = 1.80 \] (ASD)
\[ \phi_b = 0.85 \] (LRFD)
\[ = 0.80 \] (LSD)

For \( \lambda_d \leq 0.561 \)
\[ P_n = P_y \] (Eq. C4.2-1)

For \( \lambda_d > 0.561 \)
\[
P_n = \left(1 - 0.25 \left( \frac{P_{crd}}{P_y} \right)^{0.6} \left( \frac{P_{crd}}{P_y} \right)^{0.6} \right) P_y
\] (Eq. C4.2-2)

where
\[ \lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \] (Eq. C4.2-3)
\[ P_n = \text{Nominal axial strength} \]
\[ P_y = A_g F_y \] (Eq. C4.2-4)

where
\[ A_g = \text{Gross area of the cross-section} \]
\[ F_y = \text{Yield stress} \]
\[ P_{crd} = A_g F_d \] (Eq. C4.2-5)

where
\[ F_d = \text{Elastic distortional buckling stress calculated in accordance with either Section C4.2(a), (b), or (c)} \]

(a) Simplified Provision 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, Eq. C4.2-6 shall be permitted to be used to calculate a conservative prediction of distortional buckling stress, \( F_d \). See Section C4.2(b) or C4.2(c) for alternative options for members outside the dimensional limits.

The following dimensional limits shall apply:
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

\( h_o \) = Out-to-out web depth as defined in Figure B2.3-2

\( b_o \) = Out-to-out flange width as defined in Figure B2.3-2

\( D \) = Out-to-out lip dimension as defined in Figure B4-1

\( t \) = Base steel thickness

\( \theta \) = Lip angle as defined in Figure B4-1

The distortional buckling stress, \( F_d \), shall be calculated in accordance with Eq. C4.2-6:

\[
F_d = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{b_o} \right)^2
\]

(Eq. C4.2-6)

where

\( \alpha \) = A value that accounts for the benefit of an unbraced length, \( L_m \), shorter than \( L_{cr} \), but can be conservatively taken as 1.0

\( \alpha = 1.0 \) for \( L_m \geq L_{cr} \)

\( \alpha = \left( \frac{L_m}{L_{cr}} \right)^n(L_m/L_{cr}) \) for \( L_m < L_{cr} \)  \hspace{1cm} (Eq. C4.2-7)

where

\( L_m = \) Distance between discrete restraints that restrict distortional buckling

\( L_{cr} = \) Distance between continuous restraints or any Open Section with Stiffened Flanges of Equal Dimension where the Stiffener is either a Simple Lip or a Complex Edge Stiffener

The provisions of this section shall apply to any open section with stiffened flanges of equal dimension, including those meeting the geometric limits of C4.2(a).

\[
F_d = \frac{k_{\phi_{fe}} + k_{\phi_{we}} + k_{\phi}}{k_{\phi_{fe}} + k_{\phi_{we}}}
\]

(Eq. C4.2-10)

where

\( k_{\phi_{fe}} = \) Elastic rotational stiffness provided by the flange to the flange/web juncture, in accordance with Eq. C3.1.4-13

\( k_{\phi_{we}} = \) Elastic rotational stiffness provided by the web to the flange/web juncture

\[
= \frac{Et^3}{6h_o(1-\mu^2)}
\]

(Eq. C4.2-11)

\( k_{\phi} = \) Rotational stiffness provided by restraining elements (brace, panel,
sheathing) to the flange/web juncture of a member (zero if the flange is unrestrained). If rotational stiffness provided to the two flanges is dissimilar, the smaller rotational stiffness is used.

\[
\tilde{k}_{\phi g} = \text{Geometric rotational stiffness (divided by the stress } F_d) \text{ demanded by the flange from the flange/web juncture, in accordance with Eq. C3.1.4-15}
\]

\[
\tilde{k}_{\phi wg} = \text{Geometric rotational stiffness (divided by the stress } F_d) \text{ demanded by the web from the flange/web juncture}
\]

\[
= \left(\frac{\pi}{L}\right)^2 \frac{th_0^3}{60} \quad (Eq. \ C4.2-12)
\]

where

\[L = \text{Minimum of } L_{cr} \text{ and } L_m\]

where

\[
L_{cr} = \left(\frac{6\pi^4t_0^2}{t^3}\right)\left[I_{xf}(x_o - h_x)^2 + C_{wf} \frac{I_{yf}^2}{I_{yf}^2}(x_o - h_x)^2\right]^{\frac{1}{4}} \quad (Eq. \ C4.2-13)
\]

\[L_m = \text{Distance between discrete restraints that restrict distortional buckling (for continuously restrained members } L_m = L_{cr})\]

See Section C3.1.4 (b) for definition of variables in Eq. C4.2-13.

(c) Rational elastic buckling analysis

A rational elastic buckling analysis that considers distortional buckling shall be permitted to be used in lieu of the expressions given in Section C4.2(a) or (b). The safety and resistance factors in Section C4.2 shall apply.

C5 Combined Axial Load and Bending

C5.1 Combined Tensile Axial Load and Bending

C5.1.1 ASD Method

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

\[
\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} + \frac{\Omega_1 T}{T_n} \leq 1.0 \quad (Eq. \ C5.1.1-1)
\]

and

\[
\frac{\Omega_b M_x}{M_{nxt}} + \frac{\Omega_b M_y}{M_{nyt}} - \frac{\Omega_1 T}{T_n} \leq 1.0 \quad (Eq. \ C5.1.1-2)
\]

where

\[\Omega_b = 1.67\]

\[M_x, M_y = \text{Required flexural strengths with respect to centroidal axes of section}\]

\[M_{nxt}, M_{nyt} = S_{ft}F_y \quad (Eq. \ C5.1.1-3)\]

where

\[S_{ft} = \text{Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis}\]
\[ F_Y = \text{Design yield stress determined in accordance with Section A7.1} \]
\[ \Omega_t = 1.67 \]
\[ T = \text{Required tensile axial strength} \]
\[ T_n = \text{Nominal tensile axial strength determined in accordance with Section C2} \]
\[ M_{nx}, M_{ny} = \text{Nominal flexural strengths about centroidal axes determined in accordance with Section C3.1} \]

**C5.1.2 LRFD and LSD Methods**

The *required strengths* [factored tension and moments] \( T, M_x, \) and \( M_y \) shall satisfy the following interaction equations:

\[
\frac{M_x}{\phi_b M_{nxt}} + \frac{M_y}{\phi_b M_{nyt}} + \frac{T}{\phi_t T_n} \leq 1.0 \quad (Eq. \text{C5.1.2-1})
\]
\[
\frac{M_x}{\phi_b M_{nx}} + \frac{M_y}{\phi_b M_{ny}} - \frac{T}{\phi_t T_n} \leq 1.0 \quad (Eq. \text{C5.1.2-2})
\]

where
- \( M_x, M_y \) = Required flexural strengths [factored moments] with respect to centroidal axes
  \[ M_x = M_{ux}, M_y = M_{uy} \text{ (LRFD)} \]
  \[ M_x = M_{fx}, M_y = M_{fy} \text{ (LSD)} \]
- \( \phi_b \) = For flexural strength [moment resistance] (Section C3.1.1), \( \phi_b = 0.90 \) or \( 0.95 \) (LRFD) and \( 0.90 \) (LSD)
  - For laterally unbraced beams (Section C3.1.2), \( \phi_b = 0.90 \) (LRFD and LSD)
  - For closed cylindrical tubular members (Section C3.1.3), \( \phi_b = 0.95 \) (LRFD) and \( 0.90 \) (LSD)
- \( M_{nxt}, M_{nyt} = S_{ft} F_Y \) (Eq. \text{C5.1.2-3})
- \( S_{ft} \) = Section modulus of full unreduced section relative to extreme tension fiber about appropriate axis
- \( F_Y = \text{Design yield stress determined in accordance with Section A7.1} \)
- \( \bar{T} = \text{Required tensile axial strength [factored tension]} \)
  \[ = T_u \text{ (LRFD)} \]
  \[ = T_f \text{ (LSD)} \]
- \( \phi_t \) = \( 0.95 \) (LRFD)
  \[ = 0.90 \] (LSD)
- \( T_n = \text{Nominal tensile axial strength [resistance] determined in accordance with Section C2} \)
- \( M_{nx}, M_{ny} = \text{Nominal flexural strengths [moment resistances] about centroidal axes determined in accordance with Section C3.1} \)
C5.2 Combined Compressive Axial Load and Bending

C5.2.1 ASD Method

The required strengths \( P, M_x, \) and \( M_y \) shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths \( P, M_x, \) and \( M_y \) shall be determined in accordance with Appendix 2 and shall satisfy the following interaction equations using the values for \( K_x, K_y, \alpha_x, \alpha_y, C_{mx}, \) and \( C_{my} \) specified in Appendix 2. In addition, each individual ratio in Eqs. C5.2.1-1 to C5.2.1-3 shall not exceed unity.

For singly-symmetric unstiffened angle sections with unreduced effective area, \( M_y \) shall be permitted to be taken as the required flexural strength only. For other angle sections or singly-symmetric unstiffened angles for which the effective area \( (A_e) \) at stress \( F_y \) is less than the full unreduced cross-sectional area \( (A) \), \( M_y \) shall be taken either as the required flexural strength or the required flexural strength plus \( PL/1000 \), whichever results in a lower permissible value of \( P \).

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx} \alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny} \alpha_y} \leq 1.0 \quad (Eq. \ C5.2.1-1)
\]

\[
\frac{\Omega_c P}{P_{n0}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1.0 \quad (Eq. \ C5.2.1-2)
\]

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

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

where

\[
\Omega_c = 1.80
\]

\( P = \) Required compressive axial strength

\( P_n = \) Nominal axial strength determined in accordance with Section C4

\( \Omega_b = 1.67 \)

\( M_x, M_y = \) Required flexural strengths with respect to centroidal axes of effective section determined for required compressive axial strength alone.

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

\[
\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq. \ C5.2.1-4)
\]

\[
\alpha_y = 1 - \frac{\Omega_c P}{P_{Ey}} > 0 \quad (Eq. \ C5.2.1-5)
\]

where

\[
P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq. \ C5.2.1-6)
\]
\[ P_{Ey} = \frac{\pi^2 E I_y}{(K_y L_y)^2} \]  

(Eq. C5.2.1-7)

where

- \( I_x \) = Moment of inertia of full unreduced cross-section about x-axis
- \( K_x \) = Effective length factor for buckling about x-axis
- \( L_x \) = Unbraced length for bending about x-axis
- \( I_y \) = Moment of inertia of full unreduced cross-section about y-axis
- \( K_y \) = Effective length factor for buckling about y-axis
- \( L_y \) = Unbraced length for bending about y-axis

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

\( C_m \) = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

(a) For compression members in frames subject to joint translation (sidesway)

\[ C_m = 0.85 \]

(b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

\[ C_m = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) \]  

(Eq. C5.2.1-8)

where

\( M_1/M_2 \) = Ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unreduced in the plane of bending. \( M_1/M_2 \) is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

(c) For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of \( C_m \) is to be determined by rational analysis. However, in lieu of such analysis, the following values are permitted to be used:

1. For members whose ends are restrained, \( C_m = 0.85 \), and
2. For members whose ends are unrestrained, \( C_m = 1.0 \).

**C5.2.2 LRFD and LSD Methods**

The required strengths [factored compression and moments] \( \bar{P} \), \( \bar{M}_x \), and \( \bar{M}_y \) shall be determined using first order elastic analysis and shall satisfy the following interaction equations. Alternatively, the required strengths [factored axial force and moment] \( \bar{P} \), \( \bar{M}_x \), and \( \bar{M}_y \) shall be determined in accordance with Appendix 2 and shall satisfy the following interaction equations using the values for \( K_x \), \( K_y \), \( \alpha_x \), \( \alpha_y \), \( C_{mx} \), and \( C_{my} \) specified in Appendix 2. In addition, each individual ratio in Eqs. C5.2.2-1 to C5.2.2-3 shall not exceed unity.

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

\[
\frac{\bar{P}}{\phi_c P_n} + \frac{C_{mx} \bar{M}_X}{\phi_b M_{nx} \alpha_x} + \frac{C_{my} \bar{M}_Y}{\phi_b M_{ny} \alpha_y} \leq 1.0 \tag{Eq. C5.2.2-1}
\]

\[
\frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}_X}{\phi_b M_{nx}} + \frac{\bar{M}_Y}{\phi_b M_{ny}} \leq 1.0 \tag{Eq. C5.2.2-2}
\]

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

\[
\frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}_X}{\phi_b M_{nx}} + \frac{\bar{M}_Y}{\phi_b M_{ny}} \leq 1.0 \tag{Eq. C5.2.2-3}
\]

where

\(\bar{P}\) = Required compressive axial strength \([\text{factored compressive force}]\)
\(= P_u \quad (LRFD)\)
\(= P_f \quad (LSD)\)

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

\(P_n\) = Nominal axial strength \([\text{resistance}]\) determined in accordance with Section C4

\(\bar{M}_X, \bar{M}_Y\) = Required flexural strengths \([\text{factored moments}]\) with respect to centroidal axes of effective section determined for required compressive axial strength \([\text{factored axial force}]\) alone.

\(\bar{M}_X = M_{nx}\) \(\bar{M}_Y = M_{uy} \quad (LRFD)\)
\(\bar{M}_X = M_{nx}\) \(\bar{M}_Y = M_{fy} \quad (LSD)\)

\(\phi_b\) = For flexural strength \([\text{resistance}]\) (Section C3.1.1), \(\phi_b = 0.90\) or 0.95 \( (LRFD)\) and 0.90 \( (LSD)\)

\(\phi_b\) = For laterally unbraced flexural members (Section C3.1.2), \(\phi_b = 0.90\) \( (LRFD\) and LSD)

\(\phi_b\) = For closed cylindrical tubular members (Section C3.1.3), \(\phi_b = 0.95\) \( (LRFD\) and 0.90 \( (LSD)\)

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

\(\alpha_x = 1 - \frac{\bar{P}}{P_{Ex}} > 0\) \( (Eq. C5.2.2-4)\)

\(\alpha_y = 1 - \frac{\bar{P}}{P_{Ey}} > 0\) \( (Eq. C5.2.2-5)\)
where

\[ P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \]  \hspace{1cm} (Eq. C5.2.2-6)

\[ P_{Ey} = \frac{\pi^2 E I_y}{(K_y L_y)^2} \]  \hspace{1cm} (Eq. C5.2.2-7)

where

- \( I_x \) = Moment of inertia of full unreduced cross-section about x-axis
- \( K_x \) = Effective length factor for buckling about x-axis
- \( L_x \) = Unbraced length for bending about x-axis
- \( I_y \) = Moment of inertia of full unreduced cross-section about y-axis
- \( K_y \) = Effective length factor for buckling about y-axis
- \( L_y \) = Unbraced length for bending about y-axis

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

\( C_{mx}, C_{my} \) = Coefficients whose values are determined in accordance with (a), (b), or (c) as follows:

(a) For compression members in frames subject to joint translation (sidesway)

\[ C_m = 0.85 \]

(b) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

\[ C_m = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right) \]  \hspace{1cm} (Eq. C5.2.2-8)

where

\[ \frac{M_1}{M_2} = \text{Ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. } \]

\( M_1/M_2 \) is positive when the member is bent in reverse curvature and negative when it is bent in single curvature

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

(1) For members whose ends are restrained, \( C_m = 0.85 \), and

(2) For members whose ends are unrestrained, \( C_m = 1.0 \).
D. STRUCTURAL ASSEMBLIES AND SYSTEMS

D1. Built-Up Sections

D1.1 Flexural Members Composed of Two Back-to-Back C-Sections

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

$$s_{\text{max}} = \frac{L}{6} \leq \frac{2g T_s}{mq} \quad (Eq. \text{D1.1-1})$$

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 E)
- $m =$ Distance from shear center of one C-section to mid-plane of web
- $q =$ Design load [factored load] on beam for spacing of connectors (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 weld spacing, $s$, the available strength [factored resistance] of the welds or connections closest to the load or reaction shall be calculated as follows:

$$T_s = \frac{P_s m}{2g} \quad (Eq. \text{D1.1-2})$$

where

- $P_s =$ Concentrated load [factored load] or reaction based on critical load combinations for ASD, LRFD, and LSD.

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 shall be 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 $T_s$, and $g$ is taken as the depth of the beam.

D1.2 Compression Members Composed of Two Sections in Contact

For compression members composed of two sections in contact, the available axial strength [factored axial resistance] shall be determined in accordance with Section C4.1(a) subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, $KL/r$ is replaced by $(KL/r)_m$ calculated as follows:
(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 C4.1.1 for definition of other symbols.

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

1. 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.
2. 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.
3. The intermediate fastener(s) or weld(s) at any longitudinal member tie location are capable of transmitting a force in any direction of 2.5 percent of the nominal axial strength [compressive resistance] of the built-up member.

D1.3 Spacing of Connections in Cover Plated Sections

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)

\( a \) 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;

\( b \) 1.16t \( \sqrt{E/f_c} \)

where

\( t \) = Thickness of the cover plate or sheet

\( f_c \) = Compressive stress at nominal load [specified load] in the cover plate or sheet

\( c \) 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.

D2 Mixed Systems

The design of members in mixed systems using cold-formed steel components in conjunction with other materials shall conform to this Specification and the applicable specification of the other material.
D3 Lateral and Stability Bracing

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

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements. See Appendix B for additional requirements.

D3.2 C-Section and Z-Section Beams

The following provisions for bracing to restrain twisting of C-sections and Z-sections used as beams loaded in the plane of the web 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 D6.3.1. Also, see Appendix B for additional requirements.

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

D3.2.1 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 D3.2.1-1 for illustrations of coordinate systems and positive force directions.

(a) For uniform loads

$$P_{L1} = 1.5\left[WyK' - (W_x/2) + (M_z/d)\right]$$

(Eq. D3.2.1-1)

$$P_{L2} = 1.5\left[WyK' - (W_x/2) - (M_z/d)\right]$$

(Eq. D3.2.1-2)

When the uniform load, $W$, acts through the plane of the web, i.e., $W_y = W$:

$$P_{L1} = -P_{L2} = 1.5(m/d)W$$

for C-sections

(Eq. D3.2.1-3)

$$P_{L1} = P_{L2} = 1.5\left(\frac{1_{xy}}{2I_x}\right)W$$

for Z-sections

(Eq. D3.2.1-4)

where

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

where

$W =$ Design load [factored load] (applied load determined in accordance with the most critical load combinations for ASD, LRFD or LSD, whichever is applicable) within a distance of 0.5a each side of the brace
where
\[ a = \text{Longitudinal distance between centerline of braces} \]
\[ K' = \begin{cases} 0 & \text{for C-sections} \\ \frac{I_{xy}}{2I_x} & \text{for Z-sections} \end{cases} \quad (Eq. D3.2.1-5) \]

where
\[ I_{xy} = \text{Product of inertia of full unreduced section} \]
\[ I_x = \text{Moment of inertia of full unreduced section about x-axis} \]
\[ M_z = -W_xe_{sy} + W_ye_{sx}, \text{torsional moment of W about shear center} \]

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

Figure D3.2.1-1 Coordinate Systems and Positive Force Directions

(b) For concentrated loads,
\[ P_{L1} = P_yK' - (P_x/2) + (M_z/d) \quad (Eq. D3.2.1-6) \]
\[ P_{L2} = P_yK' - (P_x/2) - (M_z/d) \quad (Eq. D3.2.1-7) \]

When a design load [factored load] acts through the plane of the web, i.e., \( P_y = P \):
\[ P_{L1} = -P_{L2} = (m/d)P \quad \text{for C-sections} \quad (Eq. D3.2.1-8) \]
\[ P_{L1} = P_{L2} = \left( \frac{I_{xy}}{2I_x} \right)P \quad \text{for Z-sections} \quad (Eq. D3.2.1-9) \]

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

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whichever is applicable.

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

See Section D3.2.1(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 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 [resistance] in accordance with Section C3.1.2.1.

**D3.3 Bracing of Axially Loaded Compression Members**

The required brace strength [resistance] to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

\[ P_{br,1} = 0.01P_n \]  \( \text{(Eq. D3.3-1)} \)

The required brace stiffness to restrain lateral translation at a brace point for an individual compression member shall be calculated as follows:

\[ \beta_{br,1} = \frac{2[4 - (2/n)]P_n}{L_b} \]  \( \text{(Eq. D3.3-2)} \)

where
\[ P_{br,1} = \text{Required nominal brace strength [resistance] for a single compression member} \]
\[ P_n = \text{Nominal axial compression strength [resistance] of a single compression member} \]
\[ \beta_{br,1} = \text{Required brace stiffness for a single compression member} \]
\[ n = \text{Number of equally spaced intermediate brace locations} \]
\[ L_b = \text{Distance between braces on one compression member} \]

**D4 Cold-Formed Steel Light-Frame Construction**

The design and installation of structural members and non-structural members utilized in cold-formed steel repetitive framing applications where the specified minimum base steel thickness is between 0.0179 in. (0.455 mm) and 0.1180 in. (2.997 mm) shall be in accordance with the AISI S200 and the following, as applicable:

(a) Headers, including box and back-to-back headers, and double and single L-headers, shall be designed in accordance with AISI S212 or solely in accordance with this Specification.

(b) Trusses shall be designed in accordance with AISI S214.

(c) Wall studs shall be designed in accordance with AISI S211, or solely in accordance with this Specification either on the basis of an all-steel system in accordance with Section D4.1 or on the basis of sheathing braced design in accordance with an appropriate theory, tests, or rational engineering analysis. Both solid and perforated webs shall be permitted. Both ends of the stud shall be connected to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis.

(d) Framing for floor and roof systems in buildings shall be designed in accordance with AISI...
S210 or solely in accordance with this Specification.
See Appendix A for additional country requirements.

D4.1 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 sheathing and shall comply with the requirements of Chapter C. For compression members with circular or non-circular web perforations, the effective section properties shall be determined in accordance with Section B2.2.

D5 Floor, Roof, or Wall Steel Diaphragm Construction

The in-plane diaphragm nominal shear strength [resistance], \( S_n \), shall be established by calculation or test. The safety factors and resistance factors for diaphragms given in Table D5 shall apply to both methods. If the nominal shear strength [resistance] is only established by test without defining all limit state thresholds, the safety factors and resistance factors shall be limited by the values given in Table D5 for connection types and connection-related failure modes. The more severe factored limit state shall control the design. Where fastener combinations are used within a diaphragm system, the more severe factor shall be used.

\[
\begin{align*}
\Omega_d &= \text{As specified in Table D5 (ASD)} \\
\phi_d &= \text{As specified in Table D5 (LRFD and LSD)}
\end{align*}
\]

<table>
<thead>
<tr>
<th>Load Type or Combinations Including</th>
<th>Connection Type</th>
<th>Connection Related</th>
<th>Panel Buckling*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \Omega_d ) (ASD)</td>
<td>( \phi_d ) (LRFD)</td>
<td>( \phi_d ) (LSD)</td>
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<tr>
<td>Earthquake</td>
<td>Welds</td>
<td>3.00</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td>2.50</td>
<td>0.65</td>
</tr>
<tr>
<td>Wind</td>
<td>Welds</td>
<td>2.35</td>
<td>0.70</td>
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<tr>
<td></td>
<td>Screws</td>
<td>2.65</td>
<td>0.60</td>
</tr>
<tr>
<td>All Others</td>
<td>Welds</td>
<td>2.50</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>Screws</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note:
*Panel buckling is out-of-plane buckling and not local buckling at fasteners.

For mechanical fasteners other than screws:
(a) \( \Omega_d \) shall not be less than the Table D5 values for screws, and
(b) \( \phi_d \) shall not be greater than the Table D5 values for screws.

In addition, the value of \( \Omega_d \) and \( \phi_d \) using mechanical fasteners other than screws shall be limited by the \( \Omega \) and \( \phi \) values established through calibration of the individual fastener shear strength [resistance], unless sufficient data exist to establish a diaphragm system effect in accordance with Section F1.1. Fastener shear strength [resistance] calibration shall include the diaphragm material type. Calibration of individual fastener shear strengths [resistance] shall be in accordance with Section F1.1. The test assembly shall be such that the tested
failure mode is representative of the design. The impact of the thickness of the supporting material on the failure mode shall be considered.

**D6 Metal Roof and Wall Systems**

The provisions of Section D6.1 through D6.3 shall apply to metal roof and wall systems that include cold-formed steel *purlins, girts*, through-fastened wall/roof and wall panels, or standing seam roof panels, as applicable.

**D6.1 Purlins, Girts and Other Members**

**D6.1.1 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 [moment resistance], $M_n$, of a C- or Z-section loaded in a plane parallel to the *web*, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced, shall be calculated in accordance with Eq. D6.1.1-1. The *safety factor* and *resistance factors* given in this section shall be used to determine the allowable flexural strength or design flexural strength [factored moment resistance] in accordance with the applicable design method in Section A4, A5, or A6.

$$M_n = RS_eF_y \quad (Eq. \text{ D6.1.1-1})$$

| $\Omega_b$ | 1.67 (ASD) |
| $\phi_b$  | 0.90 (LRFD) |

where $R$ is obtained from Table D6.1.1-1 for simple span C- or Z-sections, and

- $R = 0.60$ for continuous span C-sections
- $R = 0.70$ for continuous span Z-sections

$S_e$ and $F_y$ = Values as defined in Section C3.1.1

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

1. Member depth $\leq 11.5$ in. (292 mm),
2. Member flanges with edge stiffeners,
3. $60 \leq \text{depth/thickness} \leq 170$,
4. $2.8 \leq \text{depth/flange width} \leq 4.5$,
5. $16 \leq \text{flat width/thickness of flange} \leq 43$,
6. For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than $1.5d$,
7. Member span length is not greater than 33 feet (10 m),
8. Both flanges are prevented from moving laterally at the supports,
9. 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 a maximum of 12 in. (305 mm) on centers and attached in a manner to effectively inhibit relative movement between the panel and *purlin* flange,
10. 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,
(11) 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 1/2 in. (12.7 mm) diameter,
(12) Fasteners is not standoff type screws,
(13) Fasteners are spaced not greater than 12 in. (305 mm) on centers and placed near the center of the beam flange, and adjacent to the panel high rib, and
(14) The design yield stress of the member does exceed 60 ksi (410 MPa or 4220 kg/cm²).

If variables fall outside any of the above stated limits, the user shall perform full-scale tests in accordance with Section F1 of this Specification or apply a rational engineering analysis procedure. For continuous purlin systems in which adjacent bay span lengths vary by more than 20 percent, the R values for the adjacent bays shall be taken from Table D6.1.1-1. The user shall be permitted to perform tests in accordance with Section F1 as an alternate to the procedure described in this section.

**TABLE D6.1.1-1**

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

For simple span members, R shall be reduced for the effects of compressed insulation between the sheeting and the member. The reduction shall be calculated by multiplying R from Table D6.1.1-1 by the following correction factor, r:

\[ r = 1.00 - 0.01 t_i \]  
\[ r = 1.00 - 0.0004 t_i \]  
where
\[ t_i = \text{Thickness of uncompressed glass fiber blanket insulation} \]

**D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System**

See Section D6.1.2 of Appendix A or B for the provisions of this section.

**D6.1.3 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).

(a) The weak axis nominal strength [resistance] shall be calculated in accordance with Eq. D6.1.3-1. The safety factor and resistance factors given in this section shall be used to determine the allowable axial strength or design axial strength [factored compressive resistance] in accordance with the applicable design method in Section A4, A5, or A6.
\[ P_n = \frac{C_1 C_2 C_3 AE}{29500} \quad (Eq. \ D6.1.3-1) \]

\[ \Omega = 1.80 \quad (ASD) \]

\[ \phi = 0.85 \quad (LRFD) \]

\[ = 0.80 \quad (LSD) \]

where

\[ C_1 = (0.79x + 0.54) \quad (Eq. \ D6.1.3-2) \]

\[ C_2 = (1.17\alpha t + 0.93) \quad (Eq. \ D6.1.3-3) \]

\[ C_3 = \alpha(2.5b - 1.63d) + 22.8 \quad (Eq. \ D6.1.3-4) \]

where

- \( x \) = For Z-sections, the fastener distance from the outside web edge divided by the flange width, as shown in Figure D6.1.3.
- \( = \) For C-sections, the flange width minus the fastener distance from the outside web edge divided by the flange width, as shown in Figure D6.1.3.

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

- \( = 1 \) when \( t, b, \) and \( d \) are in inches
- \( = 0.0394 \) when \( t, b, \) and \( d \) are in mm
- \( = 0.394 \) when \( t, b, \) and \( d \) are in cm

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

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

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

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

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

- \( = 29,500 \text{ ksi for U.S. customary units} \)
- \( = 203,000 \text{ MPa for SI units} \)
- \( = 2,070,000 \text{ kg/cm}^2 \text{ for MKS units} \)

Eq. D6.1.3-1 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. Flanges are edge stiffened compression elements,
4. \( 70 \leq d/t \leq 170 \),
5. \( 2.8 \leq d/b \leq 5 \),
6. \( 16 \leq \text{flange flat width} / t \leq 50 \),
7. Both flanges are prevented from moving laterally at the supports,
8. Steel roof or steel wall panels with fasteners spaced 12 in. (305 mm) on center or less and having a minimum rotational lateral stiffness of 0.0015 k/in./in. (10,300 N/m/m 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 yield stress of 33 ksi (230 MPa or 2320 kg/cm²), and
10. Span length not exceeding 33 feet (10 m).

(b) The strong axis available strength [factored resistance] shall be determined in accordance with Sections C4.1 and C4.1.1.
D6.1.4 Compression of Z-Section 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 D6.1.4 of Appendix A.

D6.2 Standing Seam Roof Panel Systems

D6.2.1 Strength [Resistance] of Standing Seam Roof Panel Systems

Under gravity loading, the nominal strength [resistance] of standing seam roof panels shall be determined in accordance with Chapters B and C 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:

(1) The Uplift Pressure Test Procedure for Class 1 Panel Roofs in FM 4471 shall be permitted.

(2) Existing tests conducted in accordance with CEGS 07416 uplift test procedure prior to the adoption of these provisions shall be permitted.

The open-open end configuration, although not prescribed by the ASTM E1592 test procedure, shall be 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 D6.2.1a of Appendix A.

When the number of physical test assemblies is 3 or more, safety factors and resistance factors shall be determined in accordance with the procedures of Section F1.1(b) 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
\[ V_M = \text{Coefficient of variation of the material factor} \]
\[ = 0.08 \text{ for anchor failure mode} \]
\[ = 0.10 \text{ for other failure modes} \]
\[ V_F = \text{Coefficient of variation of the fabrication factor} \]
\[ = 0.05 \]
\[ V_Q = \text{Coefficient of variation of the load effect} \]
\[ = 0.21 \]
\[ V_P = \text{Actual calculated coefficient of variation of the test results, without limit} \]
\[ n = \text{Number of anchors in the test assembly with same tributary area (for anchor failure) or number of panels with identical spans and loading to the failed span (for non-anchor failures)} \]

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 3, a safety factor, \( \Omega \), of 2.0 and a resistance factor, \( \phi \), of 0.8 (LRFD) and 0.70 (LSD) shall be used.

### D6.3 Roof System Bracing and Anchorage

#### D6.3.1 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 Sections C3.1 and D6.1, 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. D6.3.1-1 and shall satisfy the minimum stiffness requirement of Eq. D6.3.1-7. In addition, purlins shall be restrained laterally by the sheathing so that the maximum top flange lateral displacements between lines of lateral anchorage at nominal loads [specified loads] 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.

\[
P_{L_j} = \sum_{i=1}^{N_p} \left( \frac{P_i}{K_{total_i}} \right) \left( \frac{K_{eff,i,j}}{K_{pp}} \right) \quad (Eq. \text{ D6.3.1-1})
\]

where

\( P_{L_j} \) = 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
index for each purlin line (i=1, 2, ..., Np)
j = Index for each anchorage device (j=1, 2, ..., Na)

where

Na = Number of anchorage devices along a line of anchorage

Pi = Lateral force introduced into the system at the ith purlin

\( \text{Pi} = (C1)W_{pi} \left[ \left( \frac{C2}{1000} \right) \frac{I_{xy}L}{I_x d} + (C3) \left( \frac{m + 0.25b}{d} \right) \alpha \cos \theta - (C4) \sin \theta \right] \)  

(Eq. D6.3.1-2)

where

C1, C2, C3, and C4 = Coefficients tabulated in Tables D6.3.1-1 to D6.3.1-3

W_{pi} = Total required vertical load supported by the ith purlin in a single bay

\( w_i L \)  

(Eq. D6.3.1-3)

where

w_i = Required distributed gravity load supported by the ith purlin per unit length (determined from the critical load combination for ASD, LRFD or LSD)

I_{xy} = Product of inertia of full unreduced section about centroidal axes parallel and perpendicular to the purlin web (I_{xy} = 0 for C-sections)

L = Purlin span length

m = Distance from shear center to mid-plane of web (m = 0 for Z-sections)

b = Top flange width of purlin

t = Purlin thickness

I_x = Moment of inertia of full unreduced section about centroidal axis perpendicular to the purlin web

d = Depth of purlin

\( \alpha = +1 \) for top flange facing in the up-slope direction

\( \alpha = -1 \) for top flange facing in the down-slope direction

\( \theta = \) Angle between vertical and plane of purlin web

K_{eff,i,j} = Effective lateral stiffness of the jth anchorage device with respect to the ith purlin

\[ \frac{1}{K_{eff,i,j}} = \left[ \frac{1}{K_a} + \frac{d_{pi,j}}{(C6)A_p E} \right]^{-1} \]  

(Eq. D6.3.1-4)

where

d_{pi,j} = Distance along roof slope between the ith purlin line and the jth anchorage device

K_a = Lateral stiffness of the anchorage device

C6 = Coefficient tabulated in Tables D6.3.1-1 to D6.3.1-3

A_p = Gross cross-sectional area of roof panel per unit width

E = Modulus of elasticity of steel

K_{total,i} = Effective lateral stiffness of all elements resisting force P_i


\[
K_{\text{sys}} = \sum_{j=1}^{N_3} \left(K_{\text{eff},i,j} \right) + K_{\text{sys}} \quad (\text{Eq. D6.3.1-5})
\]

where

\[K_{\text{sys}} = \text{Lateral stiffness of the roof system, neglecting anchorage devices}\]

\[
\frac{C_5}{1000} (N_p) \frac{E t^2}{d^2} \quad (\text{Eq. D6.3.1-6})
\]

For multi-span systems, force $P_i$, calculated in accordance with Eq. D6.3.1-2 and coefficients C1 to C4 from Tables D6.3.1-1 to D6.3.1-3 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 shall be used. The values for $P_i$ in Eq. D6.3.1-1 and Eq. D6.3.1-8 shall be taken as the average of the values found from Eq. D6.3.1-2 evaluated separately for each of the two bays. The values of $K_{\text{sys}}$ and $K_{\text{eff},i,j}$ in Eq. D6.3.1-1 and Eq. D6.3.1-5 shall be calculated using Eq. D6.3.1-4 and Eq. D6.3.1-6, with $L$, $t$, and $d$ taken as the average values of the two bays.

For systems with multiple spans and anchorage devices at either 1/3 points or midpoints, where the adjacent bays have different section properties or span lengths than the bay under consideration, the following procedures shall be used to account for the influence of the adjacent bays. The values for $P_i$ in Eq. D6.3.1-1 and Eq. D6.3.1-8 shall be taken as the average of the values found from Eq. D6.3.1-2 evaluated separately for each of the three bays. The value of $K_{\text{sys}}$ in Eq. D6.3.1-5 shall be calculated using Eq. D6.3.1-6, with $L$, $t$, and $d$ taken as the average of the values from the three bays. The values of $K_{\text{eff},i,j}$ shall be calculated using Eq. D6.3.1-4, with $L$ taken as the span length of the bay under consideration. At an end bay, when computing the average values for $P_i$ or averaging the properties for computing $K_{\text{sys}}$, 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:

\[
K_{\text{total},i} \geq K_{\text{req}} \quad (\text{Eq. D6.3.1-7})
\]

where

\[
K_{\text{req}} = \Omega \frac{20 \sum_{i=1}^{N_p} P_i}{d} \quad (\text{ASD}) \quad (\text{Eq. D6.3.1-8a})
\]

\[
K_{\text{req}} = \frac{1}{\phi} \frac{20 \sum_{i=1}^{N_p} P_i}{d} \quad (\text{LRFD, LSD}) \quad (\text{Eq. D6.3.1-8b})
\]

\[
\begin{align*}
\Omega &= 2.00 \quad (\text{ASD}) \\
\phi &= 0.75 \quad (\text{LRFD})
\end{align*}
\]
\[ = 0.70 \quad \text{(LSD)} \]

In lieu of the Eqs. D6.3.1-1 through D6.3.1-6, lateral restraint forces shall be permitted to be determined from alternate analysis. Alternate 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. Alternate 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 analysis, the maximum top flange lateral displacement of the purlin between lines of lateral bracing at nominal loads 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 be calculated at factored load levels for LRFD or LSD and nominal load levels for ASD and shall be limited to:

\[
\Delta_{tf} \leq \frac{1}{20} \frac{d}{\Omega} \quad \text{(ASD)} \quad (Eq. \text{D6.3.1-9a})
\]

\[
\Delta_{tf} \leq \phi \frac{d}{20} \quad \text{(LRFD, LSD)} \quad (Eq. \text{D6.3.1-9b})
\]

<table>
<thead>
<tr>
<th>Table D6.3.1-1</th>
<th>Coefficients for Support Restraints</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple Span</td>
<td>C1</td>
</tr>
<tr>
<td>Through Fastened (TF)</td>
<td>0.5</td>
</tr>
<tr>
<td>Standing Seam (SS)</td>
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</tr>
<tr>
<td>TF</td>
<td>Exterior Frame Line</td>
</tr>
<tr>
<td></td>
<td>First Interior Frame Line</td>
</tr>
<tr>
<td></td>
<td>All Other Locations</td>
</tr>
<tr>
<td>SS</td>
<td>Exterior Frame Line</td>
</tr>
<tr>
<td></td>
<td>First Interior Frame Line</td>
</tr>
<tr>
<td></td>
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</tr>
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</table>

<table>
<thead>
<tr>
<th>Table D6.3.1-2</th>
<th>Coefficients for Mid-Point Restraints</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple Span</td>
<td>C1</td>
</tr>
<tr>
<td>Through Fastened (TF)</td>
<td>1.0</td>
</tr>
<tr>
<td>Standing Seam (SS)</td>
<td>1.0</td>
</tr>
<tr>
<td>TF</td>
<td>End Bay</td>
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<tr>
<td></td>
<td>First Interior Bay</td>
</tr>
<tr>
<td></td>
<td>All Other Locations</td>
</tr>
<tr>
<td>SS</td>
<td>End Bay</td>
</tr>
<tr>
<td></td>
<td>First Interior Bay</td>
</tr>
<tr>
<td></td>
<td>All Other Locations</td>
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</tbody>
</table>
### Table D6.3.1-3

Coefficients for One-Third Point Restraints

<table>
<thead>
<tr>
<th>Case</th>
<th>Simple Span</th>
<th>Multiple Spans</th>
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<tr>
<td></td>
<td>Through Fastened (TF)</td>
<td>End Bay Exterior Anchor</td>
</tr>
<tr>
<td></td>
<td>Standing Seam (SS)</td>
<td>End Bay Int. Anchor and 1st Int. Bay Ext. Anchor</td>
</tr>
<tr>
<td>TF</td>
<td>0.5 7.8 42 0.98 0.39 0.40</td>
<td>0.5 15 17 0.98 0.72 0.043</td>
</tr>
<tr>
<td>SS</td>
<td>0.5 7.3 21 0.73 0.19 0.18</td>
<td>0.5 2.4 50 0.96 0.82 0.20</td>
</tr>
<tr>
<td></td>
<td>End Bay Exterior Anchor</td>
<td>End Bay Int. Anchor and 1st Int. Bay Ext. Anchor</td>
</tr>
<tr>
<td></td>
<td>All Other Locations</td>
<td>All Other Locations</td>
</tr>
<tr>
<td>TF</td>
<td>0.5 6.1 41 0.96 0.69 0.12</td>
<td>0.5 3.8 45 0.65 0.10 0.014</td>
</tr>
<tr>
<td>SS</td>
<td>All Other Locations</td>
<td>All Other Locations</td>
</tr>
</tbody>
</table>

### D6.3.2 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 shall be permitted in lieu of the requirements of Section D6.3.1. 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 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 calculated at nominal load [specified load] levels or \( \phi d/20 \) for LRFD and LSD calculated at factored load levels, 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, calculated at nominal load levels, 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

- \( \Omega = 2.0 \) (ASD)
- \( \phi = 0.75 \) (LRFD)
- \( \phi = 0.70 \) (LSD)
**E. CONNECTIONS AND JOINTS**

**E1 General Provisions**

Connections shall be designed to transmit the required strength [factored loads] acting on the connected members with consideration of eccentricity where applicable.

**E2 Welded Connections**

The following design criteria shall apply to 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. For the design of welded connections in which the thickness of the thinnest connected part is greater than 3/16 in. (4.76 mm), refer to the specifications or standards stipulated in the corresponding Section E2a of Appendix A or B.

Welds shall follow the requirements of the weld standards also stipulated in Section E2a of Appendix A or B. For diaphragm applications, Section D5 shall apply.

**E2.1 Groove Welds in Butt Joints**

The nominal strength [resistance], $P_n$, of a groove weld in a butt joint, welded from one or both sides, shall be determined in accordance with (a) or (b), as applicable. The corresponding safety factor and resistance factors shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

(a) For tension or compression normal to the effective area or parallel to the axis of the weld, the nominal strength [resistance], $P_n$, shall be calculated in accordance with Eq. E2.1-1:

$$ P_n = Lt_e F_y \quad \Omega = 1.70 \quad (ASD) \quad \phi = 0.90 \quad (LRFD) \quad = 0.80 \quad (LSD) $$

(b) For shear on the effective area, the nominal strength [resistance], $P_n$, shall be the smaller value calculated in accordance with Eqs. E2.1-2 and E2.1-3:

$$ P_n = \frac{Lt_e F_y}{\sqrt{3}} \quad \Omega = 1.70 \quad (ASD) \quad \phi = 0.90 \quad (LRFD) \quad = 0.80 \quad (LSD) $$

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
\( F_{xx} = \text{Tensile strength of electrode classification} \)

**E2.2 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 connected part exceeds 0.15 in. (3.81 mm) in thickness, nor through a combination of steel sheets having a total thickness over 0.15 in. (3.81 mm).

Weld washers, as shown in Figures E2.2-1 and E2.2-2, 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 (1.27 mm) and 0.08 in. (2.03 mm) with a minimum prepunched hole of 3/8 in. (9.53 mm) diameter. Sheet-to-sheet welds shall not require weld washers.

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

**E2.2.1 Shear**

**E2.2.1.1 Minimum Edge Distance**

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of \( e_{\text{min}} \) determined in accordance with Eq. E2.2.1.1-1 or Eq. E2.2.1.1-2, as applicable. See Figures E2.2.1.1-1 and E2.2.1.1-2 for
The corresponding safety factors and resistance factors shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
\begin{align*}
    e_{\text{min}} &= \frac{P\Omega}{F_u t} \quad \text{for ASD} \\
    e_{\text{min}} &= \frac{\bar{P}}{\phi F_u t} \quad \text{for LRFD and LSD}
\end{align*}
\] (Eq. E2.2.1.1-1)

When \(\frac{F_u}{F_{sy}} \geq 1.08\)
- \(\Omega = 2.20\) (ASD)
- \(\phi = 0.70\) (LRFD)
- \(\phi = 0.60\) (LSD)

When \(\frac{F_u}{F_{sy}} < 1.08\)
- \(\Omega = 2.55\) (ASD)
- \(\phi = 0.60\) (LRFD)
- \(\phi = 0.50\) (LSD)

where
- \(P\) = Required shear strength (nominal force) transmitted by weld (ASD)
- \(F_u\) = Tensile strength as determined in accordance with A2.1, A2.2, or A2.3.2
- \(t\) = Total combined base steel thickness (exclusive of coatings) of sheet(s) involved in shear transfer above plane of maximum shear transfer
- \(\bar{P}\) = Required shear strength [factored shear load] transmitted by weld
  - \(P_u\) (LRFD)
  - \(P_f\) (LSD)
- \(F_{sy}\) = Yield stress as determined in accordance with Section A2.1, A2.2, or A2.3.2

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.

![Figure E2.2.1.1-1 Edge Distance for Arc Spot Welds – Single Sheet](image)
E2.2.1.2 Shear Strength [Resistance] for Sheet(s) Welded to a Thicker Supporting Member

The nominal shear strength [resistance], \( P_n \), 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 allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

(a) \[
P_n = \frac{\pi d_s^2}{4} 0.75 F_{xx} \]
\[
\Omega = 2.55 \quad (ASD) \\
\phi = 0.60 \quad (LRFD) \\
\phi = 0.50 \quad (LSD) \]

(b) For \((d_a/t) \leq 0.815 \sqrt{E/F_u}\)
\[
P_n = 2.20 t \, d_a \, F_{u} \] \hspace{1cm} (Eq. E2.2.1.2-2)
\[
\Omega = 2.20 \quad (ASD) \\
\phi = 0.70 \quad (LRFD) \\
\phi = 0.60 \quad (LSD) \]
For \(0.815 \sqrt{E/F_u} < (d_a/t) < 1.397 \sqrt{E/F_u}\)
\[
P_n = 0.280 \left[ 1 + 5.59 \frac{\sqrt{E/F_u}}{d_a/t} \right] t d_a F_{u} \] \hspace{1cm} (Eq. E2.2.1.2-3)
\[
\Omega = 2.80 \quad (ASD) \\
\phi = 0.55 \quad (LRFD) \\
\phi = 0.45 \quad (LSD) \]
For \((d_a/t) \geq 1.397 \sqrt{E/F_u}\)
\[
P_n = 1.40 t \, d_a \, F_{u} \] \hspace{1cm} (Eq. E2.2.1.2-4)
\[
\Omega = 3.05 \quad (ASD) \\
\phi = 0.50 \quad (LRFD) \\
\phi = 0.40 \quad (LSD) \]
where

\[ P_n = \text{Nominal shear strength [resistance] of arc spot weld} \]
\[ d_e = \text{Effective diameter of fused area at plane of maximum shear transfer} \]
\[ = 0.7d - 1.5t \leq 0.55d \]  

(Eq. E2.2.1.2-5)

where

\[ d = \text{Visible diameter of outer surface of arc spot weld} \]
\[ t = \text{Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer} \]
\[ F_{xx} = \text{Tensile strength of electrode classification} \]
\[ d_a = \text{Average diameter of arc spot weld at mid-thickness of t where } d_a = (d - t) \]

for single sheet or multiple sheets not more than four lapped sheets over a supporting member. See Figures E2.2.1.2-1 and E2.2.1.2-2 for diameter definitions.

\[ E = \text{Modulus of elasticity of steel} \]
\[ F_{tu} = \text{Tensile strength as determined in accordance with Section A2.1, A2.2, or A2.3.2} \]

**E2.2.1.3 Shear Strength [Resistance] for Sheet-to-Sheet Connections**

The nominal shear strength [resistance] for each weld between two sheets of equal thickness shall be determined in accordance with Eq. E2.2.1.3-1. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.
\[ P_n = 1.65td_aF_u \quad (Eq. \text{ E2.2.1.3-1}) \]
\[ \Omega = 2.20 \quad (ASD) \]
\[ \phi = 0.70 \quad (LRFD) \]
\[ = 0.60 \quad (LSD) \]

where

- \( P_n \) = Nominal shear strength [resistance] of sheet-to-sheet connection
- \( t \) = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer above plane of maximum shear transfer
- \( d_a \) = Average diameter of arc spot weld at mid-thickness of \( t \). See Figure E2.2.1.3-1 for diameter definitions.
  = \( (d - t) \)
- \( \Omega \) = ASD
- \( \phi \) = LRFD
- \( \phi \) = LSD

\[ d \] = Visible diameter of the outer surface of arc spot weld
\[ d_e \] = Effective diameter of fused area at plane of maximum shear transfer
  = \( 0.7d - 1.5t \leq 0.55d \) \( (Eq. \text{ E2.2.1.3-2}) \)
\[ F_u \] = Tensile strength of sheet as determined in accordance with Section A2.1 or A2.2

In addition, the following limits shall apply:

1. \( F_u \leq 59 \text{ ksi (407 MPa or 4150 kg/cm}^2) \),
2. \( F_{xx} > F_u \), and
3. \( 0.028 \text{ in. (0.71 mm)} \leq t \leq 0.0635 \text{ in. (1.61 mm)} \).

**E2.2.2 Tension**

The uplift nominal tensile strength [resistance], \( P_{nv} \), of each concentrically loaded arc spot weld connecting sheets and supporting member shall be computed as the smaller of either Eq. E2.2.2-1 or Eq. E2.2.2-2 as follows. The safety factor and resistance factors shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[ P_n = \frac{\pi d_e^2}{4} F_{xx} \quad (Eq. \text{ E2.2.2-1}) \]
\[ P_n = 0.8(F_u/F_y)^2td_aF_u \quad (Eq. \text{ E2.2.2-2}) \]

For panel and deck applications:
Ω = 2.50  (ASD)
φ = 0.60  (LRFD)
= 0.50  (LSD)

For all other applications:
Ω = 3.00  (ASD)
φ = 0.50  (LRFD)
= 0.40  (LSD)

The following limits shall apply:

\[ t \cdot d_a \cdot F_u \leq 3 \text{ kips (13.34 \text{ kN})}, \]
\[ e_{\text{min}} \geq d, \]
\[ F_{xx} \geq 60 \text{ ksi (410 MPa or 4220 kg/cm}^2\text{)}, \]
\[ F_u \leq 82 \text{ ksi (565 MPa or 5770 kg/cm}^2\text{)} \text{ (of connecting sheets), and} \]
\[ F_{xx} > F_u. \]

See Section E2.2.1 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 [resistance] shall be determined by using the sum of the sheet thicknesses as given by Eq. E2.2.2-2.

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

Where it is shown by measurement that a given weld procedure consistently gives a larger effective diameter, \( d_e \), or average diameter, \( d_a \), as applicable, this larger diameter shall be permitted to be used provided the particular welding procedure used for making those welds is followed.

### E2.3 Arc Seam Welds

Arc seam welds (See Figure E2.3-1) covered by this Specification shall apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position, and
(b) Sheet to sheet in the horizontal or flat position.

The nominal shear strength [resistance], \( P_n \), of arc seam welds shall be determined by using the smaller of either Eq. E2.3-1 or Eq. E2.3-2. The safety factor and resistance factors in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

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

\[
P_n = 2.5tF_u \left( 0.25L + 0.96d_a \right) \quad \text{(Eq. E2.3-2)}
\]

\[ \Omega = 2.55 \quad \text{(ASD)} \]
\[ \phi = 0.60 \quad \text{(LRFD)} \]
\[ = 0.50 \quad \text{(LSD)} \]

where
\[ P_n = \text{Nominal shear strength [resistance] of arc seam weld} \]
\( d_e = \) Effective width of seam weld at fused surfaces  
\[ = 0.7d - 1.5t \] (Eq. E2.3-3)

where

\( d = \) 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 seam weld  
\[ = (d - t) \] for single or double sheets (Eq. E2.3-4)

\( F_u, F_{xx}, \) and \( t = \) Values as defined in Section E2.2.1

The minimum edge distance shall be as determined for the arc spot weld in accordance with Section E2.2.1. See Figure E2.3-2 for details.

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

\( \geq e_{\text{min}} \)

\( \geq e_{\text{min}} CL \)

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

### E2.4 Fillet Welds

Fillet welds covered by this Specification shall apply to the welding of joints in any position, either sheet to sheet, or sheet to thicker steel member.

The nominal shear strength [resistance], \( P_n \), of a fillet 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 allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.
(1) For longitudinal loading:
For \( L/t < 25 \)
\[
P_n = \left( 1 - \frac{0.01L}{t} \right) L F_u
\]
\( \Omega = 2.55 \) (ASD)
\( \phi = 0.60 \) (LRFD)
\( = 0.50 \) (LSD)
For \( L/t \geq 25 \)
\[
P_n = 0.75 t L F_u
\]
\( \Omega = 3.05 \) (ASD)
\( \phi = 0.50 \) (LRFD)
\( = 0.40 \) (LSD)

(2) For transverse loading:
\[
P_n = t L F_u
\]
\( \Omega = 2.35 \) (ASD)
\( \phi = 0.65 \) (LRFD)
\( = 0.60 \) (LSD)

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

In addition, for \( t > 0.10 \) in. (2.54 mm), the nominal strength [resistance] determined in accordance with (1) and (2) shall not exceed the following value of \( P_n \):
\[
P_n = 0.75 t w L F_{xx}
\]
\( \Omega = 2.55 \) (ASD)
\( \phi = 0.60 \) (LRFD)
\( = 0.50 \) (LSD)

where
\( P_n = \) Nominal strength [resistance] of fillet weld
\( L = \) Length of fillet weld
\( F_u \) and \( F_{xx} = \) Values as defined in Section E2.2.1
\( t_w = \) Effective throat
\( = 0.707 w_1 \) or \( 0.707 w_2 \), 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
\( w_1 \) and \( w_2 = \) leg of weld (see Figures E2.4-1 and E2.4-2) and \( w_1 \leq t_1 \) in lap joints

### E2.5 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], \( P_{n\nu} \), 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 allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

(a) For flare-bevel groove welds, transverse loading (see Figure E2.5-1):

\[
P_n = 0.833tLF_u \quad (\text{Eq. E2.5-1})
\]

\( \Omega = 2.55 \) (ASD)

\( \phi = 0.60 \) (LRFD)

\( \phi = 0.50 \) (LSD)

(b) For flare groove welds, longitudinal loading (see Figures E2.5-2 through E2.5-7):

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

\[
P_n = 0.75tLF_u \quad (\text{Eq. E2.5-2})
\]

\( \Omega = 2.80 \) (ASD)

\( \phi = 0.55 \) (LRFD)

\( \phi = 0.45 \) (LSD)

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

\[
P_n = 1.50tLF_u \quad (\text{Eq. E2.5-3})
\]

\( \Omega = 2.80 \) (ASD)

\( \phi = 0.55 \) (LRFD)

\( \phi = 0.45 \) (LSD)

In addition, for \( t > 0.10 \) in. (2.54 mm), the nominal strength [resistance] determined in accordance with (a) and (b) shall not exceed the value of \( P_n \) calculated in accordance with Eq. E2.5-4.
\[ P_n = 0.75 t_w L F_{xx} \]

\[ \Omega = 2.55 \quad \text{(ASD)} \]

\[ \phi = 0.60 \quad \text{(LRFD)} \]

\[ = 0.50 \quad \text{(LSD)} \]

where

\[ P_n = \text{Nominal strength [resistance] of flare groove weld} \]

\[ t = \text{Thickness of welded member as defined in Figures E2.5-1 to E2.5-7} \]
Chapter E, Connections and Joints

L = Length of weld

$F_u$ and $F_{xx}$ = Values as defined in Section E2.2.1

h = Height of lip

t_w = Effective throat of flare groove weld filled flush to surface (See Figures E2.5-4 and E2.5-5):

- $(5/16)R$ for flare bevel groove weld
- $(1/2)R$ when $R \leq 1/2$ in. (12.7mm) for flare V-groove weld
- $(3/8)R$ when $R > 1/2$ in. (12.7mm) for flare V-groove weld

Effective throat of flare groove weld not filled flush to surface:

- $0.707w_1$ or $0.707w_2$, whichever is smaller (see Figures E2.5-6 and E2.5-7)

where

- $R$ = Radius of outside bend surface
- $w_1$ and $w_2$ = Leg of weld (see Figures E2.5-6 and E2.5-7)

### E2.6 Resistance Welds

The nominal shear strength [resistance], $P_n$, of 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 allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

- $\Omega = 2.35$ (ASD)
- $\phi = 0.65$ (LRFD)
- $= 0.55$ (LSD)

When $t$ is in inches and $P_n$ is in kips:

For $0.01 \text{ in.} \leq t < 0.14 \text{ in.}$

$$P_n = 144t^{1.47}$$

(Eq. E2.6-1)

For $0.14 \text{ in.} \leq t \leq 0.18 \text{ in.}$

$$P_n = 43.4t + 1.93$$

(Eq. E2.6-2)

When $t$ is in millimeters and $P_n$ is in kN:

For $0.25 \text{ mm} \leq t < 3.56 \text{ mm}$

$$P_n = 5.51t^{1.47}$$

(Eq. E2.6-3)

For $3.56 \text{ mm} \leq t \leq 4.57 \text{ mm}$

$$P_n = 7.6t + 8.57$$

(Eq. E2.6-4)

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

For $0.025 \text{ cm} \leq t < 0.356 \text{ cm}$

$$P_n = 16600t^{1.47}$$

(Eq. E2.6-5)

For $0.356 \text{ cm} \leq t \leq 0.457 \text{ cm}$

$$P_n = 7750t + 875$$

(Eq. E2.6-6)

where

- $P_n$ = Nominal strength [resistance] of resistance weld
- $t$ = Thickness of thinnest outside sheet
E2.7 Rupture in Net Section of Members other than Flat Sheets (Shear Lag)

The nominal tensile strength [resistance] of a welded member shall be determined in accordance with Section C2. For rupture and/or yielding in the effective net section of the connected part, the nominal tensile strength [resistance], \( P_n \), shall be determined in accordance with Eq. E2.7-1. The safety factor and resistance factors given in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
P_n = A_e F_u \quad \text{(Eq. E2.7-1)}
\]

\[
\Omega = 2.50 \quad \text{(ASD)}
\]

\[
\phi = 0.60 \quad \text{(LRFD)}
\]

\[
= 0.50 \quad \text{(LSD)}
\]

where

\( F_u \) = Tensile strength of the connected part as determined in accordance with Section A2.1 or A2.3.2

\( A_e \) = \( A U \), effective net area with \( U \) defined as follows:

When the load is transmitted only by transverse welds:

\[
A = \text{Area of directly connected elements}
\]

\[
U = 1.0
\]

When the load is transmitted only by longitudinal welds or by longitudinal welds in combination with transverse welds:

\[
A = \text{Gross area of member, } A_g
\]

\[
U = 1.0 \text{ for members when load is transmitted directly to all of the cross-sectional elements.}
\]

Otherwise the reduction coefficient \( U \) shall be determined in accordance with (a) or (b):

(a) For angle members

\[
U = 1.0 - 1.20 \frac{x}{L} < 0.9
\]

but \( U \geq 0.4 \).  \( \quad \text{(Eq. E2.7-2)} \)

(b) For channel members

\[
U = 1.0 - 0.36 \frac{\bar{x}}{L} < 0.9
\]

but \( U \geq 0.5 \).  \( \quad \text{(Eq. E2.7-3)} \)

where

\( \bar{x} \) = Distance from shear plane to centroid of cross-section

\( L \) = Length of longitudinal weld

E3 Bolted Connections

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

Bolts, nuts, and washers conforming to one of the following ASTM specifications shall be approved for use under this Specification:

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

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

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

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

See Section E3.1 of the Appendix A or B for the provisions of this section.

### E3.2 Rupture in Net Section (Shear Lag)

See Section E3.2 of the Appendix A or B for the provisions of this section.

### E3.3 Bearing

The nominal bearing strength [resistance] of bolted connections shall be determined in accordance with Sections E3.3.1 and E3.3.2. For conditions not shown, the available bearing strength [factored resistance] of bolted connections shall be determined by tests.
E3.3.1 Strength [Resistance] without Consideration of Bolt Hole Deformation

When deformation around the bolt holes is not a design consideration, the nominal bearing strength [resistance], \( P_n \), of the connected sheet for each loaded bolt shall be determined in accordance with Eq. E3.3.1-1. The safety factor and resistance factors given in this section shall be used to determine the allowable strength or design strength [factored resistance] in accordance with the applicable design method in Section A4, A5, or A6.

\[
P_n = C m_f d t F_u \tag{Eq. E3.3.1-1}
\]

<table>
<thead>
<tr>
<th>( \Omega )</th>
<th>(ASD)</th>
<th>( \phi )</th>
<th>(LRFD)</th>
<th>( \phi )</th>
<th>(LSD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50</td>
<td>0.60</td>
<td>0.50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

where

- \( C \) = Bearing factor, determined in accordance with Table E3.3.1-1
- \( m_f \) = Modification factor for type of bearing connection, which shall be determined according to Table E3.3.1-2
- \( d \) = Nominal bolt diameter
- \( t \) = Uncoated sheet thickness
- \( F_u \) = Tensile strength of sheet as defined in Section A2.1 or A2.2

Table E3.3.1-1

<table>
<thead>
<tr>
<th>Thickness of Connected Part, t, in. (mm)</th>
<th>Ratio of Fastener Diameter to Member Thickness, ( d/t )</th>
<th>( C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.024 ( \leq t &lt; 0.1875 ) ((0.61 \leq t &lt; 4.76))</td>
<td>( d/t &lt; 10 )</td>
<td>3.0</td>
</tr>
<tr>
<td>( 10 \leq d/t \leq 22 )</td>
<td>4 - 0.1(( d/t ))</td>
<td></td>
</tr>
<tr>
<td>( d/t &gt; 22 )</td>
<td>1.8</td>
<td></td>
</tr>
</tbody>
</table>

Table E3.3.1-2

<table>
<thead>
<tr>
<th>Type of Bearing Connection</th>
<th>( m_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Shear and Outside Sheets of Double Shear Connection with Washers under Both Bolt Head and Nut</td>
<td>1.00</td>
</tr>
<tr>
<td>Single Shear and Outside Sheets of Double Shear Connection without Washers under Both Bolt Head and Nut, or with only One Washer</td>
<td>0.75</td>
</tr>
<tr>
<td>Inside Sheet of Double Shear Connection with or without Washers</td>
<td>1.33</td>
</tr>
</tbody>
</table>

E3.3.2 Strength [Resistance] with Consideration of Bolt Hole Deformation

When deformation around a bolt hole is a design consideration, the nominal bearing strength [resistance], \( P_{nv} \), shall be calculated in accordance with Eq. E3.3.2-1. The safety factor and resistance factors given in this section shall be used to determine the available
strength \([\text{factored resistance}]\) in accordance with the applicable design method in Section A4, A5, or A6. In addition, the available strength shall not exceed the available strength obtained in accordance with Section E3.3.1.

\[
P_n = (4.64\alpha t + 1.53)dtF_u \quad (Eq. \ E3.3.2-1)
\]

\[
\Omega = 2.22 \quad (ASD)
\]

\[
\phi = 0.65 \quad (LRFD)
\]

\[
= 0.55 \quad (LSD)
\]

where

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

\[
= 1 \quad \text{for US customary units (with } t \text{ in inches)}
\]

\[
= 0.0394 \quad \text{for SI units (with } t \text{ in mm)}
\]

\[
= 0.394 \quad \text{for MKS units (with } t \text{ in cm)}
\]

See Section E3.3.1 for definitions of other variables.

### E3.4 Shear and Tension in Bolts

See Section E3.4 of the Appendix A or B for provisions provided in this section.

### E4 Screw Connections

All E4 requirements shall apply to screws with \(0.08 \text{ in. (2.03 mm) } \leq d \leq 0.25 \text{ in. (6.35 mm)}\). 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 Section C2.

For diaphragm applications, Section D5 shall be used.

Except where otherwise indicated, the following safety factor or resistance factor shall be used to determine the allowable strength or design strength \([\text{factored resistance}]\) in accordance with the applicable design method in Section A4, A5, or A6.

\[
\Omega = 3.00 \quad (ASD)
\]

\[
\phi = 0.50 \quad (LRFD)
\]

\[
= 0.40 \quad (LSD)
\]

Alternatively, design values for a particular application shall be permitted to be based on tests, with the safety factor, \(\Omega\), and the resistance factor, \(\phi\), determined according to Chapter F.

The following notation shall apply to Section E4:

\[
d = \text{Nominal screw diameter}
\]

\[
d_h = \text{Screw head diameter or hex washer head integral washer diameter}
\]

\[
d_w = \text{Steel washer diameter}
\]

\[
d'_w = \text{Effective pull-over resistance diameter}
\]

\[
P_{ns} = \text{Nominal shear strength [resistance] per screw}
\]

\[
P_{ss} = \text{Nominal shear strength [resistance] of screw as reported by manufacturer or determined by independent laboratory testing}
\]

\[
P_{not} = \text{Nominal pull-out strength [resistance] per screw}
\]

\[
P_{nov} = \text{Nominal pull-over strength [resistance] per screw}
\]

\[
P_{ts} = \text{Nominal tension strength [resistance] of screw as reported by manufacturer or
determined by independent laboratory testing

\[ t_1 = \text{Thickness of member in contact with screw head or washer} \]
\[ t_2 = \text{Thickness of member not in contact with screw head or washer} \]
\[ t_c = \text{Lesser of depth of penetration and thickness } t_2 \]
\[ F_{u1} = \text{Tensile strength of member in contact with screw head or washer} \]
\[ F_{u2} = \text{Tensile strength of member not in contact with screw head or washer} \]

E4.1 Minimum Spacing

The distance between the centers of fasteners shall not be less than 3d.

E4.2 Minimum Edge and End Distances

The distance from the center of a fastener to the edge of any part shall not be less than 1.5d. If the end distance is parallel to the force on the fastener, the nominal shear strength [resistance] per screw, \( P_{ns} \), shall be limited by Section E4.3.2.

E4.3 Shear

E4.3.1 Connection Shear Limited by Tilting and Bearing

The nominal shear strength [resistance] per screw, \( P_{ns} \), shall be determined in accordance with this section.
For \( t_2/t_1 \leq 1.0 \), \( P_{ns} \) shall be taken as the smallest of
\[ P_{ns} = 4.2 \left( \frac{t_2}{d} \right)^{1/2} F_{u2} \quad \text{(Eq. E4.3.1-1)} \]
\[ P_{ns} = 2.7 t_1 d F_{u1} \quad \text{(Eq. E4.3.1-2)} \]
\[ P_{ns} = 2.7 t_2 d F_{u2} \quad \text{(Eq. E4.3.1-3)} \]
For \( t_2/t_1 \geq 2.5 \), \( P_{ns} \) shall be taken as the smaller of
\[ P_{ns} = 2.7 t_1 d F_{u1} \quad \text{(Eq. E4.3.1-4)} \]
\[ P_{ns} = 2.7 t_2 d F_{u2} \quad \text{(Eq. E4.3.1-5)} \]
For \( 1.0 < t_2/t_1 < 2.5 \), \( P_{ns} \) shall be calculated by linear interpolation between the above two cases.

E4.3.2 Connection Shear Limited by End Distance

See Section E4.3.2 of the Appendix A or B for provisions of this section.

E4.3.3 Shear in Screws

The nominal shear strength [resistance] of the screw shall be taken as \( P_{ss} \).
In lieu of the value provided in Section E4, the safety factor or the resistance factor shall be permitted to be determined in accordance with Section F1 and shall be taken as 1.25Ω ≤ 3.0 (ASD), \( \phi/1.25 \geq 0.5 \) (LRFD), or \( \phi/1.25 \geq 0.4 \) (LSD).

E4.4 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). Washers shall be at least...
0.050 in. (1.27 mm) thick.

**E4.4.1 Pull-Out**

The nominal pull-out strength \( [\text{resistance}] \), \( P_{\text{not}} \), shall be calculated as follows:

\[
P_{\text{not}} = 0.85 \, t_c \, d \, F_{u2}
\]  

(Eq. E4.4.1-1)

**E4.4.2 Pull-Over**

The nominal pull-over strength \( [\text{resistance}] \), \( P_{\text{nov}} \), shall be calculated as follows:

\[
P_{\text{nov}} = 1.5 t_1 d'_w F_{u1}
\]  

(Eq. E4.4.2-1)

where

\( d'_w \) = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

(a) For a round head, a hex head (Figure E4.4.2(1)), or hex washer head (Figure E4.4.2(2)) screw with an independent and solid steel washer beneath the

![Diagram of screw pull-over configurations](image-url)
screw head
\[ d'_w = d_h + 2t_w + t_1 \leq d_w \]  \hspace{1cm} (Eq. E4.4.2-2)
where
- \( d_h \) = Screw head diameter or hex washer head integral washer diameter
- \( t_w \) = Steel washer thickness
- \( d_w \) = Steel washer diameter

(b) For a round head, a hex head, or hex washer head screw without an independent washer beneath the screw head:

\[ d'_w = d_h \text{ but not larger than } 1/2 \text{ in.} \ (12.7 \text{ mm}) \]

(c) For a domed (non-solid and independent) washer beneath the screw head (Figure E4.4.2(3)), it is permissible to use \( d'_w \) as calculated in Eq. E4.4.2-2, with \( d_h, t_w, \) and \( t_1 \) as defined in Figure E4.4.2(3). In the equation, \( d'_w \) can not exceed 5/8 in. (16 mm). Alternatively, pull-over design values for domed washers, including the safety factor, \( \Omega \), and the resistance factor, \( \phi \), shall be permitted to be determined by test in accordance with Chapter F.

E4.4.3 Tension in Screws

The nominal tension strength [resistance] of the screw shall be taken as \( P_{ts} \).

In lieu of the value provided in Section E4, the safety factor or the resistance factor shall be permitted to be determined in accordance with Section F1 and shall be taken as
\[ 1.25 \Omega \leq 3.0 \ (ASD), \ \phi / 1.25 \geq 0.5 \ (LRFD), \text{ or } \phi / 1.25 \geq 0.4 \ (LSD). \]

E4.5 Combined Shear and Pull-Over

E4.5.1 ASD Method

For screw connections subjected to a combination of shear and tension forces, the following requirement shall be met:

\[ \frac{Q}{P_{ns}} + 0.71 \frac{T}{P_{nov}} \leq \frac{1.10}{\Omega} \]  \hspace{1cm} (Eq. E4.5.1-1)

In addition, \( Q \) and \( T \) shall not exceed the corresponding allowable strength determined by Sections E4.3 and E4.4, respectively.

where
- \( Q \) = Required allowable shear strength of connection
- \( T \) = Required allowable tension strength of connection
- \( P_{ns} \) = Nominal shear strength of connection
  \[ = 2.7t_1d_{u1} \]  \hspace{1cm} (Eq. E4.5.1-2)
- \( P_{nov} \) = Nominal pull-over strength of connection
  \[ = 1.5t_1d_w F_{u1} \]  \hspace{1cm} (Eq. E4.5.1-3)

where
- \( d_w \) = Larger of screw head diameter or washer diameter
- \( \Omega \) = 2.35

Eq. E4.5.1-1 shall be valid for connections that meet the following limits:

1. \( 0.0285 \text{ in.} \ (0.724 \text{ mm}) \leq t_1 \leq 0.0445 \text{ in.} \ (1.130 \text{ mm}), \)
(2) No. 12 and No. 14 self-drilling screws with or without washers,  
(3) $d_w \leq 0.75$ in. (19.1 mm),  
(4) $F_{u1} \leq 70$ ksi (483 MPa or 4920 kg/cm²), and  
(5) $t_2/t_1 \geq 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength shall be taken as 50 percent of $P_{nov}$.

### E4.5.2 LRFD and LSD Methods

For screw connections subjected to a combination of shear and tension forces, the following requirements shall be met:

$$\frac{\bar{Q}}{P_{ns}} + 0.71 \frac{T}{P_{nov}} \leq 1.10 \phi$$  \hspace{1cm} (Eq. E4.5.2-1)

In addition, $\bar{Q}$ and $T$ shall not exceed the corresponding design strength [factored resistance] determined in accordance with Sections E4.3 and E4.4, respectively.

where

- $\bar{Q}$ = Required shear strength [factored shear force] of connection  
  $= V_u$ for LRFD  
  $= V_f$ for LSD
- $T$ = Required tension strength [factored tensile force] of connection  
  $= T_u$ for LRFD  
  $= T_f$ for LSD
- $P_{ns}$ = Nominal shear strength [resistance] of connection  
  $= 2.7t_1dF_{u1}$  \hspace{1cm} (Eq. E4.5.2-2)
- $P_{nov}$ = Nominal pull-over strength [resistance] of connection  
  $= 1.5t_1d_w F_{u1}$  \hspace{1cm} (Eq. E4.5.2-3)

where

- $d_w$ = Larger of screw head diameter or washer diameter
- $\phi$ = 0.65 (LRFD)  
  = 0.55 (LSD)

Eq. E4.5.2-1 shall be valid for connections that meet the following limits:

(1) $0.0285$ in. (0.724 mm) $\leq t_1 \leq 0.0445$ in. (1.13 mm),  
(2) No. 12 and No. 14 self-drilling screws with or without washers,  
(3) $d_w \leq 0.75$ in. (19.1 mm),  
(4) $F_{u1} \leq 70$ ksi (483 MPa or 4920 kg/cm²), and  
(5) $t_2/t_1 \geq 2.5$.

For eccentrically loaded connections that produce a non-uniform pull-over force on the fastener, the nominal pull-over strength [resistance] shall be taken as 50 percent of $P_{nov}$.

### E5 Rupture

See Section E5 of Appendix A or B for the provisions of this section.
E6 Connections to Other Materials

E6.1 Bearing

Provisions shall be made to transfer bearing forces from steel components covered by this Specification to adjacent structural components made of other materials.

E6.2 Tension

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

The nominal tensile strength [resistance] of the fastener and the nominal embedment strength [resistance] of the adjacent structural component shall be determined by applicable product code approvals, product specifications, product literature, or combination thereof.

E6.3 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 [resistance] on the steel components shall not exceed that allowed by this Specification. The available shear strength [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.
F. TESTS FOR SPECIAL CASES

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

The provisions of Chapter F shall not apply to cold-formed steel diaphragms. Refer to Section D5.

F1 Tests for Determining Structural Performance

F1.1 Load and Resistance Factor Design and Limit States Design

Any structural performance that is required to be established by tests shall be evaluated in accordance with the following performance procedure:
(a) Evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent. If such deviation from the average value exceeds 15 percent, more tests of the same kind shall be made until the deviation of any individual test result from the average value obtained from all tests does not exceed ±15 percent or until at least three additional tests have been made. No test result shall be eliminated unless a rationale for its exclusion is given. The average value of all tests made shall then be regarded as the nominal strength \( R_n \) for the series of the tests. \( R_n \) and the coefficient of variation \( V_P \) of the test results shall be determined by statistical analysis.
(b) The strength of the tested elements, assemblies, \( connections \), or members shall satisfy Eq. F1.1-1a or Eq. F1.1-1b as applicable.
\[
\Sigma \gamma_i Q_i \leq \phi R_n \quad \text{for LRFD} \tag{Eq. F1.1-1a}
\]
\[
\phi R_n \geq \Sigma \gamma_i Q_i \quad \text{for LSD} \tag{Eq. F1.1-1b}
\]
where
\[
\Sigma \gamma_i Q_i = \text{Required strength based on the most critical load combination determined in accordance with Section A5.1.2 for LRFD or A6.1.2 for LSD.} \quad \gamma_i \text{ and } Q_i \text{ are load factors and load effects, respectively.}
\]
\[
\phi = \text{Resistance factor}
\]
\[
= C_\phi (M_m F_m P_m) e^{-\beta_0 \sqrt{V_p^2 + V_P^2 + C_p V_P^2}} \tag{Eq. F1.1-2}
\]
where
\[
C_\phi = \text{Calibration coefficient}
\]
\[
= 1.52 \text{ for LRFD}
\]
\[
= 1.6 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced}
\]
\[
= 1.42 \text{ for LSD}
\]
\[
= 1.42 \text{ for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced}
\]
\[
M_m = \text{Mean value of material factor, } M, \text{ listed in Table F1 for type of component involved}
\]
\[
F_m = \text{Mean value of fabrication factor, } F, \text{ listed in Table F1 for type of component involved}
\]
\[
P_m = \text{Mean value of professional factor, } P, \text{ for tested component}
\]
\[ e = 1.0 \]
\[ e = \text{Natural logarithmic base} = 2.718 \]
\[ \beta_0 = \text{Target reliability index} \]
\[ = 2.5 \text{ for structural members and 3.5 for connections for LRFD} \]
\[ = 1.5 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]
\[ = 3.0 \text{ for structural members and 4.0 for connections for LSD} \]
\[ = 3.0 \text{ for LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]
\[ V_M = \text{Coefficient of variation of material factor listed in Table F1 for type of component involved} \]
\[ V_F = \text{Coefficient of variation of fabrication factor listed in Table F1 for type of component involved} \]
\[ C_P = \text{Correction factor} \]
\[ = (1+1/n)m/(m-2) \text{ for } n \geq 4 \]  \hspace{1cm} (Eq. F1.1-3)
\[ = 5.7 \text{ for } n = 3 \]

where
\[ n = \text{Number of tests} \]
\[ m = \text{Degrees of freedom} = n-1 \]
\[ V_P = \text{Coefficient of variation of test results, but not less than 6.5 percent} \]
\[ V_Q = \text{Coefficient of variation of load effect} \]
\[ = 0.21 \text{ for LRFD and LSD} \]
\[ = 0.43 \text{ for LRFD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]
\[ = 0.21 \text{ for the LSD for beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced} \]
\[ R_n = \text{Average value of all test results} \]

The listing in Table F1 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 A2.1, values of \( M_m \) and \( V_M \) shall be determined by the statistical analysis for the materials used. When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1.1-1a or Eq. F1.1-1b, 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. 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 F1
Statistical Data for the Determination of Resistance Factor

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>( M_m )</th>
<th>( V_M )</th>
<th>( F_m )</th>
<th>( V_F )</th>
</tr>
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<tr>
<td>Transverse Stiffeners</td>
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</tr>
<tr>
<td>Shear Stiffeners</td>
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<td>1.00</td>
<td>0.05</td>
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<td>Tension Members</td>
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<td>Flexural Members</td>
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<tr>
<td>Bending Strength</td>
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<td>1.00</td>
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<td>One Flange Through-Fastened to Deck or Sheathing</td>
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<td>Shear Strength</td>
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<td>1.00</td>
<td>0.05</td>
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<tr>
<td>Combined Bending and Shear</td>
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<td>1.00</td>
<td>0.05</td>
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<td>Web Crippling Strength</td>
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<td>Cylindrical Tubular Members</td>
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<tr>
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<td>1.00</td>
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<td>Wall Studs and Wall Stud Assemblies</td>
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<td>Wall Studs in Compression</td>
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Continued
### Table F1 (Continued)
**Statistical Data for the Determination of Resistance Factor**

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>$M_m$</th>
<th>$V_M$</th>
<th>$F_m$</th>
<th>$V_F$</th>
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<tr>
<td><strong>Welded Connections</strong></td>
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<td>Arc Spot Welds</td>
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<tr>
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<td><strong>Bolted Connections</strong></td>
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<td>Shear Strength of Bolt</td>
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<td>Tensile Strength of Bolt</td>
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</tbody>
</table>

Continued
Chapter F, Tests for Special Cases

F1.2 Allowable Strength Design

Where the composition or configuration of elements, assemblies, connections, or details of cold-formed steel structural members are such that calculation of their strength cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with Section F1.1, except as modified in this section for allowable strength design.

The allowable strength shall be calculated as follows:

\[ R = \frac{R_n}{\phi} \]

(Eq. F1.2-1)

where

- \( R_n \) = Average value of all test results
- \( \phi \) = A value evaluated in accordance with Section F1.1

\[ \phi = \frac{\sigma}{f} \]

(Eq. F1.2-2)

where

- \( \sigma \) = Tensile strength of material
- \( f \) = Tensile strength of material

The required strength shall be determined from nominal loads and load combinations as described in Section A4.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies for which the nominal strength [resistance] is computed in accordance with this Specification or its specific references, confirmatory tests shall be permitted to be made to demonstrate the strength is not less than the

}\n
TABLE F1 (Continued)
Statistical Data for the Determination of Resistance Factor

<table>
<thead>
<tr>
<th>Type of Component</th>
<th>( M_m )</th>
<th>( V_M )</th>
<th>( F_m )</th>
<th>( V_F )</th>
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<td>Shear Strength of Screw</td>
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<td>0.08</td>
<td>1.00</td>
<td>0.05</td>
</tr>
<tr>
<td>Pull-Out</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Pull-Over</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Combined Shear and Pull-Over</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.10</td>
</tr>
<tr>
<td>Connections Not Listed Above</td>
<td>1.10</td>
<td>0.10</td>
<td>1.00</td>
<td>0.15</td>
</tr>
</tbody>
</table>
nominal strength [resistance], $R_{n}$, specified in this *Specification* or its specific references for the type of behavior involved.

**F3 Tests for Determining Mechanical Properties**

**F3.1 Full Section**

Tests for determination of mechanical properties of full sections to be used in Section A7.2 shall be conducted in accordance with this section.

(a) Tensile testing procedures shall agree with 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 agrees within 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 shall be 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.

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

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of *virgin steel* to be used in Section A7.2 shall be made in accordance with 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, $F_{yf}$, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield stress to the actual virgin yield stress.
F3.3 Virgin Steel

The following provisions shall apply to steel produced to other than the ASTM Specifications listed in Section A2.1 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 A7.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each master coil for the establishment of the representative values of the virgin tensile yield stress and tensile strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.
G. DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS AND CONNECTIONS FOR CYCLIC LOADING (FATIGUE)

This design procedure shall apply to cold-formed steel 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 (fatigue).

G1 General

When cyclic loading is a design consideration, the provisions of this chapter shall apply to stresses calculated on the basis of unfactored loads. The maximum permitted tensile stress due to unfactored loads shall be 0.6 $F_Y$.

Stress range shall be defined as the magnitude of the change in stress due to the application or removal of the unfactored live load. In the case of a stress reversal, the stress range shall be computed as the sum of the absolute values of maximum repeated tensile and compressive stresses or the sum of the absolute values of maximum shearing stresses of opposite direction at the point of probable crack initiation.

Since the occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design, the evaluation of fatigue resistance shall not be required for wind load applications in buildings. If the live load stress range is less than the threshold stress range, $F_{TH}$, given in Table G1, evaluation of fatigue strength [resistance] shall also not be required.

<table>
<thead>
<tr>
<th>Description</th>
<th>Stress Category</th>
<th>Constant $C_f$</th>
<th>Threshold $F_{TH}$, ksi (MPa) $[kg/cm^2]$</th>
<th>Reference Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>As-received base metal and components with as-rolled surfaces, including sheared edges and cold-formed corners</td>
<td>I</td>
<td>$3.2 \times 10^{10}$</td>
<td>25 $(172)$ $[1760]$</td>
<td>G1-1</td>
</tr>
<tr>
<td>As-received base metal and weld metal in members connected by continuous longitudinal welds</td>
<td>II</td>
<td>$1.0 \times 10^{10}$</td>
<td>15 $(103)$ $[1050]$</td>
<td>G1-2</td>
</tr>
<tr>
<td>Welded attachments to a plate or a beam, transverse fillet welds, and continuous longitudinal fillet welds less than 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>G1-3, G1-4</td>
</tr>
<tr>
<td>Longitudinal fillet welded attachments greater than 2 in. (50.8 mm) parallel to the direction of the applied stress, and intermittent welds parallel to the direction of the applied force</td>
<td>IV</td>
<td>$1.0 \times 10^{9}$</td>
<td>9 $(62)$ $[633]$</td>
<td>G1-4</td>
</tr>
</tbody>
</table>
Evaluation of fatigue strength [resistance] shall not be required if the number of cycles of application of 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.

**G2 Calculation of Maximum Stresses and Stress Ranges**

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if applicable.

In the case of axial stress combined with bending, the maximum stresses of each kind shall be those determined for concurrent arrangements of applied load.

For members having symmetric cross-sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially stressed angle members, where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross-section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

**G3 Design Stress Range**

The range of stress at service loads [specified] shall not exceed the design stress range computed using Equation G3-1 for all stress categories as follows:

$$ F_{SR} = (\alpha C_f/N)^{0.333} \geq F_{TH} \quad (Eq. G3-1) $$

where

- $F_{SR}$ = Design stress range
- $\alpha$ = Coefficient for conversion of units
  - = 1 for US customary units
  - = 327 for SI units
  - = 352,000 for MKS units
- $C_f$ = Constant from Table G1
- $N$ = Number of stress range fluctuations in design life
- = Number of stress range fluctuations per day x 365 x years of design life
- $F_{TH}$ = Threshold fatigue stress range, maximum stress range for indefinite design life from Table G1
G4 Bolts and Threaded Parts

For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads [specified] shall not exceed the design stress range computed using Equation G3-1. The factor $C_f$ shall be taken as $22 \times 10^8$. The threshold stress, $F_{TH}$, shall be taken as $7$ ksi ($48$ MPa or $492$ kg/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 Equation G3-1. The factor $C_f$ shall be taken as $3.9 \times 10^8$. The threshold stress, $F_{TH}$, shall be taken as $7$ ksi ($48$ MPa or $492$ kg/cm$^2$). The net tensile area shall be calculated by Eq. G4-1a or G4-1b as applicable.

\[
A_t = \left( \frac{\pi}{4} \right) \left[ db - (0.9743/n) \right]^2 \quad \text{for US Customary units (Eq. G4-1a)}
\]
\[
A_t = \left( \frac{\pi}{4} \right) \left[ db - (0.9382p) \right]^2 \quad \text{for SI or MKS units (Eq. G4-1b)}
\]

where:
- $A_t$ = Net tensile area
- $db$ = 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)

G5 Special Fabrication Requirements

Backing bars in welded connections that are parallel to the stress field shall be 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
Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

2007 EDITION
PREFACE

This Appendix provides alternative design procedures to portions of the North American Specification for the Design of Cold-Formed Steel Structural Members, Chapters A through G, and Appendices A and B (herein referred to as the main Specification). The Direct Strength Method detailed in this Appendix requires determination of the elastic buckling behavior of the member, and then provides a series of nominal strength [resistance] curves for predicting the member strength based on the elastic buckling behavior. The procedure does not require effective width calculations or iteration; instead, it uses gross properties and the elastic buckling behavior of the cross-section to predict the strength. The applicability of these provisions is detailed in the General Provisions of this Appendix.
APPENDIX 1: Design of Cold-Formed Steel Structural Members Using the Direct Strength Method

1.1 General Provisions

1.1.1 Applicability

The provisions of this Appendix shall be permitted to be used to determine the nominal axial (Pn) and flexural (Mn) strengths [resistances] of cold-formed steel members. Sections 1.2.1 and 1.2.2 present a method applicable to all cold-formed steel columns and beams. Those members meeting the geometric and material limitations of Section 1.1.1.1 for columns and Section 1.1.1.2 for beams have been pre-qualified for use, and the calibrated safety factor, Ω, and resistance factor, φ, given in 1.2.1 and 1.2.2 shall be permitted to apply. The use of the provisions of Sections 1.2.1 and 1.2.2 for other columns and beams shall be permitted, but the standard Ω and φ factors for rational engineering analysis (Section A1.1(b) of the main Specification) shall apply. The main Specification refers to Chapters A through G, Appendices A and B, and Appendix 2 of the North American Specification for the Design of Cold-Formed Steel Structural Members.

Currently, the Direct Strength Method provides no explicit provisions for members in tension, shear, combined bending and shear, web crippling, combined bending and web crippling, or combined axial load and bending (beam-columns). Further, no provisions are given for structural assemblies or connections and joints. As detailed in main Specification, Section A1.1, the provisions of the main Specification, when applicable, shall be used for all cases listed above.

It shall be permitted to substitute the nominal strengths [resistances], resistance factors, and safety factors from this Appendix for the corresponding values in Sections C3.1, C4.1.1, C4.1.2, C4.1.3, C4.1.4, D6.1.1, and D6.1.2 of the main Specification.

For members or situations to which the main Specification is not applicable, the Direct Strength Method of this Appendix shall be permitted to be used, as applicable. The usage of the Direct Strength Method shall be subjected to the same provisions as any other rational engineering analysis procedure, as detailed in Section A1.1(b) of the main Specification:
(1) applicable provisions of the main Specification shall be followed when they exist, and
(2) increased safety factors, Ω, and reduced resistance factors, φ, shall be employed for strength when rational engineering analysis is conducted.

1.1.1.1 Pre-qualified Columns

Unperforated columns that fall within the geometric and material limitations given in Table 1.1.1-1 shall be permitted to be designed using the safety factor, Ω, and resistance factor, φ, defined in Section 1.2.1.
### Table 1.1.1-1

**Limits for Pre-qualified Columns**

<table>
<thead>
<tr>
<th>Lipped C-Sections</th>
<th>Simple Lips:</th>
<th>For all C-sections:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$h_0/t &lt; 472$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b_o/t &lt; 159$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$4 &lt; D/t &lt; 33$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.7 &lt; h_o/b_o &lt; 5.0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.05 &lt; D/b_o &lt; 0.41$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\theta = 90^\circ$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E/F_y &gt; 340$ [F_y &lt; 86 ksi (593 MPa or 6050 kg/cm²)]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Complex Lips:</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h_0/t &lt; 472$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b_o/t &lt; 159$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$4 &lt; D/t &lt; 33$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.7 &lt; h_o/b_o &lt; 5.0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.05 &lt; D/b_o &lt; 0.41$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\theta = 90^\circ$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E/F_y &gt; 340$ [F_y &lt; 86 ksi (593 MPa or 6050 kg/cm²)]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Lipped C-Section with Web Stiffener(s)</td>
<td>For one or two intermediate stiffeners:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$h_0/t &lt; 489$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b_o/t &lt; 160$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$6 &lt; D/t &lt; 33$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1.3 &lt; h_o/b_o &lt; 2.7$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.05 &lt; D/b_o &lt; 0.41$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E/F_y &gt; 340$ [F_y &lt; 86 ksi (593 MPa or 6050 kg/cm²)]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Z-Section</td>
<td>$h_0/t &lt; 137$</td>
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<tr>
<td></td>
<td></td>
<td>$b_o/t &lt; 56$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0 &lt; D/t &lt; 36$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1.5 &lt; h_0/b_o &lt; 2.7$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.00 &lt; D/b_o &lt; 0.73$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\theta = 50^\circ$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E/F_y &gt; 590$ [F_y &lt; 50 ksi (345 MPa or 3520 kg/cm²)]</td>
</tr>
<tr>
<td></td>
<td>Rack Upright</td>
<td>See C-Section with Complex Lips</td>
</tr>
<tr>
<td></td>
<td>Hat</td>
<td>$h_0/t &lt; 50$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b_o/t &lt; 20$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$4 &lt; D/t &lt; 6$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1.0 &lt; h_0/b_o &lt; 1.2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$D/b_o = 0.13$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E/F_y &gt; 428$ [F_y &lt; 69 ksi (476 MPa or 4850 kg/cm²)]</td>
</tr>
</tbody>
</table>

**Note:** * $r/t < 10$, where $r$ is the centerline bend radius
  
  $b_o$ = overall width; $D =$overall lip depth; $t =$ base metal thickness; $h_o =$ overall depth
1.1.1.2 Pre-qualified Beams

Unperforated beams that fall within the geometric and material limitations given in Table 1.1.1-2 shall be permitted to be designed using the safety factor, $\Omega$, and resistance factor, $\phi$, defined in Section 1.2.2.

<table>
<thead>
<tr>
<th>Table 1.1.1-2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Limitations for Pre-qualified Beams</strong></td>
</tr>
</tbody>
</table>

**C-Sections**

**Simple Lips:**

- $h_0/t < 321$
- $b_0/t < 75$
- $0 < D/t < 34$
- $1.5 < h_0/b_0 < 17.0$
- $0 < D/b_0 < 0.70$
- $44^\circ < \theta < 90^\circ$
- $E/F_y > 421$ [Fy < 70 ksi (483 MPa or 4920 kg/cm²)]

For C-sections with complex lips:
- $D_2/t < 34$
- $D_2/D < 2$
- $D_3/t < 34$
- $D_3/D_2 < 1$

Note:
- a) $\theta_2$ is permitted to vary (D2 lip is permitted to angle inward or outward)
- b) $\theta_3$ is permitted to vary (D3 lip is permitted to angle up or down).

**Complex Lips:**

**Lipped C-Sections with Web Stiffener**

- $h_0/t < 358$
- $b_0/t < 58$
- $14 < D/t < 17$
- $5.5 < h_0/b_0 < 11.7$
- $0.27 < D/b_0 < 0.56$
- $\theta = 90^\circ$
- $E/F_y > 578$ [Fy < 51 ksi (352 MPa or 3590 kg/cm²)]

**Z-Sections**

**Simple Lips:**

- $h_0/t < 183$
- $b_0/t < 71$
- $10 < D/t < 16$
- $2.5 < h_0/b_0 < 4.1$
- $0.15 < D/b_0 < 0.34$
- $36^\circ < \theta < 90^\circ$
- $E/F_y > 440$ [Fy < 67 ksi (462 MPa or 4710 kg/cm²)]

For Z-sections with complex lips:
- $D_2/t < 34$
- $D_2/D < 2$
- $D_3/t < 34$
- $D_3/D_2 < 1$

Note:
- a) $\theta_2$ is permitted to vary (D2 lip is permitted to angle inward, outward, etc.)
- b) $\theta_3$ is permitted to vary (D3 lip is permitted to angle up, down, etc.)

(Continued)
Table 1.1.1-2
Limitations for Pre-qualified Beams (Continued)

<table>
<thead>
<tr>
<th>Hats (Decks) with Stiffened Flange in Compression</th>
<th>Trapezoids (Decks) with Stiffened Flange in Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Diagram of Hats (Decks) with Stiffened Flange in Compression" /></td>
<td><img src="image2" alt="Diagram of Trapezoids (Decks) with Stiffened Flange in Compression" /></td>
</tr>
<tr>
<td>h₀/t &lt; 97</td>
<td>h₀/t &lt; 203</td>
</tr>
<tr>
<td>b₀/t &lt; 467</td>
<td>b₀/t &lt; 231</td>
</tr>
<tr>
<td>0 &lt; dₛ/t &lt; 26 (dₛ = Depth of stiffener)</td>
<td>0.42 &lt; (h₀/sinθ)/b₀ &lt; 1.91</td>
</tr>
<tr>
<td>0.14 &lt; h₀/b₀ &lt; 0.87</td>
<td>1.10 &lt; b₀/b₁ &lt; 3.38</td>
</tr>
<tr>
<td>0.88 &lt; b₀/b₁ &lt; 5.4</td>
<td>0 &lt; nₑ ≤ 2 (nₑ = Number of compression flange stiffeners)</td>
</tr>
<tr>
<td>0 &lt; n ≤ 4 (n = Number of compression flange stiffeners)</td>
<td>0 &lt; nₑ ≤ 2 (nₑ = Number of web stiffeners and/or folds)</td>
</tr>
<tr>
<td>E/Fᵧ &gt; 492 [Fᵧ &lt; 60 ksi (414 MPa or 4220 kg/cm²)]</td>
<td>0 &lt; nₑ ≤ 2 (nₑ = Number of tension flange stiffeners)</td>
</tr>
<tr>
<td></td>
<td>52° &lt; θ &lt; 84° (θ = Angle between web and horizontal plane)</td>
</tr>
<tr>
<td></td>
<td>E/Fᵧ &gt; 310 [Fᵧ &lt; 95 ksi (655 MPa or 6680 kg/cm²)]</td>
</tr>
</tbody>
</table>

Note:
* r/t < 10, where r is the centerline bend radius.

See Section 1.1.1.1 for definitions of other variables given in Table 1.1.1-2.

1.1.2 Elastic Buckling

Analysis shall be used for the determination of the elastic buckling loads and/or moments used in this Appendix. For columns, this includes the local, distortional, and overall buckling loads (P_cr₀, P_cr₉, and P_crₑ of Section 1.2.1). For beams, this includes the local, distortional, and overall buckling moments (M_cr₀, M_cr₉, and M_crₑ of Section 1.2.2). In some cases, for a given column or beam, all three modes do not exist. In such cases, the non-existent mode shall be ignored in the calculations of Sections 1.2.1 and 1.2.2. The commentary to this Appendix provides guidance on appropriate analysis procedures for elastic buckling determination.

1.1.3 Serviceability Determination

The bending deflection at any moment, M, due to nominal loads shall be permitted to be determined by reducing the gross moment of inertia, I₉, to an effective moment of inertia for deflection, as given in Eq. 1.1.3-1:

\[ I_{eff} = I_9(M_d/M) \leq I_9 \]  

(Eq. 1.1.3-1)

where

- \( M_9 \) = Nominal flexural strength [resistance], \( M_{ny} \) defined in Section 1.2.2, but with \( M_y \) replaced by \( M \) in all equations of Section 1.2.2
- \( M \) = Moment due to nominal loads [specified loads] on member to be considered (\( M \leq M_y \))

1.2 Members

1.2.1 Column Design

The nominal axial strength [resistance], \( P_n \), shall be the minimum of \( P_{nw}, P_{nt}, \) and \( P_{nd} \) as given in Sections 1.2.1.1 to 1.2.1.3. For columns meeting the geometric and material criteria of...
Section 1.1.1.1, \( \Omega_c \) and \( \phi_c \) shall be as follows:
\[
\Omega_c = 1.80 \quad (ASD) \\
\phi_c = 0.85 \quad (LRFD) \\
= 0.80 \quad (LSD)
\]
For all other columns, \( \Omega \) and \( \phi \) of the main Specification, Section A1.1(b), shall apply. The available strength [factored resistance] shall be determined in accordance with applicable method in Section A4, A5, or A6 of the main Specification.

1.2.1.1 Flexural, Torsional, or Flexural-Torsional Buckling

The nominal axial strength [resistance], \( P_{ne} \), for flexural, torsional, or flexural-torsional buckling shall be calculated in accordance with the following:
(a) For \( \lambda_c \leq 1.5 \)
\[
P_{ne} = \left( 0.658 \frac{\lambda_c^2}{\Omega_c} \right) P_y
\]
(Eq. 1.2.1-1)
(b) For \( \lambda_c > 1.5 \)
\[
P_{ne} = \left( \frac{0.877}{\lambda_c^2} \right) P_y
\]
(Eq. 1.2.1-2)

where
\[
\lambda_c = \sqrt{\frac{P_y}{P_{cre}}}
\]
(Eq. 1.2.1-3)

where
\[
P_y = A_g F_y
\]
(Eq. 1.2.1-4)
\[
P_{cre} = \text{Minimum of the critical elastic column buckling load in flexural, torsional, or flexural-torsional buckling determined by analysis in accordance with Section 1.1.2}
\]

1.2.1.2 Local Buckling

The nominal axial strength [resistance], \( P_{nl} \), for local buckling shall be calculated in accordance with the following:
(a) For \( \lambda_{\ell} \leq 0.776 \)
\[
P_{nl} = P_{ne}
\]
(Eq. 1.2.1-5)
(b) For \( \lambda_{\ell} > 0.776 \)
\[
P_{nl} = \left[ \frac{P_{cre}^\ell}{P_{ne}} \right]^{0.4} P_{ne}^{0.4}
\]
(Eq. 1.2.1-6)

where
\[
\lambda_{\ell} = \sqrt{\frac{P_{ne}}{P_{cre}^\ell}}
\]
(Eq. 1.2.1-7)
\[
P_{ne} = \text{A value as defined in Section 1.2.1.1}
\]
\[
P_{cre}^\ell = \text{Critical elastic local column buckling load determined by analysis in accordance with Section 1.1.2}
\]
1.2.1.3 Distortional Buckling

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

(a) For \( \lambda_d \leq 0.561 \)
\[
P_{nd} = P_y \quad (Eq. 1.2.1-8)
\]

(b) For \( \lambda_d > 0.561 \)
\[
P_{nd} = \left(1 - 0.25 \left( \frac{P_{crd}}{P_y} \right)^{0.6} \right)^{0.6} P_y \quad (Eq. 1.2.1-9)
\]

where
\[
\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \quad (Eq. 1.2.1-10)
\]

where
\[
P_y \text{ = A value as given in Eq. 1.2.1-4}
\]
\[
P_{crd} \text{ = Critical elastic distortional column buckling load determined by analysis in accordance with Section 1.1.2}
\]

1.2.2 Beam Design

The nominal flexural strength [resistance], \( M_{ne} \), shall be the minimum of \( M_{ne} \), \( M_{n/t} \), and \( M_{nd} \) as given in Sections 1.2.2.1 to 1.2.2.3. For beams meeting the geometric and material criteria of Section 1.1.1.2, \( \Omega_b \) and \( \phi_b \) shall be as follows:

\[
\Omega_b = 1.67 \quad (ASD)
\]
\[
\phi_b = 0.90 \quad (LRFD)
\]
\[
= 0.85 \quad (LSD)
\]

For all other beams, \( \Omega \) and \( \phi \) of the main Specification, Section A1.1(b), shall apply. The available strength [factored resistance] shall be determined in accordance with applicable method in Section A4, A5, or A6 of the main Specification.

1.2.2.1 Lateral-Torsional Buckling

The nominal flexural strength [resistance], \( M_{ne} \), for lateral-torsional buckling shall be calculated in accordance with the following:

(a) For \( M_{cre} < 0.56M_y \)
\[
M_{ne} = M_{cre} \quad (Eq. 1.2.2-1)
\]

(b) For \( 2.78M_y \geq M_{cre} \geq 0.56M_y \)
\[
M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_y}{36M_{cre}} \right) \quad (Eq. 1.2.2-2)
\]

(c) For \( M_{cre} > 2.78M_y \)
\[
M_{ne} = M_y \quad (Eq. 1.2.2-3)
\]

where
\[
M_{cre} \text{ = Critical elastic lateral-torsional buckling moment determined by analysis in accordance with Section 1.1.2}
\]
\[
M_y = S_tF_y \quad (Eq. 1.2.2-4)
\]

[72x748]Appendix 1, Design of Cold-Formed Steel Structural Members Using the Direct Strength Method
1-8 July 2007
where

\[ S_f = \text{Gross section modulus referenced to the extreme fiber in first yield} \]

### 1.2.2.2 Local Buckling

The nominal flexural strength [resistance], \( M_{nl} \), for local buckling shall be calculated in accordance with the following:

(a) For \( \lambda_l \leq 0.776 \)

\[ M_{nl} = M_{ne} \quad (Eq. 1.2.2-5) \]

(b) For \( \lambda_l > 0.776 \)

\[ M_{nl} = \left( 1 - 0.15 \left( \frac{M_{cr}\ell}{M_{ne}} \right)^{0.4} \right) \left( \frac{M_{cr}\ell}{M_{ne}} \right)^{0.4} M_{ne} \quad (Eq. 1.2.2-6) \]

where

\[ \lambda_l = \sqrt{\frac{M_{ne}}{M_{cr}\ell}} \quad (Eq. 1.2.2-7) \]

\[ M_{ne} = \text{A value as defined in Section 1.2.2.1} \]

\[ M_{cr}\ell = \text{Critical elastic local buckling moment determined by analysis in accordance with Section 1.1.2} \]

### 1.2.2.3 Distortional Buckling

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

(a) For \( \lambda_d \leq 0.673 \)

\[ M_{nd} = M_y \quad (Eq. 1.2.2-8) \]

(b) For \( \lambda_d > 0.673 \)

\[ M_{nd} = \left( 1 - 0.22 \left( \frac{M_{crd}}{M_y} \right)^{0.5} \right) \left( \frac{M_{crd}}{M_y} \right)^{0.5} M_y \quad (Eq. 1.2.2-9) \]

where

\[ \lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \quad (Eq. 1.2.2-10) \]

\[ M_y = \text{A value as given in Eq. 1.2.2-4} \]

\[ M_{crd} = \text{Critical elastic distortional buckling moment determined by analysis in accordance with Section 1.1.2} \]
Appendix 2

Second-Order Analysis

2007 EDITION
APPENDIX 2: Second-Order Analysis

This Appendix addresses *second-order analysis* for structural systems comprised of *moment frames*, *braced frames*, *shear walls*, or combinations thereof.

### 2.1 General Requirements

Members shall satisfy the provisions of Section C5 with the nominal column strengths [nominal axial resistance], $P_{n}$, determined using $K_x$ and $K_y = 1.0$, as well as $\alpha_x = 1.0$, $\alpha_y = 1.0$, $C_{mx} = 1.0$, and $C_{my} = 1.0$. The required strengths [factored forces and moments] for members, connections, and other structural elements shall be determined using a second-order analysis as specified in this Appendix. All component and connection deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

### 2.2 Design and Analysis Constraints

#### 2.2.1 General

The second-order analysis shall consider both the effect of loads acting on the deflected shape of a member between joints or nodes (P-δ effects) and the effect of loads acting on the displaced location of joints or nodes in a structure (P-Δ effects). It shall be permitted to perform the analysis using any general second-order analysis method. Analyses shall be conducted according to the design and loading requirements specified in Chapter A. For the ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations and the results shall be divided by 1.6 to obtain the required strengths at allowable load levels.

#### 2.2.2 Types of Analysis

It shall be permissible to carry out the second-order analysis 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 Section 2.2.4.

For second-order elastic analysis, axial and flexural stiffnesses shall be reduced as specified in Section 2.2.3.

#### 2.2.3 Reduced Axial and Flexural Stiffnesses

Flexural and axial stiffnesses shall be reduced by using $E^*$ in place of $E$ as follows for all members whose flexural and axial stiffnesses are considered to contribute to the lateral stability of the structure:

$$E^* = 0.8 \tau_b E$$  \hspace{1cm} (Eq. 2-1)

where

- $\tau_b = 1.0$ for $\alpha P_r / P_y \leq 0.5$
- $\tau_b = 4[\alpha P_r / P_y (1 - \alpha P_r / P_y)]$ for $\alpha P_r / P_y > 0.5$

- $P_r$ = Required axial compressive strength [factored axial compressive force], kips (N)
- $P_y$ = Member yield strength [resistance] (=AF_y, where A is the full unreduced cross-sectional area), kips (N)
- $\alpha = 1.0$ (LRFD and LSD)
= 1.6 (ASD)

In cases where flexibility of other structural components such as connections, flexible column base details, or horizontal trusses acting as diaphragms is modeled explicitly in the analysis, the stiffnesses of the other structural components shall be reduced by a factor of 0.8.

If notional loads are used, in lieu of using $\tau_b < 1.0$ where $\alpha P_t/P_y > 0.5$, $\tau_b = 1.0$ shall be permitted to be used for all members, provided that an additional notional load of $0.001Y_i$ is added to the notional load required in Section 2.2.4.

### 2.2.4 Notional loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections. Notional loads are lateral loads that are applied at each framing level and specified in terms of the gravity loads applied at that level. The gravity load used to determine the notional load shall be equal to or greater than the gravity load associated with the load combination being evaluated. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

A notional load, $N_i = (1/240) Y_i$, shall be applied independently in two orthogonal directions as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any.

- $N_i$ = Notional lateral load applied at level i, kips (N)
- $Y_i$ = Gravity load from the LRFD or LSD load combination or 1.6 times the ASD load combination applied at level i, kips (N)

The notional load coefficient of 1/240 is based on an assumed initial story out-of-plumbness ratio of 1/240. Where a different assumed out-of-plumbness is justified, the notional load coefficient shall be permitted to be adjusted proportionally to a value not less than 1/500.
Appendix A:
Provisions Applicable to
the United States and Mexico

2007 EDITION
PREFACE TO APPENDIX A

Appendix A provides specification provisions that apply to the United States and Mexico. Included are provisions of a broad nature relating to the design method used, ASD or LRFD, and use of ASCE/SEI 7 for loads and load combinations where there is not an applicable building code. Reference documents that are used by both countries are listed here as well.

Also included in Appendix A are technical items where full agreement between countries was not reached. Such items included certain provisions pertaining to the design of

- Beams and compression members (C and Z sections) for standing seam roofs,
- Bolted connections, and
- Tension members

Efforts are being made to minimize these differences in future editions of the Specification.
APPENDIX A: PROVISIONS APPLICABLE TO THE UNITED STATES AND MEXICO

This Appendix provides design provisions or supplements to Chapters A through G that specifically applies to the United States and Mexico. This appendix is considered mandatory for applications in the United States and Mexico.

A section number ending with a letter indicates that the provisions herein supplement the corresponding section in Chapters A through G of the Specification. A section number not ending with a letter indicates that the section gives the entire design provision.

A1.1a Scope

Designs shall be made in accordance with the provisions for Load and Resistance Factor Design, or with the provisions for Allowable Strength Design.

A2.2 Other Steels

The listing in Section A2.1 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:
(1) The steel shall conform to the chemical and mechanical requirements of one of the listed specifications or other published specification.
(2) The chemical and mechanical properties shall be determined by the producer, the supplier, or the purchaser, in accordance with the following specifications. For coated sheets, ASTM A924/A924M; for hot-rolled or cold-rolled sheet and strip, ASTM A568/A568M; for plate and bar, ASTM A6/A6M; for hollow structural sections, such tests shall be made in accordance with the requirements of A500 (for carbon steel) or A847 (for HSLA steel).
(3) The coating properties of coated sheet shall be determined by the producer, the supplier, or the purchaser, in accordance with ASTM A924/A924M.
(4) The steel shall meet the requirements of Section A2.3.
(5) 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 or D1.3 as applicable.

If the identification and documentation of the production of the steel have not been established, then in addition to requirements (1) through (5), 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.

A2.3.1a Ductility

In seismic design category D, E or F (as defined by ASCE/SEI 7), when material ductility is determined on the basis of the local and uniform elongation criteria of Section A2.3.1, curtain wall studs shall be limited to the dead load of the curtain wall assembly divided by its surface area, but no greater than 15 psf (0.72 kN/m² or 7.32 g/cm²).
A3 Loads

A3.1 Nominal Loads

The nominal loads shall be as stipulated by the applicable building code under which the structure is designed or as dictated by the conditions involved. In the absence of a building code, the nominal loads shall be those stipulated in the ASCE/SEI 7.

A4.1.2 Load Combinations for ASD

The structure and its components shall be designed so that allowable strengths equal or exceed the effects of the nominal loads and load combinations as stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the ASCE/SEI 7.

A5.1.2 Load Factors and Load Combinations for LRFD

The structure and its components shall be designed so that design strengths equal or exceed the effects of the factored loads and load combinations stipulated by the applicable building code under which the structure is designed or, in the absence of an applicable building code, as stipulated in the ASCE/SEI 7.

A9a Referenced Documents

The following documents are referenced in Appendix A:

1. American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 700, Chicago, Illinois 60601-1802:
   ANSI/AISC 360-05, Specification for Structural Steel Buildings

2. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:
   AISI S213-07, North American Standard for Cold-Formed Steel Framing – Lateral Design
   AISI S908-04, Base Test Method for Purlins Supporting a Standing Seam Roof System

3. American Society of Civil Engineers (ASCE), 1801 Alexander Bell Drive, Reston VA, 20191:
   ASCE/SEI 7-05, Minimum Design Loads in Buildings and Other Structures

4. American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135:
   AWS D1.3-98, Structural Welding Code - Sheet Steel
   AWS C1.1/C1.1M-2000, Recommended Practices for Resistance Welding

C2 Tension Members

For axially loaded tension members, the nominal tensile strength, $T_{nv}$, shall be the smallest value obtained in accordance with the limit states of (a), (b) and (c). Unless otherwise specified, the corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.

(a) For yielding in gross section

$$ T_n = A_g F_y $$

$$ \Omega_t = 1.67 \quad (ASD) $$
$\phi_t = 0.90 \quad (LRFD)$
where
$T_n = \text{Nominal strength of member when loaded in tension}$
$A_g = \text{Gross area of cross section}$
$F_y = \text{Design yield stress as determined in accordance with Section A7.1}$

(b) For rupture in net section away from connection

$T_n = A_n F_u \quad (Eq. C2-2)$

$\phi_t = 2.00 \quad (ASD)$
$\phi_t = 0.75 \quad (LRFD)$

where
$A_n = \text{Net area of cross section}$
$F_u = \text{Tensile strength as specified in either Section A2.1 or A2.3.2}$

(c) For rupture in net section at connection

The available tensile strength shall also be limited by Sections E2.7, E3, and E5 for tension members using welded connections, bolted connections, and screw connections.

**D4a Light-Frame Steel Construction**

In addition to the cold-formed steel framing standards listed in Section D4, the following standard shall be followed, as applicable:

(e) Light-framed shear walls, diagonal strap bracing (that is part of a structural wall) and diaphragms to resist wind, seismic and other in-plane lateral loads shall be designed in accordance with AISI S213.

**D6.1.2 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 C3.1.2.1, or shall be calculated in accordance with this section. The safety factor and the resistance factor provided in this section shall be applied to the nominal strength, $M_n$, calculated by Eq. D6.1.2-1 to determine the available strengths in accordance with the applicable method in Section A4 or A5.

$M_n = R_s F_y \quad (Eq. D6.1.2-1)$

$\Omega_b = 1.67 \quad (ASD)$
$\phi_b = 0.90 \quad (LRFD)$

where
$R = \text{Reduction factor determined in accordance with AISI S908}$
See Section C3.1.1 for definitions of $S_e$ and $F_y$.

**D6.1.4 Compression of Z-Section 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 shall be permitted to be based on 
discrete point bracing locations, or on tests in accordance with Chapter F.

The nominal axial strength of simple span or continuous Z-sections shall be calculated 
in accordance with (a) and (b). 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 method in Section A4 or A5.

(a) For weak axis available strength

\[ P_n = k_{af} R F_y A \]  
\[ \Omega = 1.80 \quad (ASD) \]
\[ \phi = 0.85 \quad (LRFD) \]

where

For \( d/t \leq 90 \)

\[ k_{af} = 0.36 \]

For \( 90 < d/t \leq 130 \)

\[ k_{af} = 0.72 - \frac{d}{250t} \]  
(\text{Eq. D6.1.4-2})

For \( d/t > 130 \)

\[ k_{af} = 0.20 \]

\( R \) = 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

See Section C3.1.1 for definition of \( F_y \).

Eq. D6.1.4-1 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) \( 70 \leq d/t \leq 170 \)
(5) \( 2.8 \leq d/b < 5 \), where \( b \) = Z section flange width.
(6) \( 16 \leq \frac{\text{flange flat width}}{t} < 50 \)
(7) Both flanges are prevented from moving laterally at the supports
(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 
Section C4.1 and C4.1.1.

D6.2.1a - Strength [Resistance] of Standing Seam Roof Panel Systems

In addition to the provisions provided in Section D6.2.1, for load combinations that 
include wind uplift, the nominal wind load shall be permitted to be multiplied by 0.67 
provided the tested system and wind load evaluation satisfies 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, Method 
1 (Simplified Procedure) or Method 2 (Analytical Procedure).
(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).

(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:
   (i) The standing seam roof clip mechanically fails by separating from the panel sidelap.
   (ii) The standing seam roof clip mechanically fails by the sliding tab separating from the stationary base.

E2a Welded Connections

Welded connections in which the thickness of the thinnest connected part is greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.

Except as modified herein, arc welds on steel where at least one of the connected parts is 3/16 in. (4.76 mm) or less in thickness shall be made in accordance with AWS D1.3. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as listed in Table E2a.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1 or AWS C1.3.

<table>
<thead>
<tr>
<th>TABLE E2a</th>
<th>Welding Positions Covered</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Square Groove Butt Weld</td>
</tr>
<tr>
<td>Sheet to Sheet</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>–</td>
</tr>
<tr>
<td>H</td>
<td>–</td>
</tr>
<tr>
<td>V</td>
<td>–</td>
</tr>
<tr>
<td>OH</td>
<td>–</td>
</tr>
<tr>
<td>Sheet to Supporting Member</td>
<td></td>
</tr>
<tr>
<td>–</td>
<td>F</td>
</tr>
<tr>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

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

E3a Bolted Connections

In addition to the design criteria given in Section E3 of the Specification, the following design requirements shall also be followed for bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm). Bolted connections in which the thickness of the thinnest connected part is equal to or
greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.

The holes for bolts shall not exceed the sizes specified in Table E3a, except that larger holes are permitted to be used in column base details or structural systems connected to concrete walls.

Standard holes shall be used in bolted connections, except that oversized and slotted holes shall be permitted to be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or slotted holes in an outer ply unless suitable performance is demonstrated by tests in accordance with Chapter F. In the situation where the hole occurs within the lap of lapped and nested zee members, the above requirements regarding the direction of the slot and the use of washers shall be permitted not to apply, subject to the following limits:

1) 1/2 in. (12.7 mm) diameter bolts only,
2) Maximum slot size is 9/16 in. x 7/8 in. (14.3 mm x 22.2 mm) slotted vertically,
3) Maximum oversize hole is 5/8 in. (15.9 mm) diameter,
4) Minimum member thickness is 0.060 in. (1.52 mm) nominal,
5) Maximum member yield stress is 60 ksi (410 MPa, and 4220 kg/cm²),
6) Minimum lap length measured from center of frame to end of lap is 1.5 times the member depth.

### TABLE E3a
Maximum Size of Bolt Holes, inches

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1/2</td>
<td>d + 1/32</td>
<td>d + 1/16</td>
<td>(d + 1/32) by (d + 1/4)</td>
<td>(d + 1/32) by (21/2 d)</td>
</tr>
<tr>
<td>≥ 1/2</td>
<td>d + 1/16</td>
<td>d + 1/8</td>
<td>(d + 1/16) by (d + 1/4)</td>
<td>(d + 1/16) by (21/2 d)</td>
</tr>
</tbody>
</table>

### TABLE E3a
Maximum Size of Bolt Holes, millimeters

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d mm</th>
<th>Standard Hole Diameter, dh mm</th>
<th>Oversized Hole Diameter, dh mm</th>
<th>Short-Slotted Hole Dimensions mm</th>
<th>Long-Slotted Hole Dimensions mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 12.7</td>
<td>d + 0.8</td>
<td>d + 1.6</td>
<td>(d + 0.8) by (d + 6.4)</td>
<td>(d + 0.8) by (21/2 d)</td>
</tr>
<tr>
<td>≥ 12.7</td>
<td>d + 1.6</td>
<td>d + 3.2</td>
<td>(d + 1.6) by (d + 6.4)</td>
<td>(d + 1.6) by (21/2 d)</td>
</tr>
</tbody>
</table>

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

The nominal shear strength, \( P_{nv} \), of the connected part as affected by spacing and edge distance in the direction of applied force shall be calculated in accordance with Eq. E3.1-1. The corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.
P_n = teF_u \hspace{1cm} \text{(Eq. E3.1-1)}

(a) When \( F_u / F_{sy} \geq 1.08 \)
\[ \Omega = 2.00 \ \text{(ASD)} \]
\[ \phi = 0.70 \ \text{(LRFD)} \]

(b) When \( F_u / F_{sy} < 1.08 \)
\[ \Omega = 2.22 \ \text{(ASD)} \]
\[ \phi = 0.60 \ \text{(LRFD)} \]

where

\( P_n \) = Nominal strength per bolt
\( e \) = Distance measured in line of force from center of a standard hole to nearest edge of adjacent hole or to end of connected part
\( t \) = Thickness of thinnest connected part
\( F_u \) = Tensile strength of connected part as specified in Section A2.1, A2.2 or A2.3.2
\( F_{sy} \) = Yield stress of connected part as specified in Section A2.1, A2.2 or A2.3.2

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, \( d \). Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than \( 11/2 \ d \).

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of \( e-(d_h/2) \), in which \( e \) is the required distance used in Eq. E3.1-1, and \( d_h \) is the diameter of a standard hole defined in Table E3a. In no case shall the clear distance between edges of two adjacent holes be less than 2\( d \) and the distance between the edge of the hole and the end of the member be less than \( d \).

### E3.2 Rupture in Net Section (Shear Lag)

The nominal tensile strength of a bolted member shall be determined in accordance with Section C2. For rupture in the effective net section of the connected part, the nominal tensile strength [resistance], \( P_n \), shall be determined in accordance with this section. Unless otherwise specified, the corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.

(a) For flat sheet connections not having staggered hole patterns
\[ P_n = A_n F_t \hspace{1cm} \text{(Eq. E3.2-1)} \]

(1) When washers are provided under both the bolt head and the nut
For a single bolt, or a single row of bolts perpendicular to the force
\[ F_t = (0.1 + 3d/s) F_u \leq F_u \hspace{1cm} \text{(Eq. E3.2-2)} \]

For multiple bolts in the line parallel to the force
\[ F_t = F_u \hspace{1cm} \text{(Eq. E3.2-3)} \]

For double shear:
\[ \Omega = 2.00 \ \text{(ASD)} \]
\[ \phi = 0.65 \ \text{(LRFD)} \]
For single shear:
\[ \Omega = 2.22 \text{ (ASD)} \]
\[ \phi = 0.55 \text{ (LRFD)} \]

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

For a single bolt, or a single row of bolts perpendicular to the force
\[ F_t = (2.5d/s) F_u \leq F_u \quad (Eq. E3.2-4) \]

For multiple bolts in the line parallel to the force
\[ F_t = F_u \quad (Eq. E3.2-5) \]
\[ \Omega = 2.22 \text{ (ASD)} \]
\[ \phi = 0.65 \text{ (LRFD)} \]

where
- \( A_n \) = *Net area* of connected part
- \( F_t \) = Nominal tensile stress in flat sheet
- \( d \) = Nominal bolt diameter
- \( s \) = Sheet width divided by number of bolt holes in cross section being analyzed (when evaluating \( F_t \))
- \( F_u \) = *Tensile strength* of connected part as specified in Section A2.1, A2.2 or A2.3.2

(b) For flat sheet connections having staggered hole patterns
\[ P_n = A_n F_t \quad (Eq. E3.2-6) \]
\[ \Omega = 2.22 \text{ (ASD)} \]
\[ \phi = 0.65 \text{ (LRFD)} \]

where \( F_t \) is determined in accordance with Eqs. E3.2-2 to E3.2-5.

\[ A_n = 0.90 \left[ A_g - n_b d_h t + \left( \sum s'^2 / 4g \right) t \right] \quad (Eq. E3.2-7) \]

- \( A_g \) = *Gross area* of member
- \( s' \) = Longitudinal center-to-center spacing of any two consecutive holes
- \( g \) = Transverse center-to-center spacing between fastener gage lines
- \( n_b \) = Number of bolt holes in the cross section being analyzed
- \( d_h \) = Diameter of a standard hole

See Section E3.1 for the definition of \( t \).

(c) For other than flat sheet
\[ P_n = A_e F_u \quad (Eq. E3.2-8) \]
\[ \Omega = 2.22 \text{ (ASD)} \]
\[ \phi = 0.65 \text{ (LRFD)} \]

where
- \( A_e \) = \( A_n U \), *effective net area* with \( U \) defined as follows:
  - \( U = 1.0 \) for members when the *load* is transmitted directly to all of the cross-sectional elements. Otherwise, the reduction coefficient \( U \) is determined as follows:
    (1) For angle members having two or more bolts in the line of force
    \[ U = 1.0 - 1.20 \frac{x}{L} < 0.9 \quad (Eq. E3.2-9) \]
    but \( U \geq 0.4 \).
(2) For channel members having two or more bolts in the line of force

\[ U = 1.0 - 0.36 \frac{x}{L} < 0.9 \]  

(Eq. E3.2-10)

but \( U \geq 0.5 \).

where
\( x = \text{Distance from shear plane to centroid of the cross section} \)
\( L = \text{Length of the connection} \)

**E3.4 Shear and Tension in Bolts**

The nominal bolt strength, \( P_n \), resulting from shear, tension or a combination of shear and tension shall be calculated in accordance with this section. The corresponding *safety factor* and the *resistance factor* provided in Table E3.4-1 shall be used to determine the *available strengths* in accordance with the applicable method in Section A4 or A5.

\[ P_n = A_b F_n \]  

(Eq. E3.4-1)

where
\( A_b = \text{Gross cross-sectional area of bolt} \)
\( F_n = \text{Nominal strength ksi (MPa), is determined in accordance with (a) or (b) as follows:} \)

(a) When bolts are subjected to shear only or tension only

\( F_n \) shall be given by \( F_{nv} \) or \( F_{nt} \) in Table E3.4-1.

Corresponding *safety* and *resistance factors*, \( \Omega \) and \( \phi \), shall be in accordance with Table E3.4-1.

The pullover strength of the connected sheet at the bolt head, nut or washer shall be considered where bolt tension is involved. See Section E6.2.

(b) When bolts are subjected to a combination of shear and tension, \( F_n \), is given by \( F'_{nt} \) in Eq. E3.4-2 or E3.4-3 as follows

For ASD

\[ F'_{nt} = 1.3 F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt} \]  

(Eq. E3.4-2)

For LRFD

\[ F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \]  

(Eq. E3.4-3)

where
\( F'_{nt} = \text{Nominal tensile stress modified to include the effects of required shear stress, ksi (MPa)} \)
\( F_{nt} = \text{Nominal tensile stress from Table E3.4-1} \)
\( F_{nv} = \text{Nominal shear stress from Table E3.4-1} \)
\( f_v = \text{Required shear stress, ksi (MPa)} \)
\( \Omega = \text{Safety factor for shear from Table E3.4-1} \)
\( \phi = \text{Resistance factor for shear from Table E3.4-1} \)

In addition, the required shear stress, \( f_v \), shall not exceed the allowable shear stress, \( F_{nv} / \Omega \) (ASD) or the design shear stress, \( \phi F_{nv} \) (LRFD), of the fastener.
In Table E3.4-1, the shear strength shall apply to bolts in holes as limited by Table E3a. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Chapter F.

<table>
<thead>
<tr>
<th>Type of Bolt</th>
<th>Diameter</th>
<th>Tensile Strength</th>
<th>Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 Bolts, Grade A</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td>2.25</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>≥ 1/2 in.</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td></td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TABLE E3.4-1
Nominal Tensile and Shear Strengths for Bolts

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>Diameter</th>
<th>Tensile Strength</th>
<th>Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 Bolts, Grade A</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td>2.25</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>≥ 1/2 in.</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>A325 bolts, when threads are not excluded from shear planes</td>
<td></td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>A354 Grade BD Bolts</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A449 Bolts</td>
<td>≤ 1/2 in. (12.7 mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A490 Bolts, when threads are not excluded from shear planes</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In Table E3.4-1, the shear strength shall apply to bolts in holes as limited by Table E3a. Washers or back-up plates shall be installed over long-slotted holes and the capacity of connections using long-slotted holes shall be determined by load tests in accordance with Chapter F.
E4.3.2 **Connection Shear Limited by End Distance**

The nominal shear strength per screw, $P_{ns}$ shall not exceed that calculated in accordance with Eq. E4.3.2-1 where the distance to an end of the connected part is parallel to the line of the applied force. The safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.

$$P_{ns} = t e F_u \quad (Eq. \ E4.3.2-1)$$

$$\Omega = 3.00 \ (ASD)$$

$$\phi = 0.50 \ (LRFD)$$

where

$t = Thickness$ of part in which end distance is measured

e = Distance measured in line of force from center of a standard hole to nearest end of connected part.

$F_u = Tensile \ strength$ of part in which end distance is measured.

E5 **Rupture**

E5.1 **Shear Rupture**

At beam-end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the nominal shear strength, $V_n$, shall be calculated in accordance with Eq. E5.1-1. The safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.

$$V_n = 0.6 F_u A_{wn} \quad (Eq. \ E5.1-1)$$

$$\Omega = 2.00 \ (ASD)$$

$$\phi = 0.75 \ (LRFD)$$

where

$$A_{wn} = (h_{wc} - n d_h) t \quad (Eq. \ E5.1-2)$$

$h_{wc} = Coped \ flat \ web \ depth$

$n = Number \ of \ holes \ in \ critical \ plane$

$d_h = Hole \ diameter$

$F_u = Tensile \ strength$ of connected part as specified in Section A2.1 or A2.2

$t = Thickness$ of coped web

E5.2 **Tension Rupture**

The available tensile rupture strength along a path in the affected elements of connected members shall be determined by Section E2.7 or E3.2 for welded or bolted connections, respectively.

E5.3 **Block Shear Rupture**

When the thickness of the thinnest connected part is less than 3/16 in. (4.76 mm), the block shear rupture nominal strength, $R_n$, shall be determined in accordance with this section. Connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 in. (4.76 mm) shall be in accordance with ANSI/AISC-360.
The nominal block shear rupture strength, \( R_n \), shall be determined as the lesser of Eqs. E5.3-1 and E5.3-2. The corresponding safety factor and the resistance factor provided in this section shall be used to determine the available strengths in accordance with the applicable method in Section A4 or A5.

\[
R_n = 0.6F_Y A_{gv} + F_u A_{nt} \quad \text{(Eq. E5.3-1)}
\]

\[
R_n = 0.6F_u A_{nv} + F_u A_{nt} \quad \text{(Eq. E5.3-2)}
\]

For bolted connections
\[
\Omega = 2.22 \quad \text{(ASD)}
\]
\[
\phi = 0.65 \quad \text{(LRFD)}
\]

For welded connections
\[
\Omega = 2.50 \quad \text{(ASD)}
\]
\[
\phi = 0.60 \quad \text{(LRFD)}
\]

where
\[
A_{gv} = \text{Gross area subject to shear}
\]
\[
A_{nv} = \text{Net area subject to shear}
\]
\[
A_{nt} = \text{Net area subject to tension}
\]
Appendix B:

Provisions Applicable to

Canada

2007 EDITION
PREFACE TO APPENDIX B:

Appendix B provides specification provisions that are applicable only to Canada. Included are items of a general nature such as specific reference documents and provisions on loads and load combinations in accordance with the National Building Code of Canada.

While this document is referred to as a “Specification”, in Canada it is considered a “Standard”.

Also included in Appendix B are technical items where full agreement between the three countries was not reached. The most noteworthy of these items are

• Beams (C- and Z- sections) for standing seam roofs,
• Bolted connections, and
• Tension members

Efforts will be made to minimize these differences in future editions of the Specification.
APPENDIX B: PROVISIONS APPLICABLE TO CANADA

The material contained in this Appendix provides design provisions and supplements that, in addition to those in Chapters A through G, are mandatory for use in Canada. A section number ending with the letter “a” indicates that the provisions herein supplement the corresponding section in Chapters A through G of the Specification. A section number not ending with the letter “a” indicates that the section presents the entire design provision.

A1.3a Definitions

The following additional definition applies in Appendix B:

Importance Factor. A factor applied to the specified loads, other than dead load, to take into account the consequences of failure as related to the limit state and the use and occupancy of the building.

Load factor. A factor applied to a specified load that, for the limit states under consideration, takes into account the variability in magnitude of the load, the loading patterns, and the analysis of their effects.

A2.1a Applicable Steels

These steels are in addition to those listed in Section A2.1:

CSA Standards G40.20/G40.21-03, General requirements for rolled or welded structural quality steel/Structural quality steel.

A2.2 Other Steels

A2.2.1 Other Structural Quality Steels

For structural quality steels not listed in Section A2.1, $F_y$ and $F_u$ shall be the specified minimum values as given in the material standard or published specification. These steels shall also meet the requirements of Section A2.3.

A2.2.2 Other Steels

For steels not covered by Section A2.1 of the Specification and A2.2.1 of this Appendix, tensile tests shall be conducted in accordance with Section F3. $F_y$ and $F_u$ shall be 0.8 times the yield strength and 0.8 times the tensile strength determined from the tests. These steels shall also meet the requirements of Section A2.3.

A2.3.1a Ductility

In buildings with specified short-period spectral acceleration ratios greater than 0.35, and when material ductility is determined on the basis of the local and uniform elongation criteria of Section A2.3.1, the use of curtain wall studs shall be limited to wall assemblies whose dead load divided by its surface area is not greater than 0.72 kN/m².

The specified short-period acceleration ratio is given by the expression $I_E F_a S_a(0.2)$. The terms $I_E$, $F_a$, and $S_a(0.2)$ are defined in Volume 1, Division B, Part 4 earthquake load and effects of the National Building Code of Canada.
A3 Loads

The resistance factors adopted in this Specification are correlated with the loads and load factors for buildings specified in the National Building Code of Canada. For other cases, load factors shall be established in such a way that, in conjunction with the resistance factors used in this Specification, the required level of reliability is maintained.

A3.1 Loads and Effects

The following loads, forces, and effects shall be considered in the design of cold-formed steel structural members and their connections:

D = Dead load (a permanent load due to the weight of building components, including the mass of the member and all permanent materials of construction, partitions, permanent equipment, and supported earth, plants and trees, multiplied by the acceleration due to gravity to convert mass (kg) to force (N)),
E = Earthquake load and effects (a rare load due to earthquake),
H = A permanent load due to lateral earth pressure, including groundwater,
L = Live load (a variable load depending on intended use and occupancy, including loads due to movable equipment, cranes, and pressure of liquids in containers),
S = Variable load due to snow, including ice and associated rain, or rain,
T = Effects due to contraction, expansion, or deflection caused by temperature changes, shrinkage, moisture changes, creep, ground settlement, or any combination thereof,
W = Wind load (a variable load due to wind).

A3.2 Temperature, Earth, and Hydrostatic Pressure Effects

Where the effects due to lateral earth pressure, H, and imposed deformation, T, affect structural safety, they shall be taken into account in the calculations. H shall have a load factor of 1.5, and T shall have a load factor of 1.25.

A6.1.2 Load Factors and Load Combinations for LSD

The effect of factored loads for a building or structural component shall be determined in accordance with the load combination cases listed in Table A6.1.2-1, and the applicable combination being that which results in the most critical effect.
Table A6.1.2-1
Load Combinations for Ultimate Limit States

<table>
<thead>
<tr>
<th>CASE</th>
<th>Load Combination</th>
<th>Companion Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Principal Loads</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.4D</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>(1.25D or 0.9D) + 1.5L</td>
<td>0.5S or 0.4W</td>
</tr>
<tr>
<td>3</td>
<td>(1.25D or 0.9D) + 1.5S</td>
<td>0.5L or 0.4W</td>
</tr>
<tr>
<td>4</td>
<td>(1.25D or 0.9D) + 1.4W</td>
<td>0.5L or 0.5S</td>
</tr>
<tr>
<td>5</td>
<td>1.0D + 1.0E</td>
<td>0.5L + 0.25S</td>
</tr>
</tbody>
</table>

Notes to Table A6.1.2-1:
(1) Except for rocking footings, the counteracting factored dead load, 0.9D in load combination cases (2), (3), and (4), and 1.0D in load combination case (5), shall be used when the dead load acts to resist overturning, uplift, sliding, failure due to stress reversal, and to determine anchorage requirements and the factored resistance of members.
(2) The principal-load factor 1.5 for live load, L, may be reduced to 1.25 for liquids in tanks.
(3) The companion-load factor 0.5 for live load, L, shall be increased to 1.0 for storage areas, equipment areas, and service rooms.
(4) The load factor 1.25 for dead load, D, for soil, superimposed earth, plants, and trees shall be increased to 1.5, except that when the soil depth exceeds 1.2 m, the factor may be reduced to 1+0.6/hs but not less than 1.25, where hs is the depth of soil in metres supported by the structure.
(5) Earthquake load, E, in load combination case (5) includes horizontal earth pressure due to earthquake.

A6.1.2.1 Importance Categories

For the purpose of determining specified loads S, W, or E, buildings shall be assigned an importance category, based on intended use and occupancy, in accordance with Table A6.1.2.1-1.
### Table A6.1.2.1-1
Importance Categories for Buildings

<table>
<thead>
<tr>
<th>Use and Occupancy</th>
<th>Importance Category</th>
</tr>
</thead>
</table>
| Buildings that represent a low direct or indirect hazard to human life in the event of failure, including:  
  - low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences  
  - minor storage buildings | Low                 |
| All buildings except those listed in Categories Low, High, and Post-disaster      | Normal              |
| Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is:  
  - as an elementary, middle, and secondary school  
  - as a community centre | High                |
| Manufacturing and storage facilities containing toxic, explosive, or other hazardous substances in sufficient quantities to be dangerous to the public if released |                    |
| Post-disaster buildings are buildings that are essential to the provision of services in the event of a disaster, and include:  
  - hospitals, emergency treatment facilities, and blood banks  
  - telephone exchanges  
  - power generating stations and electrical substations  
  - control centres for air, land, and marine transportation  
  - public water treatment and storage facilities and pumping stations  
  - sewage treatment facilities and buildings having critical national defense functions  
  - buildings of the following types, unless exempted from this designation by the authority having jurisdiction:  
    - emergency response facilities  
    - fire, rescue, and police stations, and housing for vehicles, aircraft, or boats used for such purposes  
    - communications facilities, including radio and television stations | Post-disaster       |

For buildings in the Low Importance Category, a factor of 0.8 may be applied to the live load.

#### A6.1.2.2 Importance Factor (I)

The *importance factor* for snow, wind, and earthquake shall be as provided for in Table A6.1.2.2-1.
### Table A6.1.2.2-1
Importance Factors for Snow, Wind, and Earthquake

<table>
<thead>
<tr>
<th>Importance Category</th>
<th>Importance Factor for Ultimate Limit States</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Snow, $I_S$</td>
<td>Wind, $I_W$</td>
</tr>
<tr>
<td>Low</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Normal</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>High</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td>Post-disaster</td>
<td>1.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>

A9a Reference Documents

This Appendix refers to the following publications, and where such reference is made, it shall be to the edition listed below including all amendments published thereto:

1. Canadian Standards Association (CSA), 5060 Spectrum Way, Suite 100, Mississauga, ON, Canada, L4W 5N6:
   - G40.20-04/G40.21-04, General requirements for rolled or welded structural quality steel/Structural quality steel
   - CAN/CSA-S16-01 (including 2005 Supplement), Limit states design of steel structures
   - W47.1-03, Certification of companies for fusion welding of steel
   - W55.3-1965 (R2003), Resistance Welding Qualification Code for Fabricators of Structural Members Used in Buildings
   - W59-03, 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, 2005

C2 Tension Members

The nominal tensile resistance, $T_{nv}$, shall be the lesser of the values determined in Sections C2.1 and C2.2 of this Appendix. The nominal tensile resistance shall also be limited by Sections E2.7 of the Specification, E3.2 of this Appendix, and E3.3 of the Specification for tension members using welded, bolted, and screw connections.

**C2.1 Yielding of Gross Section**

The nominal tensile resistance, $T_{nv}$, due to yielding of the gross section shall be determined as follows:

$$T_{nv} = A_g F_y$$  \(\text{(Eq. C2.1-1)}\)

where

- $A_g = \text{Gross area of cross-section}$
- $F_y = \text{Yield stress defined in Section A7.1}$

**C2.2 Rupture of Net Section**

The nominal tensile resistance, $T_{nv}$, due to rupture of the net section shall be determined as
follows:

\[ T_n = A_n F_u \]  \hspace{1cm} (Eq. C2.2-1)

\[ \phi_u = 0.75 \]

where

\( A_n = \text{Critical net area of connected part} \)

\( = L_{ct} \)  \hspace{1cm} (Eq. C2.2-2)

where

\( L_c = \text{Summation of critical path lengths of each segment along a potential failure path of minimum strength.} \)

\( L_c \) shall be determined as follows:

(a) For failure normal to force due to direct tension:

\[ L_c = L_t \quad \text{not involving stagger} \]  \hspace{1cm} (Eq. C2.2-3)

\[ L_c = 0.9 L_s \quad \text{involving stagger} \]  \hspace{1cm} (Eq. C2.2-4)

(b) For failure parallel to force due to shear:

\[ L_c = 0.6 L_{nv} \]  \hspace{1cm} (Eq. C2.2-5)

(c) For failure due to block tear-out at end of member:

\[ L_c = L_t + 0.6 L_v \quad \text{not involving stagger} \]  \hspace{1cm} (Eq. C2.2-6)

\[ L_c = 0.9(L_t + L_s) + 0.6 L_v \quad \text{involving stagger} \]  \hspace{1cm} (Eq. C2.2-7)

(d) For failure of coped beams:

\[ L_c = 0.5 L_t + 0.6 L_v \quad \text{not involving stagger} \]  \hspace{1cm} (Eq. C2.2-8)

\[ L_c = 0.45(L_t + L_s) + 0.6 L_v \quad \text{involving stagger} \]  \hspace{1cm} (Eq. C2.2-9)

where

\( L_v = \text{the lesser of } C L_{gv} \text{ and } L_{nv} \) in (c) and (d)

\[ C = \frac{F_y}{F_u} \]  \hspace{1cm} (Eq. C2.2-10)

\( L_t = \text{Net failure path length normal to force due to direct tension} \)

\( L_s = \text{Net failure path length inclined to force (including } \frac{s^2}{4g} \text{ allowance for staggered holes)} \)

\( L_{gv} = \text{Gross failure path length parallel to force (i.e., in shear)} \)

\( L_{nv} = \text{Net failure path length parallel to force (i.e., in shear)} \)

\( s = \text{Pitch, spacing of fastener parallel to force} \)

\( g = \text{Gauge, spacing of fastener perpendicular to force} \)

\( t = \text{Base steel thickness} \)

\( F_u = \text{Tensile strength as specified in Section A2} \)

**D3a 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 assure the effectiveness of the bracing or bracing system.

The spacing of braces shall not be greater than the unbraced length assumed in the design of the member or component being braced.
D3.1a  Symmetrical Beams and Columns

The provisions of Sections D3.1.1 and D3.1.2 of this Appendix apply to symmetric sections in compression or bending in which the applied load does not induce twist.

D3.1.1 Discrete Bracing for Beams

The factored resistance of braces shall be at least 2% 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.

D3.1.2 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% of either the maximum factored compressive force in a compressive member or the maximum factored compressive force in the compressive flange of a member in bending.

D3.2a  C-Section and Z-Section Beams

The provisions of Sections D3.2.2, D3.2.3, and D3.2.4 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.

D3.2.2 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 analysis, testing, or Section D6.1.1 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 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.

D3.2.3 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 D3.1.2 of this Appendix. Discrete bracing shall be provided to restrain the flange that is not braced by the deck, slab, or sheathing. The spacing of discrete bracing shall be in accordance with Section D3.2.2 of this Appendix.
D3.2.4 Both Flanges Braced by Deck, Slab, or Sheathing

The factored resistance of the attachment shall be as given by Section D3.1.2 of this Appendix.

D6.1.2 Flexural Members Having One Flange Fastened to a Standing Seam Roof System

This type of member shall have discrete bracing in accordance with Section D3.2.2 of this Appendix.

E2a Welded Connections

Arc welding shall be performed by a fabricator or erector certified in accordance with CSA W47.1. Resistance welding shall be performed by a fabricator or erector certified in accordance with CSA W55.3.

Where each connected part is over 4.76 mm in base steel thickness, welding shall conform to CSA W59. Where at least one of the connected parts is between 0.70 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 for in Section E2.2, where at least one of the connected parts is less than 0.70 mm in base steel thickness, welds shall be considered to have no structural value unless a value is substantiated by appropriate tests.

The resistance in tension or compression of butt welds shall be the same as that prescribed for the lower strength of base metal being joined. The butt weld shall fully penetrate the joint.

E2.2a Arc Spot Welds

This section replaces the first paragraph of Section E2.2 but does not pertain to Section E2.2.1.3.

Arc spot welds (circular in shape) covered by this Specification are for welding sheet steel to thicker supporting members in the flat position. The weld is formed by melting through the steel sheet to fuse with the underlying supporting member, whose thickness at the weld location shall be at least 2.5 times the steel sheet thickness (aggregate sheet thickness in the case of multiple plies). The materials to be joined shall be of weldable quality, and the electrodes to be used shall be suited to the materials, the welding method, and the ambient conditions during welding.

The following maximum and minimum sheet thicknesses shall apply:
(a) maximum single sheet thickness shall be 2.0 mm;
(b) minimum sheet thickness shall be 0.70 mm; and
(c) maximum aggregate sheet thickness of double sheets shall be 2.5 mm.

E2.3a Arc Seam Welds

The information in Section E2.2a also applies to arc seam welds that are oval in shape.

E3a Bolted Connections

In addition to the design criteria given in Section E3 of the Specification, the design requirements given in Sections E3.1 and E3.2 of this Appendix shall be followed for bolted connections where the thickness of the thinnest connected part is 4.76 mm or less, there are no
gaps between connected parts, and fasteners are installed with sufficient tightness to achieve satisfactory performance of the connection under anticipated service conditions. Refer to CSA S16 for the design of mechanically fastened connections in which the thickness of all connected parts exceeds 4.76 mm.

Unless otherwise specified, circular holes for bolts shall not be greater than the nominal bolt diameter, d, plus 1 mm for bolt sizes up to 13 mm and plus 2 mm for bolt sizes over 13 mm.

Slotted or oversized holes may be used when the hole occurs within the lap of lapped or nested Z-members, subject to the following restrictions:

1. 12.7 mm diameter bolts only, with or without washers,
2. Maximum slot size is 14.3 x 22.2 mm slotted vertically,
3. Maximum oversize hole is 15.9 mm diameter,
4. Minimum member thicknesses is 1.52 mm nominal,
5. Maximum member yield stress is 410 MPa, and
6. Minimum lap length measured from centre of frame to end of lap is 1.5 times the member depth.

E3.1 Shear, Spacing, and Edge Distance

The nominal shear resistance per bolt as affected by spacing and edge distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix.

The center-to-center distance between fasteners shall not be less than 2.5d, and the distance from the center of a fastener to an edge or end shall not be less than 1.5d, where d = nominal diameter of fastener.

E3.2 Rupture of Net Section (Shear Lag)

The nominal tensile resistance, \( P_n \), of a tension member other than a flat sheet shall be determined as follows:

\[
P_n = A_e F_u \phi \quad (Eq. E3.2-1)
\]

where

- \( F_u \) = Tensile strength of connected part as specified in Section A2
- \( A_e = A_n U \), effective net area with reduction coefficient, U

where

- \( U = 1.0 \) for members when the load is transmitted directly to all of the cross-sectional elements. Otherwise, U shall be determined as follows:
  - a) For angle members having two or more bolts in the line of force
    \[
    U = 1.0 - 1.2 \frac{\bar{x}}{L} < 0.9 \quad (Eq. E3.2-2)
    \]
    \[
    U \geq 0.4
    \]
  - b) For channel members having two or more bolts in the line of force
    \[
    U = 1.0 - 0.36 \frac{\bar{x}}{L} < 0.9 \quad (Eq. E3.2-3)
    \]
    \[
    U \geq 0.5.
    \]

- \( \bar{x} \) = Distance from shear plane to centroid of cross-section
- \( L \) = Length of connection
- \( A_n \) = Net area of connected part
E3.3a Bearing

When the thickness of connected steels is equal to or larger than 4.76 mm, the requirements of CSA S16 shall be met for connection design.

E3.4 Shear and Tension in Bolts

For ASTM A 307 bolts less than 12.7 mm in diameter, refer to Tables E3.4-1 and E3.4-2 of this Appendix. For all other bolts, refer to CSA S16.

The nominal bolt resistance, \( P_n \), resulting from shear, tension, or a combination of shear and tension shall be calculated as follows:

\[
P_n = A_b F_n \quad (Eq. \ E3.4-1)
\]

where

\( A_b = \) Gross cross-sectional area of bolt

\( F_n = \) A value determined in accordance with i) and ii) below, as applicable:

i) When bolts are subjected to shear or tension

\( F_n \) is given by \( F_{nt} \) or \( F_{nv} \) in Table E3.4-1, as well as the \( \phi \) values

ii) When bolts are subjected to a combination of shear and tension

\( F_n \) is given by \( F'_{nt} \) in Table E3.4-2, 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 E6.2 of the Specification.

### TABLE E3.4-1
Nominal Tensile and Shear Stresses for Bolts

<table>
<thead>
<tr>
<th>Description of Bolts</th>
<th>Nominal Tensile Stress, ( F_{nt} ) (MPa)</th>
<th>Resistance Factor, ( \phi )</th>
<th>Nominal Shear Stress, ( F_{nv} ) (MPa)</th>
<th>Resistance Factor, ( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 Bolts, Grade A 6.4 mm ( \leq d &lt; 12.7 ) mm</td>
<td>279</td>
<td>0.65</td>
<td>165</td>
<td>0.55</td>
</tr>
</tbody>
</table>

### TABLE E3.4-2
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 ( \leq d &lt; 12.7 ) mm</td>
<td>( 324 - 2.4f_v \leq 279 )</td>
<td>0.65</td>
</tr>
</tbody>
</table>

The actual shear stress, \( f_v \), shall also satisfy Table E3.4-1 of this Appendix.
E4.3.2 Connection Shear Limited by End Distance

The nominal shear resistance per screw as affected by end distance in the direction of the applied force shall be calculated in accordance with the requirements of Section C2.2 of this Appendix. For spacing requirements, see Section E3.1 of this Appendix.

E5 Rupture

Shear rupture, tension rupture, and block shear rupture shall be determined in accordance with the requirements of Section C2.2 of this Appendix.