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:

<u>ACI International (ACI)</u>

ACI 318-02 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-02 *Metric Building Code Requirements for Structural Concrete and Commentary*

<u>American Institute of Steel Construction, Inc. (AISC)</u>

AISC 303-05 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement No. 2*

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

<u>American Society of Civil Engineers (ASCE)</u>

SEI/ASCE 7-02 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-99 *Standard Calculation Methods for Structural Fire Protection*

<u>*American Society of Mechanical Engineers (ASME)*</u>

ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

ASME B46.1-95 *Surface Texture, Surface Roughness, Waviness, and Lay*

<u>*American Iron and Steel Institute (AISI)*</u>

North American Specification for the Design of Cold Formed Steel Structural Members (AISI/COS/NASPEC 2001).

Code of Standard Practice for Cold-Formed Steel Structural Framing, 2005

<u>ASTM International (ASTM)</u>

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*

A216/A216M-93(2003) *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

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*

A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Test- ing of Structural Steel*

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*

A847-99a(2003) *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

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 Pro- cess (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*

<u>American Welding Society (AWS)</u>

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*

AWS A5.29:1998 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

<u>Research Council on Structural Connections (RCSC)</u>

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

ASTM A500, ASTM A501, ASTM A618, ASTM A847, BDS 1031:2006

(iii) Pipe

ASTM A53/A53M, Gr. B, BDS 1031:2006

(iv) Plates

ASTM A36/A36M, ASTM A242/A242M, ASTM A283/A283M, ASTM A514/A514M, ASTM A529/A529M, ASTM A572/A572M, ASTM A588/A588M, ASTM A709/A709M, ASTM A852/A852M, ASTM A1011/A1011M, BDS 1122:1987 Reaffirmed 2007

(v) Bars

ASTM A36/A36M, ASTM A529/A529M, ASTM A572/A572M, ASTM A709/A709M, BDS ISO 6935-1:2006, BDS ISO 6935-2:2006

(vi) Sheets

ASTM A606, A1011/A1011M SS, HSLAS, AND HSLAS-F, BDS 1122:1987 Reaffirmed 2007

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 $+21^0$ 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 $+21^0$ 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:

ASTM A36/A36M, ASTM A193/A193M, ASTM A354, ASTM A449, ASTM A572/A572M, ASTM A588/A588M, ASTM F1554

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:

$$\leq \qquad (10.2.1)$$

where

R_u= required strength (LRFD)

R_n= nominal strength, specified in Sections 10.2 through 10.20

Ø = resistance factor, specified in Sections 10.2 through 10.20

ØR_n= 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:

$$\leq /\Omega \qquad (10.2.2)$$

where

R_a= required strength (ASD)

R_n= nominal strength, specified in Sections 10.2 through 10.20

Ω = safety factor, specified in Sections 10.2 through 10.20

R_n/Ω= 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 , 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 λ_p from Table 10.2. 1. If the width-thickness ratio of one or more compression elements exceeds λ_p , but does not exceed λ_r from Table 10.2.1, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r, 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 welds 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 welds 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 welds 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-Δ and P-δ *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 10.2.1
Limiting Width-Thickness Ratios for Compression Elements

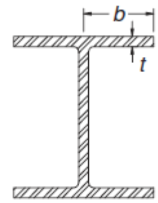
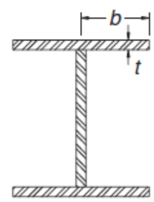
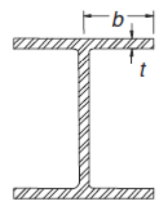
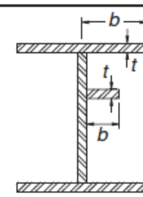
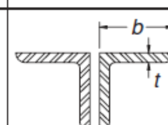
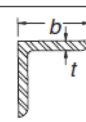
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	Limiting Width-Thickness Ratios λ_r (noncompact)	Example
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$ [a],[b]	
	3	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	b/t	NA	$0.56\sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_c E/F_y}$ [a]	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

TABLE 10.2.1 (cont.) Limiting Width-Thickness Ratios for Compression Elements

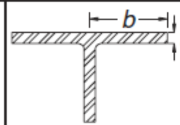
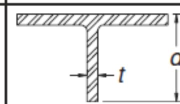
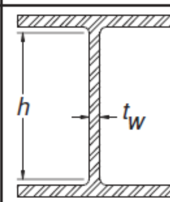
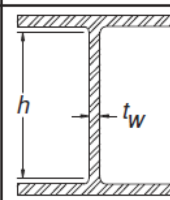
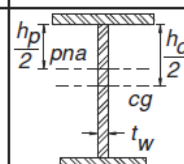
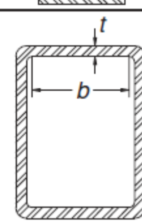
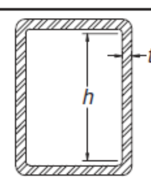
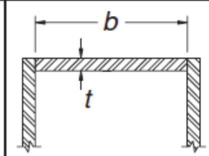
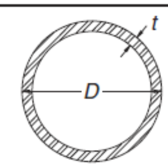
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	Limiting Width-Thickness Ratios λ_r (noncompact)	Example
	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
Stiffened Elements	9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
Stiffened Elements	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
Stiffened Elements	11	Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left(0.54\dfrac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
Stiffened Elements	12	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	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
Stiffened Elements	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

TABLE 10.2.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	Limiting Width-Thickness Ratios λ_r (noncompact)	Example
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections				
	In uniform compression	D/t	NA	$0.11\,E/F_y$	
	In flexure	D/t	$0.07\,E/F_y$	$0.31\,E/F_y$	

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

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_e, 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-Δ 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-Δ* and *P-δ 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:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \quad (10.3.2.1a)$$

$$P_r = P_{nt} + B_2 P_{lt} \quad (10.3.2.1b)$$

Where,

$$B_1 = \frac{C_m}{1-\alpha P_r / P_{e1}} \geq 1 \quad (10.3.2.2)$$

For members subjected to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.

For members in which $B_1 \leq 1.05$, 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, $M_r = B_2 (M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1-\frac{\alpha \sum P_{nt}}{\sum P_e^2}} \geq 1 \quad (10.3.2.3)$$

And

$\alpha = 1.00$(LRFD) $\alpha = 1.60$ (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_{lt} = 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,

$$C_m = 0.6 - 0.4(M_1/M_2) \tag{10.3.2.4}$$

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_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, N

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \tag{10.3.2.5}$$

$\sum P_{e2}$ = elastic critical buckling resistance for the story determined by sideway buckling analysis, N

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\sum P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2}$$

(10.3.2.6a)

For all types of lateral load resisting systems, it is permitted to use

$$\sum P_{e2} = R_M \frac{\sum HL}{\Delta_H}$$

(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

I = 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 sideway buckling analysis

Δ_H = first-order interstory drift due to lateral forces, mm. Where Δ_H varies over the plan area of the structure, Δ_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 Δ_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 *required strengths* of members, *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 $0.002Y_i$, 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_e , determined from a sidesway 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:

$$\alpha P_r \le 0.5 P_y \qquad (10.3.2.7)$$

Where,

$\alpha = 1.0$(LRFD) $\alpha = 1.6$ (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

$$N_i = 2.1(\Delta/L)Y_i \ge 0.0042Y_i \qquad (10.3.2.8)$$

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level *i*, N

Δ/L = the maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the design loads, mm. Where Δ varies over the plan area of the structure, Δ 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, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_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:

$$P_n = F_y A_g \qquad (10.4.2.1)$$

$\phi_t = 0.90$ (LRFD) $\Omega_t = 1.67$ (ASD)

g) For tensile rupture in the net section:

$$P_n = F_u A_e \qquad (10.4.2.2)$$

$\phi_t = 0.75$ (LRFD) $\Omega_t = 2.00$ (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_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 , 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.

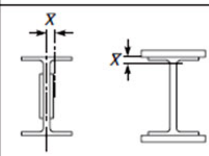
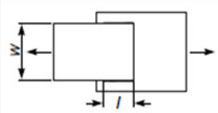
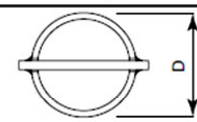
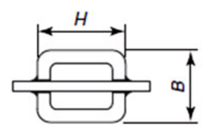
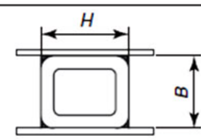
10.4.3.3 **Effective Net Area:** The effective area of tension members shall be determined as follows:

$$A_e = A_n U \qquad (10.4.3.1)$$

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

Case	Description of Element		Shear Lag Factor, U	Example
1	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)		$U = 1.0$	——
2	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.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	——
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B + H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B + H)}$	
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)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	——
		with web connected with 4 or more fasteners per line in the direction of loading	$U = 0.70$	——
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	——
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	——

l = length of connection, mm ; w = plate width, mm ; $\bar{x}$ = connection eccentricity, mm ; B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, mm ; H = 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, $\emptyset_t P_n$ and the allowable tensile strength, P_n/Ω_t, of pin-connected members, shall be the lower value obtained according to the limit states of tensile rupture, shear rupture, bearing, and yielding.

a) For tensile rupture on the net effective area:

$$P_n = 2tb_{eff}F_u \quad (10.4.5.1)$$

$\emptyset_t = 0.75$ (LRFD) $\Omega_t = 2.00$ (ASD)

h) For shear rupture on the effective area:

$$P_n = 0.6F_uA_{sf} \quad (10.4.5.2)$$

$\emptyset_{sf} = 0.75$ (LRFD) $\Omega_{sf} = 2.00$ (ASD)

where

$A_{sf} = 2t(a + d/2)$, 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

$b_{eff} = 2t + 16$, 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 = pin diameter, mm

t = 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 $2b_{eff} + d$ 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 × b_{eff}.

The corners beyond the pin hole are permitted to be cut at 45^0 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, $\phi_c P_n$, and the allowable compressive strength, P_n / Ω_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.

$\phi_c = 0.90$ (LRFD) $\Omega_c = 1.67$ (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 = 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.

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.

$$P_n = F_{cr} A_g \qquad (10.5.3.1)$$

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

(a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ (or $F_e \geq 0.44 F_y$)

$$= [0.658] \qquad (10.5.3.2)$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ (or $F_e < 0.44 F_y$)

$$= 0.877 \qquad (10.5.3.3)$$

where

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,

$$= \frac{2}{()^2} \qquad (10.5.3.4)$$

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:

$$P_n = F_{cr} A_g \qquad (10.5.4.1)$$

For double-angle and tee-shaped compression members:

$$= \left(\frac{+}{2}\right)\left[1 - \sqrt{1 - \frac{4}{(+)^2}}\right] \qquad (10.5.4.2)$$

where F_{cry} is taken as F_{cr} from Equation 10.5.3.2 or 10.5.3.3, for *flexural buckling*

about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$= \frac{}{\bar{}_0^2} \qquad (10.5.4.3)$$

For all other cases, F_{cr} 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:

$$= \left[\frac{2}{()^2} + \right]\frac{1}{+} \qquad (10.5.4.4)$$

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

$$= \left(\frac{+}{2}\right)\left[1 - \sqrt{1 - \frac{4}{(+)^2}}\right] \qquad (10.5.4.5)$$

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

$$(-)(-)(-) - {}^2(-)\left(\frac{0}{\bar{}_0}\right)^2 - {}^2(-)\left(\frac{0}{\bar{}_0}\right)^2 = 0 \qquad (10.5.4.6)$$

where

A_g = gross area of member, mm^2

C_w = warping constant, mm^6

$$\bar{}_0^2 = {}_0^2 + {}_0^2 + \frac{+}{} \qquad (10.5.4.7)$$

$$= 1 - \frac{{}_0^2 + {}_0^2}{\bar{}_0^2} \qquad (10.5.4.8)$$

$$= \frac{2}{()^2} \qquad (10.5.4.9)$$

$$= \frac{2}{()^2} \qquad (10.5.4.10)$$

$$= \left(\frac{2}{()^2} + \right)\frac{1}{\bar{}_0^2} \qquad (10.5.4.11)$$

G = shear modulus of elasticity of steel = 77 200 MPa

I_x, I_y = moment of inertia about the principal axes, mm^4

J = torsional constant, mm^4

K_z = *effective length factor* for torsional buckling

x_o, y_o = coordinates of shear center with respect to the centroid, mm

$\bar{r}_0$ = 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, Pn , 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 $0 \leq \frac{L}{r_x} \leq 80$

$= 72 + 0.75$ (10.5.5.1)

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

$= 32 + 1.25 \leq 200$ (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 $0 \leq \frac{L}{r_x} \leq 75$

$= 60 + 0.8$ (10.5.5.3)

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

$= 45 + \leq 200$ (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:

$$\left(\right) = \sqrt{\left(\right)_0^2 + \left(\cdot\right)^2} \qquad (10.5.6.1)$$

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

$$\left(\right) = \sqrt{\left(\right)_0^2 + 0.82\frac{^2}{(1+^2)}\left(\cdot\right)^2} \qquad (10.5.6.2)$$

Where

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

$\left(\frac{KL}{r}\right)_0$ = 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

α = separation ratio = h/2rib

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.*

$$P_n = F_{cr} A_g \tag{10.5.7.1}$$

a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$ (or $F_e \geq 0.44\, QF_y$)

$$= \left[0.658\right] \tag{10.5.7.2}$$

m) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{QF_y}}$ (or $F_e < 0.44\, QF_y$)

$$= 0.877 \tag{10.5.7.3}$$

Where

F_e = 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_e 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

= $Q_s\, Q_a$ 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 $\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}$

$$= 1.0 \tag{10.5.7.4}$$

When $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$

$$= 1.415 - 0.74\,(\;)\sqrt{\;} \quad (10.5.7.5)$$

When $b/t \geq 1.03\sqrt{E/F_y}$

$$= \frac{0.69}{(\;)^2} \quad (10.5.7.6)$$

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

When $\frac{b}{t} \leq 0.64\sqrt{\frac{Ek_c}{F_y}}$

$$= 1.0 \quad (10.5.7.7)$$

When $0.64\sqrt{\frac{Ek_c}{F_y}} < b/t \leq 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$= 1.415 - 0.65\,(\;)\sqrt{\;} \quad (10.5.7.8)$$

when $\frac{b}{t} > 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$= \frac{0.90}{(\;)^2} \quad (10.5.7.9)$$

Where $k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

For single angles

When $\frac{b}{t} \leq 0.45\sqrt{\frac{E}{F_y}}$

$$= 1.0 \quad (10.5.7.10)$$

When $0.45\sqrt{E/F_y} < b/t \leq 0.91\sqrt{E/F_y}$

$$= 1.34 - 0.76\,(\;)\sqrt{\;} \quad (10.5.7.11)$$

When $b/t > 0.91\sqrt{E/F_y}$

$$= \frac{0.53}{(\;)^2} \quad (10.5.7.12)$$

Where

b = full width of longest angle leg, mm

For stems of tees

When $\frac{d}{t} \leq 0.75\sqrt{\frac{E}{F_y}}$

$$= 1.0 \quad (10.5.7.13)$$

When $0.75\sqrt{\frac{E}{F_y}} < d/t \leq 1.03\sqrt{\frac{E}{F_y}}$

$$= 1.908 - 1.22\,(\;)\sqrt{\;} \quad (10.5.7.14)$$

When $d/t > 1.03\sqrt{\frac{E}{F_y}}$

$$= \frac{0.69}{(\;)^2} \quad (10.5.7.15)$$

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, Qa , for slender stiffened elements is defined as follows:

$$= \quad (10.5.7.16)$$

A = total cross-sectional area of member, mm2

Aeff = summation of the effective areas of the cross section based on the reduced effective width, be , mm2

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

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

$$= 1.92\sqrt{}\left[1 - \frac{0.34}{(/)}\sqrt{}\right] \leq \quad (10.5.7.17)$$

Where

f is taken as Fcr with Fcr calculated based on Q = 1.0.

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

$$= 1.92\sqrt{}\left[1 - \frac{0.38}{(/)}\sqrt{}\right] \leq \quad (10.5.7.18)$$

Where $f = P_n/A_{eff}$

For axially-loaded circular sections:

When $0.11\frac{E}{F_y} < \frac{D}{t} < 0.45\frac{E}{F_y}$

$$= \; = \frac{0.038}{(/)} + \frac{2}{3} \quad (10.5.7.19)$$

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, $\emptyset_b M_n$, and the allowable flexural strength, M_n/Ω_b, shall be determined as follows:

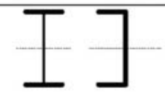
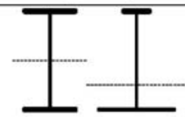
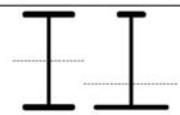
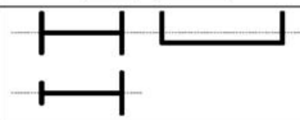
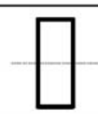
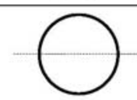
a) For all provisions in this Section 10.6

   ϕ_b = 0.90 (LRFD) Ω_b = 1.67 (ASD)

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

n) 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

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

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

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0 \qquad (10.6.1.1)$$

Where

M_{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

$= 0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$, 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**

$$M_n = M_p = F_y Z_x \tag{10.6.2.1}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, MPa

Z_x = plastic section modulus about the x-axis, mm^3

10.6.2.2 **Lateral –Torsional Buckling**

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

o) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \tag{10.6.2.2}$$

p) When $L_b > L_r$

$$M_n = F_{cr}S_x \le M_p \tag{10.6.2.3}$$

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

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{10.6.2.4}$$

and where

E = modulus of elasticity of steel = 200 000 MPa

J = torsional constant, mm^4

S_x = elastic section modulus taken about the x-axis, mm^3

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{10.6.2.5}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_o}{Jc}\right)^2}} \tag{10.6.2.6}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{10.6.2.7}$$

and

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

For a channel: $c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$ (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

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

q) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_x}{\lambda^2} \tag{10.6.3.2}$$

Where

$\lambda = \frac{b_f}{2t_f}$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table 10.2.4.1

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

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**

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \tag{10.6.4.1}$$

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 $L_p < L_b \leq L_r$

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

s) When $L_b > L_r$

$$M_n = F_{cr}S_{xc} \leq R_{pc}M_{yc} \tag{10.6.4.3}$$

Where

$$M_{yc} = F_yS_{xc} \tag{10.6.4.4}$$

$$F_{cr} = \frac{C_b\pi^2E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h_0}\left(\frac{L_b}{r_t}\right)^2} \tag{10.6.4.5}$$

For $\frac{I_{yc}}{I_y} \leq 0.23$, J shall be taken as zero.

The stress, F_L , is determined as follows:

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

$$F_L = 0.7F_y \tag{10.6.4.6a}$$

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

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

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

$$L_p = 1.1r_t\sqrt{\frac{E}{F_y}} \tag{10.6.4.7}$$

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

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

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

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

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

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

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yc}} \tag{10.6.4.9b}$$

Where

$M_p = Z_x F_y \leq 1.6 S_{xc} F_y$

S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, mm^3

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

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

$\lambda_{rw} = \lambda_r$ 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:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_0}{d} + \frac{1}{6} a_w \frac{h^2}{h_0 d}\right)}} \qquad (10.6.4.10)$$

Where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \qquad (10.6.4.11)$$

b_{fc}= compression flange width, mm

t_{fc}= compression flange thickness, mm

For I-shapes with channel caps or cover plates attached to the compression flange:

r_t = 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 = 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.

t) For sections with noncompact flanges

$$M_n = \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \qquad (10.6.4.12)$$

u) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \qquad (10.6.4.13)$$

Where

F_L is defined in Equations 10.6.4.6a and 10.6.4.6b

R_{pc} = 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

$\lambda = \frac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$ limiting slenderness for a compact flange, Table 10.2.4.1

$\lambda_{rf} = \lambda_r$ limiting slenderness for a noncompact flange, Table 10.2.4.1

10.6.4.4 **Tension Flange Yielding**

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

v) When $S_{xt} < S_{xc}$

$$M_n = R_{pt} M_{yt} \qquad (10.6.4.14)$$

Where

$M_{yt} = F_y S_{xt}$

The web *plastification* factor corresponding to the tension flange yielding limit state, R_{pt} , is determined as follows:

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

$$R_{pt} = \frac{M_p}{M_{yt}} \tag{10.6.4.15a}$$

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

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \leq \frac{M_p}{M_{yt}} \tag{10.6.4.15b}$$

Where

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

$\lambda_{pw} = \lambda_p$, 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**

$$M_n = R_{pg} F_y S_{xc} \tag{10.6.5.1}$$

10.6.5.2 **Lateral-Torsional Buckling**

$$M_n = R_{pg} F_{cr} S_{xc} \tag{10.6.5.2}$$

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

w) When $L_p < L_b \leq L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq F_y \tag{10.6.5.3}$$

x) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \leq F_y \tag{10.6.5.4}$$

where

L_p is defined by Equation 10.6.4.7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \tag{10.6.5.5}$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \tag{10.6.5.6}$$

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**

$$M_n = R_{pg} F_{cr} S_{xc} \quad (10.6.5.7)$$

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

y) For sections with noncompact flanges

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

z) For sections with slender flanges

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \quad (10.6.5.9)$$

Where

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \frac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, 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

$$M_n = \left[M_p - (M_p - 0.7F_y S_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \quad (10.6.6.2)$$

bb) For sections with slender flanges

$$M_n = F_{cr} S_y \quad (10.6.6.3)$$

where

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f}\right)^2} \quad (10.6.6.4)$$

$\lambda = \frac{b}{t}$

$\lambda_{pf} = \lambda_p$, 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

S_y for a channel shall be taken as the minimum section modulus

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.

10.6.7.1 **Yielding**

$$M_n = M_p = F_y Z \qquad (10.6.7.1)$$

Where

Z = plastic section modulus about the axis of bending, mm^3

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

$$M_n = M_p - \left(M_p - F_y S\right)\left(3.57\frac{b}{t}\sqrt{\frac{F_y}{E}} - 4.0\right) \leq M_p \qquad (10.6.7.2)$$

cc) For sections with slender flanges

$$M_n = F_y S_{eff}$$

(10.6.7.3)

where

S_{eff} is the *effective section modulus* determined with the *effective width* of the compression flange taken as:

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

10.6.7.3 **Web Local Buckling**

a) For compact sections, the limit state of web local buckling does not apply.
dd) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S_x\right)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \leq M_p \qquad (10.6.7.5)$$

10.6.8 Round HSS

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

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

10.6.8.1 **Yielding**

$$M_n = M_p = F_y Z \qquad (10.6.8.1)$$

10.6.8.2 **Local Buckling**

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

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y\right)S \qquad (10.6.8.2)$$

ee) For sections with slender walls

$M_n = F_{cr} S$

(10.6.8.3)

where

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \tag{10.6.8.4}$$

S = 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**

$$M_n = M_p \tag{10.6.9.1}$$

Where

$$M_p = F_y Z_x \leq 1.6\, M_y \text{ for stems in tension} \tag{10.6.9.2}$$

$$\leq M_y \quad \text{for stems in compression} \tag{10.6.9.3}$$

10.6.9.2 **Lateral-Torsional Buckling**

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_yGJ}}{L_b}\left[B + \sqrt{1 + B^2}\right] \tag{10.6.9.4}$$

Where

$$B = \pm 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \tag{10.6.9.5}$$

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**

$$M_n = F_{cr} S_{xc} \tag{10.6.9.6}$$

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

$$F_{cr} = F_y\left(1.19 - 0.50\left(\frac{b_f}{2t_f}\right)\sqrt{\frac{F_y}{E}}\right) \tag{10.6.9.7}$$

For slender sections

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

10.6.10 Single Angle

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

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**

$$M_n = 1.5 M_y \tag{10.6.10.1}$$

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 $M_e \le M_y$

$$M_n = \left(0.92 - \frac{0.17M_e}{M_y}\right)M_e \tag{10.6.10.2}$$

When $M_e > M_y$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right)M_y \le 1.5M_y \tag{10.6.10.3}$$

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

$$M_e = \frac{0.66Eb^4tC_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} - 1\right) \tag{10.6.10.4a}$$

With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} + 1\right) \tag{10.6.10.4b}$$

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:

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \tag{10.6.10.5}$$

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

$$M_e = \frac{4.9EI_zC_b}{L^2}\left(\sqrt{\beta_w^2 + 0.052\left(\frac{Lt}{r_z}\right)^2} + \beta_w\right) \tag{10.6.10.6}$$

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

β_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 β_w shall be used.

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

$$M_n = F_y S_c \left(2.43 - 1.72 \left(\frac{b}{t}\right) \sqrt{\frac{F_y}{E}} \right) \quad (10.6.10.7)$$

For sections with slender legs

$$M_n = F_{cr} S_c \quad (10.6.10.8)$$

Where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \quad (10.6.10.9)$$

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 $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis and rounds:

$$M_n = M_p = F_y Z \leq 1.6 M_y \quad (10.6.11.1)$$

10.6.11.2 **Lateral-Torsional Buckling**

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

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2}\right) \frac{F_y}{E} \right] M_y \leq M_P \quad (10.6.11.2)$$

gg) For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \leq M_P \quad (10.6.11.3)$$

Where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \quad (10.6.11.4)$$

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

d = depth of rectangular bar, mm

L_b = length between points that are either braced against lateral displace- ment 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

$$M_n = F_n S \quad (10.6.12.1)$$

Where

S = lowest elastic section modulus relative to the axis of bending, mm^3

10.6.12.1 **Yielding**

$$F_n = F_y \quad (10.6.12.2)$$

10.6.12.2 **Lateral-Torsional Buckling**

$$F_n = F_{cr} \le F_y \quad (10.6.12.3)$$

Where

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

10.6.12.3 **Local Buckling**

$$F_n = F_{cr} \le F_y \quad (10.6.12.4)$$

Where

F_{cr} = 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 $F_u A_{fn} \ge Y_t F_y A_{fg}$, 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:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad (10.6.13.1)$$

Where

A_{fg} = gross tension flange area, calculated in accordance with the provisions of Section 10.4.3.1, mm^2

A_{fn} = net tension flange area, calculated in accordance with the provisions of Section 10.4.3.2, mm^2

Y_t = 1.0 for $F_y / F_u \le 0.8$

= 1.1 otherwise

10.6.13.2 **Proportioning Limits for I-Shaped Members**

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

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \qquad (10.6.13.2)$$

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

For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 11.7\sqrt{\frac{E}{F_y}} \qquad (10.6.13.3)$$

For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.42E}{F_y} \qquad (10.3.13.4)$$

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 strength 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

$$a' = w \qquad (10.6.13.5)$$

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

$$a' = 1.5w \qquad (10.6.13.6)$$

When there is no weld across the end of the plate

$$a' = 2w \qquad (10.6.13.7)$$

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*, $\emptyset_v V_n$ and the *allowable shear strength*, V_n/Ω_v, shall be determined as follows.

For all provisions in this section except Section 10.7.2.1a:

$\emptyset_v$ =0.90 (LRFD) $\Omega_v = 1.67$ (ASD)

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

$$V_n = 0.6F_y A_w C_w \tag{10.7.2.1}$$

For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$

$\emptyset_v$ =1.00 (LRFD) $\Omega_v = 1.50$ (ASD)

And

$$C_v = 1.0 \tag{10.7.2.2}$$

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 $h/t_w \le 1.10\sqrt{k_v\,E/F_y}$

$$C_v = 1.0 \tag{10.7.2.3}$$

For $1.10\sqrt{k_v\,E/F_y} < h/t_w \le 1.37\sqrt{k_v\,E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v\,E/F_y}}{h/t_w} \tag{10.7.2.4}$$

For $h/t_w > 1.37\sqrt{k_v\,E/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \tag{10.7.2.5}$$

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 $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \qquad (10.7.2.6)$$

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 $[260/(h/t_w)]^2$;

$$2\,A_w/\left(A_{fc} + A_{ft}\right) > 2.5; \text{ or}$$

$$h/b_{fc} \text{ or } h/b_{ft} > 6.0$$

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 $h/t_w \leq 1.10\sqrt{k_v\,E/F_y}$

$$V_n = 0.6F_yA_w \tag{10.7.3.1}$$

For $h/t_w > 1.10\sqrt{k_v\,E/F_y}$

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

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:

$$(b/t)_{st} \le 0.56\sqrt{\frac{E}{F_{yst}}}$$

$$A_{st} > \frac{F_y}{F_{yst}}\left[0.15D_sht_w(1-C_v)\frac{V_r}{V_c} - 18t_w^2\right] \ge 0 \tag{10.7.3.3}$$

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;* $\emptyset_v, V_n$ (LRFD) or V_n/Ω_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 = 2*ht* 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

$$V_n = F_{cr}A_g/2 \tag{10.7.6.1}$$

Where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{10.7.6.2a}$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \qquad (10.7.6.2b)$$

but shall not exceed 0.6 F_y

A_g = gross area of section based on design wall thickness, mm^2

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 ≤ (Iyc / Iy) ≤ 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 Iyc is the moment of inertia about the y-axis referred to the compression flange, mm4 .

For $\frac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad (10.8.1.1a)$$

For $\frac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad (10.8.1.1b)$$

Where

Pr = required axial compressive strength, N

Pc = available axial compressive strength, N

Mr = required flexural strength, N-mm

Mc = 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)

Pr = required axial compressive strength using LRFD load combinations, N

Pc = ϕc Pn = design axial compressive strength, determined in accordance with Section10.5, 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 /Ωc = 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 $\sqrt{1+\frac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

Where

$$P_{ey} = \frac{\pi^2 E I_y}{L_b^2}$$

For design according to Section 10.2.3.4 (ASD)

Pr = required tensile strength using ASD load combinations, N

Pc = Pn /Ωt = 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 $\sqrt{1+\frac{1.5P_a}{P_{ey}}}$ for axial tension that acts concurrently with flexure

Where

$$P_{ey} = \frac{\pi^2 E I_y}{L_b^2}$$

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 Pc , Mr , and Mc determined in the plane of bending.

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

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

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 ≥ 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.

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

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

$F_{bw}, F_{bz} = \frac{\phi_b M_n}{S} =$ *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.

ϕ_c = *resistance factor* for compression = 0.90

ϕ_t = resistance factor for tension (Section 10.4.2)

ϕ_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

$F_a = \frac{F_{cr}}{\Omega_c}$ = *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

$F_{bw}, F_{bz} = \frac{M_n}{\Omega_b S} =$ *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.

Ω_c = *safety factor* for compression = 1.67

Ω_t = safety factor for tension (Section 10.4.2)

Ω_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, $\emptyset_T T_n$ and the allowable torsional strength, T_n/Ω_T, for round and rectangular HSS shall be determined as follows:

$\emptyset_T$ =0.90 (LRFD) Ω_T =1.67 (ASD)

The nominal torsional strength, Tn , according to the limit states of torsional yielding and torsional buckling is:

$$T_n = F_{cr} C \qquad (10.8.3.1)$$

Where

C is the HSS torsional constant

Fcr shall be determined as follows:

For round HSS, Fcr shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \qquad (10.8.3.2a)$$

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \qquad (10.8.3.2b)$$

but shall not exceed 0.6 F_y ,

where

L = length of the member, mm

D = outside diameter, mm

For rectangular *HSS*

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

$$F_{cr} = 0.6F_y \qquad (10.8.3.3)$$

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

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

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

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

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

$$\left(\frac{P_r}{P_c}+\frac{M_r}{M_c}\right)+\left(\frac{V_r}{V_c}+\frac{T_r}{T_c}\right)^2 \leq 1.0 \quad (10.8.3.6)$$

Where

For design according to Section 10.2.3.3 (LRFD)

P_r = *required axial strength* using *LRFD load combinations*, N

P_c =$\emptyset P_n$, *design tensile* or *compressive strength* in accordance with Section 10.4 or 10.5, N

M_r = *required flexural strength* using *LRFD load combinations*, N-mm

M_c =$\emptyset_b M_n$, *design flexural strength* in accordance with Section 10.6, N-mm

V_r = *required shear strength* using LRFD load combinations, N

V_c = $\emptyset_v V_n$, *design shear strength* in accordance with Section 10.7, N

T_r = *required torsional strength* using LRFD load combinations, N-mm

T_c =$\emptyset_T T_n$, *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 = required axial strength using *ASD load combinations*, N

P_c = P_n/Ω, *allowable tensile* or *compressive strength* in accordance with Section 10.4 or 10.5, N

M_r = required flexural strength using ASD load combinations determined in accordance with Section 10.2.5, N-mm

M_c = M_n / Ω_b , *allowable flexural strength* in accordance with Section 10.6, N-mm

V_r = required shear strength using ASD load combinations, N

V_c = V_n /Ω_v , *allowable shear strength* in accordance with Section 10.7, N

T_r = required torsional strength using ASD load combinations, N-mm

T_c = T_n /Ω_T , *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*, $\emptyset_T F_n$, and the *allowable torsional strength*, F_n/Ω_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:

$\emptyset_T$ = 0.90 (LRFD) Ω_T = 1.67 (ASD)

a) For the limit state of yielding under normal stress

$$F_n = F_y \quad (10.8.3.7)$$

ii) For the limit state of shear yielding under shear stress

$$F_n = 0.6\,F_y \quad (10.8.3.8)$$

jj) For the limit state of buckling

$$F_n = F_{cr} \quad (10.8.3.9)$$

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 s*hall 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_r, 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,* ϕR_n, and the *allowable strength* R_n / Ω , 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 *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

Welding Process	Welding Position F (flat), H (horiz.), V (vert.),OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove 60^0 V	Depth of Groove
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	All		
Submerged Arc (SAW)	F	J or U Groove 60^0 Bevel or V	
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	F, H	45^0 Bevel	Depth of Groove
Shielded Metal Arc (SMAW)	All	45^0 Bevel	Depth of Groove Minus 3 mm
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45^0 Bevel	Depth of Groove Minus 3 mm

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 90^0 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

Welding Process	Flare Bevel Groove[a]	Flare V Groove
GMAW and FCAW-G	$\frac{5}{8}R$	$\frac{3}{4}R$
SMAW and FCAW-S	$\frac{5}{16}R$	$\frac{5}{8}R$
SAW	$\frac{5}{16}R$	$\frac{1}{2}R$

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

Material Thickness of Thinner Part Joined, mm	Minimum Effective Throat Thickness,[a] mm
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6
Over 19 to 38	8
Over 38 to 57	10
Over 57 to 150	13

[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.*

10.10.2.2.2 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

Material Thickness of Thinner	Minimum Size of Fillet [a]
To 6 inclusive	3
Over 6 to 13	5
Over 13 to 19	6

[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, β ,

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \qquad (10.10.2.1)$$

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 β 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 be not 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 $2\frac{1}{4}$ 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,* ϕR_n and the allowable strength, R_n / Ω , 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>
<tr><th>Load Type and Direction Relative to Weld Axis</th><th>Pertinent Metal</th><th>φ and Ω</th><th>Nominal Strength (F_{BM} or F_w)</th><th>Effective Area (A_{BM} or A_w) mm^2</th><th>Required Filler Metal Strength Level[a][b]</th></tr>
<tr><td colspan="6">COMPLETE-JOINT-PENETRATION GROOVE WELDS</td></tr>
<tr><td>Tension
Normal to weld axis</td><td colspan="4">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></tr>
<tr><td>Compression
Normal to weld axis</td><td colspan="4">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></tr>
<tr><td>Tension or Compression
Parallel to weld axis</td><td colspan="4">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></tr>
<tr><td>Shear</td><td colspan="4">Strength of the joint is controlled by the base metal</td><td>Matching filler metal shall be used.[c]</td></tr>
<tr><td colspan="6">PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS</td></tr>
<tr><td rowspan="2">Tension
Normal to weld axis</td><td></td><td>φ = 0.90</td><td></td><td>See</td><td rowspan="13">Filler metal with a strength level equal to or less than matching filler metal is permitted.</td></tr>
<tr><td></td><td>φ = 0.80</td><td></td><td>See</td></tr>
<tr><td>Compression Column to Base Plate and column splices designed per 10.10.1.4(a)</td><td colspan="4">Compressive stress need not be considered in design of welds joining the parts.</td></tr>
<tr><td rowspan="2">Compression Connections of members designed to bear other than columns as described in 10.10.1.4(b)</td><td>Base</td><td>φ = 0.90
Ω = 1.67</td><td>F_y</td><td>See 10.10.4</td></tr>
<tr><td>Weld</td><td>φ = 0.80
Ω = 1.88</td><td>0.60 F_{EXX}</td><td>See 10.10.2.1.1</td></tr>
<tr><td rowspan="2">Compression Connections not finished-to-bear</td><td></td><td>φ = 0.90</td><td></td><td>See</td></tr>
<tr><td></td><td>φ = 0.80</td><td></td><td>See</td></tr>
<tr><td>Tension or Compression
Parallel to weld axis</td><td colspan="4">Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the</td></tr>
<tr><td rowspan="2">Shear</td><td>Base</td><td colspan="3">Governed by 10.10.4</td></tr>
<tr><td></td><td>φ =</td><td>0.60 F_{EXX}</td><td>See</td></tr>
</table>

TABLE 10.10.2.5 (Contd..): Available Strength of Welded Joints, N

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	**Nominal Strength**	**Effective Area**	**Required Filler Metal Strength Level**[a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS					
Shear	Base	Governed by 10.10.4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	ϕ = 0..75 Ω = 2.00	$0.60F_{EXX}^{[d]}$	See 10.10.2.2.1	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear Parallel to faying surface on the effective	Base	Governed by 10.10.4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
		ϕ =	$0.60F_{EXX}^{[d]}$		

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, ϕ = 0.80,Ω= 1.88 and 0.60 F_{EXX} as the nominal strength.

For the base metal

$$R_n = F_{BM}A_{BM} \qquad (10.10.2.2)$$

For the weld metal

$$R_n = F_w A_w \qquad (10.10.2.3)$$

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, mm^2

A_w = effective area of the weld, mm^2

The values of ϕ, Ω, 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,* ϕR_n and the allowable strength, R_n/Ω, of welds is permitted to be determined as follows:

ϕ = 0.75 (LRFD) Ω= 2.00 (ASD)

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

$$R_n = F_w A_w \qquad (10.10.2.4)$$

where

$$F_w = 0.60F_{EXX}(1.0 + 0.50\sin^{1.5}\theta) \qquad (10.10.2.5)$$

And

F_{EXX} = electrode classification number, MPa

Θ = angle of loading measured from the weld longitudinal axis, degrees

A_w = effective area of the weld, mm^2

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:

$$R_{nx} = \sum F_{wix} A_{wi} \qquad R_{ny} = \sum F_{wiy} A_{wi} \tag{10.10.2.6}$$

where

A_{wi} = effective area of weld throat of any *i*th weld element, mm^2

$$F_{wi} = 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta) f(p) \tag{10.10.2.7}$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \tag{10.10.2.8}$$

F_{wi} = nominal stress in any ith weld element, MPa

F_{wix} = x component of stress, F_{wi}

F_{wiy} = y component of stress, F_{wi}

p = Δ_i / Δ_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 Δ_u / r_i ratio, mm

Δ_I = 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 , mm $= r_i \Delta_u / r_{crit}$

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

$\Delta_u = 1.087(\theta + 6)^{-0.65} w \le 0.17w$, 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

$$R_n = R_{wl} + R_{wt} \tag{10.10.2.9a}$$

or

$$R_n = 0.85 R_{wl} + 1.5 R_{wt} \tag{10.10.2.9b}$$

where

R_{wl} = 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 ^{0}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

bearing-type connections.

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*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334

* 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 $2\frac{2}{3}$ 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 10.10.3.2: Nominal Stress of Fasteners and Threaded Parts, MPa

Description of Fasteners	Nominal Tensile Stress, F_{nt} , MPa	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, MPa
A307 bolts	310 [a][b]	165 [b] [c] [f]
A325 or A325M bolts, when threads are not excluded from shear planes	620 [e]	330 [f]
A325 or A325M bolts, when threads are excluded from shear planes	620 [e]	414 [f]
A490 or A490M bolts, when threads are not excluded from shear planes	780 [e]	414 [f]
A490 or A490M bolts, when threads are excluded from shear planes	780 [e]	520 [f]
Threaded parts meeting the requirements of Section 10.1.3.4, when threads are not excluded from shear planes	0.75 F_u [a][d]	0.40F_u
Threaded parts meeting the requirements of Section 10.1.3.4, when threads are excluded from shear planes	0.75 F_u [a][d]	0.50F_u

[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 Fy .

TABLE 10.10.3.3: Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27 [a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75

[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

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
16	28	22
20	34	26
22	38 [d]	28
24	42 [d]	30
27	48	34

[a] Lesser edge distances are permitted to be used provided provisions of Section 10.10.3.10, as appro- priate, 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

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
≤22	2	3	0.75d	0
24	3	3		
≥27	3	5		

[a] When length of slot is less than maximum allowable (see Table **10.10.3.3**), 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,* ϕR_n and the *allowable tension* or *shear strength,* R_n/Ω, 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:

$$R_n = F_n A_b \tag{10.10.3.1}$$

ϕ = 0.75 (LRFD) Ω= 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, mm^2

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:

$$R_n = F'_{nt} A_b \tag{10.10.3.2}$$

ϕ= 0.75 (LRFD) Ω= 2.00 (ASD)

where

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

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\emptyset F_{nv}} f_v \leq F_{nt}(\text{LRFD}) \tag{10.10.3.3a}$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{\emptyset F_{nv}} f_v \leq F_{nt}(\text{ASD}) \tag{10.10.3.3b}$$

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 trans- verse 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, ϕR_n and the allowable slip resistance, R_n /Ω , shall be determined for the *limit state* of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \qquad (10.10.3.4)$$

For connections in which prevention of slip is a serviceability limit state

ϕ= 1.00 (LRFD) Ω= 1.50 (ASD)

For connections designed to prevent slip at the required strength level

ϕ= 0.85 (LRFD) Ω= 1.76 (ASD)

Where

μ = 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:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \quad \text{(LRFD)} \qquad (10.10.3.5a)$$

$$k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} \quad \text{(ASD)} \qquad (10.10.3.5b)$$

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, ϕR_n and R_n/Ω, at bolt holes shall be determined for the *limit state* of bearing as follows:

ϕ = 0.75 (LRFD) Ω = 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

$$R_n = 1.2 L_c t F_u \leq 2.4 d t F_u \qquad (10.10.3.6a)$$

e) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5 L_c t F_u \leq 3.0 d t F_u \qquad (10.10.3.6b)$$

For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0 L_c t F_u \leq 2.0 d t F_u \qquad (10.10.3.6c)$$

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 = nominal bolt diameter, mm

F_u = specified minimum tensile strength of the connected material, MPa

L_c = 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 = 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,* ϕR_n and the *allowable strength,* R_n/Ω, 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:

$$R_n = F_y A_g \tag{10.10.4.1}$$

ϕ= 0.90 (LRFD) Ω = 1.67 (ASD)

For tensile rupture of connecting elements:

$$R_n = F_u A_e \tag{10.10.4.2}$$

ϕ= 0.75 (LRFD) Ω = 2.00 (ASD)

where

A_e = *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:

$$R_n = 0.60 F_y A_g \tag{10.10.4.3}$$

ϕ= 1.00 (LRFD) Ω = 1.50 (ASD)

For shear rupture of the element:

$$R_n = 0.60 F_u A_{nv} \tag{10.10.4.4}$$

ϕ= 0.75 (LRFD) Ω = 2.00 (ASD)

where

A_{nv} = 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

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (10.10.4.5)$$

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

where

A_{gv} = gross area subject to shear, mm^2

A_{nt} = *net area* subject to tension, mm^2

A_{nv} = 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 $KL/r \leq 25$

$$P_n = F_yA_g \qquad (10.10.4.6)$$

ϕ= 0.90 (LRFD) Ω = 1.67 (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, ϕR_n and the allowable bearing strength, R_n/Ω, of surfaces in contact shall be determined for the limit state of bearing (local compressive yielding) as follows:

ϕ= 0.75 (LRFD) Ω = 2.00 (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:

$$R_n = 1.8F_y A_{Pb} \quad (10.10.7.1)$$

where

F_y = specified minimum yield stress, MPa

A_{pb} = projected bearing area, mm^2

For expansion rollers and rockers:

If d ≤ 635 mm

$$R_n = 1.2(F_y - 90)ld/20 \quad (10.10.7.2)$$

If d > 635 mm

$$R_n = 30.2(F_y - 90)l\sqrt{d}/20 \quad (10.10.7.3)$$

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, $\phi_c P_p$, and the allowable bearing strength, P_p / Ω_c , for the *limit state* of *concrete crushing* are permitted to be taken as follows:

ϕ_c = 0.60 (LRFD) Ω_c = 2.50 (ASD)

The nominal bearing strength, P_p , is determined as follows:

On the full area of a concrete support:

$$P_P = 0.8f_c' A_1 \quad (10.10.8.1)$$

On less than the full area of a concrete support:

$$P_P = 0.8f_c' A_1\sqrt{A_2/A_1} \leq 1.7f_c' A_1 \quad (10.10.8.2)$$

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,* ϕR_n and the *allowable strength,* R_n / Ω, for the *limit state* of flange *local bending* shall be determined as follows:

$$R_n = 6.25 t_f^2 F_{yf} \qquad (10.10.10.1)$$

ϕ= 0.90 (LRFD) Ω = 1.67 (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 $0.15b_f$, where b_f is the member flange width, Equation 10.10.10.1 need not be checked.

When the concentrated *force* to be resisted is applied at a distance from the member end that is less than $10t_f$, 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:

ϕ= 1.00 (LRFD) Ω = 1.50 (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,*

$$R_n = (5k + N) F_{yw} t_w \qquad (10.10.10.2)$$

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* ,

$$R_n = (2.5k + N) F_{yw} t_w \qquad (10.10.10.3)$$

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:

ϕ = 0.75 (LRFD) Ω = 2.00 (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:*

$$R_n = 0.80t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad (10.10.10.4)$$

When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than *d /2:*

For $N/d \le 0.2$

$$R_n = 0.40t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad (10.10.10.5a)$$

For $N/d > 0.2$

$$R_n = 0.40t_w^2\left[1+\left(\frac{4N}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \qquad (10.10.10.5b)$$

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 $(h/t_w)/(l/b_f) \le 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1+0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \qquad (10.10.10.6)$$

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 $(h/t_w)/(l/b_f) \le 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \qquad (10.10.10.7)$$

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 × 10^6 MPa when $M_u < M_y$ (LRFD) or 1.5 $M_a < M_y$ (ASD) at the location of the force

= 3.31×10^6 MPa when $M_u \geq M_y$ (LRFD) or $1.5\,M_a \geq M_y$ (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:

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \qquad (10.10.10.8)$$

ϕ = 0.90 (LRFD) Ω = 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:

ϕ = 0.90 (LRFD) Ω = 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 $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \qquad (10.10.10.9)$$

For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w\left(1.4 - \frac{P_r}{P_c}\right) \qquad (10.10.10.10)$$

When frame stability, including plastic panel-zone deformation, is considered in the analysis:

For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w\left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \qquad (10.10.10.11)$$

For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w\left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \qquad (10.10.10.12)$$

In Equations 10.10.10.9 through 10.10.10.12, the following definitions apply:

A = *column* cross-sectional area, mm^2

b_{cf} = width of column flange, mm

d_b = *beam* depth, mm

d_c = column depth, 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 = *required strength*, N

P_y = $F_y A$, axial *yield strength* of the column, N

t_{cf} = thickness of the column flange, mm

t_w = 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.

Double*r* 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.