Chapter 10

STEEL STRUCTURES

10.1 General Provisions for Structural Steel Buildings and Structures

This section states the scope of the Specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

10.1.1 Scope

The specification contained in chapter 10 of Part 6 of this code sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other steel-structures are defined as those structures designed, fabricated, and erected in a manner similar to steel-buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by this specification, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

10.1.1.1 Low-Seismic Applications: When the seismic response modification coefficient, \( R \), (as specified in Chapter 2 of Part 6) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other steel-structures shall comply with this specification except that such structures need not to comply with the specifications set forth in Section 10.20 Seismic Provisions.

10.1.1.2 High-Seismic Applications: When the seismic response modification coefficient, \( R \), (as specified in Chapter 2 of Part 6) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the Section 10.20 Seismic Provisions, in addition to the provisions of other sections (whichever applicable) this specification.

10.1.2 Referenced Specifications, Codes and Standards

The following specifications, codes and standards are referenced in this Specification:

ACI International (ACI)
ACI 318-02 Building Code Requirements for Structural Concrete and Commentary
ACI 318M-02 Metric Building Code Requirements for Structural Concrete and Commentary
American Institute of Steel Construction, Inc. (AISC)
AISC 303-05 Code of Standard Practice for Steel Buildings and Bridges
ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings
ANSI/AISC N690L-03 Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities
American Society of Civil Engineers (ASCE)
SEI/ASCE 7-02 Minimum Design Loads for Buildings and Other Structures
American Society of Mechanical Engineers (ASME)
ASME B18.2.6-96 Fasteners for Use in Structural Applications
ASME B46.1-95 Surface Texture, Surface Roughness, Waviness, and Lay
American Iron and Steel Institute (AISI)
Code of Standard Practice for Cold-Formed Steel Structural Framing, 2005
ASTM International (ASTM)
A6/A6M-04a Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling
A36/A36M-04 Standard Specification for Carbon Structural Steel
A53/A53M-02 Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
A193/A193M-04a Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service
A194/A194M-04 Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both
A242/A242M-04 Standard Specification for High-Strength Low-Alloy Structural Steel
A283/A283M-03 Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates
A307-03 Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength
A325-04 Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
A325M-04 Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)
A354-03a Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners
A370-03a Standard Test Methods and Definitions for Mechanical Testing of Steel Products
A449-04 Standard Specification for Quenched and Tempered Steel Bolts and Studs
A490-04 Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength
A490M-04 Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)
A500-03a Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A501-01 Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A502-03 Standard Specification for Steel Structural Rivets
A514/A514M-00a Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding
A529/A529M-04 Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality
A563-04 Standard Specification for Carbon and Alloy Steel Nuts
A563M-03 Standard Specification for Carbon and Alloy Steel Nuts [Metric]
A568/A568M-03 Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for
A572/A572M-04 Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
A588/A588M-04 Standard Specification for High-Strength Low-Alloy Structural Steel with 345 MPa Minimum Yield Point to 100 mm Thick
A606-04 Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance
A618/A618M-04 Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing
A668/A668M-04 Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use
A709/A709M-04 Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges
A751-01 Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products
A852/A852M-03 Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 485 MPa Minimum Yield Strength to 100 mm Thick
A913/A913M-04 Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)
A992/A992M-04 Standard Specification for Structural Steel Shapes
A1011/A1011M-04 Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability
C33-03 Standard Specification for Concrete Aggregates
C330-04 Standard Specification for Lightweight Aggregates for Structural Concrete
E119-00a Standard Test Methods for Fire Tests of Building Construction and Materials
E709-01 Standard Guide for Magnetic Particle Examination
F436-03 Standard Specification for Hardened Steel Washers
F959-02 Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners
F1554-99 Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength
F1852-04 Standard Specification for “Twist-Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
American Welding Society (AWS)
AWS D1.1/D1.1M-2004 Structural Welding Code–Steel
AWS A5.1-2004 Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding
AWS A5.5-96 Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding
AWS A5.17/A5.17M-97 Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding
AWS A5.18:2001 Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding
AWS A5.20-95 Specification for Carbon Steel Electrodes for Flux Cored Arc Welding
AWS A5.23/A5.23M-97 Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding
AWS A5.25/A5.25M-97 Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding
AWS A5.26/A5.26M-97 Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding
AWS A5.28-96 Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding
Research Council on Structural Connections (RCSC)
Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004
Bangladesh Standards and Testing Institute (Bangladesh Standards or BDS)

10.1.3 Material

10.1.3.1 Structural Steel Materials

a) Regular Structural Steel: Material test reports from an acceptable testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those
product forms. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

Structural steel material conforming to one of the following specifications is approved for use under this Specification:

(i) Hot-rolled structural shapes
ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A913/A913M, ASTM A992/ A992M

(ii) Structural tubing

(iii) Pipe

(iv) Plates

(v) Bars

(vi) Sheets

b) Unidentified Steel: Unidentified steel free of injurious defects is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

c) Rolled Heavy Shapes: ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm, used as members subject to primary (computed) tensile forces due to tension or flexure and spliced using complete-joint-penetration groove welds that fuse through the thickness of the member, shall be specified as follows. The contract documents shall require that such shapes be supplied with Charpy V-Notch (CVN) impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S30, Charpy V-Notch Impact Test for Structural Shapes — Alternate Core Location. The impact test shall meet a minimum average value of 27 J absorbed energy at +21°C.

The above requirements do not apply if the splices and connections are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 50 mm that have shapes with flange or web elements less than 50 mm thick welded with complete-joint-penetration groove welds to the face of the shapes with thicker elements.

d) Built-Up Heavy Shapes: Built-up cross-sections consisting of plates with a thickness exceeding 50 mm, used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected to other members using complete-joint-penetration groove welds that fuse through the thickness of the plates, shall be specified as follows. The contract documents shall require that the steel be supplied with Charpy V-Notch impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 27 J absorbed energy at +21°C.

The above requirements also apply to built-up cross-sections consisting of plates exceeding 50 mm that are welded with complete-joint-penetration groove welds to the face of other sections.

e) Cold Form Sections: Specifications for cold form shapes regarding their use as structural members is not covered in Section 10. For such type of structural steel, AISI standard (AISI/COS/NASPEC 2001) or equivalent may be followed.

10.1.3.2 Steel Castings and Forgings: Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports
produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

10.1.3.3 Bolts, Washers and Nuts: Bolt, washer, and nut material conforming to one of the following ASTM specifications is approved for use under this specification:

(1) **Bolts**: ASTM A307, ASTM A325, ASTM A325M, ASTM A449, ASTM A490, ASTM A490M, ASTM F1852
(2) **Nuts**: ASTM A194/A194M, ASTM A563, ASTM A563M
(3) **Washers**: ASTM F436, ASTM F436M
(4) **Compressible-Washer-Type Direct Tension Indicators**: ASTM F959, ASTM F959M

10.1.3.4 **Anchor rods and Threaded rods**: Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:


A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

10.1.3.5 **Filler metal and Flux for Welding**: Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society: AWS A5.1, AWS A5.5, AWS A5.17/A5.17M, AWS A5.18, AWS A5.20, AWS A5.23/A5.23M, AWS A5.25/A5.25M, AWS A5.26/A5.26M, AWS A5.28, AWS A5.29, AWS A5.32/A5.32M

10.1.3.6 **Stud Shear Connectors**: Steel stud shear connectors shall conform to the requirements of *Structural Welding Code–Steel*, AWS D1.1.

10.1.4 Structural Design Drawings and Specifications
The design drawings and specifications shall meet the requirements specified in this specification (Sections 10.1 through 10.20) and shall be prepared and presented in an internationally approved standard in accordance with the provisions of Section 10.13, except for deviations specifically identified in the design drawings and/or specifications and approved by an appropriate authority.

10.2 General Design Requirements
The general requirements for the analysis and design of steel buildings and structures that are applicable to all sections of Chapter 10 of Part 6 are given in this section.

10.2.1 General Provisions
The design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis. Unless restricted by the applicable code, lateral load resistance and stability may be provided by any combination of members and connections.

10.2.2 Loads and Load Combinations
The loads and load combinations shall be as stipulated in Chapter 2 of Part 6 of this code. For design purposes, the nominal loads shall be taken as the loads stipulated in the Chapter 2.

10.2.3 Design Basis
Designs shall be made according to the provisions for Load and Resistance Factor Design (LRFD) or to the provisions for Allowable Strength Design (ASD).

10.2.3.1 **Required Strength**: The required strength of structural members and connections shall be determined by structural analysis for the appropriate load combinations as stipulated in Chapter 2 of Part 6. Design by elastic, inelastic or plastic analysis is permitted. Provisions for inelastic and plastic analysis
are as stipulated in Section 10.15, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Section 10.15.3 are permitted for elastic analysis only.

10.2.3.2 **Limit States:** Design shall be based on the principle that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to all appropriate load combinations.

10.2.3.3 **Design for Strength Using Load and Resistance Factor Design (LRFD):** Design according to the provisions for Load and Resistance Factor Design (LRFD) 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 LRFD load combinations as specified in Chapter 2 of Part 6. All provisions of this Specification, except for those in Section 10.2.3.4, shall apply. Design shall be performed in accordance with Equation 10.2.1:

\[
\leq (10.2.1)
\]

where

\[
R_u = \text{required strength (LRFD)}
\]

\[
R_n = \text{nominal strength, specified in Sections 10.2 through 10.20}
\]

\[
\varnothing = \text{resistance factor, specified in Sections 10.2 through 10.20}
\]

\[
\varnothing R_n = \text{design strength}
\]

10.2.3.4 **Design for Strength Using Allowable Strength Design (ASD):** Design according to the provisions for Allowable Strength Design (ASD) 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 ASD load combinations as specified in Chapter 2 of Part 6. All provisions of this Specification, except those of Section 10.2.3.3, shall apply. Design shall be performed in accordance with Equation 10.2.2:

\[
\leq /\Omega (10.2.2)
\]

where

\[
R_o = \text{required strength (ASD)}
\]

\[
R_n = \text{nominal strength, specified in Sections 10.2 through 10.20}
\]

\[
\Omega = \text{safety factor, specified in Sections 10.2 through 10.20}
\]

\[
R_n /\Omega = \text{allowable strength}
\]

10.2.3.5 **Design for Stability:** Stability of the structure and its elements shall be determined in accordance with Section 10.3.

10.2.3.6 **Design for Connection:** Connection elements shall be designed in accordance with the provisions of Section 10.10 and 10.11. The forces and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the structural analysis.

10.2.3.6.1 **Simple Connection:** A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted.

10.2.3.6.2 **Moment Connection:** A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

a) **Fully-Restrained (FR) Moment Connections:** A fully-restrained (FR) moment connection transfers moment with a negligible relative rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.

b) **Partially-Restrained (PR) Moment Connections:** Partially-restrained (PR) moment connections transfer moments, but the relative rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall
be included. The response characteristics of a PR connection shall be documented in the technical
literature or established by analytical or experimental means. The component elements of a PR
connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit
states.

10.2.3.7 Design for Connection: The overall structure and the individual members, connections and connectors
shall be checked for serviceability. Performance requirements for serviceability design are given in
Section 10.12.

10.2.3.8 Design for Ponding: The roof system shall be investigated through structural analysis to assure
adequate strength and stability under ponding conditions, unless the roof surface is provided with a
slope of 20 mm per meter or greater toward points of free drainage or an adequate system of
drainage is provided to prevent the accumulation of water. Methods of checking ponding are given in
section 10.16.

10.2.3.9 Design for Fatigue: Fatigue shall be considered in accordance with Section 10.17, Design for Fatigue,
for members and their connections subject to repeated loading. Fatigue need not be considered for
seismic effects or for the effects of wind loading on normal building lateral load resisting systems and
building enclosure components.

10.2.3.10 Design for Fire Conditions: Two methods of design for fire conditions are provided in Section 10.18,
Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis. Compliance with
the fire protection requirements in Part 4 of this code shall be required in addition to satisfying the
requirements of Section 10.18.

10.2.3.11 Design for Corrosion Effects: Where corrosion may impair the strength or serviceability of a structure,
structural components shall be designed to tolerate corrosion or shall be protected against corrosion.

10.2.3.12 Design Wall Thickness for HSS: The design wall thickness, t, shall be used in calculations involving the
wall thickness of hollow structural sections (HSS). The design wall thickness, t, shall be taken equal to
0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and equal to the
nominal thickness for submerged-arc-welded (SAW) HSS.

10.2.3.13 Gross and Net Area Determination
10.2.3.13.1 Gross Area: The gross area, $A_g$, of a member is the total cross-sectional area.
10.2.3.13.2 Net Area: The net area, $A_n$, of a member is the sum of the products of the thickness and the net
width of each element computed as follows:
In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the
nominal dimension of the hole.
For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be
obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section
10.10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/(4g)$
where
$s$ = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.
g = transverse center-to-center spacing (gage) between fastener gage lines, mm.
For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles
less the thickness.
For slotted HSS welded to a gusset plate, the net area, $A_{n_g}$, is the gross area the product of the thickness and the
total width of material that is removed to form the slot.
In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the
net area.
10.2.4 Classification of Sections for Local Buckling

Sections are classified as compact, noncompact, or slender-element sections. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios $\lambda_p$ from Table 10.2.1. If the width-thickness ratio of one or more compression elements exceeds $\lambda_p$, but does not exceed $\lambda_c$ from Table 10.2.1, the section is noncompact. If the width-thickness ratio of any element exceeds $\lambda_c$, the section is referred to as a slender-element section.

10.2.4.1 Unstiffened Elements: For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width $b$ is one-half the full-flange width, $b_f$.
(b) For legs of angles and flanges of channels and zees, the width $b$ is the full nominal dimension.
(c) For plates, the width $b$ is the distance from the free edge to the first row of fasteners or line of welds.
(d) For stems of tees, $d$ is taken as the full nominal depth of the section.

10.2.4.2 Stiffened Elements: For stiffened elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

a) For webs of rolled or formed sections, $h$ is the clear distance between flanges less the fillet or corner radius at each flange; $h_c$ is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

b) For webs of built-up sections, $h$ is the distance between adjacent lines of fasteners or the clear distance between flanges when webs are used, and $h_c$ is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when webs are used; $h_p$ is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when webs are used.

c) For flange or diaphragm plates in built-up sections, the width $b$ is the distance between adjacent lines of fasteners or lines of welds.

d) For flanges of rectangular hollow structural sections (HSS), the width $b$ is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, $h$ is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, $b$ and $h$ shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, $t$, shall be taken as the design wall thickness per Section 10.2.3.12.

e) For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

10.2.5 Fabrication, Erection and Quality

Shop drawings, fabrication, shop painting, erection, and quality control shall meet the requirements stipulated in Section 10.13, Fabrication, Erection, and Quality Control.

10.3 Stability Analysis and Design

This section addresses general requirements for the stability analysis and design of members and frames of steel buildings and structures.

10.3.1 Stability Design Requirements

General Requirements: Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of second-order effects (including $P-\Delta$ and $P-\delta$ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this Section and Section 10.14: Direct Analysis Method, satisfy these requirements. All component and connection deformations that contribute to the lateral displacements shall be considered in the stability analysis.
<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Width Thickness Ratio</th>
<th>Limiting Width-Thickness Ratios</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Flexure in flanges of rolled I-shaped sections and channels</td>
<td>$b/t$</td>
<td>$\lambda_p = 0.38 \sqrt{E/F_y}$</td>
<td>![Example 1]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 1.0 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Flexure in flanges of doubly and singly symmetric I-shaped built-up sections</td>
<td>$b/t$</td>
<td>$\lambda_p = 0.38 \sqrt{E/F_y}$</td>
<td>![Example 2]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 0.95 \sqrt{k_c E/F_L}$</td>
<td>![Formula]</td>
</tr>
<tr>
<td>3</td>
<td>Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels</td>
<td>$b/t$</td>
<td>NA</td>
<td>![Example 3]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 0.56 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections</td>
<td>$b/t$</td>
<td>NA</td>
<td>![Example 4]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 0.64 \sqrt{k_c E/F_L}$</td>
<td>![Formula]</td>
</tr>
<tr>
<td>5</td>
<td>Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements</td>
<td>$b/t$</td>
<td>NA</td>
<td>![Example 5]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 0.45 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Flexure in legs of single angles</td>
<td>$b/t$</td>
<td>$\lambda_p = 0.54 \sqrt{E/F_y}$</td>
<td>![Example 6]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\lambda_y = 0.91 \sqrt{E/F_y}$</td>
<td></td>
</tr>
<tr>
<td>Case</td>
<td>Description of Element</td>
<td>Width Thickness Ratio</td>
<td>Limiting Width-Thickness Ratios</td>
<td>Example</td>
</tr>
<tr>
<td>------</td>
<td>------------------------</td>
<td>-----------------------</td>
<td>---------------------------------</td>
<td>---------</td>
</tr>
<tr>
<td>7</td>
<td>Flexure in flanges of tees</td>
<td>b/t</td>
<td>$\lambda_p = 0.38\sqrt{E/F_y}$</td>
<td><img src="example1.png" alt="image" /></td>
</tr>
<tr>
<td>8</td>
<td>Uniform compression in stems of tees</td>
<td>d/t</td>
<td>$\lambda_p = 0.75\sqrt{E/F_y}$</td>
<td><img src="example2.png" alt="image" /></td>
</tr>
<tr>
<td>9</td>
<td>Flexure in webs of doubly symmetric I-shaped sections and channels</td>
<td>h/tw</td>
<td>$\lambda_p = 3.76\sqrt{E/F_y}$</td>
<td><img src="example3.png" alt="image" /></td>
</tr>
<tr>
<td>10</td>
<td>Uniform compression in webs of doubly symmetric I-shaped sections</td>
<td>h/tw</td>
<td>$\lambda_p = 1.49\sqrt{E/F_y}$</td>
<td><img src="example4.png" alt="image" /></td>
</tr>
<tr>
<td>11</td>
<td>Flexure in webs of singly-symmetric I-shaped sections</td>
<td>$h_c/t_w$</td>
<td>$\lambda_p = 5.70\sqrt{E/F_y}$</td>
<td><img src="example5.png" alt="image" /></td>
</tr>
<tr>
<td>12</td>
<td>Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds</td>
<td>b/t</td>
<td>$\lambda_p = 1.40\sqrt{E/F_y}$</td>
<td><img src="example6.png" alt="image" /></td>
</tr>
<tr>
<td>13</td>
<td>Flexure in webs of rectangular HSS</td>
<td>h/t</td>
<td>$\lambda_p = 5.70\sqrt{E/F_y}$</td>
<td><img src="example7.png" alt="image" /></td>
</tr>
</tbody>
</table>
In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

a) Calculation of the required strengths for members, connections and other elements using one of the methods specified in Section 10.3.2.2, and
f) Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Section 10.15 shall be satisfied.

10.3.1.1 Member Stability Design Requirements: Individual member stability is provided by satisfying the provisions of Section 10.5, 10.6, 10.7, 10.8, 10.10 and 10.11 etc. Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Section 10.19.

10.3.1.2 System Stability Design Requirements

Lateral stability shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. The overturning effects of drift and the destabilizing influence of gravity loads shall be considered. Force transfer and load sharing between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and combined systems shall satisfy the following specific requirements:

10.3.1.3.1 Braced-Frame and Shear-Wall Systems: In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the effective length factor, $K$, for compression members shall be taken as 1.0, unless structural analysis indicates that a smaller value is appropriate. In braced-frame systems, it is permitted to design the columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

10.3.1.3.2 Moment Frame Systems: In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor $K$ or elastic critical buckling stress, $F_y$, for columns and beam-columns shall be determined as specified in Section 10.3.2.
10.3.1.3.3 Gravity Framing Systems: Columns in gravity framing systems shall be designed based on their actual length \((K = 1.0)\) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. \(P-\Delta\) effects due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

10.3.1.3.4 Combined Systems: The analysis and design of members, connections and other elements in combined systems of moment frames, braced frames, and/or shear walls and gravity frames shall meet the requirements of their respective systems.

**10.3.2 Calculation of Required Strengths**

Except as permitted in Section 10.3.2.2.2, **required strengths** shall be determined using a **second-order analysis** as specified in Section 10.3.2.1. Design by either second- **order** or first-order analysis shall meet the requirements specified in Section 10.3.2.2.

10.3.2.1 **Methods of Second-order Analysis:** Second-order analysis shall conform to the requirements in this section.

10.3.2.1.1 General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both \(P-\Delta\) and \(P-\delta\) effects may be used. The Amplified First-Order Elastic Analysis Method defined in Section 10.3.2.1.2 is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

10.3.2.1.2 Second-Order Analysis by Amplified first-Order Elastic Analysis

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of lateral load resisting systems. The required second-order flexural strength, \(M_r\), and axial strength, \(P_r\), shall be determined as follows:

\[
M_r = B_1 M_{nt} + B_2 M_{lt} \tag{10.3.2.1a}
\]

\[
P_r = P_{nt} + B_2 P_{lt} \tag{10.3.2.1b}
\]

Where,

\[
B_1 = \frac{C_m}{1 - \alpha P_r/P_{el}} \geq 1 \tag{10.3.2.2}
\]

For members subjected to axial compression, \(B_2\) may be calculated based on the first-order estimate \(P_r = P_{nt} + P_{lt}\).

For members in which \(B_2 \leq 1.05\), it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the \(B_2\) amplifier, in other words, \(M_r = B_2 (M_{nt} + M_{lt})\).

\[
B_2 = \frac{1}{1 - \alpha \sum P_{nt}^{\delta}} \geq 1 \tag{10.3.2.3}
\]

And

\[
\alpha = 1.00\text{(LRFD)} \quad \alpha = 1.60 \text{(ASD)}
\]

\(M_r\) = required second-order flexural strength using LRFD or ASD load combinations, N-mm

\(M_{nt}\) = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N-mm

\(M_{lt}\) = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm

\(P_r\) = required second-order axial strength using LRFD or ASD load combinations, N

\(P_{nt}\) = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, N

\(\sum P_{nt}\) = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, N
\(P_a\) = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, N  
\(C_m\) = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:  
For beam-columns not subject to transverse loading between supports in the plane of bending,  
\[C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right)\]  
(10.3.2.4)  
where \(M_1\) and \(M_2\), calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. \(M_1 / M_2\) is positive when the member is bent in reverse curvature, negative when bent in single curvature.  
For beam-columns subjected to transverse loading between supports, the value of \(C_m\) shall be determined either by analysis or conservatively taken as 1.0 for all cases.  
\(P_{el}\) = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sideways, N  
\[P_{el} = \frac{\pi^2EI}{(K_1L)^2}\]  
(10.3.2.5)  
\(\sum P_{e2}\) = elastic critical buckling resistance for the story determined by sideways buckling analysis, N.  
For moment frames, where sideways buckling effective length factors \(K_2\) are determined for the columns, it is permitted to calculate the elastic story sideways buckling resistance as  
\[\sum P_{e2} = \sum \frac{\pi^2EI}{(K_2L)^2}\]  
(10.3.2.6a)  
For all types of lateral load resisting systems, it is permitted to use  
\[\sum P_{e2} = R_M \sum \frac{HL}{\Delta_H}\]  
(10.3.2.6b)  
where  
\(E\) = modulus of elasticity of steel = 200 000 MPa  
\(R_M\) = 1.0 for braced-frame systems;  
= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis  
\(l\) = moment of inertia in the plane of bending, mm\(^4\)  
\(L\) = story height, mm  
\(K_1\) = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used  
\(K_2\) = effective length factor in the plane of bending, calculated based on a sideways buckling analysis  
\(\Delta_H\) = first-order interstory drift due to lateral forces, mm. Where \(\Delta_H\) varies over the plan area of the structure, \(\Delta_H\) shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.  
\(\sum H\) = story shear produced by the lateral forces used to compute \(\Delta_H\), N.  

### 10.3.2.2 Design Requirements  
These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the \textit{required strengths} of members, \textit{connections} and other elements shall be determined by one of the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2, or by the Direct Analysis Method of Section 10.14. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the Direct Analysis Method of Section 10.14.  
For the methods specified in Sections 10.3.2.2.1 or 10.3.2.2.2:  
Analyses shall be conducted according to the design and loading requirements specified in either Section 10.2.3.3 (LRFD) or Section 10.2.3.4 (ASD).  
The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.  

#### 10.3.2.2.1 Design By Second-Order Analysis
Where required strengths are determined by a second-order analysis:
The provisions of Section 10.3.2.1 shall be satisfied.

For design by ASD, analyses 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.

All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of 0.002\(Y_i\), where \(Y_i\) is the design gravity load applied at level \(i\). This minimum lateral load shall be considered independently in two orthogonal directions.

Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using \(K = 1.0\). Otherwise, columns and beam-columns in moment frames shall be designed using a \(K\) factor or column buckling stress, \(F_c\), determined from a sideways buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the \(K\) factor. For braced frames, \(K\) for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

10.3.2.2.2 Design Requirements

Required strengths are permitted to be determined by a first-order analysis, with all members designed using \(K = 1.0\), provided that

The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

\[
a P_r \leq 0.5P_y
\]  

Where,

\[\alpha = 1.0 \text{(LRFD)} \quad \alpha = 1.6 \text{ (ASD)}\]

\(P_r\) = required axial compressive strength under LRFD or ASD load combinations

\(P_y\) = member yield strength (= \(AF_y\)), N.

All load combinations include an additional lateral load, \(N_i\), applied in combination with other loads at each level of the structure, where

\[
N_i = 2.1(\Delta/L)Y_i \geq 0.0042Y_i
\]  

\(Y_i\) = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level \(i\), N
\(\Delta /L\) = the maximum ratio of \(\Delta\) to \(L\) for all stories in the structure
\(\Delta\) = first-order interstory drift due to the design loads, mm. Where \(\Delta\) varies over the plan area of the structure, \(\Delta\) shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.
\(L\) = story height, mm

This additional lateral load shall be considered independently in two orthogonal directions.

The non-sway amplification of beam-column moments is considered by applying the \(B_1\) amplifier of Section 10.3.2.1 to the total member moments.

### 10.4 Design of Members for Tension

This Section applies to steel members subject to axial tension caused by static forces acting through the centroidal axis.

#### 10.4.1 Slenderness Limitations

The maximum slenderness \((KL/r)\) limit for design of structural members (except cables and hanger rods) in tension shall be 300 unless it is justified by a comprehensive dynamic analysis (including 2nd order effects if applicable) that a higher slenderness ratio is satisfactory.

Here,

\(L\) = laterally unbraced length of the member, mm
\(r\) = governing radius of gyration, mm
\(K\) = the effective length factor determined in accordance with Section 10.3.2.
10.4.2 Tensile Strength

The design tensile strength, $\Phi_{L} P_{n}$, and the allowable tensile strength, $P_{n}/\Omega_{t}$, of tension members, shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

a) For tensile yielding in the gross section:

$$P_{n} = F_{y} A_{g}$$  \hspace{1cm} (10.4.2.1)

$$\Phi_{L} = 0.90 \text{ (LRFD)} \quad \Omega_{t} = 1.67 \text{ (ASD)}$$

b) For tensile rupture in the net section:

$$P_{n} = F_{y} A_{e}$$  \hspace{1cm} (10.4.2.2)

$$\Phi_{L} = 0.75 \text{ (LRFD)} \quad \Omega_{t} = 2.00 \text{ (ASD)}$$

where

- $A_{e}$ = effective net area, mm$^2$
- $A_{g}$ = gross area of member, mm$^2$
- $F_{y}$ = specified minimum yield stress of the type of steel being used, MPa
- $F_{u}$ = specified minimum tensile strength of the type of steel being used, MPa

When members without holes are fully connected by welds, the effective net area used in Equation 10.4.2.2 shall be as defined in Section 10.4.3. When holes are present in a member with welded end connections, or at the welded connection in the case of plug or slot welds, the effective net area through the holes shall be used in Equation 10.4.2.2.

10.4.3 Area Determination

10.4.3.1 Gross Area: The gross area, $A_{g}$, of a member is the total cross-sectional area.

10.4.3.2 Net Area: The net area, $A_{e}$, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as 2 mm greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section 10.10.3.2, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/(4g)$

where

- $s$ = longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.
- $g$ = transverse center-to-center spacing (gage) between fastener gage lines, mm.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted HSS welded to a gusset plate, the net area, $A_{n}$, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

10.4.3.3 Effective Net Area: The effective area of tension members shall be determined as follows:

$$A_{e} = A_{n} U$$  \hspace{1cm} (10.4.3.1)

where $U$, the shear lag factor, is determined as shown in Table 10.4.3.1.

Members such as single angles, double angles and WT sections shall have connections proportioned such that $U$ is equal to or greater than 0.60. Alternatively, a lesser value of $U$ is permitted if these tension members are designed for the effect of eccentricity in accordance with Sections 10.8.1.2 or 10.8.2.
# TABLE 10.4.3.1
Shear Lag Factors for Connections to Tension Members

<table>
<thead>
<tr>
<th>Case</th>
<th>Description of Element</th>
<th>Shear Lag Factor, $U$</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)</td>
<td>$U = 1.0$</td>
<td>——</td>
</tr>
<tr>
<td>2</td>
<td>All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)</td>
<td>$U = 1 - \frac{X}{L}$</td>
<td>——</td>
</tr>
<tr>
<td>3</td>
<td>All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.</td>
<td>$U = 1.0$ and $A_n = \text{area of the directly connected elements}$</td>
<td>——</td>
</tr>
</tbody>
</table>
| 4    | Plates where the tension load is transmitted by longitudinal welds only. | $I \geq 2w \ldots U = 1.0$  
$2w > I \geq 1.5w \ldots U = 0.87$  
$1.5w > I \geq w \ldots U = 0.75$ | —— |
| 5    | Round HSS with a single concentric gusset plate | $I \geq 1.3D \ldots U = 1.0$  
$D \leq I < 1.3D \ldots U = 1 - \frac{X}{L}$  
$X = D / \pi$ | —— |
| 6    | Rectangular HSS with a single concentric gusset plate | $I \geq H \ldots U = 1 - \frac{X}{L}$  
$X = \frac{B^2 + 2BH}{4(B + H)}$ | —— |
|      | with two side gusset plates | $I \geq H \ldots U = 1 - \frac{X}{L}$  
$X = \frac{B^2}{4(B + H)}$ | —— |
| 7    | W, M, S or HP Shapes or Tees cut from these shapes. (If $U$ is calculated per Case 2, the larger value is permitted to be used) | $b_f \geq 2/3d \ldots U = 0.90$  
$b_f < 2/3d \ldots U = 0.85$ | —— |
|      | with flange connected with 3 or more fasteners per line in direction of loading | $U = 0.70$ | —— |
|      | with web connected with 4 or more fasteners per line in the direction of loading | $U = 0.80$ | —— |
| 8    | Single angles (If $U$ is calculated per Case 2, the larger value is permitted to be used) | $U = 0.80$ | —— |
|      | with 4 or more fasteners per line in direction of loading | $U = 0.60$ | —— |

$I = \text{length of connection, mm}$  
$w = \text{plate width, mm}$  
$X = \text{connection eccentricity, mm}$  
$B = \text{overal width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm}$  
$H = \text{overall height of rectangular HSS member, measured in the plane of the connection, mm}$
10.4.4 Built-Up Members

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.10.3.5.

Either perforated cover plates or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 150 mm.

10.4.5 Pin-Connected Members

10.4.5.1 Tensile Strength: The design tensile strength, \( P_n \), and the allowable tensile strength, \( P_{n}/\Omega_t \), of pin-connected members, shall be the lower value obtained according to the limit states of tensile rupture, shear rupture, bearing, and yielding.

\[ P_n = 2t_{\text{eff}} F_u \]  
(10.4.5.1)

\[ \varphi_t = 0.75 \text{ (LRFD)} \quad \Omega_t = 2.00 \text{ (ASD)} \]

h) For shear rupture on the effective area:

\[ P_n = 0.6 F_u A_{\text{eff}} \]  
(10.4.5.2)

\[ \varphi_{\text{eff}} = 0.75 \text{ (LRFD)} \quad \Omega_{\text{eff}} = 2.00 \text{ (ASD)} \]

where

\[ A_{\text{eff}} = 2t (a + d /2), \text{ mm}^2 \]

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, mm

\[ b_{\text{eff}} = 2t + 16, \text{ mm} \]

but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

\[ d = \text{pin diameter, mm} \]

\[ t = \text{thickness of plate, mm} \]

i) For bearing on the projected area of the pin, see Section 10.10.7.

j) For yielding on the gross section, use Equation 10.4.2.1.

10.4.5.2 Dimensional Requirements: The pin hole shall be located midway between the edges of the member in the direction normal to the applied force. When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1 mm greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than \( 2b_{\text{eff}} + d \) and the minimum extension, \( a \), beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than 1.33 \( \times b_{\text{eff}} \).

The corners beyond the pin hole are permitted to be cut at 45\(^\circ\) to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

10.4.6 Eyebars

10.4.6.1 Tensile Strength: The available tensile strength of eyebars shall be determined in accordance with Section 10.4.2, with \( A_g \) taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

10.4.6.2 Dimensional Requirements: Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.
The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than 1 mm greater than the pin diameter.

For steels having $F_y$ greater than 485 MPa, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

A thickness of less than 13 mm is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

10.5 Design of Members for Compression

This Section addresses members subject to axial compression through the centroidal axis.

10.5.1 General Provisions

The design compressive strength, $\phi c \cdot P_n$, and the allowable compressive strength, $P_n / \Omega c$, are determined as follows:

The nominal compressive strength, $P_n$, shall be the lowest value obtained according to the limit states of flexural buckling, torsional buckling and flexural-torsional buckling.

For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.

For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, the limit states of torsional or flexural-torsional buckling are also applicable.

$$\phi c = 0.90 \text{ (LRFD)} \quad \Omega c = 1.67 \text{ (ASD)}$$

10.5.2 Slenderness Limitations and effective Length

The effective length factor, $K$, for calculation of column slenderness, $KL/r$, shall be determined in accordance with Section 10.3,

where

$L = \text{laterally unbraced length of the member, mm}$

$r = \text{governing radius of gyration, mm}$

$K = \text{the effective length factor determined in accordance with Section 10.3.2.}$

The maximum limit of slenderness, $KL/r$, for compression members shall be 150 unless a comprehensive analysis including second order effects (including dynamic effects if any) shows that a higher value is justified.

10.5.3 Compressive Strength for Flexural Buckling of Members without Slender elements

This section applies to compression members with compact and noncompact sections, as defined in Section 10.2.4, for uniformly compressed elements.

The nominal compressive strength, $P_n$, shall be determined based on the limit state of flexural buckling.

$$P_n = F_{cr} \cdot A_g$$  \hspace{1cm} (10.5.3.1)

The flexural buckling stress, $F_{cr}$, is determined as follows:

(a) When

$$\frac{KL}{r} \leq 4.7 \sqrt{\frac{E}{F_y}} \quad \text{(or Fe} \geq 0.44 \text{ Fy )}$$

$$= 0.658$$  \hspace{1cm} (10.5.3.2)

(b) When

$$\frac{KL}{r} > 4.7 \sqrt{\frac{E}{F_y}} \quad \text{(or Fe} < 0.44 \text{ Fy )}$$

$$= 0.877$$  \hspace{1cm} (10.5.3.3)

where
Steel Structures

Chapter 10

Bangladesh National Building Code 2011

Fe = elastic critical buckling stress determined according to Equation 10.5.3.4, Section 10.5.4, or the provisions of Section 10.3.2, as applicable,

\[ Fe = \frac{2}{(\frac{1}{G})} \]  

(10.5.3.4)

### 10.5.4 Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender elements

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns with compact and noncompact sections, as defined in Section 10.2.4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section 10.5.5.

The nominal compressive strength, \( P_n \), shall be determined based on the limit states of flexural-torsional and torsional buckling, as follows:

\[ P_n = F_c A_g \]  

(10.5.4.1)

For double-angle and tee-shaped compression members:

\[ = \left( \frac{1}{2} \right) \left[ 1 - \frac{4}{\sqrt{1 - \frac{4}{(+)^2}}} \right] \]  

(10.5.4.2)

where \( F_{cxy} \) is taken as \( F_c \) from Equation 10.5.3.2 or 10.5.3.3, for flexural buckling about the y-axis of symmetry and \( \frac{K_I}{r_y} = \frac{K_I}{r_y} \), and

\[ = \frac{1}{6} \]  

(10.5.4.3)

For all other cases, \( F_c \) shall be determined according to Equation 10.5.3.2 or 10.5.3.3, using the torsional or flexural-torsional elastic buckling stress, \( F_e \), determined as follows:

For doubly symmetric members:

\[ = \frac{1}{2} + \frac{1}{2} \]  

(10.5.4.4)

For singly symmetric members where y is the axis of symmetry:

\[ = \left( \frac{1}{2} \right) \left[ 1 - \frac{4}{\sqrt{1 - \frac{4}{(+)^2}}} \right] \]  

(10.5.4.5)

For unsymmetric members, \( F_e \) is the lowest root of the cubic equation:

\[ (-) (-) (-) - 2(-) \left( \frac{2}{g} \right)^2 - 2(-) \left( \frac{2}{g} \right)^2 = 0 \]  

(10.5.4.6)

where

\( A_g = \) gross area of member, \( \text{mm}^2 \)

\( C_w = \) warping constant, \( \text{mm}^6 \)

\[ \frac{2}{g} = \frac{1}{2} + \frac{1}{6} + \frac{1}{2} \]  

(10.5.4.7)

\[ = 1 - \frac{1}{2} + \frac{1}{2} \]  

(10.5.4.8)

\[ = \frac{2}{(\frac{1}{g})} \]  

(10.5.4.9)

\[ = \frac{2}{(\frac{1}{g})} \]  

(10.5.4.10)

\[ = \left( \frac{1}{2} + \frac{1}{2} \right) \]  

(10.5.4.11)

\( G = \) shear modulus of elasticity of steel = 77 200 MPa

\( I_x, I_y = \) moment of inertia about the principal axes, \( \text{mm}^4 \)

\( K_z = \) torsional constant, \( \text{mm}^4 \)

\( K_z = \) effective length factor for torsional buckling

\( x_0, y_0 = \) coordinates of shear center with respect to the centroid, mm
\( r_0 \)  \( \equiv \) polar radius of gyration about the shear center, mm
\( r_y \)  = radius of gyration about \( y \)-axis, mm

**10.5.5 Single Angle Compression Members**

The nominal compressive strength, \( P_n \), of single angle members shall be determined in accordance with Section 10.5.3 or Section 10.5.7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Section 10.5.5(a) or 10.5.5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt connections; and (3) there are no intermediate transverse loads.

a) For equal-leg angles or unequal-leg angles connected through the longer leg that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

(i) When \( 0 \leq \frac{L}{r_x} \leq 80 \)

\[
= 72 + 0.75 \cdot \quad (10.5.5.1)
\]

(ii) When \( \frac{L}{r_x} > 80 \)

\[
= 32 + 1.25 \cdot \leq 200 \quad (10.5.5.2)
\]

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, \( KL/r \) from Equations 10.5.5.1 and 10.5.5.2 shall be increased by adding \( 4[(bl/bs)^2 - 1] \), but \( KL/r \) of the members shall not be less than 0.95L/rz.

k) For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

(i) When \( 0 \leq \frac{L}{r_x} \leq 75 \)

\[
= 60 + 0.8 \cdot \quad (10.5.5.3)
\]

(ii) When \( \frac{L}{r_x} > 75 \)

\[
= 45 + \cdot \leq 200 \quad (10.5.5.4)
\]

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, \( KL/r \) from Equations 10.5.5.3 and 10.5.5.4 shall be increased by adding \( 6[(bl/bs)^2 - 1] \), but \( KL/r \) of the members shall not be less than 0.82L/rz.

where

\( L \)  = length of member between work points at truss chord centerlines, mm
\( bl \)  = longer leg of angle, mm
\( bs \)  = shorter leg of angle, mm
\( rx \)  = radius of gyration about geometric axis parallel to connected leg, mm
\( rz \)  = radius of gyration for the minor principal axis, mm

Single angle members with different end conditions from those described in Section 10.5.5(a) or (b), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial load and flexure using the provisions of Section 10.8. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the gusset plate or chord shall constitute different end conditions requiring the use of Section 10.8 provisions.

**10.5.6 Built-up Members**

**10.5.6.1 Compressive Strength**
a) The nominal compressive strength of built-up members composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, \( KL/r \) is replaced by \( (KL/r)m \) determined as follows:

(i) For intermediate connectors that are snug-tight bolted:

\[
() = \sqrt{\left(\frac{KL}{r}\right)^2 + \left(\frac{KL}{r}m\right)^2}
\]  
(10.5.6.1)

(ii) For intermediate connectors that are welded or pretensioned bolted:

\[
() = \sqrt{\left(\frac{KL}{r}\right)^2 + 0.82 \frac{2}{(1+\alpha)} \left(\frac{KL}{r}m\right)^2}
\]  
(10.5.6.2)

Where

\[
\left(\frac{KL}{r}\right)_m = \text{modified column slenderness of built-up member}
\]

\[
\left(\frac{KL}{r}\right)_0 = \text{column slenderness of built-up member acting as a unit in the buckling direction being considered}
\]

\( a \) = distance between connectors, mm

\( ri \) = minimum radius of gyration of individual component, mm

\( rib \) = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm

\( \alpha \) = separation ratio = \( h/2\text{rib} \)

\( h \) = distance between centroids of individual components perpendicular to the member axis of buckling, mm

l) The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated cover plates or lacing with tie plates shall be determined in accordance with Sections 10.5.3, 10.5.4, or 10.5.7 subject to the modification given in Section 10.5.6.1 (a).

10.5.6.2 **Dimensional Requirements**

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, \( a \), such that the effective slenderness ratio \( Ka/ri \) of each of the component shapes, between the fasteners, does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration, \( ri \), shall be used in computing the slenderness ratio of each component part. The end connection shall be welded or pre-tensioned bolted with Class A or B faying surfaces.

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to \( 1\frac{1}{2} \) times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required forces. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section 10.10.3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times \( 0.75\sqrt{E/F_y} \), nor 305 mm, when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times \( 1.12\sqrt{E/F_y} \) nor 460 mm.

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section 10.2.4, is assumed to contribute to the available strength provided the following requirements are met:
The width-thickness ratio shall conform to the limitations of Section 10.2.4.

The ratio of length (in direction of stress) to width of hole shall not exceed two.

The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

The periphery of the holes at all points shall have a minimum radius of 38 mm.

As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the available compressive strength of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, L is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing.

For additional spacing requirements, see Section 10.10.3.5.

10.5.7 Members with Slender Elements

This section applies to compression members with slender sections, as defined in Section 10.2.4 for uniformly compressed elements.

The nominal compressive strength, \( P_n \), shall be determined based on the limit states of flexural, torsional and flexural-torsional buckling.

\[
P_n = F_{cr} A_g
\]

\[(10.5.7.1)\]

a) When \( \frac{KL}{r} \leq 4.71 \left( \frac{E}{Qf_y} \right) \) (or \( F_{cr} \geq 0.44 Qf_y \))

\[= [0.658] \]

\[(10.5.7.2)\]

m) When \( \frac{KL}{r} > 4.71 \left( \frac{E}{Qf_y} \right) \) (or \( F_{cr} < 0.44 Qf_y \))

\[= 0.877 \]

\[(10.5.7.3)\]

Where

\( F_{cr} \) = elastic critical buckling stress, calculated using Equations 10.5.3.4 and 10.5.4.4 for doubly symmetric members, Equations 10.5.3.4 and 10.5.4.5 for singly symmetric members, and Equation 10.5.4.6 for unsymmetric members, except for single angles where \( F_{cr} \) is calculated using Equation 10.5.3.4.

\( Q \) = 1.0 for members with compact and noncompact sections, as defined in Section 10.2.4, for uniformly compressed elements

\( = Q_s \) for members with slender-element sections, as defined in Section 10.2.4, for uniformly compressed elements.

10.5.7.1 Slender Unstiffened Elements, \( Q_s \)

The reduction factor \( Q_s \) for slender unstiffened elements is defined as follows:

For flanges, angles, and plates projecting from rolled columns or other compression members:

When \( \frac{b}{r} \leq 0.56 \left( \frac{E}{Qf_y} \right) \)

\[= 1.0 \]

\[(10.5.7.4)\]
When \(0.56\sqrt[6]{E/F_y} < b/t < 1.03\sqrt[6]{E/F_y}\)
\[
\left(1.415 - 0.74 \sqrt[6]{\frac{E}{F_y}}\right)
\]  
(10.5.7.5)

When \(b/t \geq 1.03\sqrt[6]{E/F_y}\)
\[
\frac{0.69}{t}
\]  
(10.5.7.6)

For flanges, angles, and plates projecting from built-up columns or other compression members:

When \(\frac{b}{t} \leq 0.64 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.0
\]  
(10.5.7.7)

When \(0.64 \sqrt[8]{\frac{E}{F_y}} < \frac{b}{t} \leq 1.17 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.415 - 0.65 \sqrt[6]{\frac{E}{F_y}}
\]  
(10.5.7.8)

When \(\frac{b}{t} > 1.17 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 0.90 \sqrt[6]{\frac{E}{F_y}}
\]  
(10.5.7.9)

Where \(k_c = \frac{4}{\sqrt{b/t}}\), and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

For single angles

When \(\frac{b}{t} \leq 0.45 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.0
\]  
(10.5.7.10)

When \(0.45 \sqrt[8]{\frac{E}{F_y}} < \frac{b}{t} \leq 0.91 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.34 - 0.76 \sqrt[6]{\frac{E}{F_y}}
\]  
(10.5.7.11)

When \(\frac{b}{t} > 0.91 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 0.53 \sqrt[6]{\frac{E}{F_y}}
\]  
(10.5.7.12)

Where

\(b = \) full width of longest angle leg, mm

For stems of tees

When \(\frac{d}{t} \leq 0.75 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.0
\]  
(10.5.7.13)

When \(0.75 \sqrt[8]{\frac{E}{F_y}} < \frac{d}{t} \leq 1.03 \sqrt[8]{\frac{E}{F_y}}\)
\[
= 1.908 - 1.22 \sqrt[6]{\frac{E}{F_y}}
\]  
(10.5.7.14)

When \(\frac{d}{t} > 1.03 \sqrt[8]{\frac{E}{F_y}}\)
\[
= \frac{0.69}{t}
\]  
(10.5.7.15)

Where

\(b = \) width of unstiffened compression element, as defined in Section 10.2.4, mm

\(d = \) the full nominal depth of tee, mm

\(t = \) thickness of element, mm
10.5.7.2 **Slender Unstiffened Elements, \( Q_s \)**

The reduction factor, \( Q_a \), for slender stiffened elements is defined as follows:

\[
  = .
\]  \hspace{1cm} (10.5.7.16)

\( A \) = total cross-sectional area of member, mm\(^2\)

\( A_{eff} \) = summation of the effective areas of the cross section based on the reduced effective width, be , mm\(^2\)

The reduced effective width, be , is determined as follows:

For uniformly compressed slender elements, with \( \frac{b}{t} \geq 1.49 \frac{E}{f} \), except flanges of square and rectangular sections of uniform thickness:

\[
  = 1.92 \left[ 1 - \frac{0.34}{(f)} \right] \leq
\]  \hspace{1cm} (10.5.7.17)

Where

\( f \) is taken as \( F_{cr} \) with \( F_{cr} \) calculated based on \( Q = 1.0 \).

For flanges of square and rectangular slender-element sections of uniform thickness with \( \frac{b}{t} \geq 1.40 \frac{E}{f} \):

\[
  = 1.92 \left[ 1 - \frac{0.38}{(f)} \right] \leq
\]  \hspace{1cm} (10.5.7.18)

Where \( f = \frac{P_n}{A_{eff}} \)

For axially-loaded circular sections:

When \( 0.11 \frac{E}{F_y} < \frac{D}{t} < 0.45 \frac{E}{F_y} \):

\[
  = \frac{0.038}{(f)} + \frac{2}{3}
\]  \hspace{1cm} (10.5.7.19)

Where

\( D \) = outside diameter, mm

\( t \) = wall thickness, mm

10.6 **Design of Members for Flexure**

This section applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports.

10.6.1 **General Provisions**

The design flexural strength, \( \phi_b M_n \), and the allowable flexural strength, \( M_n/\Omega_b \), shall be determined as follows:

\( \phi_b = 0.90 \) (LRFD) \( \Omega_b = 1.67 \) (ASD)

and the nominal flexural strength, \( M_n \), shall be determined according to Sections 10.6.2 through 10.6.12.

\( \phi_b \) = The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.
### TABLE 10.6.1.1
Selection Table for the Application of Sub-sections of Section 10.6

<table>
<thead>
<tr>
<th>Sub-section in this provision</th>
<th>Cross Section</th>
<th>Flange Slenderness</th>
<th>Web Slenderness</th>
<th>Limit States</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.6.2</td>
<td></td>
<td>C</td>
<td>C</td>
<td>Y, LTB</td>
</tr>
<tr>
<td>10.6.3</td>
<td></td>
<td>NC, S</td>
<td>C</td>
<td>LTB, FLB</td>
</tr>
<tr>
<td>10.6.4</td>
<td></td>
<td>C, NC, S</td>
<td>C, NC</td>
<td>Y, LTB, FLB, TFY</td>
</tr>
<tr>
<td>10.6.5</td>
<td></td>
<td>C, NC, S</td>
<td>S</td>
<td>Y, LTB, FLB, TFY</td>
</tr>
<tr>
<td>10.6.6</td>
<td></td>
<td>C, NC, S</td>
<td>N/A</td>
<td>Y, FLB</td>
</tr>
<tr>
<td>10.6.7</td>
<td></td>
<td>C, NC, S</td>
<td>C, NC</td>
<td>Y, FLB, WLB</td>
</tr>
<tr>
<td>10.6.8</td>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>Y, LB</td>
</tr>
<tr>
<td>10.6.9</td>
<td></td>
<td>C, NC, S</td>
<td>N/A</td>
<td>Y, LTB, FLB</td>
</tr>
<tr>
<td>10.6.10</td>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>Y, LTB, LLB</td>
</tr>
<tr>
<td>10.6.11</td>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>Y, LTB</td>
</tr>
<tr>
<td>10.6.12</td>
<td>Unsymmetrical shapes</td>
<td>N/A</td>
<td>N/A</td>
<td>All limit states</td>
</tr>
</tbody>
</table>

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

The following terms are common to the equations in this chapter except where noted:

- $C_B$ = lateral-torsional buckling modification factor for non-uniform moment diagrams when both ends of the unsupported segment are braced

\[
C_B = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_A + 4M_B + 3M_C}R_m \leq 3.0
\]  

(10.6.1.1)

Where

- $M_{\text{max}}$ = absolute value of maximum moment in the unbraced segment, N-mm
- $M_A$ = absolute value of moment at quarter point of the unbraced segment, N-mm
M_b = absolute value of moment at centerline of the unbraced segment, N-mm

M_c = absolute value of moment at three-quarter point of the unbraced segment, N-mm

R_m = cross-section monosymmetry parameter

= 1.0, doubly symmetric members

= 1.0, singly symmetric members subjected to single curvature bending

= 0.5 + 2 \left( \frac{I_y}{I_{yc}} \right)^2, singly symmetric members subjected to reverse curvature bending

I_y = moment of inertia about the principal y-axis, mm^4

I_{yc} = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, mm^4

In singly symmetric members subjected to reverse curvature bending, the lateral - torsional buckling strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, C_b = 1.0.

**10.6.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis**

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section 10.2.4.

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

10.6.2.1 Yielding

\[ M_n = M_p = f_y Z_x \]  

where

- \( f_y = \text{specified minimum yield stress} \) of the type of steel being used, MPa
- \( Z_x = \text{plastic section modulus about the x-axis, mm}^3 \)

10.6.2.2 Lateral -Torsional Buckling

a) When \( L_b \leq L_p \), the limit state of lateral-torsional buckling does not apply.

b) When \( L_p < L_b \leq L_r \)

\[ M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \]  

(10.6.2.2)

\( F_{cr} \) when \( L_b > L_r \)

\[ M_n = F_{cr} S_x \leq M_p \]  

(10.6.2.3)

Where

- \( L_b \) = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm

\[ F_{cr} = \frac{C_b \pi^2 E}{(L_b)^2} \left[ 1 + 0.078 \frac{Jc}{S_x R_0 \tau_{ts}} \right] \]  

(10.6.2.4)

and where

- \( E \) = modulus of elasticity of steel = 200 000 MPa
- \( J = \text{torsional constant, mm}^4 \)
- \( S_x = \text{elastic section modulus taken about the x-axis, mm}^3 \)

The limiting lengths \( L_p \) and \( L_r \) are determined as follows:
\[ L_p = 1.76r_y \frac{E}{F_y} \]  
(10.6.2.5)

\[ L_v = 1.95t_{is} \frac{E}{0.7F_y} \sqrt{\frac{J_c}{S_x h_o}} \left[ 1 + \sqrt{1 + 6.76 \left( \frac{0.7F_y S_x h_o}{E J_c} \right)^2} \right] \]  
(10.6.2.6)

where

\[ r_{ts}^2 = \frac{\sqrt{J_y C_w}}{S_x} \]  
(10.6.2.7)

and

For a doubly symmetric I-shape: \( c = 1 \)  
(10.6.2.8a)

For a channel: \[ c = \frac{h_o}{2} \sqrt{\frac{J_y}{C_w}} \]  
(10.6.2.8b)

Where

\( h_o = \) distance between the flange centroids, mm

**10.6.3 Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis**

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section 10.2.4.

The nominal flexural strength, \( M_n \), shall be the lower value obtained according to the *limit states of lateral-torsional buckling* and compression flange local buckling.

**10.6.3.1 Lateral–Torsional Buckling**

For lateral-torsional buckling, the provisions of Section 10.6.2.2 shall apply.

**10.6.3.2 Compression Flange Local Buckling**

a) For sections with noncompact flanges

\[ M_n = M_p - \left( M_p - 0.7F_y S_x \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right) \]  
(10.6.3.1)

q) For sections with slender flanges

\[ M_n = \frac{0.9E k_c S_x}{\lambda^2} \]  
(10.6.3.2)

Where

\[ \lambda = \frac{b_f}{2t_f} \]

\( \lambda_{pf} \) is the limiting slenderness for a compact flange, Table 10.2.4.1

\( \lambda_{rf} \) is the limiting slenderness for a noncompact flange, Table 10.2.4.1

\[ k_c = \frac{4}{\sqrt{h_o C_w}} \] and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

**10.6.4 Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis**

This section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.
The nominal flexural strength, $M_n$, shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

10.6.4.1 *Compression Flange Yielding*

$$ M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} $$ (10.6.4.1)

10.6.4.2 *Lateral-Torsional Buckling*

a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.

r) When $L_p < L_b \leq L_r$

$$ M_n = C_b \left[ R_{pc}M_{yc} - (R_{pc}M_{yc} - F_tS_{xc}) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc}M_{yc} $$ (10.6.4.2)

s) When $L_b > L_r$

$$ M_n = F_c r S_{xc} \leq R_{pc}M_{yc} $$ (10.6.4.3)

Where

$$ M_{yc} = F_y S_{xc} $$ (10.6.4.4)

$$ F_c r = \frac{C_p \pi^2 E (L_b/L_r)^2}{1 + 0.078 \frac{J}{S_{xc} h_0} \left( \frac{L_b}{r} \right)^2} $$ (10.6.4.5)

For $\frac{L_p}{L_y} \leq 0.23$, $J$ shall be taken as zero.

The stress, $F_L$, is determined as follows:

For $\frac{S_{xt}}{S_{xc}} \geq 0.7$

$$ F_L = 0.7 F_y $$ (10.6.4.6a)

For $\frac{S_{xt}}{S_{xc}} < 0.7$

$$ F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5 F_y $$ (10.6.4.6b)

The limiting laterally unbraced length for the limit state of yielding, $L_p$ is,

$$ L_p = 1.1 r_t \frac{E}{F_y} $$ (10.6.4.7)

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, $L_r$, is

$$ L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_0}} \left[ 1 + \sqrt{1 + 6.76 \left( \frac{F_L S_{xc} h_0}{E J} \right)^2} \right] $$ (10.6.4.8)

The web plastification factor, $R_{pc}$, is determined as follows:

For $\frac{t_w}{h_c} \leq \lambda_{pw}$

$$ R_{pc} = \frac{M_p}{M_{yc}} $$ (10.6.4.9a)

For $\frac{t_w}{h_c} > \lambda_{pw}$

$$ R_{pc} = \left[ \frac{M_p}{M_{yc}} - \left( \frac{M_p}{M_{yc}} - 1 \right) \left( \frac{\lambda}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yc}} $$ (10.6.4.9b)
Where
\[ M_p = Z_x F_y \leq 1.6 S_{xc} F_y \]
\[ S_{xc}, S_{xt} = \text{elastic section modulus referred to tension and compression flanges, respectively, mm}^3 \]
\[ \lambda = \frac{h_c}{t_w} \]
\[ \lambda_{pw} = \lambda_p \text{ limiting slenderness for a compact web, Table 10.2.4.1} \]
\[ \lambda_{rw} = \lambda_r \text{ limiting slenderness for a noncompact web, Table 10.2.4.1} \]
The effective radius of gyration for lateral-torsional buckling, \( r_t \), is determined as follows:

For I-shapes with a rectangular compression flange:
\[ r_t = \frac{b_{fc}}{\sqrt{12 \left( \frac{h_w}{d} + \frac{1}{6} a_w \frac{h^2}{h_d d} \right)}} \quad (10.6.4.10) \]

Where
\[ a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \quad (10.6.4.11) \]

\( b_{fc} = \text{compression flange width, mm} \)
\( t_{fc} = \text{compression flange thickness, mm} \)

For I-shapes with channel caps or cover plates attached to the compression flange:
\[ r_t = \text{radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, mm} \]
\( a_w = \text{the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components.} \)

10.6.4.1 **Compression Flange Local Buckling**

a) For sections with compact flanges, the limit state of local buckling does not apply.

10.6.4.4 **Tension Flange Yielding**

a) When \( S_{xt} \geq S_{xc} \), the limit state of tension flange yielding does not apply.

\[ M_n = R_{pc} M_{yc} \quad (10.6.4.14) \]

Where
\( F_L \) is defined in Equations 10.6.4.6a and 10.6.4.6b
\( R_{pc} = \text{is the web plastification factor, determined by Equations 10.6.4.9} \)
\( k_c = \frac{4}{\sqrt{h/t_w}} \) and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes
\( \lambda = \frac{b_{fc}}{2t_{fc}} \)
\( \lambda_{pf} = \lambda_p \text{ limiting slenderness for a compact flange, Table 10.2.4.1} \)
\( \lambda_{rf} = \lambda_r \text{ limiting slenderness for a noncompact flange, Table 10.2.4.1} \)
The web plastification factor corresponding to the tension flange yielding limit state, $R_{pt}$, is determined as follows:

For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}}$$

(10.6.4.15a)

For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[ \frac{M_p}{M_{yt}} - \left( \frac{M_p}{M_{yt}} - 1 \right) \left( \frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}} \right) \right] \leq \frac{M_p}{M_{yt}}$$

(10.6.4.15b)

Where

$$\lambda = \frac{h_c}{t_w}$$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table 10.2.4.1

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table 10.2.4.1

### 10.6.5 Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Section 10.2.4.

The nominal flexural strength, $M_n$, shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling and tension flange yielding.

#### 10.6.5.1 Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc}$$

(10.6.5.1)

#### 10.6.5.2 Lateral-Torsional Buckling

$$M_n = R_{pg} F_y S_{xc}$$

(10.6.5.2)

a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.

w) When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[ F_y - (0.3 F_y) \left( \frac{L_p - L_b}{L_r - L_p} \right) \right] \leq F_y$$

(10.6.5.3)

x) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b r_t)^2} \leq F_y$$

(10.6.5.4)

where

$L_p$ is defined by Equation 10.6.4.7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7 F_y}}$$

(10.6.5.5)

$R_{pg}$ is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300 a_w} \left( \frac{h_c}{t_w} - 5.7 \sqrt{\frac{E}{F_y}} \right) \leq 1.0$$

(10.6.5.6)

$a_w$ is defined by Equation 10.6.4.11 but shall not exceed 10 and

$r_t$ is the effective radius of gyration for lateral buckling as defined in Section 10.6.4.
10.6.5.3 **Compression Flange Local Buckling**

\[ M_n = R_{pg} F_c S_{xc} \quad (10.6.5.7) \]

a) For sections with compact flanges, the limit state of compression flange local buckling does not apply.

y) For sections with noncompact flanges

\[ F_{cr} = \left[ F_y - (0.3 F_y) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (10.6.5.8) \]

z) For sections with slender flanges

\[ F_{cr} = \frac{0.9 E k_c}{\left( \frac{b_f}{2F_f} \right)^2} \quad (10.6.5.9) \]

Where

\[ k_c = \frac{6}{\sqrt{h/t_w}} \]

and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

\[ \lambda = \frac{b_f}{2F_f} \]

\[ \lambda_{pf} = \lambda_p \]

the limiting slenderness for a compact flange, Table 10.2.4.1

\[ \lambda_{rf} = \lambda_r \]

the limiting slenderness for a noncompact flange, Table 10.2.4.1

10.6.5.4 **Tension Flange Yielding**

(a) When \( S_{xt} \geq S_{xc} \), the *limit state* of tension flange *yielding* does not apply.

(b) When \( S_{xt} < S_{xc} \)

\[ M_n = F_y S_{xt} \quad (10.6.5.10) \]

10.6.6 **I-Shaped Members and Channels Bent about Their Minor Axis**

This section applies to i-shaped members and channels bent about their minor axis.

The nominal flexural strength, \( M_n \), shall be the lower value obtained according to the *limit states of yielding* (plastic moment) and flange *local buckling*.

10.6.6.1 **Yielding**

\[ M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \quad (10.6.6.1) \]

10.6.6.2 **Flange Local Buckling**

a) For sections with compact flanges the limit state of yielding shall apply.

aa) For sections with noncompact flanges

\[ M_n = M_p - \left( M_p - 0.7 F_y S_y \right) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \]

bb) For sections with slender flanges

\[ M_n = F_{cr} S_y \]

(10.6.6.3)

where

\[ F_{cr} = \frac{0.69 E}{\left( \frac{b_f}{2F_f} \right)^2} \quad (10.6.6.4) \]

\[ \lambda = \frac{b}{t} \]

\[ \lambda_{pf} = \lambda_p \]

the limiting slenderness for a compact flange, Table 10.2.4.1

\[ \lambda_{rf} = \lambda_r \]

the limiting slenderness for a noncompact flange, Table 10.2.4.1
10.6.7 Square and Rectangular HSS and Box-Shaped Members

This section applies to square and rectangular HSS, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section 10.2.4.

The nominal flexural strength, \( M_n \), shall be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling and web local buckling under pure flexure.

\[
M_n = M_p = F_y Z \quad \text{(10.6.7.1)}
\]

Where

\( Z = \) plastic section modulus about the axis of bending, mm³

\[
10.6.7.1 \text{ Yielding}
\]

\[
M_n = M_p = F_y Z
\]

\[
M_n = M_p - \left( M_p - F_y S \right) \left( 3.57 \frac{b}{t} \sqrt{\frac{F_y}{E} - 4.0} \right) \leq M_p \quad \text{(10.6.7.2)}
\]

cc) For sections with slender flanges

\[
M_n = F_y S_{\text{eff}}
\]

\[
S_{\text{eff}} \text{is the effective section modulus determined with the effective width of the compression flange taken as:}
\]

\[
b_e = 1.92 t \sqrt{\frac{E}{F_y}} \left[ 1 - 0.38 \frac{b}{t} \sqrt{\frac{E}{F_y}} \right] \leq b \quad \text{(10.6.7.4)}
\]

10.6.7.2 Flange Local Buckling

a) For compact sections, the limit state of flange local buckling does not apply.

(b) For sections with noncompact flanges

\[
M_n = M_p - \left( M_p - F_y S \right) \left( 0.305 \frac{h}{t_w} \sqrt{\frac{F_y}{E} - 0.738} \right) \leq M_p \quad \text{(10.6.7.5)}
\]

10.6.8 Round HSS

This section applies to round HSS having D/t ratios of less than 0.45 \( \frac{E}{F_y} \).

The nominal flexural strength, \( M_n \), shall be the lower value obtained according to the limit states of yielding (plastic moment) and local buckling.

\[
M_n = M_p = F_y Z \quad \text{(10.6.8.1)}
\]

10.6.8.2 Local Buckling

a) For compact sections, the limit state of flange local buckling does not apply. (b) For noncompact sections

\[
M_n = \left( 0.021 \frac{E}{D} + F_y \right) S \quad \text{(10.6.8.2)}
\]
ee) For sections with slender walls

\[ M_n = F_{cr}S \]  
(10.6.8.3)

where

\[ F_{cr} = \frac{0.33E}{D} \]  
(10.6.8.4)

\[ S = \text{elastic section modulus, mm}^3 \]

**10.6.9 Tees and Double Angles Loaded in the Plane of Symmetry**

This section applies to tees and double angles loaded in the plane of symmetry. The nominal flexural strength, \( M_n \), shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and flange local buckling.

**10.6.9.1 Yielding**

\[ M_n = M_p \]  
(10.6.9.1)

Where

\[ M_p = F_Y Z_x \leq \frac{1.6 M_p}{\text{for stems in tension}} \]  
(10.6.9.2)

\[ \leq M_p \quad \text{for stems in compression} \]  
(10.6.9.3)

**10.6.9.2 Lateral-Torsional Buckling**

\[ M_n = M_{cr} = \frac{\pi \sqrt{EI_y G_T}}{L_B} \left[ B + \sqrt{1 + B^2} \right] \]  
(10.6.9.4)

Where

\[ B = \pm 2.3 \left( \frac{d}{L_B} \right) \sqrt{\frac{l_y}{f}} \]  
(10.6.9.5)

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the unbraced length, the negative value of B shall be used.

**10.6.9.3 Flange Local Buckling of Tees**

\[ M_n = F_{cr} S_{xc} \]  
(10.6.9.6)

\( S_{xc} \) is the elastic section modulus referred to the compression flange.

\( F_{cr} \) is determined as follows:

For compact sections, the limit state of flange local buckling does not apply.

For noncompact sections

\[ F_{cr} = F_Y \left( 1.19 - 0.50 \left( \frac{b_f}{2t_f} \right) \sqrt{\frac{F_Y}{E}} \right) \]  
(10.6.9.7)

For slender sections

\[ F_{cr} = \frac{0.69E}{\left( \frac{b_f}{2t_f} \right)^2} \]  
(10.6.9.8)

**10.6.10 Single Angle**

This section applies to single angles with and without continuous lateral restraint along their length.
Part 6  
Structural Design

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of geometric axis (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for principal axis bending except where the provision for bending about a geometric axis is permitted.

The nominal flexural strength, \( M_n \), shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling and leg local buckling.

10.6.10.1 Yielding

\[ M_n = 1.5 M_y \]  \hspace{1cm} (10.6.10.1)

where

\( M_y = \) yield moment about the axis of bending, N-mm

10.6.10.2 Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

When \( M_e \leq M_y \)

\[ M_n = \left( 0.92 - \frac{0.17 M_e}{M_y} \right) M_e \]  \hspace{1cm} (10.6.10.2)

When \( M_e > M_y \)

\[ M_n = \left( 1.92 - 1.17 \frac{M_y}{M_e} \right) M_y \leq 1.5 M_y \]  \hspace{1cm} (10.6.10.3)

Where

\( M_e \), the elastic lateral-torsional buckling moment, is determined as follows:

For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint

With maximum compression at the toe

\[ M_e = \frac{0.66E b^4 t C_b}{L^2} \left( 1 + 0.78 \left( \frac{L t}{b^2} \right)^2 - 1 \right) \]  \hspace{1cm} (10.6.10.4a)

With maximum tension at the toe

\[ M_e = \frac{0.66E b^4 t C_b}{L^2} \left( 1 + 0.78 \left( \frac{L t}{b^2} \right)^2 + 1 \right) \]  \hspace{1cm} (10.6.10.4b)

\( M_y \) shall be taken as 0.80 times the yield moment calculated using the geometric section modulus.

For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

\( M_e \) shall be taken as 1.25 times \( M_e \) computed using Equation 10.6.10.4a or 10.6.10.4b.

\( M_y \) shall be taken as the yield moment calculated using the geometric section modulus.

For bending about the major principal axis of equal-leg angles:

\[ M_e = \frac{0.46E b^2 t^2 C_b}{L} \]  \hspace{1cm} (10.6.10.5)

For bending about the major principal axis of unequal-leg angles:

\[ M_e = \frac{4.9E I_w C_b}{L^2} \left( \beta_w^2 + 0.052 \left( \frac{L t}{r_w} \right)^2 + \beta_w \right) \]  \hspace{1cm} (10.6.10.6)

Where
Steel Structures

10.6.10.3 Leg Local Buckling

The limit state of leg local buckling applies when the toe of the leg is in compression.

a) For compact sections, the limit state of leg local buckling does not apply.

ff) For sections with noncompact legs

\[ M_n = F_C S_c \left( 2.43 - 1.72 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \right) \]  

For sections with slender legs

\[ M_n = F_{cr} S_c \]  

Where

\[ F_{cr} = \frac{0.71E}{\left( \frac{b}{T} \right)^2} \]

b = outside width of leg in compression, mm

\( S_c \) = elastic section modulus to the toe in compression relative to the axis of bending, mm\(^3\). For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint, \( S_c \) shall be 0.80 of the geometric axis section modulus.

10.6.11 Rectangular Bars and Rounds

This section applies to rectangular bars bent about either geometric axis and rounds. The nominal flexural strength, \( M_n \), shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling, as required.

10.6.11.1 Yielding

For rectangular bars with \( \frac{L_{bd}}{t^2} \leq \frac{0.08E}{F_y} \) bent about their major axis, rectangular bars bent about their minor axis and rounds:

\[ M_n = M_p = F_y Z \leq 1.6 M_y \]  

10.6.11.2 Lateral-Torsional Buckling

a) For rectangular bars with \( \frac{0.08E}{F_y} < \frac{L_{bd}}{t^2} \leq \frac{1.9E}{F_y} \) bent about their major axis:

\[ M_n = C_b \left[ 1.52 - 0.274 \left( \frac{L_{bd}}{t^2} \right) \left( \frac{F_y}{E} \right) M_y \right] \leq M_p \]  

gg) For rectangular bars with \( \frac{L_{bd}}{t^2} > \frac{1.9E}{F_y} \) bent about their major axis:

\[ M_n = F_{cr} S_x \leq M_p \]  

Where

\( C_b \) is computed using Equation 10.6.1.1 with a maximum value of 1.5.

\( L \) = laterally unbraced length of a member, mm

\( I_z \) = minor principal axis moment of inertia, mm\(^4\)

\( r_z \) = radius of gyration for the minor principal axis, mm

\( t \) = angle leg thickness, mm

\( \beta_w \) = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of \( \beta_w \) shall be used.
\[ F_{cr} = \frac{1.9EG_b}{L_b d} \]  \hspace{1cm} (10.6.11.4)

t = width of rectangular bar parallel to axis of bending, mm

d = depth of rectangular bar, mm

\[ L_b = \text{length between points that are either braced against lateral displacement of the compression region or braced against twist of the cross section, mm} \]

(c) For rounds and rectangular bars bent about their minor axis, the limit state of lateral-torsional buckling need not be considered.

### 10.6.12 Unsymmetrical Shapes

This section applies to all unsymmetrical shapes, except single angles.

The nominal flexural strength, \( M_n \), shall be the lowest value obtained according to the limit states of yielding (yield moment), lateral-torsional buckling and local buckling where

\[ M_n = F_n S \]  \hspace{1cm} (10.6.12.1)

Where

\( S = \text{lowest elastic section modulus relative to the axis of bending, mm}^3 \)

**10.6.12.1 Yielding**

\[ F_n = F_y \]  \hspace{1cm} (10.6.12.2)

**10.6.12.2 Lateral-Torsional Buckling**

\[ F_n = F_{cr} \leq F_y \]  \hspace{1cm} (10.6.12.3)

Where

\( F_{cr} = \text{buckling stress for the section as determined by analysis, MPa} \)

**10.6.12.3 Local Buckling**

\[ F_n = F_{cr} \leq F_y \]  \hspace{1cm} (10.6.12.4)

Where

\( F_{cr} = \text{buckling stress for the section as determined by analysis, MPa} \)

### 10.6.13 Proportions of Beams and Girders

#### 10.6.13.1 Hole reductions

This section applies to rolled or built-up shapes, and cover-plated beams with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the limit states specified in other sections of this Chapter, the nominal flexural strength, \( M_n \), shall be limited according to the limit state of tensile rupture of the tension flange.

For \( F_u A_{fn} \geq Y_t F_y A_{fg} \), the limit state of tensile rupture does not apply.

For \( F_u A_{fn} < Y_t F_y A_{fg} \), the nominal flexural strength, \( M_n \), at the location of the holes in the tension flange shall not be taken greater than:

\[ M_n = \frac{F_u A_{fn}}{A_{fg}} S_k \]  \hspace{1cm} (10.6.13.1)

Where

\( A_{fn} = \text{gross tension flange area, calculated in accordance with the provisions of Section 10.4.3.1, mm}^2 \)

\( A_{fg} = \text{net tension flange area, calculated in accordance with the provisions of Section 10.4.3.2, mm}^2 \)

\( Y_t = \text{1.0 for } F_y / F_u \leq 0.8 \)

\( = 1.1 \text{ otherwise} \)
10.6.13.2 Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

\[
0.1 \leq \frac{l_{yc}}{l_y} \leq 0.9 
\]  

(10.6.13.2)

I-shaped members with slender webs shall also satisfy the following limits:

For \( \frac{a}{h} \leq 1.5 \)

\[
\left( \frac{h}{t_w} \right)_{max} = 11.7 \sqrt{\frac{E}{F_y}} 
\]  

(10.6.13.3)

For \( \frac{a}{h} > 1.5 \)

\[
\left( \frac{h}{t_w} \right)_{max} = \frac{0.42E}{F_y} 
\]  

(10.3.13.4)

Where

\( a = \) clear distance between transverse stiffeners, mm

In unstiffened girders \( h/t_w \) shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

10.6.13.3 Cover Plates

Flanges of welded beams or girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section 10.5.6 or 10.4.4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection or fillet welds. The attachment shall be adequate, at the applicable stress given in Sections 10.10.2.2, 10.10.3.8, or 10.2.3.9 to develop the cover plate’s portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length \( a' \), defined below, and shall be adequate to develop the cover plate’s portion of the strength of the beam or girder at the distance \( a' \) from the end of the cover plate.

When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

\[
a' = w 
\]  

(10.6.13.5)

Where

\( w = \) width of cover plate, mm.

When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

\[
a' = 1.5w 
\]  

(10.6.13.6)

When there is no weld across the end of the plate

\[
a' = 2w 
\]  

(10.6.13.7)
10.6.13.4 Built-Up Beams
Where two or more beams or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section 10.5.6.2. When concentrated loads are carried from one beam to another, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be welded or bolted between the beams.

10.7 Design of Members for Shear
This section addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS sections, and shear in the weak direction of singly or doubly symmetric shapes.

10.7.1 General Provisions
Two methods of calculating shear strength are presented below. The method presented in Section 10.7.2 does not utilize the post buckling strength of the member (tension field action). The method presented in Section 10.7.3 utilizes tension field action.

The design shear strength, $\Phi_{v}V_{n}$, and the allowable shear strength, $V_{n}/\Omega_{v}$, shall be determined as follows.
For all provisions in this section except Section 10.7.2.1a:

10.7.2 Members with Unstiffened or Stiffened Webs

10.7.2.1 Nominal Shear Strength
This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The nominal shear strength, $V_{n}$, of unstiffened or stiffened webs, according to the limit states of shear yielding and shear buckling, is

$$V_{n} = 0.6F_{y}A_{w}C_{w}$$  \hspace{1cm} (10.7.2.1)

For webs of rolled I-shaped members with $h/t_{w} \leq 2.24\sqrt{E/F_{y}}$

$\Phi_{v} = 1.00$ (LRFD) \hspace{0.5cm} $\Omega_{v} = 1.50$ (ASD)

And

$$C_{v} = 1.0$$  \hspace{1cm} (10.7.2.2)

For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, the web shear coefficient, $C_{v}$, is determined as follows:

For $h/t_{w} \leq 1.10\sqrt{k_{v}E/F_{y}}$

$$C_{v} = 1.0$$  \hspace{1cm} (10.7.2.3)

For $1.10\sqrt{k_{v}E/F_{y}} < h/t_{w} \leq 1.37\sqrt{k_{v}E/F_{y}}$

$$C_{v} = \frac{1.10\sqrt{k_{v}E/F_{y}}}{h/t_{w}}$$  \hspace{1cm} (10.7.2.4)

For $h/t_{w} > 1.37\sqrt{k_{v}E/F_{y}}$

$$C_{v} = \frac{1.51E}{(h/t_{w})^{2}F_{y}}$$  \hspace{1cm} (10.7.2.5)

Where

$A_{w}$ = the overall depth times the web thickness, $dt_{w}$, mm$^{2}$

The web plate buckling coefficient, $k_{v}$, is determined as follows:

For unstiffened webs with $h/t_{w} < 260$, $k_{v} = 5$ except for the stem of tee shapes where $k_{v} = 1.2$.
For stiffened webs,
\[ k_v = 5 + \frac{5}{(a/h)^2} \]
\[ = 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[ \frac{260}{(h/t_w)} \right]^2 \]

Where

\( a = \) clear distance between transverse stiffeners, mm
\( h = \) for rolled shapes, the clear distance between flanges less the fillet or corner radii, mm
\( = \) for built-up welded sections, the clear distance between flanges, mm
\( = \) for built-up bolted sections, the distance between fastener lines, mm
\( = \) for tees, the overall depth, mm.

10.7.2.2 Transverse Stiffeners

Transverse stiffeners are not required where \( h/t_w \leq 2.46\sqrt{E/F_y} \), or where the required shear strength is less than or equal to the available shear strength provided in accordance with Section 10.7.2.1 for \( k_v = 5 \).

Transverse stiffeners used to develop the available web shear strength, as provided in Section 10.7.2.1, shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than \( at^w_j \), where

\[ j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \]  \hspace{1cm} (10.7.2.6)

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange force, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 305 mm on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 250 mm.

10.7.3 Tension Field Action

10.7.3.1 Limits on the Use of Tension Field Action

Consideration of tension field action is permitted for flanged members when the web plate is supported on all four sides by flanges or stiffeners. Consideration of tension field action is not permitted for:

- end panels in all members with transverse stiffeners;
- members when \( a/h \) exceeds 3.0 or \( \left[ 260/(h/t_w) \right]^2 \);

\[ 2 A_w/(A_{fc} + A_{ft}) > 2.5; \text{ or} \]
\[ h/b_{fc} \text{ or } b_{ft} > 6.0 \]

where

\( A_{fc} = \) area of compression flange, mm\(^2\)
\( A_{ft} = \) area of tension flange, mm\(^2\)
\( b_{fc} = \) width of compression flange, mm
\( b_{ft} = \) width of tension flange, mm

In these cases, the nominal shear strength, \( V_n \), shall be determined according to the provisions of Section 10.7.2.

10.7.3.2 Nominal Shear Strength with Tension Field Action

When tension field action is permitted according to Section 10.7.3.1, the nominal shear strength, \( V_n \), with tension field action, according to the limit state of tension field yielding, shall be

For \( h/t_w \leq 1.10\sqrt{k_v E/F_y} \)
\[ V_{nt} = 0.6F_yA_w \]  
(10.7.3.1)

For \( h/t_w > 1.10\sqrt{k_v E/F_y} \)

\[ V_n = 0.6F_yA_w \left( C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \]  
(10.7.3.2)

Where

\( k_v \) and \( C_v \) are as defined in Section 10.7.2.1.

### 10.7.3.3 Transverse Stiffeners

Transverse stiffeners subject to tension field action shall meet the requirements of Section 10.7.2.2 and the following limitations:

\[ (b/t)_{st} \leq 0.56 \frac{E}{F_{yst}} \]  
(10.7.3.3)

Where

\( (b/t)_{st} \) = the width-thickness ratio of the stiffener
\( F_{yst} \) = specified minimum yield stress of the stiffener material, MPa
\( C_v \) = coefficient defined in Section 10.7.2.1
\( D_s \) = 1.0 for stiffeners in pairs
     = 1.8 for single angle stiffeners
     = 2.4 for single plate stiffeners
\( V_r \) = required shear strength at the location of the stiffener, N
\( V_c \) = available shear strength; \( \Phi_v, V_n \) (LRFD) or \( V_n/\Omega_v \) (ASD) with \( V_n \) as defined in Section 10.7.3.2, N

### 10.7.4 Single Angles

The nominal shear strength, \( V_n \), of a single angle leg shall be determined using Equation 10.7.2.1 with \( C_v = 1.0 \), \( A_w = bt \) where \( b \) = width of the leg resisting the shear force, mm and \( k_v = 1.2 \).

### 10.7.5 Rectangular HSS and Box Members

The nominal shear strength, \( V_n \), of rectangular HSS and box members shall be determined using the provisions of Section 10.7.2.1 with \( A_w = 2ht \) where \( h \) for the width resisting the shear force shall be taken as the clear distance between the flanges less the inside corner radius on each side and \( t_w = t \) and \( k_v = 5 \). If the corner radius is not known, \( h \) shall be taken as the corresponding outside dimension minus three times the thickness.

### 10.7.6 Round HSS

The nominal shear strength, \( V_n \), of round HSS, according to the limit states of shear yielding and shear buckling, is

\[ V_n = F_{cr}A_g/2 \]  
(10.7.6.1)

Where

\( F_{cr} \) shall be the larger of

\[ F_{cr} = \frac{1.60E}{L_e/\sqrt{D/T}} \]  
(10.7.6.2a)

and
Steel Structures

Chapter 10

Bangladesh National Building Code 2011

6-527

\[ F_{cr} = \frac{0.78E}{(bT)^2} \]  

(10.7.6.2b)

but shall not exceed 0.6 \( F_y \)

\( A_g \) = gross area of section based on design wall thickness, mm²

\( D \) = outside diameter, mm

\( L_v \) = the distance from maximum to zero shear force, mm

\( T \) = design wall thickness, equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal thickness for SAW HSS, mm

10.7.7 Weak Axis Shear in Singly and Doubly Symmetric Shapes

For singly and doubly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength, \( V_n \), for each shear resisting element shall be determined using Equation 10.7.2.1 and Section 10.7.2.1b with \( A_w = b_f t_f \) and \( k_v = 1.2 \).

10.7.8 Beams and girders with Web Openings

The effect of all web openings on the nominal shear strength of steel and composite beams shall be determined. Adequate reinforcement shall be provided when the required strength exceeds the available strength of the member at the opening.

10.8 Design of Members for Combined Forces and Torsion

This section addresses members subject to axial force and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

10.8.1 Doubly and Singly Symmetric Members Subject to Flexure and Axial Force

10.8.1.1 Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which \( 0.1 \leq \left( \frac{I_{yc}}{I_y} \right) \leq 0.9 \), that are constrained to bend about a geometric axis (x and/or y) shall be limited by Equations 10.8.1.1a and 10.8.1.1b, where \( I_{yc} \) is the moment of inertia about the y-axis referred to the compression flange, mm⁴.

For \( \frac{P_r}{P_c} \geq 0.2 \)

\[ \frac{P_r}{P_c} + 8 \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \]  

(10.8.1.1a)

For \( \frac{P_r}{P_c} < 0.2 \)

\[ \frac{P_r}{2P_c} + \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0 \]  

(10.8.1.1b)

Where

\( P_r \) = required axial compressive strength, N

\( P_c \) = available axial compressive strength, N

\( M_r \) = required flexural strength, N-mm

\( M_c \) = available flexural strength, N-mm

\( x \) = subscript relating symbol to strong axis bending

\( y \) = subscript relating symbol to weak axis bending

For design according to Section 10.2.3.3 (LRFD)

\( P_r \) = required axial compressive strength using LRFD load combinations, N
Pc = φc Pn = design axial compressive strength, determined in accordance with Section 10.5, N
Mr = required flexural strength using LRFD load combinations, N-mm
Mc = φb Mn = design flexural strength determined in accordance with Section 10.6, N-mm
φc = resistance factor for compression = 0.90
φb = resistance factor for flexure = 0.90
For design according to Section 10.2.3.4 (ASD)
Pr = required axial compressive strength using ASD load combinations, N
Pc = Pn /φc = allowable axial compressive strength, determined in accordance with Section 10.5, N
Mr = required flexural strength using ASD load combinations, N-mm
Mc = Mn /Ωb = allowable flexural strength determined in accordance with Section 10.6, N-mm
Ωc = safety factor for compression = 1.67
Ωb = safety factor for flexure = 1.67

10.8.1.2 Doubly and Singly Symmetric Members in Flexure and Tension
The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (x and/or y) shall be limited by Equations 10.8.1.1a and 10.8.1.1b,
Where
For design according to Section 10.2.3.3 (LRFD)
Pr = required tensile strength using LRFD load combinations, N
Pc = φt Pn = design tensile strength, determined in accordance with Section 10.4.2, N
Mr = required flexural strength using LRFD load combinations, N-mm
Mc = φb Mn = design flexural strength determined in accordance with Section 10.6, N-mm
φt = resistance factor for tension (see Section 10.4.2)
φb = resistance factor for flexure = 0.90
For doubly symmetric members, Cb in Section 10.6 may be increased by \(\sqrt{1 + \frac{P_{cy}}{P_{ey}}}\) for axial tension that acts concurrently with flexure,
Where
\[P_{ey} = \frac{\pi^2EI_y}{L_b^2}\]
For design according to Section 10.2.3.4 (ASD)
Pr = required tensile strength using ASD load combinations, N
Pc = Pn /φt = allowable tensile strength, determined in accordance with Section 10.4.2, N
Mr = required flexural strength using ASD load combinations, N-mm
Mc = Mn /Ωb = allowable flexural strength determined in accordance with Section 10.6, N-mm
Ωt = safety factor for tension (see Section 10.4.2)
Ωb = safety factor for flexure = 1.67
For doubly symmetric members, Cb in Section 10.6 may be increased by \(\sqrt{1 + \frac{1.5P_{cy}}{P_{ey}}}\) for axial tension that acts concurrently with flexure
Where
\[P_{ey} = \frac{\pi^2EI_y}{L_b^2}\]
A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations 10.8.1.1a and 10.8.1.1b.

10.8.1.3 Doubly Symmetric Members in Single Axis Flexure and Compression
For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent limit states, in-plane instability and out-of-plane buckling or flexural-torsional buckling, separately in lieu of the combined approach provided in Section 10.8.1.1.

a) For the limit state of in-plane instability, Equations 10.8.1.1 shall be used with $P_c$, $M_r$, and $M_c$ determined in the plane of bending.

hh) For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 \leq 1.0$$ (10.8.1.2)

Where

$P_{co}$ = available compressive strength out of the plane of bending, N

$M_{cx}$ = available flexural-torsional strength for strong axis flexure determined from Section 10.6, N-mm

If bending occurs only about the weak axis, the moment ratio in Equation 10.8.1.2 shall be neglected.

For members with significant biaxial moments ($Mr/Mc \geq 0.05$ in both directions), the provisions of Section 10.8.1.1 shall be followed.

### 10.8.2 Unsymmetric and Other Members Subject to Flexure and Axial Force

This section addresses the interaction of flexure and axial stress for shapes not covered in Section 10.8.1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section 10.8.1.

$$\frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \leq 1.0$$ (10.8.2.1)

Where

$f_a$ = required axial stress at the point of consideration, MPa

$F_a$ = available axial stress at the point of consideration, MPa

$f_{bw}, f_{bz}$ = required flexural stress at the point of consideration, MPa

$F_{bw}, F_{bz}$ = available flexural stress at the point of consideration, MPa

$w$ = subscript relating symbol to major principal axis bending

$z$ = subscript relating symbol to minor principal axis bending

**For design according to Section 10.2.3.3 (LRFD)**

$f_a$ = required axial stress using LRFD load combinations, MPa

$F_a = \phi_c F_cr$ = design axial stress, determined in accordance with Section 10.5 for compression or Section 10.4.2 for tension, MPa

$f_{bw}, f_{bz}$ = required flexural stress at the specific location in the cross section using LRFD load combinations, MPa

$F_{bw}, F_{bz} = \frac{\phi_b M_b}{S} = design$ flexural stress determined in accordance with Section 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.

$\phi_c = resistance$ factor for compression = 0.90

$\phi_t = resistance$ factor for tension (Section 10.4.2)

$\phi_b = resistance$ factor for flexure = 0.90

**For design according to Section 10.2.3.4 (ASD)**

$f_a$ = required axial stress using ASD load combinations, MPa

$F_a = \frac{P_c}{\Omega_c}$ = allowable axial stress determined in accordance with Section 10.5 for compression, or Section 10.4.2 for tension, MPa

$f_{bw}, f_{bz}$ = required flexural stress at the specific location in the cross section using ASD load combinations, MPa

$F_{bw}, F_{bz} = \frac{M_b}{\Omega_b S} = allowable$ flexural stress determined in accordance with Section 10.6, MPa. Use the section modulus for the specific location in the cross section and consider the sign of the stress.

$\Omega_c = safety$ factor for compression = 1.67
\( \Omega_t \) = safety factor for tension (Section 10.4.2)
\( \Omega_b \) = safety factor for flexure = 1.67

Equation 10.8.2.1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial force is compression, second order effects shall be included according to the provisions of Section 10.3.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equation 10.8.2.1.

### 10.8.3 Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force

#### 10.8.3.1 Torsional Strength of Round and Rectangular HSS

The design torsional strength, \( \Phi_T T_n \) and the allowable torsional strength, \( T_n / \Omega_T \), for round and rectangular HSS shall be determined as follows:

\( \Phi_T = 0.90 \) (LRFD) \hspace{1em} \( \Omega_T = 1.67 \) (ASD)

The nominal torsional strength, \( T_n \), according to the limit states of torsional yielding and torsional buckling is:

\[ T_n = F_{cr} C \] (10.8.3.1)

Where

\( C \) is the HSS torsional constant
\( F_{cr} \) shall be determined as follows:

For round HSS, \( F_{cr} \) shall be the larger of

\[ F_{cr} = \frac{1.23E}{L \sqrt{D (D/2)^2}} \] (10.8.3.2a)

and

\[ F_{cr} = \frac{0.60E}{(D/2)^2} \] (10.8.3.2b)

but shall not exceed 0.6 \( F_y \),

where

\( L \) = length of the member, mm
\( D \) = outside diameter, mm

For rectangular HSS

For \( h/t \leq 2.45 \sqrt{E/F_y} \)

\[ F_{cr} = 0.6F_y \] (10.8.3.3)

For \( 2.45 \sqrt{E/F_y} < h/t \leq 3.07 \sqrt{E/F_y} \)

\[ F_{cr} = 0.6F_y \left( 2.45 \sqrt{E/F_y} / (h/t) \right) \] (10.8.3.4)

For \( 3.07 \sqrt{E/F_y} < h/t \leq 260 \)

\[ F_{cr} = 0.458\pi^2E / (h/t)^2 \] (10.8.3.5)

#### 10.8.3.2 HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the required torsional strength, \( T_r \), is less than or equal to 20 percent of the available torsional strength, \( T_c \), the interaction of torsion, shear, flexure and/or axial force for HSS shall be determined by Section 10.8.1 and the torsional effects shall be neglected. When \( T_r \) exceeds 20 percent of \( T_c \), the interaction of torsion, shear, flexure and/or axial force shall be limited by...
\[
\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0
\] (10.8.3.6)

Where

For design according to Section 10.2.3.3 (LRFD)

\[ P_r = \text{required axial strength using LRFD load combinations, N} \]
\[ P_c = 0.6 P_n, \text{ design tensile or compressive strength in accordance with Section 10.4 or 10.5, N} \]
\[ M_r = \text{required flexural strength using LRFD load combinations, N-mm} \]
\[ M_c = 0.9 M_n, \text{ design flexural strength in accordance with Section 10.6, N-mm} \]
\[ V_r = \text{required shear strength using LRFD load combinations, N} \]
\[ V_c = 0.9 V_n, \text{ design shear strength in accordance with Section 10.7, N} \]
\[ T_r = \text{required torsional strength using LRFD load combinations, N-mm} \]
\[ T_c = 0.7 T_n, \text{ design torsional strength in accordance with Section 10.8.3.1, N-mm} \]

For design according to Section 10.2.3.4 (ASD)

\[ P_r = \text{required axial strength using ASD load combinations, N} \]
\[ P_c = P_n/\Omega, \text{ allowable tensile or compressive strength in accordance with Section 10.4 or 10.5, N} \]
\[ M_r = \text{required flexural strength using ASD load combinations determined in accordance with Section 10.2.5, N-mm} \]
\[ M_c = M_n/\Omega_b, \text{ allowable flexural strength in accordance with Section 10.6, N-mm} \]
\[ V_r = \text{required shear strength using ASD load combinations, N} \]
\[ V_c = V_n/\Omega_b, \text{ allowable shear strength in accordance with Section 10.7, N} \]
\[ T_r = \text{required torsional strength using ASD load combinations, N-mm} \]
\[ T_c = T_n/\Omega_T, \text{ allowable torsional strength in accordance with Section 10.8.3.1, N-mm} \]

10.8.3.3 Strength of Non-HSS Members under Torsion and Combined Stress

The design torsional strength, \( \Omega_T P_n \), and the allowable torsional strength, \( P_n/\Omega_T \), for non-HSS members shall be the lowest value obtained according to the limit states of yielding under normal stress, shear yielding under shear stress, or buckling, determined as follows:

\[ \Omega_T = 0.90 \text{ (LRFD)} \quad \Omega_T = 1.67 \text{ (ASD)} \]

i) For the limit state of yielding under normal stress
\[ F_n = F_y \] (10.8.3.7)

ii) For the limit state of shear yielding under shear stress
\[ F_n = 0.6 F_y \] (10.8.3.8)

jj) For the limit state of buckling
\[ F_n = F_{cr} \] (10.8.3.9)

where

\( F_{cr} \) = buckling stress for the section as determined by analysis, MPa

Some constrained local yielding is permitted adjacent to areas that remain elastic.

10.9 EVALUATION OF EXISTING STRUCTURES

This section applies to the evaluation of the strength and stiffness under static vertical (gravity) loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section 10.1.3.1. This section does not address load testing for the effects of seismic loads or moving loads (vibrations).

10.9.1 GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the available strength of a load resisting...
member or system. The evaluation shall be performed by structural analysis (Section 10.9.3), by load tests (Section 10.9.4), or by a combination of structural analysis and load tests, as specified in the contract documents. Where load tests are used, the engineer of record shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

10.9.2 MATERIAL PROPERTIES

Determination of Required Tests: The engineer of record shall determine the specific tests that are required from Section 10.9.2.2 through 10.9.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

10.9.2.1 Tensile Properties

Tensile properties of members shall be considered in evaluation by structural analysis (Section 10.9.3) or load tests (Section 10.9.4). Such properties shall include the yield stress, tensile strength and percent elongation. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

10.9.2.2 Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

10.9.2.3 Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section 10.1.3.1d are critical to the performance of the structure, the Charpy V-Notch toughness shall be determined in accordance with the provisions of Section 10.1.3.1d. If the notch toughness so determined does not meet the provisions of Section 10.1.3.1d, the engineer of record shall determine if remedial actions are required.

10.9.2.4 Weld Metal

Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the engineer of record shall determine if remedial actions are required.

10.9.2.5 Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

10.9.3 EVALUATION BY STRUCTURAL ANALYSIS

10.9.3.1 Dimensional Data

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross section dimensions, thicknesses and connection details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.
10.9.3.2 **Strength Evaluation**

Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section 10.2.2.

The available strength of members and connections shall be determined from applicable provisions of Sections 10.2 through 10.19 of this Specification.

10.9.3.3 **Serviceability Evaluation**

Where required, the deformations at service loads shall be calculated and reported.

10.9.4 **EVALUATION BY LOAD TESTS**

10.9.4.1 **Determination of Load Rating by Testing**

To determine the load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the engineer of record’s plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to \(1.2D + 1.6L\), where \(D\) is the nominal dead load and \(L\) is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, \(L\), \(S\), or \(R\) as defined in the Symbols, shall be substituted for \(L\). More severe load combinations shall be used if required by specifications of Chapter 2 of Part 6.

Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

10.9.4.2 **Serviceability Evaluation**

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

10.9.5 **EVALUATION REPORT**

After the evaluation of an existing structure has been completed, the engineer of record shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and connections, is adequate to withstand the load effects.

10.10 **Connections**

This section addresses connecting elements, connectors, and the affected elements of the connected members not subject to fatigue loads.
10.10.1 General Provisions

10.10.1.1 Design Basis
The design strength, $\phi R_m$, and the allowable strength $R_e/\Omega$, of connections shall be determined in accordance with the provisions of this section and the provisions of Section 10.2.

The required strength of the connections shall be determined by structural analysis for the specified design loads, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

10.10.1.2 Simple Connection
Simple connections of beams, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.

10.10.1.3 Moment Connection
End connections of restrained beams, girders, and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section 10.2.6.3.2.

10.10.1.4 Compression Members with Bearing Joints
(a) When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.

(b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii) below. It is permissible to use the less severe of the two conditions:

(i) An axial tensile force of 50 percent of the required compressive strength of the member; or

(ii) The moment and shear resulting from a transverse load equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

10.10.1.5 Splices in Heavy Sections
When tensile forces due to applied tension or flexure are to be transmitted through splices in heavy sections, as defined in Section 10.1.3.1c and 10.1.3.1d, by complete- joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section 10.1.3.1c and 10.1.3.1d, weld access hole details as given in Section 10.10.1.6 and thermal cut surface preparation and inspection requirements as given in 10.1.3.2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

10.10.1.6 Beam Copes and Weld Access Holes
All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than $1\frac{1}{2}$ times the thickness of the material in which the hole is made. The height of the access hole shall be $1\frac{1}{2}$ times the thickness of the material with the access hole, $t_w$, but not less than 25 mm nor does it need to exceed 50 mm. The access hole shall be detailed to provide room for weld backing as needed.

For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the root of the reentrant surface of the access hole. In hot-rolled shapes, and built-up shapes with CJP groove welds that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. No arc of the weld access hole shall have a radius less than 10 mm.

In built-up shapes with fillet or partial-joint-penetration groove welds that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. The access hole shall be
permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

For heavy sections as defined in 10.1.3.1c and 10.1.3.1d, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of splice welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

10.10.1.7 Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single angle, double angle, and similar members.

10.10.1.8 Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Section 10.1.3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

In making welded alterations to structures, existing rivets and high strength bolts tightened to the requirements for slip-critical connections are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional required strength.

10.10.1.9 High-Strength Bolts in Combination with Rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section 10.10.3, high-strength bolts are permitted to be considered as sharing the load with existing rivets.

10.10.1.10 Limitations on Bolted and Welded Connections

Pretensioned joints, slip-critical joints or welds shall be used for the following connections:

Column splices in all multi-story structures over 38 m in height

Connections of all beams and girders to columns and any other beams and girders on which the bracing of columns is dependent in structures over 38 m in height

In all structures carrying cranes of over 50 kN capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports

Connections for the support of machinery and other live loads that produce impact or reversal of load

Snug-tightened joints or joints with ASTM A307 bolts shall be permitted except where otherwise specified.

10.10.2 Welds

All provisions of AWS D1.1 apply under this Specification, with the exception that the provisions of the listed sections apply under this specification in lieu of the cited AWS provisions as follows:

Section 10.10.1.6 in lieu of AWS D1.1 Section 5.17.1
Section 10.10.2.2.1 in lieu of AWS D1.1 Section 2.3.2
Table 10.10.2.2 in lieu of AWS D1.1 Table 2.1
Table 10.10.2.5 in lieu of AWS D1.1 Table 2.3
Table 10.17.1.1 in lieu of AWS D1.1 Table 2.4
Section 10.2.3.9 and Section 10.17 in lieu of AWS D1.1 Section 2, Part C
Section 10.13.2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

10.10.2.1 Groove Welds
10.10.2.1.1 Effective Area
The effective area of groove welds shall be considered as the length of the weld times the effective throat thickness.

The effective throat thickness of a complete-joint-penetration (CJP) groove weld shall be the thickness of the thinner part joined.

Table 10.10.2.1: Effective Throat of Partial-Joint-Penetration Groove Welds

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)</th>
<th>Groove Type (AWS D1.1, Figure 3.3)</th>
<th>Effective Throat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shielded Metal Arc (SMAW)</td>
<td>All</td>
<td>J or U Groove</td>
<td>Depth of Groove</td>
</tr>
<tr>
<td>Gas Metal Arc (GMAW)</td>
<td>All</td>
<td>60° V</td>
<td></td>
</tr>
<tr>
<td>Flux Cored Arc (FCAW)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Submerged Arc (SAW)</td>
<td>F</td>
<td>J or U Groove</td>
<td>Depth of Groove</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60° Bevel or V</td>
<td></td>
</tr>
<tr>
<td>Gas Metal Arc (GMAW)</td>
<td>F, H</td>
<td>45° Bevel</td>
<td>Depth of Groove</td>
</tr>
<tr>
<td>Flux Cored Arc (FCAW)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shielded Metal Arc (SMAW)</td>
<td>All</td>
<td>45° Bevel</td>
<td>Depth of Groove</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Minus 3 mm</td>
</tr>
<tr>
<td>Gas Metal Arc (GMAW)</td>
<td>V, OH</td>
<td>45° Bevel</td>
<td>Depth of Groove</td>
</tr>
<tr>
<td>Flux Cored Arc (FCAW)</td>
<td></td>
<td></td>
<td>Minus 3 mm</td>
</tr>
</tbody>
</table>

The effective throat thickness of a partial-joint-penetration (PJP) groove weld shall be as shown in Table 10.10.2.1.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a formed section, or rectangular HSS shall be as shown in Table 10.10.2.2, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table 10.10.2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

**Table 10.10.2.2: Effective Weld Sizes of Flare Groove Welds**

<table>
<thead>
<tr>
<th>Welding Process</th>
<th>Flare Bevel Groove[a]</th>
<th>Flare V Groove</th>
</tr>
</thead>
<tbody>
<tr>
<td>GMAW and FCAW-G</td>
<td>(\sqrt[4]{R})</td>
<td>(\sqrt[4]{R})</td>
</tr>
<tr>
<td>SMAW and FCAW-S</td>
<td>(\sqrt[4]{6R})</td>
<td>(\sqrt[4]{R})</td>
</tr>
<tr>
<td>SAW</td>
<td>(\sqrt[4]{6R})</td>
<td>(\sqrt{2R})</td>
</tr>
</tbody>
</table>

[a] For Flare Bevel Groove with \(R < 10\) mm use only reinforcing fillet weld on filled flush joint. General Note: \(R = \text{radius of joint surface (can be assumed to be } 2t \text{ for HSS), mm}\)

Larger effective throat thicknesses than those in Table 10.10.2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

10.10.2.1.2 Limitations
The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated forces nor the size shown in Table 10.10.2.3. Minimum weld size is determined by the thinner of the two parts joined.
TABLE 10.10.2.3: Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

<table>
<thead>
<tr>
<th>Material Thickness of Thinner Part Joined, mm</th>
<th>Minimum Effective Throat Thickness, ([\text{mm}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>To 6 inclusive</td>
<td>3</td>
</tr>
<tr>
<td>Over 6 to 13</td>
<td>5</td>
</tr>
<tr>
<td>Over 13 to 19</td>
<td>6</td>
</tr>
<tr>
<td>Over 19 to 38</td>
<td>8</td>
</tr>
<tr>
<td>Over 38 to 57</td>
<td>10</td>
</tr>
<tr>
<td>Over 57 to 150</td>
<td>13</td>
</tr>
</tbody>
</table>

[a] See Table 10.10.2.1.

10.10.2.2 Fillet Welds

10.10.2.2.1 Effective Area

The effective area of a fillet weld shall be the effective length multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

10.10.2.2 Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated forces nor the size as shown in Table 10.10.2.4. These provisions do not apply to fillet weld reinforcements of partial- or complete-joint-penetration groove welds.

TABLE 10.10.2.4: Minimum Size of Fillet Welds

<table>
<thead>
<tr>
<th>Material Thickness of Thinner ([\text{mm}])</th>
<th>Minimum Size of Fillet ([\text{mm}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>To 6 inclusive</td>
<td>3</td>
</tr>
<tr>
<td>Over 6 to 13</td>
<td>5</td>
</tr>
<tr>
<td>Over 13 to 19</td>
<td>6</td>
</tr>
</tbody>
</table>

[a] Leg dimension of fillet welds. Single pass welds must be used.

The maximum size of fillet welds of connected parts shall be:

Along edges of material less than 6 mm thick, not greater than the thickness of the material.

Along edges of material 6 mm or more in thickness, not greater than the thickness of the material minus 2 mm, unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 2 mm provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed 1/4 of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section 10.4.3.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, \(\beta\),

\[
\beta = 1.2 - 0.002(L/w) \leq 1.0
\]  

(10.10.2.1)

where
\( L \) = actual length of end-loaded weld, mm  
\( w \) = weld leg size, mm  

When the length of the weld exceeds 300 times the leg size, the value of \( \beta \) shall be taken as 0.60.

Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces when the required strength is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall not be less than four times the weld size, with a minimum of 38 mm.

In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 25 mm. Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following:

For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.

For connections where flexibility of the outstanding elements is required, when end returns are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.

Fillet welds joining transverse stiffeners to plate girder webs 19 mm thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of stiffeners are welded to the flange.

Fillet welds that occur on opposite sides of a common plane shall be interrupted at the corner common to both welds.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section 10.10.2. Fillet welds in holes or slots are not to be considered plug or slot welds.

10.10.2.3 Plug and Slot Welds

10.10.2.3.1 Effective Area

The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

10.10.2.3.2 Limitations

Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus 8 mm, rounded to the next larger odd even mm, nor greater than the minimum diameter plus 3 mm or \( 2\frac{1}{2} \) times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 8 mm rounded to the next larger odd even mm, nor shall it be larger than \( 2\frac{1}{4} \) times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material 16 mm or less in thickness shall be equal to the thickness of the material. In material over 16 mm thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than 16 mm.

10.10.2.4 Strength
The design strength, \( \phi R_n \) and the allowable strength, \( R_n / \Omega \), of welds shall be the lower value of the base material and the weld metal strength determined according to the limit states of tensile rupture, shear rupture or yielding as follows:

**TABLE 10.10.2.5: Available Strength of Welded Joints, N**

<table>
<thead>
<tr>
<th>Load Type and Direction Relative to Weld Axis</th>
<th>Pertinent Metal ( \phi ) and ( \Omega )</th>
<th>Nominal Strength ( (F_{BM} \text{ or } F_{W}) )</th>
<th>Effective Area ( (A_{BM} \text{ or } A_{W}) ) ( \text{mm}^2 )</th>
<th>Required Filler Metal Strength Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Normal to weld axis</td>
<td>Strength of the joint is controlled by the base metal</td>
<td>Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section 10.10.2.6.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression Normal to weld axis</td>
<td>Strength of the joint is controlled by the base metal</td>
<td>Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or Compression Parallel to weld axis</td>
<td>Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.</td>
<td>Filler metal with a strength level equal to or less than matching filler metal is permitted.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>Strength of the joint is controlled by the base metal</td>
<td>Matching filler metal shall be used.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS**

<table>
<thead>
<tr>
<th>Load Type and Direction Relative to Weld Axis</th>
<th>Pertinent Metal ( \phi ) and ( \Omega )</th>
<th>Nominal Strength ( (F_{BM} \text{ or } F_{W}) )</th>
<th>Effective Area ( (A_{BM} \text{ or } A_{W}) ) ( \text{mm}^2 )</th>
<th>Required Filler Metal Strength Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Normal to weld axis</td>
<td>( \phi = 0.90 ) See</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression Column to Base Plate and column splices designed per 10.10.1.4(a)</td>
<td></td>
<td>Compressive stress need not be considered in design of welds joining the parts.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression Connections of members designed to bear other than columns as described in 10.10.1.4(b)</td>
<td>Base ( \phi = 0.90 ) ( F_y ) See</td>
<td>( \phi = 0.80 ) 0.60 See</td>
<td>( \Omega = 1.67 ) 10.10.4</td>
<td></td>
</tr>
<tr>
<td>Compression Connections not finished-to-bear</td>
<td>( \phi = 0.90 ) See</td>
<td>( \phi = 0.80 ) See</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or Compression Parallel to weld axis</td>
<td>Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.</td>
<td></td>
<td></td>
<td>Filler metal with a strength level equal to or less than matching filler metal is permitted.</td>
</tr>
<tr>
<td>Shear</td>
<td>Base Governed by 10.10.4</td>
<td></td>
<td>( \phi = 0.60 F_{XX} ) See</td>
<td></td>
</tr>
</tbody>
</table>
Table 10.10.2.5 (Contd.): Available Strength of Welded Joints, N

<table>
<thead>
<tr>
<th>Load Type and Direction Relative to Weld Axis</th>
<th>Pertinent Metal</th>
<th>Φ and Ω</th>
<th>Nominal Strength</th>
<th>Effective Area</th>
<th>Required Filler Metal Strength Level[a][b]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKewed T-JOINTS</td>
<td>Base</td>
<td>Governed by 10.10.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td>Filled by 10.10.4</td>
<td></td>
<td></td>
<td></td>
<td>Filler metal with a strength level equal to or less than matching filler metal is permitted.</td>
</tr>
<tr>
<td>Weld</td>
<td>0.75</td>
<td>0.60F_{EXX}^{[d]}</td>
<td>10.10.2.1</td>
<td>See</td>
<td></td>
</tr>
<tr>
<td>Ω = 2.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or Compression Parallel to weld axis</td>
<td>Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Parallel to faying surface on the effective</td>
<td>Base</td>
<td>Governed by 10.10.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Φ = 0.60F_{EXX}^{[d]}</td>
<td></td>
<td></td>
<td>Filler metal with a strength level equal to or less than matching filler metal is permitted.</td>
</tr>
</tbody>
</table>

For matching weld metal see AWS D1.1, Section 3.3.

Filler metal with a strength level one strength level greater than matching is permitted.

Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, \( \phi = 0.80, \Omega = 1.88 \) and \( 0.60F_{EXX}^{[d]} \) as the nominal strength.

For the base metal

\[
R_n = F_{BM}A_{BM} \quad (10.10.2.2)
\]

For the weld metal

\[
R_n = F_wA_w \quad (10.10.2.3)
\]

Where

\( F_{BM} = nominal strength \) of the base metal per unit area, MPa
\( F_w = nominal strength \) of the weld metal per unit area, MPa
\( A_{BM} = cross-sectional area \) of the base metal, mm²
\( A_w = effective area \) of the weld, mm²

The values of \( \phi, \Omega, F_{BM}, \) and \( F_w \) and limitations thereon are given in Table 10.10.2.5.

Alternatively, for fillet welds loaded in-plane the design strength, \( \phi R_n \) and the allowable strength, \( R_n / \Omega \), of welds is permitted to be determined as follows:

\( \phi = 0.75 \) (LRFD) \( \Omega = 2.00 \) (ASD)

For a linear weld group loaded in-plane through the center of gravity

\[
R_n = F_wA_w \quad (10.10.2.4)
\]

where

\[
F_w = 0.60F_{EXX}^{[d]}(1.0 + 0.50\sin^{1.5}\theta) \quad (10.10.2.5)
\]

And

\( F_{EXX} \) = electrode classification number, MPa
\( \theta \) = angle of loading measured from the weld longitudinal axis, degrees
\( A_w \) = effective area of the weld, mm²
For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the nominal strength, \( R_{nx} \) and \( R_{ny} \), are permitted to be determined as follows:

\[
R_{nx} = \sum F_{wx}A_{wi} \quad R_{ny} = \sum F_{wy}A_{wi}
\]

(10.10.2.6)

where

\( A_{wi} = \) effective area of weld throat of any \( i \)th weld element, \( \text{mm}^2 \)

\[
F_{wi} = 0.60F_{EX}[(1.0 + 0.50\sin^{1.5}\theta)]f(p)
\]

(10.10.2.7)

\[
f(p) = [(p(1.9 - 0.9p)]^{0.3}
\]

(10.10.2.8)

\( F_{wi} \) = nominal stress in any \( i \)th weld element, MPa

\( F_{wx} \) = \( x \) component of stress, \( F_{wi} \)

\( F_{wy} \) = \( y \) component of stress, \( F_{wi} \)

\( p = \Delta_l/\Delta_m \), ratio of element \( i \) deformation to its deformation at maximum stress

\( w = \) weld leg size, mm

\( r_{crit} = \) distance from instantaneous center of rotation to weld element with minimum \( \Delta_l/r_i \) ratio, mm

\( \Delta_l = \) deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, \( r_i, \text{mm} = r_i \Delta_u/r_{crit} \)

\( \Delta_m = 0.209(0+2)^{-0.32}w \), deformation of weld element at maximum stress, mm

\( \Delta_u = 1.087(0+6)^{-0.65}w \leq 0.17w \), deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm

For fillet weld groups concentrically loaded and consisting of elements that are oriented both longitudinally and transversely to the direction of applied load, the combined strength, \( R_n \), of the fillet weld group shall be determined as the greater of

\[
R_n = R_{wi} + R_{wt}
\]

(10.10.2.9a)

or

\[
R_n = 0.85R_{wi} + 1.5R_{wt}
\]

(10.10.2.9b)

where

\( R_{wt} \) = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 10.10.2.5, N

\( R_{wt} \) = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 10.10.2.5 without the alternate in Section 10.10.2.4(a), N

10.10.2.5 Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

10.10.2.6 Filler Metal Requirements

The choice of electrode for use with complete-joint-penetration groove welds subject to tension normal to the effective area shall comply with the requirements for matching filler metals given in AWS D1.1.

Filler metal with a specified Charpy V-Notch (CVN) toughness of 27.12 N·m (27 J) at 4 °C shall be used in the following joints:

Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor as applicable for a PJP weld.

Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in 10.1.3.1c and 10.1.3.1d.
10.10.2.7 Mixed Weld Metal
When Charpy V-Notch toughness is specified, the process consumables for all weld metal, tack welds, root pass and subsequent passes deposited in a joint shall be compatible to ensure notch-tough composite weld metal.

10.10.3 Bolts and Threaded Parts

10.10.3.1 High-Strength Bolts
Use of high-strength bolts shall conform to the provisions of the Specification for Structural Joints Using ASTM A325 or A490 Bolts, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. All ASTM A325 or A325M and A490 or A490M bolts shall be tightened to a bolt tension not less than that given in Table 10.10.3.1, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

Bolts are permitted to be installed to only the snug-tight condition when used in tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations.

| TABLE 10.10.3.1 : Minimum Bolt Pretension, kN<sup>x</sup> |
|--------------------------|------------------|------------------|
| Bolt Size, mm | A325M Bolts | A490M Bolts |
| M16            | 91              | 114             |
| M20            | 142             | 179             |
| M22            | 176             | 221             |
| M24            | 205             | 257             |
| M27            | 267             | 334             |

<sup>x</sup> Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When ASTM A490 or A490M bolts over 25 mm in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with 8 mm minimum thickness, shall be used in lieu of the standard washer.

In slip-critical connections in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Section 10.10.3.10.

When bolt requirements cannot be provided by ASTM A325 and A325M, F1852, or A490 and A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding 38 mm, bolts or threaded rods conforming to ASTM A354 Gr. BC, A354 Gr. BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table 10.10.3.2.

When ASTM A354 Gr. BC, A354 Gr. BD, or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the head and nut(s) shall be equal to or (if larger in diameter) proportional to that provided by ASTM A325 and A325M, or ASTM A490 and A490M bolts. Installation shall comply with all applicable requirements of the RCSC Specification with modifications as required for the increased diameter and/ or length to provide the design pretension.

10.10.3.2 Size and Use of Holes
The maximum sizes of holes for bolts are given in Table 10.10.3.3, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in column base details.

Standard holes or short-slotted holes transverse to the direction of the load shall be provided in accordance with the provisions of this specification, unless over-sized holes, short-slotted holes parallel to the load or long-slotted holes are approved by the engineer of record. Finger shims up to 6 mm are permitted in slip-critical
connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 8 mm thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

10.10.3.3 Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than \( \frac{2}{5} d \) times the nominal diameter, \( d \), of the fastener; a distance of 3d is preferred.

10.10.3.4 Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table 10.10.3.4, or as required in Section 10.10.3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment \( C_2 \) from Table 10.10.3.5.

<table>
<thead>
<tr>
<th>TABLE 10.10.3.2: Nominal Stress of Fasteners and Threaded Parts, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Description of Fasteners</strong></td>
</tr>
<tr>
<td>A307 bolts</td>
</tr>
<tr>
<td>A325 or A325M bolts, when threads are not excluded from shear planes</td>
</tr>
<tr>
<td>A325 or A325M bolts, when threads are excluded from shear planes</td>
</tr>
<tr>
<td>A490 or A490M bolts, when threads are not excluded from shear planes</td>
</tr>
<tr>
<td>A490 or A490M bolts, when threads are excluded from shear planes</td>
</tr>
<tr>
<td>Threaded parts meeting the requirements of Section 10.1.3.4, when threads are not excluded from shear planes</td>
</tr>
<tr>
<td>Threaded parts meeting the requirements of Section 10.1.3.4, when threads are excluded from shear planes</td>
</tr>
</tbody>
</table>
[a] Subject to the requirements of Section 10.17.

[b] For A307 bolts the tabulated values shall be reduced by 1 percent for each 2 mm over 5 diameters of length in the grip.

[c] Threads permitted in shear planes.

[d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, \( AD \), which shall be larger than the nominal body area of the rod before upsetting times \( F_y \).

### TABLE 10.10.3.3: Nominal Hole Dimensions, mm

<table>
<thead>
<tr>
<th>Bolt Diameter</th>
<th>Hole Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard (Dia.)</td>
</tr>
<tr>
<td>M16</td>
<td>18</td>
</tr>
<tr>
<td>M20</td>
<td>22</td>
</tr>
<tr>
<td>M22</td>
<td>24</td>
</tr>
<tr>
<td>M24</td>
<td>27 [a]</td>
</tr>
<tr>
<td>M27</td>
<td>30</td>
</tr>
<tr>
<td>M30</td>
<td>33</td>
</tr>
</tbody>
</table>

[a] Clearance provided allows the use of a 25mm. bolt if desirable.

### TABLE 10.10.3.4: Minimum Edge Distance, [a] mm, from Center of Standard Hole [b] to Edge of Connected Part

<table>
<thead>
<tr>
<th>Bolt Diameter (mm)</th>
<th>At Sheared Edges</th>
<th>At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>28</td>
<td>22</td>
</tr>
<tr>
<td>20</td>
<td>34</td>
<td>26</td>
</tr>
<tr>
<td>22</td>
<td>38 [d]</td>
<td>28</td>
</tr>
<tr>
<td>24</td>
<td>42 [d]</td>
<td>30</td>
</tr>
<tr>
<td>27</td>
<td>48</td>
<td>34</td>
</tr>
</tbody>
</table>

[a] Lesser edge distances are permitted to be used provided provisions of Section 10.10.3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table 10.10.3.5.

[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.

### TABLE 10.10.3.5: Values of Edge Distance Increment \( C_2 \), mm

<table>
<thead>
<tr>
<th>Nominal Diameter of Fastener (mm)</th>
<th>Oversized Holes</th>
<th>Short Slots</th>
<th>Long Slots</th>
<th>Long Axis Perpendicular to Edge</th>
<th>Long Axis Parallel to Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td>0.75d</td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>3</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[When length of slot is less than maximum allowable (see Table 10.10.3.1)], \( C_2 \) is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.
10.10.3.5 Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 150 mm. The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a

For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 305 mm.

For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 180 mm.

10.10.3.6 Tension and Shear Strength of Bolts and Threaded Parts

The design tension or shear strength, \( \phi R_n \), and the allowable tension or shear strength, \( R_n/\Omega \), of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the limit states of tensile rupture and shear rupture as follows:

\[
R_n = F_n A_b
\]

\( \phi = 0.75 \) (LRFD) \( \Omega = 2.00 \) (ASD)

where

\( F_n \) = nominal tensile stress \( F_{nt} \), or shear stress, \( F_{nv} \) from Table 10.10.3.2, MPa

\( A_b \) = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table 10.10.3.2, mm²

The required tensile strength shall include any tension resulting from prying action produced by deformation of the connected parts.

10.10.3.7 Combined Tension and Shear in Bearing-Type Connection

The available tensile strength of a bolt subjected to combined tension and shear shall be determined according to the limit states of tension and shear rupture as follows:

\[
R_n = F'_{nt} A_b
\]

\( \phi = 0.75 \) (LRFD) \( \Omega = 2.00 \) (ASD)

where

\( F'_{nt} \) = nominal tensile stress modified to include the effects of shearing stress, MPa

\[
F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\Omega} f_v \leq F_{nt}(LRFD)
\]  

\[
F'_{nt} = 1.3 F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt}(ASD)
\]

\( F_{nt} \) = nominal tensile stress from Table 10.10.3.2, MPa

\( F_{nv} \) = nominal shear stress from Table 10.10.3.2, MPa

\( f_v \) = the required shear stress, MPa

The available shear stress of the fastener shall equal or exceed the required shear strength per unit area, \( f_v \).

10.10.3.7 High-Strength Bolts in Slip-Critical Connections

High-strength bolts in slip-critical connections are permitted to be designed to prevent slip either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections 10.10.3.6 and 10.10.3.7 and bearing strength in accordance with Sections 10.10.3.1 and 10.10.3.10.

Slip-critical connections shall be designed as follows, unless otherwise designated by the engineer of record. Connections with standard holes or slots transverse to the direction of the load shall be designed for slip as a serviceability limit state. Connections with oversized holes or slots parallel to the direction of the load shall be designed to prevent slip at the required strength level.

The design slip resistance, \( \phi R_n \), and the allowable slip resistance, \( R_n/\Omega \), shall be determined for the limit state of slip as follows:
\[ R_n = \mu D_u h_{sc} T_b N_s \]  
\( (10.10.3.4) \)

For connections in which prevention of slip is a serviceability limit state
\[ \phi = 1.00 \) (LRFD) \quad \Omega = 1.50 \) (ASD) \)

For connections designed to prevent slip at the required strength level
\[ \phi = 0.85 \) (LRFD) \quad \Omega = 1.76 \) (ASD) \)

Where
\[ \mu \] = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests
\[ = 0.35 \) for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)
\[ = 0.50 \) for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)
\[ D_u = 1.13 \) a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values may be approved by the engineer of record.
\[ h_{sc} \] = hole factor determined as follows:
  a) For standard size holes \( h_{sc} = 1.00 \)
  c) For oversized and short-slotted holes \( h_{sc} = 0.85 \)
  d) For long-slotted holes \( h_{sc} = 0.70 \)

\( N_s \) = number of slip planes
\( T_b \) = minimum fastener tension given in Table 10.10.3.1, kN

10.10.3.8 Combined Tension and Shear in Slip-Critical Connections

When a slip-critical connection is subjected to an applied tension that reduces the net clamping force, the available slip resistance per bolt, from Section 10.10.3.8, shall be multiplied by the factor, \( k_s \), as follows:

\[ k_s = 1 - \frac{T_u}{D_u T_b N_b} \) (LRFD) \)
\[ (10.10.3.5a) \]
\[ k_s = 1 - \frac{1.5 T_a}{D_u T_b N_b} \) (ASD) \)
\[ (10.10.3.5b) \]

where
\( N_b \) = number of bolts carrying the applied tension
\( T_a \) = tension force due to ASD load combinations, kN
\( T_b \) = minimum fastener tension given in Table 10.10.3.1, kN
\( T_u \) = tension force due to LRFD load combinations, kN

10.10.3.10 Bearing Strength at Bolt Holes

The available bearing strength, \( \phi R_n \) and \( R_n/\Omega \) at bolt holes shall be determined for the limit state of bearing as follows:
\[ \phi = 0.75 \) (LRFD) \quad \Omega = 2.00 \) (ASD) \)

For a bolt in a connection with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force:

  a) When deformation at the bolt hole at service load is a design consideration
\[ R_n = 1.2 L_c t F_u \leq 2.4 dt F_u \) \)
\[ (10.10.3.6a) \]
  e) When deformation at the bolt hole at service load is not a design consideration
\[ R_n = 1.5 L_c t F_u \leq 3.0 dt F_u \) \)
\[ (10.10.3.6b) \]

For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:
\[ R_n = 1.0 L_c t F_u \leq 2.0 dt F_u \]  
\[ (10.10.3.6c) \]
For connections made using bolts that pass completely through an unstiffened box member or HSS see Section 10.10.7 and Equation 10.10.7.1,

where
\[ d = \text{nominal bolt diameter, mm} \]
\[ F_s = \text{specified minimum tensile strength of the connected material, MPa} \]
\[ L_c = \text{clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, mm} \]
\[ t = \text{thickness of connected material, mm} \]

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

Bearing strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section 10.10.3.2.

10.10.3.9 Special Fasteners
The nominal strength of special fasteners other than the bolts presented in Table 10.10.3.2 shall be verified by tests.

10.10.3.10 Tension Fasteners
When bolts or other fasteners in tension are attached to an unstiffened box or HSS wall, the strength of the wall shall be determined by rational analysis.

10.10.4 Affected Elements of Members and Connecting Elements
This section applies to elements of members at connections and connecting elements, such as plates, gussets, angles, and brackets.

10.10.4.1 Strength of Elements in Tension
The design strength, \( \phi R_n \) and the allowable strength, \( R_n/\Omega \), of affected and connecting elements loaded in tension shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture.

For tensile yielding of connecting elements:

\[ R_n = F_y A_g \]  
(10.10.4.1)

\( \phi = 0.90 \) (LRFD) \( \Omega = 1.67 \) (ASD)

For tensile rupture of connecting elements:

\[ R_n = F_u A_e \]  
(10.10.4.2)

\( \phi = 0.75 \) (LRFD) \( \Omega = 2.00 \) (ASD)

where
\[ A_e = \text{effective net area as defined in Section 10.4.3.3, mm}^2 \] ; for bolted splice plates, \( A_e = A_n \leq 0.85 A_g \)

10.10.4.2 Strength of Elements in Shear
The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

For shear yielding of the element:

\[ R_n = 0.60F_y A_g \]  
(10.10.4.3)

\( \phi = 1.00 \) (LRFD) \( \Omega = 1.50 \) (ASD)

For shear rupture of the element:

\[ R_n = 0.60F_u A_{nv} \]  
(10.10.4.4)

\( \phi = 0.75 \) (LRFD) \( \Omega = 2.00 \) (ASD)
where

\[ A_{nv} = \text{net area subject to shear, mm}^2 \]

10.10.4.3 Block Shear Strength

The available strength for the limit state of block shear rupture along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

\[ R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.6F_y A_{gy} + U_{bs}F_u A_{nt} \quad (10.10.4.5) \]

\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where

\[ A_{gy} = \text{gross area subject to shear, mm}^2 \]
\[ A_{nt} = \text{net area subject to tension, mm}^2 \]
\[ A_{nv} = \text{net area subject to shear, mm}^2 \]

Where the tension stress is uniform, \( U_{bs} = 1 \); where the tension stress is non-uniform, \( U_{bs} = 0.5 \).

10.10.4.3 Strength of Elements in Compression

The available strength of connecting elements in compression for the limit states of yielding and buckling shall be determined as follows.

For \( KL/r \leq 25 \)

\[ P_n = F_y A_g \quad (10.10.4.6) \]

\[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]

For \( KL/r > 25 \) the provisions of Section 10.5 apply.

10.10.5 Fillers

In welded construction, any filler 6 mm or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than 6 mm thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 6 mm thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 6 mm thick, one of the following requirements shall apply:

For fillers that are equal to or less than 19 mm thick, the shear strength of the bolts shall be multiplied by the factor \([1 - 0.0154(t - 6)]\), where \( t \) is the total thickness of the fillers up to 19 mm;

The fillers shall be extended beyond the joint and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers;

The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

The joint shall be designed to prevent slip at required strength levels in accordance with Section 10.10.3.8.

10.10.6 Splices

Groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

10.10.7 Bearing Strength

The design bearing strength, \( \phi R_n \), and the allowable bearing strength, \( R_n/\Omega \), of surfaces in contact shall be determined for the limit state of bearing (local compressive yielding) as follows:

\[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]
The nominal bearing strength, $R_n$, is defined as follows for the various types of bearing:

For milled surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners:

$$ R_n = 1.8F_y A_{pb} $$  \hspace{1cm} (10.10.7.1)

where

$F_y$ = specified minimum yield stress, MPa

$A_{pb}$ = projected bearing area, mm$^2$

For expansion rollers and rockers:

If $d \leq 635$ mm

$$ R_n = 1.2(F_y - 90)d/20 $$  \hspace{1cm} (10.10.7.2)

If $d > 635$ mm

$$ R_n = 30.2(F_y - 90)\sqrt{d}/20 $$  \hspace{1cm} (10.10.7.3)

where

$d$ = diameter, mm

$l$ = length of bearing, mm

### 10.10.8 Column Bases and Bearing on Concrete

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the design bearing strength, $P_p$, and the allowable bearing strength, $P_{p}/\Omega_c$, for the limit state of concrete crushing are permitted to be taken as follows:

$$ \phi_c = 0.60 \text{ (LRFD)} \quad \Omega_c = 2.50 \text{ (ASD)} $$

The nominal bearing strength, $P_p$, is determined as follows:

On the full area of a concrete support:

$$ P_p = 0.8f'_c A_1 $$  \hspace{1cm} (10.10.8.1)

On less than the full area of a concrete support:

$$ P_p = 0.8 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7f'_c A_1 $$  \hspace{1cm} (10.10.8.2)

where

$A_1$ = area of steel concentrically bearing on a concrete support, mm$^2$

$A_2$ = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm$^2$

### 10.10.9 Anchor Rods and Embedments

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section 10.2.2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table 10.10.3.2.

Larger oversized and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using structural or plate washers to bridge the hole.

When horizontal forces are present at column bases, these forces should, where possible, be resisted by bearing against concrete elements or by shear friction between the column base plate and the foundation. When anchor rods are designed to resist horizontal force the base plate hole size, the anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

### 10.10.10 Flanges and Webs with Concentrated Forces

This section applies to single- and double-concentrated forces applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single- concentrated force can be either tensile or compressive. Double-
concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the required strength exceeds the available strength as determined for the limit states listed in this section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Section 10.10.10.8. Doublers shall also meet the design requirement in Section 10.10.10.9.

Stiffeners are required at unframed ends of beams in accordance with the requirements of Section 10.10.10.7.

10.10.10.1 Flange Local Bending

This section applies to tensile single-concentrated forces and the tensile component of double-concentrated forces.

The design strength, $R_n$, and the allowable strength, $R_n / \Omega$, for the limit state of flange local bending shall be determined as follows:

$$R_n = 6.25t_f^2F_{yf}$$  \hspace{1cm} (10.10.1.1)

$\phi = 0.90 \text{ (LRFD)} \hspace{0.5cm} \Omega = 1.67 \text{ (ASD)}$

where

$F_{yf}$ = specified minimum yield stress of the flange, MPa

$t_f$ = thickness of the loaded flange, mm

If the length of loading across the member flange is less than 0.15$b_f$, where $b_f$ is the member flange width, Equation 10.10.1.1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than 10$t_f$, $R_n$ shall be reduced by 50 percent.

When required, a pair of transverse stiffeners shall be provided.

10.10.10.2 Web Local Yielding

This section applies to single-concentrated forces and both components of double-concentrated forces.

The available strength for the limit state of web local yielding shall be determined as follows:

$\phi = 1.00 \text{ (LRFD)} \hspace{0.5cm} \Omega = 1.50 \text{ (ASD)}$

The nominal strength, $R_n$, shall be determined as follows:

When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member $d$,

$$R_n = (5k + N)F_{yw}t_w$$  \hspace{1cm} (10.10.2.1)

When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member $d$,

$$R_n = (2.5k + N)F_{yw}t_w$$  \hspace{1cm} (10.10.3.1)

where

$k$ = distance from outer face of the flange to the web toe of the fillet, mm

$F_{yw}$ = specified minimum yield stress of the web, MPa

$N$ = length of bearing (not less than $k$ for end beam reactions), mm

$t_w$ = web thickness, mm

When required, a pair of transverse stiffeners or a doubler plate shall be provided.

10.10.10.3 Web Crippling

This section applies to compressive single-concentrated forces or the compressive component of double-concentrated forces.

The available strength for the limit state of web local crippling shall be determined as follows:

$\phi = 0.75 \text{ (LRFD)} \hspace{0.5cm} \Omega = 2.00 \text{ (ASD)}$

The nominal strength, $R_n$, shall be determined as follows:
When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to \( d/2 \):

\[
R_n = 0.80 \frac{t_w^2}{t_f} \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \frac{E_f t_f}{t_w} \tag{10.10.10.4}
\]

When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than \( d/2 \):
For \( N/d \leq 0.2 \)

\[
R_n = 0.40 \frac{t_w^2}{t_f} \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \frac{E_f t_f}{t_w} \tag{10.10.10.5a}
\]

For \( N/d > 0.2 \)

\[
R_n = 0.40 \frac{t_w^2}{t_f} \left[ 1 + \left( \frac{4N}{d} - 0.2 \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \frac{E_f t_f}{t_w} \tag{10.10.10.5b}
\]

where
\( d \) = overall depth of the member, mm
\( t_f \) = flange thickness, mm

When required, a transverse stiffener or pair of transverse stiffeners, or a doubler plate extending at least one-half the depth of the web shall be provided.

10.10.10.4 Web Sidesway Buckling

This Section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The available strength of the web shall be determined as follows:
\( \phi = 0.85 \) (LRFD) \( \Omega = 1.76 \) (ASD)

The nominal strength, \( R_n \), for the limit state of web sidesway buckling shall be determined as follows:

If the compression flange is restrained against rotation:
For \((h/t_w)/(l/b_f) \leq 2.3\)

\[
R_n = C_r t_w t_f \left[ 1 + 0.4 \left( \frac{h}{t_w} \right) \left( \frac{t_w}{l/b_f} \right)^3 \right] \tag{10.10.10.6}
\]

For \((h/t_w)/(l/b_f) > 2.3\), the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate shall be provided.

If the compression flange is not restrained against rotation
For \((h/t_w)/(l/b_f) \leq 1.7\)

\[
R_n = C_r t_w t_f \left[ 0.4 \left( \frac{h}{t_w} \right) \left( \frac{t_w}{l/b_f} \right)^3 \right] \tag{10.10.10.7}
\]

For \((h/t_w)/(l/b_f) > 1.7\) the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations 10.10.10.6 and 10.10.10.7, the following definitions apply:

\( b_f \) = flange width, mm
\( C_r = 6.62 \times 10^6 \) MPa when \( M_o < M_r \) (LRFD) or \( 1.5 \ M_o < M_r \) (ASD) at the location of the force
= 3.31 × 10^6 MPa when \( M_c \geq M_y \) (LRFD) or 1.5 \( M_c \geq M_y \) (ASD) at the location of the force
\( h \) = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, mm
\( l \) = largest laterally unbraced length along either flange at the point of load, mm
\( t_f \) = flange thickness, mm
\( t_w \) = web thickness, mm

10.10.10.5 **Web Compression Buckling**

This Section applies to a pair of compressive *single-concentrated forces* or the compressive components in a pair of *double-concentrated forces*, applied at both flanges of a member at the same location.

The available strength for the limit state of web local buckling shall be determined as follows:

\[
R_n = \frac{24t_w^3 \sqrt{E t_f}}{h} \tag{10.10.10.8}
\]

\( \phi = 0.90 \) (LRFD) \( \Omega = 1.67 \) (ASD)

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than \( d/2 \), \( R_n \) shall be reduced by 50 percent.

When required, a single transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending the full depth of the web shall be provided.

10.10.10.6 **Web Panel Zone Shear**

This section applies to *double-concentrated forces* applied to one or both flanges of a member at the same location.

The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

\( \phi = 0.90 \) (LRFD) \( \Omega = 1.67 \) (ASD)

The nominal strength, \( R_n \), shall be determined as follows:

When the effect of panel-zone deformation on frame stability is not considered in the analysis:

For \( P_t \leq 0.4P_c \)

\[ R_n = 0.60F_yd_c t_w \tag{10.10.10.9} \]

For \( P_t > 0.4P_c \)

\[ R_n = 0.60F_yd_c t_w \left( 1 + \frac{1.4}{P_t/P_c} \right) \tag{10.10.10.10} \]

When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For \( P_t \leq 0.75P_c \)

\[ R_n = 0.60F_yd_c t_w \left( 1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w} \right) \tag{10.10.10.11} \]

For \( P_t > 0.75P_c \)

\[ R_n = 0.60F_yd_c t_w \left( 1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w} \right) \left( 1.9 - \frac{1.2P_t}{P_c} \right) \tag{10.10.10.12} \]

In Equations 10.10.10.9 through 10.10.10.12, the following definitions apply:

\( A \) = column cross-sectional area, \( \text{mm}^2 \)

\( b_{cf} \) = width of column flange, \( \text{mm} \)

\( d_b \) = beam depth, \( \text{mm} \)

\( d_c \) = column depth, \( \text{mm} \)

\( F_y \) = specified minimum yield stress of the column web, MPa

\( P_c \) = \( P_y \), N (LRFD)

\( P_c \) = 0.6 \( P_y \), N (ASD)
\[ P_r = \text{required strength}, \text{N} \]
\[ P_y = F_y A, \text{axial yield strength of the column, N} \]
\[ t_{fl} = \text{thickness of the column flange, mm} \]
\[ t_w = \text{column web thickness, mm} \]

When required, doubler plate(s) or a pair of diagonal stiffeners shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

See Section 10.10.10.9 for doubler plate design requirements.

10.10.10.7 Unframed Ends of Beams and Girders

At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided.

10.10.10.8 Additional Stiffeners Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Section 10.4 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available limit state strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections 10.5.6.2 and 10.10.4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section 10.10.7.

Transverse full depth bearing stiffeners for compressive forces applied to a beam or plate girder flange(s) shall be designed as axially compressed members (columns) in accordance with the requirements of Sections 10.5.6.2 and 10.10.4.4.

The member properties shall be determined using an effective length of 0.75h and a cross section composed of two stiffeners and a strip of the web having a width of 25t_w at interior stiffeners and 12t_w at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and diagonal stiffeners shall comply with the following additional criteria:

The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.

The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, and greater than or equal to the width divided by 15.

Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in 10.10.10.5 and 10.10.10.7.

10.10.10.9 Additional doubler Plate Requirements for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Section 10.5.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Section 10.4.

Doubler plates required for shear strength (see Section 10.10.10.6) shall be designed in accordance with the provisions of Section 10.7.

In addition, doubler plates shall comply with the following criteria:

The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.